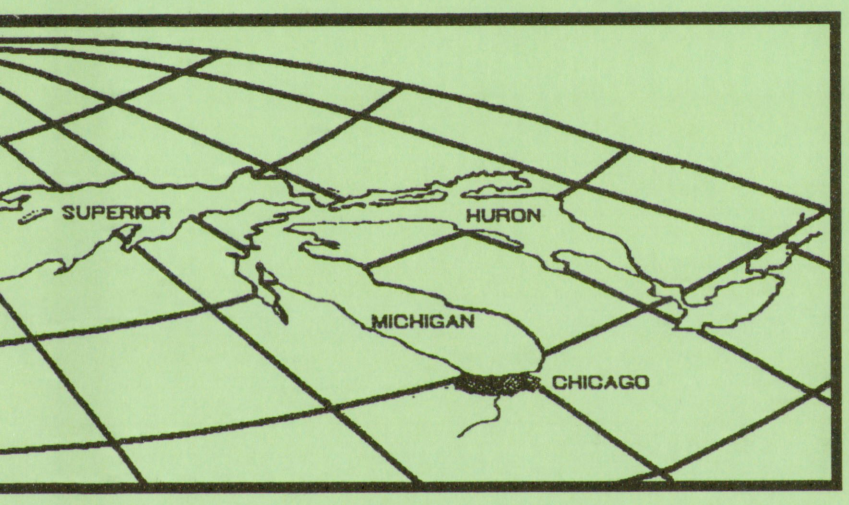
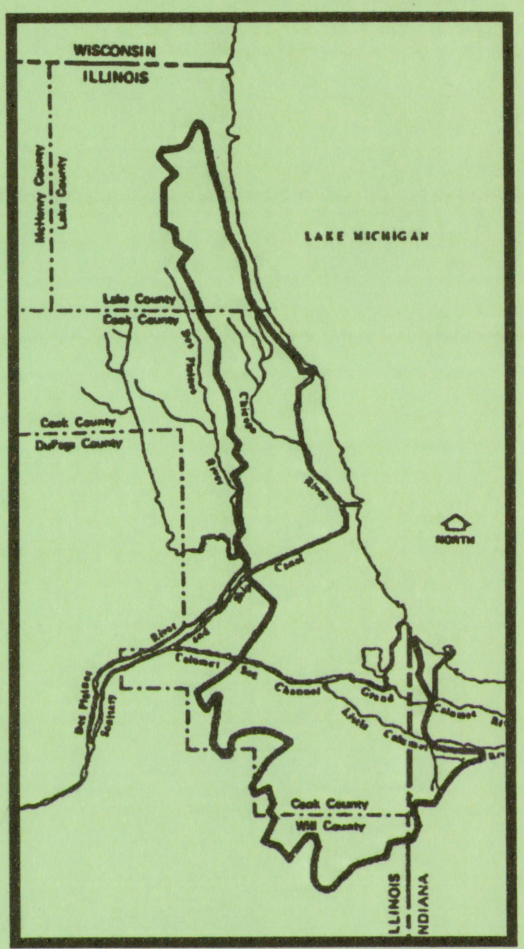
US Army Corps
Of Engineers
Chicago District

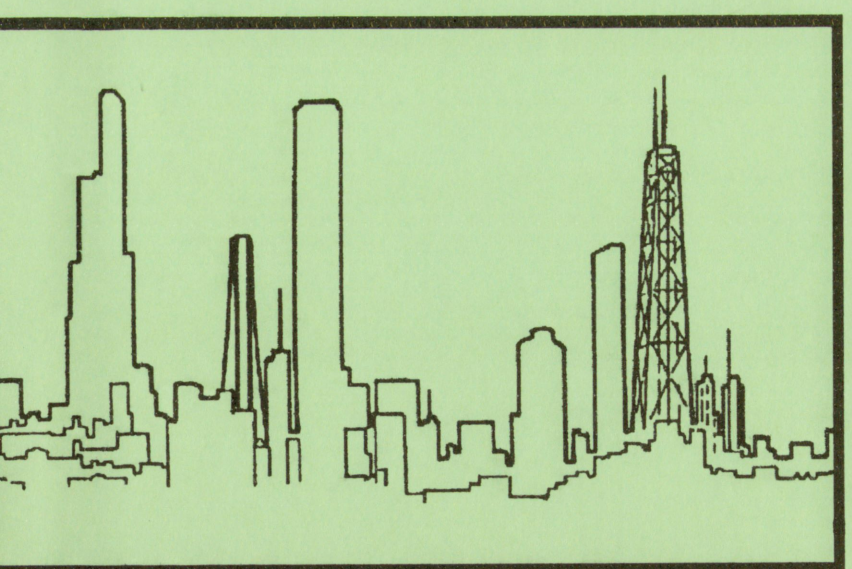
No. 1 Orig

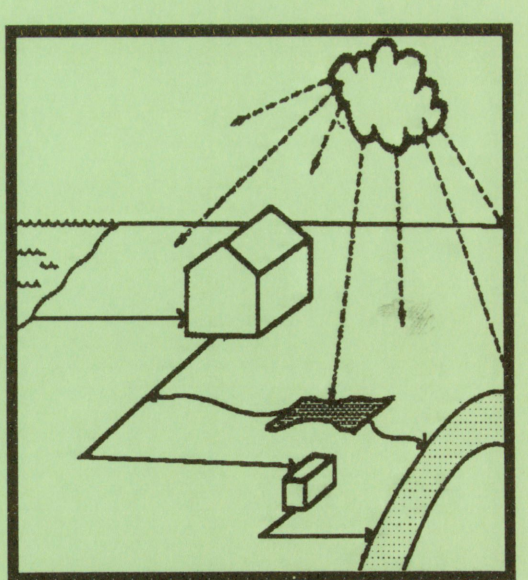
RECEIVED
OCT 11 1994
OFFICE OF THE CLERK
SUPREME COURT, U.S.

LAKE MICHIGAN DIVERSION ACCOUNTING WATER YEAR 1993 ANNUAL REPORT









U. S. Department of Justice

Office of the Solicitor General

Washington, D.C. 20530

October 11, 1994

RECEIVED
OCT 11 1994
OFFICE OF THE CLERK
SUPREME COURT, U.S.

Honorable William K. Suter
Clerk
Supreme Court of the United States
Washington, D.C. 20543

Re: Wisconsin v. Illinois, No. 1 Original (O.T. Term 1980)

Dear Mr. Suter:

The Supreme Court's Amended Decree in the above-captioned case directs the Army Corps of Engineers to prepare an annual report on the measure of water that the State of Illinois diverts from Lake Michigan pursuant to the Decree. See Wisconsin v. Illinois, 449 U.S. 48, 49 (1980). See also Water Resources Development Act of 1986, Pub. L. No. 99-662, § 1142, 100 Stat. 4253. The Corps provides a copy of that report to the parties in the litigation as well as to other interested entities.

In past years, the Army Corps of Engineers has sent a copy of the annual report directly to the Court. As we explained in our May 19, 1994 letter (copy attached), we have determined that the Solicitor General's Office, which customarily transmits government documents to the Clerk's Office, should assume the responsibility for providing the Clerk with copies of these and other similar reports for the Court's records. In accordance with that practice, we enclose a copy of the Corps' Lake Michigan Diversion Accounting Water Year 1993 Annual Report .

Sincerely,

Drew S. Days, III

Drew S. Days, III
Solicitor General

cc (without attachment or enclosure): See attached list

U. S. Department of Justice

Office of the Solicitor General

Washington, D.C. 20530

May 19, 1994

Honorable William K. Suter, Clerk
United States Supreme Court
Washington, D.C. 20543

Re: Wisconsin v. Illinois, No. 1 Original (O.T. Term 1980)

Dear Mr. Suter:

The Supreme Court's Amended Decree in the above-captioned case directs the Army Corps of Engineers to prepare an annual report on the measure of water that the State of Illinois diverts from Lake Michigan pursuant to the Decree. See Wisconsin v. Illinois, 449 U.S. 48, 49 (1980). See also Water Resources Development Act of 1986, Pub. L. No. 99-662, § 1142, 100 Stat. 4253. The Corps provides a copy of that report to the parties in the litigation, as well as to other interested entities.

In previous years, the Army Corps of Engineers has also sent a copy of the annual report directly to the Court. We have determined that the Solicitor General's Office, which customarily transmits government documents to the Clerk's Office, should assume the responsibility for providing the Clerk with copies of these and other similar reports for the Court's records. In accordance with that practice, we enclose a copy of the Corps' Lake Michigan Diversion Accounting Annual Report, Water Years 1990-1992.

Sincerely,

Drew S. Days, III
Solicitor General

cc: (without enclosure): See attached list

NO. 1 ORIGINAL

MS. LOUISE THOMPSON, ESQ.
CHIEF COUNSEL
PENNSYLVANIA DEPARTMENT
OF ENVIRONMENTAL RESOURCES
555 NORTH LANE STREET, SUITE 6015
CONSHOHOCKEN, PA 19428-2233

MR. P. GALLO, ESQ.
ASST. ATTORNEY GENERAL
OFFICE OF THE ATTORNEY GENERAL
NATURAL RESOURCES DIVISION,
STATE OF MINNESOTA
520 LAFAYETTE ROAD, 2ND FLOOR
ST. PAUL, MN 55155

MR. MARK MATUS, ESQ.
ASST. ATTORNEY GENERAL,
STATE OF MICHIGAN
640 LAW BUILDING
525 WEST OTTAWA STREET
LANSING, MI 48913

MS. S. EGGLESON, ESQ.
ASST. ATTORNEY GENERAL
STATE OF WISCONSIN
P.O. BOX 7857
MADISON, WI 53707-7857

MR. JAMES MORRIS
CHIEF, DIVISION OF WATER
OHIO DEPARTMENT OF NATURAL RESOURCES
1930 BELCHER DRIVE
FOUNTAIN SQUARE, E-3
COLUMBUS, OH 43224

MR. ALAN COOK, ESQ.
METROPOLITAN WATER RECLAMATION DISTRICT
OF GREATER CHICAGO
100 EAST ERIE STREET
CHICAGO, IL 60611

MR. DANIEL INJERED
CHIEF, LAKE MICHIGAN MANAGEMENT SECTION
ILLINOIS DEPARTMENT OF TRANSPORTATION
DIVISION OF WATER RESOURCES
310 SOUTH MICHIGAN AVENUE, ROOM 1606
CHICAGO, IL 60604

MR. GORDEN JOHNSON, ESQ.
ASST. ATTORNEY GENERAL
DEPARTMENT OF LAW
ENVIRONMENTAL PROTECTION BUREAU
120 BROADWAY, 26TH FLOOR, ROOM 102
NEW YORK, NY 10271

MR. MATTHEW DUNN, ESQ.
ASST. ATTORNEY GENERAL
CHIEF, ENVIRONMENTAL CONTROL DIVISION
STATE OF ILLINOIS CENTER
100 WEST RANDOLPH STREET, 12TH FLOOR
CHICAGO, IL 60601

MR. DONALD R. VONNAHME
DIRECTOR, WATER RESOURCES DIVISION
ILLINOIS DEPARTMENT OF TRANSPORTATION
3215 EXECUTIVE PARK DRIVE
SPRINGFIELD, IL 62794-9484

MR. STEPHEN BLANCHARD
ILLINOIS, DISTRICT CHIEF
UNITED STATES GEOLOGICAL SURVEY
WATER RESOURCES DIVISION
102 EAST MAIN STREET
URBANA, IL 61801

MR. RANDY A. PEPPLER
OFFICE OF SPECIAL PROGRAMS
ILLINOIS STATE WATER SURVEY
2204 GRIFFITH DRIVE
CHAMPAIGN, IL 61820-7495

MR. WILLIAM MORRIS
HYDROLOGIST
NATIONAL WEATHER SERVICE FORECAST OFFICE
333 WEST UNIVERSITY DRIVE
ROMEOVILLE, IL 60441

TABLE OF CONTENTS

	PAGE
INTRODUCTION	1
AUTHORITY FOR REPORT	1
HISTORY OF THE DIVERSION	1
SIGNIFICANT HYDROLOGIC EVENTS	6
OTHER SIGNIFICANT EVENTS	6
STATUS OF ACCOUNTING REPORTS	7
SOURCES OF DIVERSION	8
ACCOUNTING PROCEDURES	9
DIVERSION COMPUTATION	9
DIVERSION BUDGET CHECKS	10
CHANGES AND UPGRADES	10
ACCOMPLISHMENTS DURING FY 1993	11
REVISION OF COMPUTER MODELS	11
AVM GAGE REGRESSION MODELS	11
LAKEFRONT FLOW MEASUREMENTS	11
THIRD TECHNICAL COMMITTEE	13

PAGE

ACTIVITIES FOR FY 1994 AND 1995 14

ACCOUNTING REPORTS 14

THIRD TECHNICAL COMMITTEE 14

FLOW MEASUREMENTS 14

DIVERSION ACCOUNTING MANUAL 14

SOFTWARE IMPROVEMENT 14

SUMMARY AND CONCLUSIONS 15

SUMMARY 15

CONCLUSIONS 15

LIST OF TABLES

	PAGE
WY 1993 MONTHLY AND ANNUAL PRECIPITATION	6
STATUS OF STATE OF ILLINOIS' DIVERSION	7

LIST OF FIGURES

	PAGE
CANAL SYSTEM (BEFORE AND AFTER)	3
DIVERSION AT CHICAGO	4

LIST OF APPENDICES

	APPENDIX
SIGNIFICANT EVENTS (NON-HYDROLOGIC)	A
WY 1990 DIVERSION ACCOUNTING REPORT	B
INSTALLATION AND OPERATION OF THE 25-GAGE PRECIPITATION NETWORK	C
DIVERSION ACCOUNTING MODEL UPDATE FOR THE 25-GAGE PRECIPITATION NETWORK	D
COMPARISON, ANALYSIS, AND ESTIMATION OF DISCHARGE DATA FROM TWO ACOUSTIC VELOCITY METERS	E
LAKEFRONT LEAKAGE MEASUREMENTS	F
SLUICE GATE MEASUREMENTS	G
THIRD THREE MEMBER TECHNICAL COMMITTEE REPORT	H

EXECUTIVE SUMMARY

This document is the Water Year (WY) 1993 Annual Report of the Chicago District, U. S. Army Corps of Engineers activities in the monitoring and review of the accounting of Lake Michigan diversion flows through Chicago, Illinois as directed by 1980 amendment to the U. S. Supreme Court Decree. Additionally, this report serves to summarize the Corps' major accomplishment with respect to the mission as mandated by the Water Resources Development Act of 1986, PL99-662, Section 1142. This act gave the Corps complete responsibility for diversion accounting effective 1 October 1987. This report provides an overview and audit of flow measurements and accounting conducted by the Corps of Engineers from 1 October 1992 through 30 September 1993.

During WY 1993 the District has made some major changes to improve the accuracy and efficiency of the diversion accounting as listed below.

Modifications were made to the hydrologic runoff models and hydraulic sewer routing models in order to incorporate the 25-gage precipitation network into the WY 1990 diversion accounting procedures and computations. This included review and correction of map delineations for combined sewer special contributing areas, delineation of precipitation assigned areas for the 25-gage network, revisions and updates of land-use/land-cover delineations, and modifications to the hydraulic sewer routing models to reflect the revised precipitation network and land cover assignments.

Additionally, minor modifications were made to the hydrologic runoff model parameters to correct for past inconsistencies and improve parameter accuracy.

The Lake Michigan Diversion Accounting Report for WY 1990 has been completed. The State of Illinois diverted 3,531 cfs during WY 1990. This is 331 cfs greater than the 3,200 cfs 40 year average diversion specified in the modified decree. The running average of the diversion for WY 1981 through WY 1990 is 3,452 cfs, or 252 cfs over the annual allocation. The cumulative deviation is now -2,520 cfs-years. The negative sign indicates a cumulative flow deficit. The maximum allowable cumulative flow deficit specified in the decree is 2,000 cfs-years.

INTRODUCTION

The diversion of water from the Lake Michigan watershed is important to the Great Lake states and to the Canadian province of Ontario. The states and province that border the Great Lakes have concerns with diversions during periods of low lake levels and the long term effects of diversion. To insure these concerns are considered, the U.S. Army Corps of Engineers is responsible for the accounting of flow diverted from the Lake Michigan watershed.

The Water Year (WY) 1993 Annual Report on Lake Michigan Diversion Accounting presents activities by the Corps of Engineers in accounting for the diversion from Lake Michigan by the State of Illinois. The accounting of the diversion is performed according to the guidelines established in the 1980 modified U.S. Supreme Court Decree concerning the diversion.

Presented in this report is the history of the diversion and its accounting, the certification of WY 1990 diversion flows, a description of the sources of the diversion, a description of the accounting procedures, and a summary of all significant activities that occurred during WY 1993.

AUTHORITY FOR REPORT

Under the provisions of the U.S. Supreme Court Decree in the Wisconsin, et al v. Illinois et al, 388 U.S. 426, 87 S.Ct. 1774 (1967) as modified 449 U.S. 48, 101 S. CT. 557 (1980), the Corps of Engineers monitors the measurement and computation Lake Michigan diversion by the State of Illinois. The terms of the modified decree require the Corps of Engineers to prepare an annual report on the accounting of the Lake Michigan water diverted by the State of Illinois and actions taken by the involved agencies.

HISTORY OF THE DIVERSION

Water was first diverted from Lake Michigan at Chicago into the Mississippi River Basin with the completion of the Illinois and Michigan (I & M) Canal in 1848. The Illinois and Michigan Canal was primarily for transportation and diverted up to 500 cubic feet per second (cfs).

Development of the Chicago sewer system led to severe sanitation problems in the Chicago River by the mid to late 1800's. The newly constructed sewers moved water and wastes into the Chicago River, which until 1900 drained to Lake Michigan. The water quality of Lake Michigan deteriorated and contaminated the city's primary water supply.

A second problem during this time was an increase in the overbank flooding within the city. The sewer system expanded as more roads and buildings were built. This construction increased the rate and volume of runoff and resulted in increased flooding.

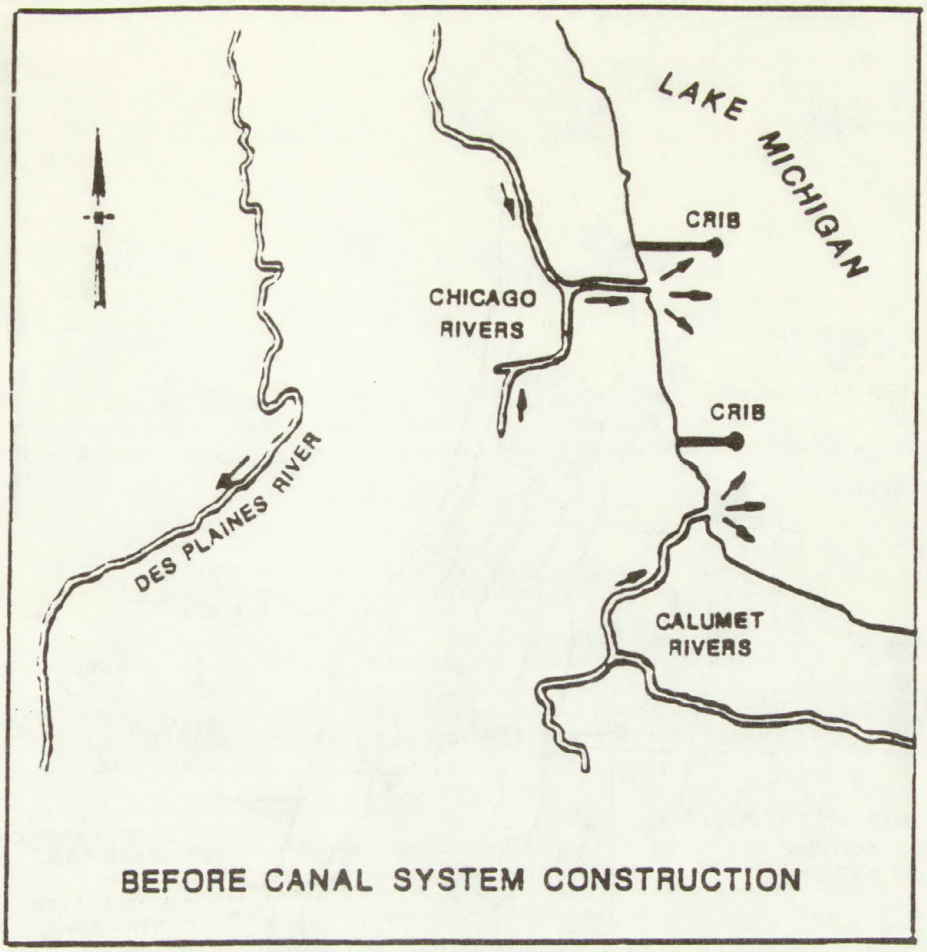
As a solution to the sanitation and flooding problems, the Chicago Sanitary and Ship Canal (CSSC) was built. The construction reversed the flow direction of the Chicago River (figure 1). The Chicago Sanitary And Ship Canal was completed in 1900 by the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC, formerly the Metropolitan Sanitary District of Greater Chicago, MSDGC). The Sanitary and Ship Canal followed the course of the older I & M Canal. This canal is much larger than the I & M Canal and can handle the Chicago River flow as well as increased shipping. The Chicago River Controlling Works were constructed at the mouth of the Chicago River in the 1930s. The lock and sluice gates regulate the amount of Lake Michigan water allowed to pass into the river and restricts river flooding entering Lake Michigan.

Between 1907 and 1910, the MWRDGC constructed a second sanitary canal called the North Shore Canal. This canal extends from Lake Michigan at Wilmette south 6.14 miles to the North Branch of the Chicago River. The Wilmette Controlling Works regulate the amount of Lake Michigan flow allowed down the channel.

Construction of a third canal, the Calumet Sag Canal, was completed in 1922. The canal connects Lake Michigan, through the Grand Calumet River, to the Sanitary and Ship Canal. This canal carried sewage from South Chicago, Illinois and East Chicago, Indiana. The O'Brien Lock and Dam located on the Calumet River, regulates the flow of Lake Michigan waters down the canal. Figure 2 shows the affected watershed.

Upon completion of the Chicago Sanitary and Ship Canal in 1901, the Secretary of War issued a permit authorizing a diversion of 4,167 cfs. In 1908 and 1913, the United States brought actions to enjoin the MWRDGC from diverting more than the 4,167 cfs previously authorized in 1901. The two actions were consolidated and the Supreme Court entered a decree on 5 January 1925 allowing the Secretary of War to issue diversion permits. In March 1925, the permit issued limited diversion 8,500 cfs, about the average then being used.

In 1922, 1925, and 1926, several Great Lakes States filed similar original actions in the U.S. Supreme Court seeking to restrict the diversion at Chicago. A Special Master, appointed by the U.S. Supreme Court to hear the combined three suits, found the 1925 permit to be valid and recommended dismissal of the action. The U.S. Supreme Court, however, reversed the Special Master's finding. Subsequently, the Court instructed the Special Master to determine the steps necessary for Illinois and MWRDGC to reduce the diversion. Consequently, a 1930 decree reduced the allowable diversion (which did not include domestic pumpage) in three steps: to 6,500 cfs after 1 July 1930; to 5,000 cfs after 30 December 1935; and to 1,500 cfs after 31 December 1938.


LAKE MICHIGAN
CRIB
CHICAGO RIVERS
CRIB
DES PLAINES RIVER
CALUMET RIVERS
BEFORE CANAL SYSTEM CONSTRUCTION

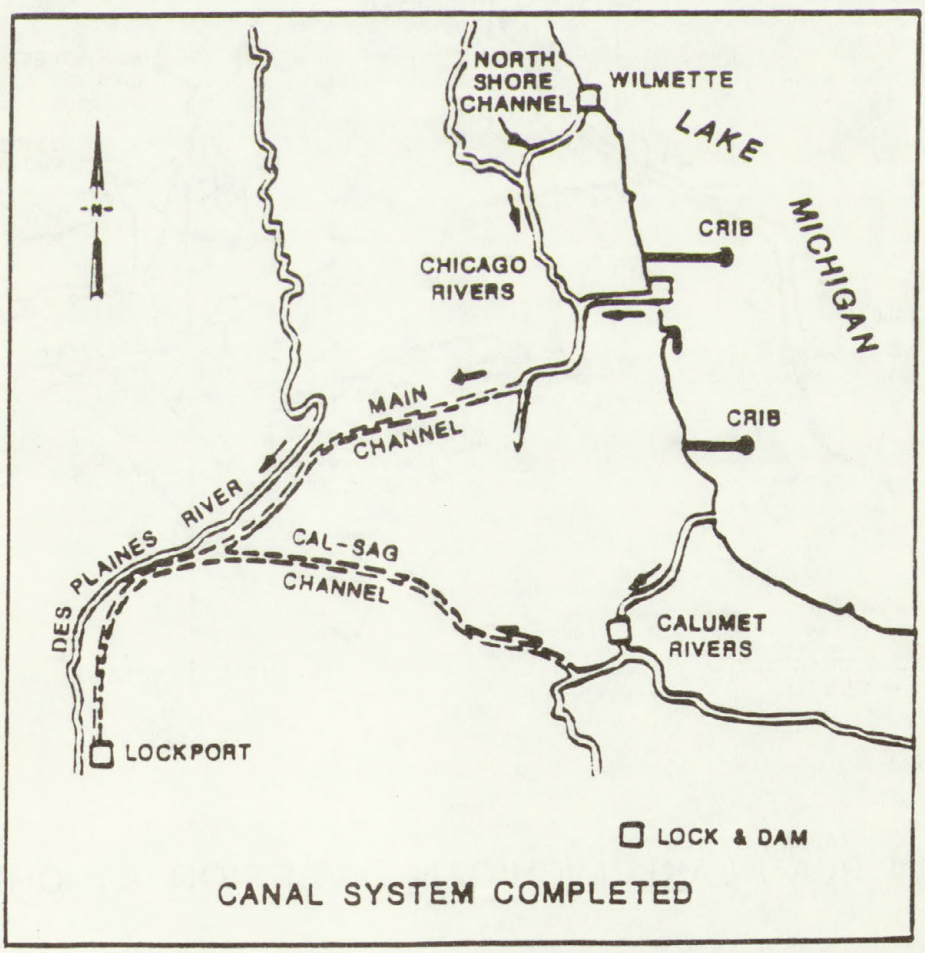
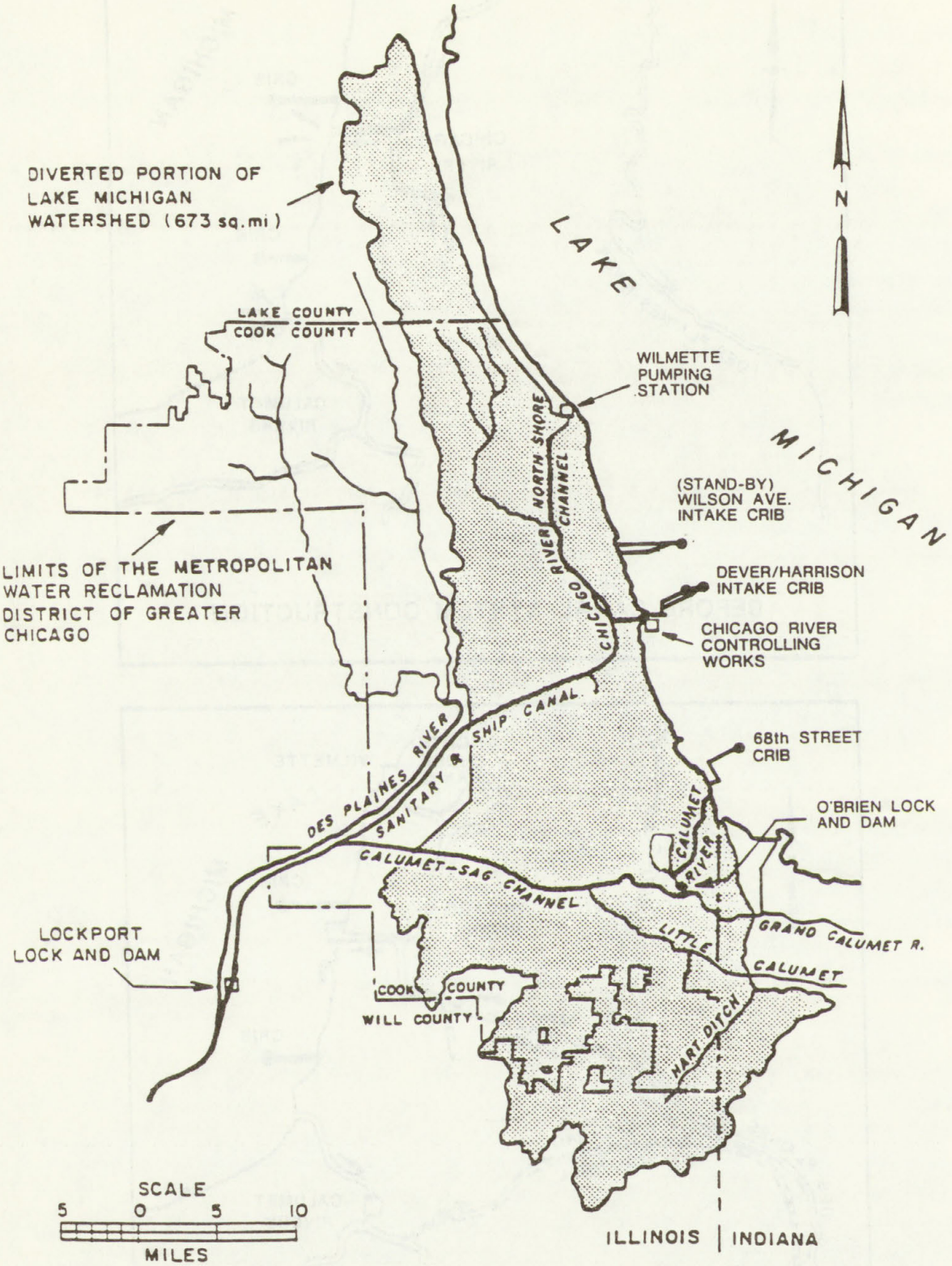

NORTH SHORE CHANNEL
WILMETTE
LAKE MICHIGAN
CRIB
CHICAGO RIVERS
MAIN CHANNEL
CRIB
DES PLAINES RIVER
CAL-SAG CHANNEL
CALUMET RIVERS
LOCKPORT
LOCK & DAM
CANAL SYSTEM COMPLETED

Figure 1



LOCATION PLAN-LAKE MICHIGAN DIVERSION AT CHICAGO

Figure 2

In 1967, an additional Supreme Court Decree limited the diversion of Lake Michigan water by the State of Illinois and its municipalities, including domestic pumpage, to a five year average of 3,200 cfs effective 1 March 1970. The 1967 Supreme Court Decree gave full responsibility to the State of Illinois for diversion measurements and computations. The role of the Corps of Engineers, as specified in the decree, was to be one of "general supervision and direction."

The 1967 decree was modified on 1 December 1980. This modified decree changed the beginning of the accounting year from 1 March to 1 October. The modified decree also extended the period for the running average diversion from five years to forty years beginning with WY 1981.

The amended decree contains three provisions that affected the role of the Corps of Engineers in the diversion accounting program. First, although the State of Illinois was primarily responsible for measurement and computation of diversion flows, the decree allowed the Corps of Engineers to participate in the function, subject to agreement and cost sharing with the State of Illinois. Negotiations were held on cost sharing the computation of the diversion. No agreement was reached due to lack of funding. The measurement and computation of the diversion continued to be done by the Illinois Department of Transportation (IDOT) through its consultants, the Northeastern Illinois Planning Commission (NIPC), MWRDGC, and the United States Geological Survey (USGS).

Second, the supervisory role for the Corps of Engineers increased so the Corps of Engineers was responsible for auditing the computations and measurements performed by the State of Illinois.

Third, the modified decree states that the Chief of Engineers shall appoint a Three Member Technical Committee to determine the best current engineering practice and scientific knowledge for measuring the diversion and to make recommendations as appropriate. The decree states that "...the members should be selected on the basis of recognized experience and technical expertise in flow measurement or hydrology." A technical committee is to be reconvened at least once every five years. The first Technical Committee convened in June 1981 and completed its work in April 1982. The second Technical Committee convened in July 1986, and completed their final report in November 1987. The third Technical Committee convened in February 1993.

The Water Resources Development Act of 1986 gave the Corps of Engineers responsibility for the computation of diversion flows as formerly done by the State of Illinois. The Corps of Engineers' new mission became effective 1 October 1987.

SIGNIFICANT HYDROLOGIC EVENTS

On 7-10 June 1993, heavy thunderstorms flooded southern Cook County. The 25 gage precipitation network gages in the area measured 3.5 to 4.0 inches of rain. One gage registered 6.4 inches over 24 hours and 16.5 inches for the month of June. Above average rainfall tends to increase the diversion, especially when severe storms, such as in June, fall.

Monthly precipitation compared to 30 year averages are shown in Table 1.

TABLE 1
WY 1993 Monthly And Annual Precipitation (Inches)
National Weather Service O'Hare Weather Station

MONTH	PRECIPITATION	1951-80 AVG. PRECIPITATION	DEVIATION	% DEVIATION
10/92	1.79	2.28	- 0.49	- 21
11/92	5.41	2.06	+ 3.35	+163
12/92	0.39	2.10	- 1.71	- 81
01/93	3.83	1.60	+ 2.23	+139
02/93	0.82	1.31	- 0.49	- 37
03/93	4.52	2.59	+ 1.93	+ 75
04/93	4.57	3.66	+ 0.91	+ 25
05/93	1.83	3.15	- 1.32	- 42
06/93	9.96	4.08	+ 5.88	+144
07/93	4.45	3.63	+ 0.82	+ 23
08/93	5.74	3.53	+ 2.21	+ 63
09/93	4.47	3.35	+ 1.12	+ 33
ANNUAL	47.78	33.34	+14.44	+ 43

OTHER SIGNIFICANT EVENTS

Three major non-hydrologic events occurred during WY 1993. The first major event was the convening of the third Technical Committee during February 1993. The committee's mission is to review the diversion accounting procedures and assure that the "best current engineering practice and scientific knowledge'" is being applied by the Corps of Engineers in computing the diversion. The second major event was the USGS flow measurements at the three lakefront structures from April through October 1993. The USGS used a highly accurate Acoustic Doppler Current Profiler (ADCP) to measure leakage and sluice gate flows at the Chicago River Controlling Works, O'Brien Lock and Dam, and the Wilmette Controlling Works. The third major event was the modifications to the hydrologic runoff and hydraulic sewer routing models for the purpose of incorporating the 25-gage precipitation network into the diversion accounting procedures. A large portion of this work was accomplished by Rust Environment and Infrastructure

under contract to the Corps of Engineers. The three major events listed above are documented within individual reports that are included as appendices to this WY 1993 Annual Report. A separate, detailed chronology of significant non-hydrologic events is included in appendix A.

STATUS OF ACCOUNTING REPORTS

Lake Michigan diversion flow data is summarized in accounting reports prepared on an annual basis as flows are certified. Since implementation of the modified Supreme Court Decree of 1 December 1980 and before this report, the Corps of Engineers has certified diversion flows for WY 1981 through WY 1989. The WY 1990 Lake Michigan Diversion Accounting Report is certified and included as appendix B of this Water Year 1993 Annual Report. The State of Illinois diverted 3,531 cfs during WY 1990. This is 331 cfs greater than the 3,200 cfs 40 year average diversion specified in the 1980 modified decree. Table 2 shows the accounting year, the certified flows, the running average flows, and the cumulative deviation from the allowable diversion of 3,200 cfs.

Table 2 Status of the State of Illinois' Diversion Under the 1980 Modified U.S. Supreme Court Decree

Accounting Year	Certified Flow (cfs)	Running Average (cfs)	Cumulative Deviation (cfs)
1981	3,106	3,106	94
1982	3,087	3,097	207
1983	3,613	3,269	-206
1984	3,432	3,310	-438
1985	3,472	3,342	-710
1986	3,751	3,410	-1,261
1987	3,774	3,462	-1,835
1988	3,376	3,451	-2,011
1989	3,378	3,443	-2,189
1990	3,531	3,452	-2,520

The running average diversion for the period WY 1981 through WY 1990 is 3,452 cfs, 252 cfs greater than the 3,200 cfs 40 year average diversion specified by the modified decree. Also, the annual average diversion has twice exceeded the 3680 cfs annual limit, the maximum number of times allowed in the decree. None of the years have exceeded the absolute annual maximum of 3840 cfs. The cumulative deviation, the sum of the differences between the annual average flows and 3,200 cfs, is -2,520 cfs-years. The negative cumulative deviation indicates a cumulative flow deficit. The decree specifies a

maximum allowable deficit of 2,000 cfs- years over the first 39 years of the 40 year averaging period.

Data collection and preparation for the WY 1991 and WY 1992 accounting reports will be performed by the Corps. Christopher B. Burke Engineering, under contract to the Corps of Engineers, will compute the diversion and prepare the accounting reports for WY 1991 and WY 1992 with assistance provided by the Corps of Engineers. Data collection and preparation for these reports began in Fiscal Year (FY) 1993. Certification of the WY 1991 and WY 1992 accounting is scheduled for FY 1995.

SOURCES OF DIVERSION

The Lake Michigan diversion consists of three primary components. These components are domestic pumpage from Lake Michigan used for water supply and not returned to Lake Michigan, stormwater runoff from the diverted Lake Michigan watershed, and direct diversions through the three lakefront control structures.

Direct diversion locations are at the Chicago River Controlling Works (CRCW), the O'Brien Lock and Dam, and the Wilmette Controlling Works. These controlling structures are located downtown, at the south end, and at the north end of the Chicago area, respectively.

The direct diversion consists of four components; lockage, discretionary flow, navigation makeup flow, and leakage. The lockage component is the flow used in locking vessels to and from the lake. The purpose of the discretionary diversion is to dilute effluent from sewage discharges. When large storms are forecast, the canal is drawn down before the storm to prevent flooding. If the runoff is not enough to refill the canal, navigation makeup water is passed. The leakage component is water estimated to pass, in an uncontrolled way, through or around the lakefront structures.

Domestic pumpage from Lake Michigan is used for water supply and its effluent is discharged to various Water Reclamation Plants (WRP's). Currently, the WRP's that divert domestic pumpage from the lake either discharge to the canal system or to the Des Plaines River and its tributaries. In the future as more communities convert to Lake Michigan water supply, water supply effluent may also discharge to the Fox River. The Fox River is approximately 35 miles west of downtown Chicago.

Stormwater runoff that previously drained to Lake Michigan through the Chicago River and the Calumet River now drains to the Chicago Sanitary and Ship Canal (CSSC) and the Calumet Sag Channel, respectively. The Calumet Sag Channel drains to the CSSC to ultimately drain into the Illinois River and the Mississippi River. The drainage area of the diverted Lake Michigan watershed is approximately 673 square miles.

ACCOUNTING PROCEDURES

Diversion accounting uses both measured and estimated flows. A series of hydrologic and hydraulic computer models use various meteorological data to simulate flows not measured. These simulated flows as well as measured flows are used to compute the diversion. Along with the diversion calculation, a number of water budgets verify simulated flows and estimate of the reliability of the computed diversion.

DIVERSION COMPUTATION

An acoustic velocity meter (AVM) was installed and has been operating at Romeoville (five miles upstream of the Lockport Powerhouse and three miles upstream of the Lockport Controlling Works) since 12 June 1984. The AVM directly measures total flow through the canal above both the Powerhouse and the Controlling Works. The overwhelming majority of the Lake Michigan diversion and some non-Lake Michigan flows pass through the AVM. The diversion accounting procedure uses the flow measured at Romeoville and deducts flows not accountable in the diversion. Diversion flows which bypass Lockport are added to yield the net computed diversion of water from Lake Michigan. This procedure represents the accounting technique as required by the modified Supreme Court Decree.

During WY 1990, the flow measured at Romeoville was approximately 106% of the annual diversion. Approximately 97% of the diversion was measured by the AVM.

Deductions from the Romeoville flow include runoff from 217 square miles of the Des Plaines River watershed discharged to the canal, groundwater supply effluent and groundwater seepage into the Tunnel and Reservoir Plan (TARP) tunnels discharged to the canal, and Indiana water supply discharged to the canal through the Calumet River system and the Calumet Sag Channel (see figure 1 for locations). The computer models of the Des Plaines watershed area estimate the runoff deduction. The groundwater pumpage deductions are obtained directly from pumping records. The Indiana water supply is computed from pumping records and a calculation to determine the portion of the water supply draining west to the Calumet Sag Channel.

The additions for diversion flow that do not flow through Romeoville are primarily Lake Michigan water supply pumpage effluent treated and released to the Des Plaines River or its tributaries. This flow is obtained directly through pumping records of the communities involved and accounts for approximately 3% of the diversion. As more communities convert to Lake Michigan water supply, the percentage will increase.

DIVERSION BUDGET CHECKS

Water budgets verify those flows not measured. Most of the budgets compare simulated flows to recorded flows and these comparisons indicate the accuracy of the diversion accounting. The four primary budgets are the budgets for the three major Water Reclamation Plants (WRP's) that serve the area involved in diversion accounting and the canal balance budget for the CSSC. The Upper Des Plaines pump station budget will also become a significant budget after the measurement problems are resolved. The remaining budgets estimate runoff from stream gaged areas in the Lake Michigan watershed or are budgets of non-simulated flows such as water supply pumpage. The budgets are discussed in more detail later in the WY 1990 accounting report in appendix B.

PROCEDURAL CHANGES AND UPGRADES

This section will highlight the changes and upgrades in the accounting procedure and their impacts on the computation of Lake Michigan diversion. The accounting of the diversion consists of measuring the water that flows past Lockport, subtracting the deductions and adding back the by-pass flows.

a. The installation of the AVM has produced the single largest impact of any change in the accounting procedures. The AVM flow measurements, plus the use of the regression equations (that translate Lockport flows into AVM flows) for filling in missing records, have significantly increased the flow records, and therefore the flows accountable to the State of Illinois. The average difference between the Lockport flows and the AVM record is equal to 192 cfs for WY 1990. The average difference was 313 cfs from WY 1986 through WY1989.

b. The new 25-gage precipitation network was introduced into the WY 1990 diversion accounting procedures during FY 1993. A report by the ISWS describing the installation and operation of this 25-gage precipitation network constitutes appendix C of this WY 1993 Annual Report. Subsequent modifications were made to the hydrologic runoff models and hydraulic sewer routing models in order to incorporate the 25-gage precipitation network into the diversion accounting modeling. Additionally, improvements and updates were made for land-use/land-cover delineations used in the models. Input parameters used in the hydrologic runoff models were slightly revised as well. All of these modifications have resulted in improved water balances at the primary calibration points, the three major MWRDGC water reclamation plants and the Upper Des Plaines pump station. Consequently, the deductible Des Plaines River watershed runoff is simulated with increased accuracy.

ACCOMPLISHMENTS DURING FY 1993

In each accounting year, various changes to the diversion procedures and other activities help to improve the accuracy and efficiency of the diversion accounting.

REVISION OF COMPUTER MODELS

As previously discussed, modifications were made to the hydrologic runoff models and hydraulic sewer routing models in order to incorporate the new WY 1990 25-gage precipitation network into the WY 1990 diversion accounting. The procedures for the model modifications are outlined in a report by Rust Environment and Infrastructure that constitutes appendix D of this report. Additional improvements and updates were made for land-use/land-cover delineations and hydrologic runoff input parameters. These revisions have resulted in improved water balances at the MWRDGC water reclamation plants and the Upper Des Plaines pumping station. Consequently, the Des Plaines River watershed runoff deduction is simulated with increased accuracy.

AVM GAGE REGRESSION EQUATIONS

New regression equations were developed for the Romeoville AVM gage for calculating flows when the AVM is not functioning. The USGS has reviewed the AVM flows for WY 1986 through WY 1992. The USGS revised the regression equations and the updated flows have been included in the Accounting Reports for WY 1986 through WY 1990. The USGS report included as appendix E of this report provides a complete description of the review process and the development of the revised regression equations. An additional update of the regression equations is underway to include the AVM's installed by MWRDGC in the Lockport Powerhouse during WY 1992.

LAKEFRONT FLOW MEASUREMENTS

The accuracy of the canal balance budget, used to check the diversion modeling, depends on the accuracy of the reported lakefront flows. While errors in the lakefront flows do not change the diversion accounting flows, errors reduce the accuracy and usefulness of the budget to indicate modeling errors. To quantify errors in the reported flows through and around the lakefront structures a series of measurements have been taken at all three lakefront structures, the Chicago Lock, the O'Brien Lock and Dam and the Wilmette Controlling Works.

The first series of measurements were taken in WY 1990. The report by the Corps of Engineers documenting the measurements is included in the WY 1990-92 Annual Report. Flow velocities were measured downstream of each of the three structures between 23 July and 27 July. Five flow measurements were taken at each site while the nearby structures maintained constant gate openings with all locks closed. Measured flows were compared to flows calculated using the MWRDGC methods. The measured flows were about twice the calculated flows.

Given the limited nature of the study, in time and equipment, the results are only a guide to the possible errors in the present lakefront flow calculations.

In WY 1993 a far more extensive series of measurements were undertaken from April through October 1993. The USGS conducted a detailed program utilizing an Acoustic Doppler Current Profiler (ADCP) and dye tests. The measurements focused on leakage and sluice gate flows. Detailed results of the leakage measurements can be found in the USGS report contained in appendix F. The report detailing USGS results on sluice gate measurements will be available during the summer of 1994. A brief letter report on the sluice gate measurements is included in this report as appendix G. A brief discussion of the leakage measurements is summarized below.

Chicago River Controlling Works: During a majority of the measurements the lock gates were in a state of disrepair which resulted in increased leakage. The lake gates began experiencing closing difficulties sometime during WY 1990. The river gates began experiencing closing difficulties sometime between the winter of 1992 and the winter of 1993. During September and October 1993 both lock gates were rehabilitated. This involved roller maintenance and side seal replacement. The effect of this maintenance was that the lake gates now fully close and the leakage through the side seals of both sets of gates has been reduced. Additional lock rehabilitation is currently under design. This future rehabilitation will include replacement of the bottom and front seals of both pairs of gates. This should further reduce the leakage passing through the lock chamber.

The river gates are of greater importance to diversion accounting since these gates act as the control gates. This is because the river gates are closed for the majority of the time during normal operations. The only time these gates are open during normal operations is shortly before an upstream lockage or immediately after a downstream lockage.

Over the period of measurement, the most significant leakage occurred through the lake gates while the river gates were open. Although there is considerable leakage during this gate configuration the annual effect is minimal since this gate setting occurs only for a short time prior to an upstream lockage or immediately after a downstream lockage. A better estimate of the normal leakage through the lock is obtained with the river gates closed. With the river gates closed the measured leakage passing through the lock chamber was approximately six times less than the measured leakage under the former gate configuration. The total leakage, including leakage through the sluice gates and harbor walls, was also measured. The measurements indicated that leakage through the river gates accounted for well over half the total leakage.

O'Brien Lock and Dam: Leakage passing through the lock chamber with both gates closed was very minimal. Velocities were near the minimal threshold velocities which the ADCP can accurately measure. The leakage measured through the lock chamber was reasonable for this structure. Attempts to measure total leakage which includes leakage through the sluice gates were unsuccessful due to extremely low velocities. This indicates that the leakage through the sluice gates is minimal.

Wilmette Controlling Works: Total leakage, leakage through the old abandoned lock and pump bays, was measured before and after MWRDGC sealed the old pump bays. The total measured leakage prior to the sealing of the pump bays was significantly greater than the reported leakage. After the pump bays were sealed, the total measured leakage dropped by a factor of approximately four.

The leakage and sluice gate measurements have accounted for a significant portion of the imbalance of flows determined in the canal system balance budget. Refer to appendix B for a detailed discussion of the canal system balance budget. Additional measurements and evaluations have been recommended by the third Three Member Technical Committee for the purpose of additional verification and to obtain measurements over a wide range of lake elevations and sluice gate settings. Further, since these measurements were made in WY 1993, they may not represent the level of leakage occurring during previous years. Without further review of how the lakefront structures have changed over time, it would be inappropriate to generalize these results.

THIRD TECHNICAL COMMITTEE

The third Three Member Technical Committee was convened during February 1993. The committee's mission is to review and assess the diversion accounting procedures and to assure that the "best current engineering practice and scientific knowledge" is applied by the Corps of Engineers in computing the diversion. Their work culminated in a report that constitutes appendix H of this WY 1993 Annual Report. The primary recommendations of the third Technical Committee are summarized below.

a. Release diversion accounting and annual reports in a more timely fashion.

b. Consider revising the WY81 through WY83 accounting to reflect AVM based flows through the use of regression equations.

c. A detailed manual of procedures for diversion accounting should be developed.

d. Update the diversion accounting and AVM quality assurance plans.

e. Improve the accuracy and reliability of measured flows at the Upper Des Plaines pumping station.

ACTIVITIES FOR FY 1994 AND 1995

ACCOUNTING REPORTS

The Accounting Report for WY 1990 was completed in FY 1994 and the Accounting Reports for WY 1991 and WY 1992 will be completed in FY 1995. The Accounting Reports for WY 1993 and WY 1994 will be completed in FY 1996. Thereafter, additional accounting reports are expected to be completed in the second fiscal year following the end of the water year for which the diversion is computed.

THIRD TECHNICAL COMMITTEE

The Third Technical Committee was under contract at the close of FY 1992. The Committee began its work in February 1993 which culminated in a report included as appendix H of this report.

FLOW MEASUREMENTS

Due to significant measurement problems that exist at the Upper Des Plaines pumping station, flow measurements will be conducted during WY 1995 to assess the accuracy of existing pump measurements. Based of the measurements, either the existing pumps will be recalibrated or additional measurement devices will be permanently installed to provide a consistently accurate and reliable means of measuring the flows. Accurate measurements are necessary so that the full advantage of this facility as a calibration point for the diversion models may be realized.

DIVERSION ACCOUNTING MANUAL

A manual will be written during FY 1995 to describe in detail the steps in the diversion accounting procedure. The manual will include any updates and modifications up to and including the WY 1992 Accounting Report.

SOFTWARE IMPROVEMENT

The conversion of the Hydrologic Simulation Program - FORTRAN (HSPF) from the TSS (Time Series Storage) database to a DSS (Data Storage System) database was completed in January 1994 by Aqua Terra Consultants, under contract to the Corps of Engineers through the USGS. HSPF uses hydrologic and meteorological data to calculate runoff. The result of this conversion will be an increase in computational efficiency which should reduce the time required to compute the diversion for future water years.

Beginning with the WY 1991 accounting, the DSS database will be used as the sole database for all modeling and computations. This will require numerous conversions of HSPF and SCALP model input decks as well as conversions of all mathematical computational input decks since all of the input decks, excluding TNET, used through the WY 1990 accounting read from and write to a TSS database.

SUMMARY AND CONCLUSIONS

SUMMARY

The Lake Michigan Diversion Accounting procedure continues to evolve and improve. Further improvements will occur during the WY 1991 and WY1992 diversion accounting. Among the changes is the inclusion of the more efficient DSS database as the sole database for all modeling and computations. A comprehensive manual will also be written to include all the improvements.

CONCLUSIONS

The Lake Michigan Diversion Accounting Report for WY 1990 has been completed as required by the Supreme Court Decree.

The State of Illinois diverted 3,531 cfs during WY 1990. This is 331 cfs greater than the 3,200 cfs limit specified in the decree. The running average of the diversion for WY 1981 through WY 1990 is 3,452 cfs, or 252 cfs over the annual allocation. The cumulative deviation is now -2,520 cfs-years. The negative sign indicates a cumulative flow deficit. The maximum allowable cumulative flow deficit specified in the decree is 2,000 cfs-years.

APPENDIX A

SIGNIFICANT EVENTS (NON HYDROLOGIC)

07 October 1992 Memorandum from CENCD-ED-WH to CENCC-ED-H.
Subject: Status of Reservoir Regulation Manuals.

15 October 1992 Memorandum from CENCD-ED-WH to CENCC-ED-H.
Subject: Update of he procedures to transfer real Time Hydrologic Data to CENCD.

26 October 1992 Letter to Tom Fogarty from USGS (Kevin Oberg).
Steve Melching and Kevin Oberg prepared brief Statement of Work and a cost estimate for the Romeoville AVM record analysis.

26 October 1992 Letter to Tom Fogarty from USGS (Kevin Oberg)
Completion of estimate of the cost to do work at Romeoville.

26 October 1992 Letter to Tom Fogarty from USGS (Kevin Oberg).
Analysis of Romeoville AVM Discharge Record for inconsistencies.

26 October 1992 Letter to Tom Fogarty from USGS (Kevin Oberg)
Preliminary Investigation of Romeoville Record consistency

26 October 1992 Letter to Tom Fogarty from USGS (Stephen Blanchard)
Funding needed for gaging station in Kishwaukee River near Perryville. USGS wanted to know if ACE was interested in funding the gage station..

28 October 1992 Memorandum from CENCC-ED-HW to CENCD-PE-ED-WH.
Subject: Comments on draft WCDS Field Guidance and ER1110-2-249.

29 October 1992 Memorandum from CENCD-PE-ED-WH to CENCC-ED-HW.
Subject: Lake Michigan diversion Accounting Concerns.

02 November 1992 Memorandum from CENCC-ED-HW to CENCC-ED.
Subject: Status of Reservoir Regulation Manual

03 November 1992 Memorandum from CENCD-PE-ED-WH to CENCC-ED-H.
Subject: Problems with Cooperative Stream Gaging Program, Gage, FY93.

03 November 1992 Memorandum from CENCD-PE-ED-WH to CENCC-ED-H.
Subject: Update on procedures to transfer Real Time Data to the NCD PC Server using TCP-IP protocol.

09 November 1992 Letter from David Kiel CENCC-ED-HW sent to: R.H. Gabey (AMG Resources Corp.), Russell Harrison (Cal Flexicore Corp.), Ron Santelik (CITGO Petroleum Co.), J.N. Orban (E.I. Dupont), Hon. Robert Pastrick (E. Chicago Mnicipal WWYP), Thomas Gordon (Gary WWTP), Nicholas Menninga (Hammond Municipal STP), S.L. Hocker (Indresco), Mark Knight (9nth Ave Dump Superfund Site), Landy Herndon Jr. (Roll Center Inc.), Pat Murphy (USX Corp): Subject: Request for locations and volumes of discharge to the river for the development of the flow model for Grand Cal Riv. including Indiana Harbor/Lake George Canals.

16 November 1992 Letter to Gerald Kauffman (Sec Donohue) from Dave Kiel . Forwarded Final Scope of Work dated 16 Nov 92 for work under contract # DACW23-91-D-0001. Work consist of Hydrologic Modeling of Grand Cal River Basin.

17 November 1992 Memorandum from CENCC-ED-HW to CENCC-ED.
Subject: Reservoir Regulation Manual for CUP O'Hare.

18 November 1992 Memorandum from CENCD-PE-ED-WH to CENCC-ED-HW.
Subject: Update on {procedures to transfer Real; Time Data to the NCD PC Server using TCP-IP Protocol.

20 November 1992 Memorandum for Record
Subject: Technical and Cost Price Analysis of SEC Donohue's proposal for Hydrologic Modeling of Grand Calumet River Basin - Contract DACW-23-91-D-0001.

24 November 1992 Letter to LTC David Reed from Illinois Department of Transportation (Donald Vonnahme). IDOT received written confirmation from NCD Commander that water diverted into canal system during freight tunnel flooding is not to be included as part of Illinois Diversion. Discusses WY87 Annual Report and has remarks.

01 December 1992 Contract Acknowledgment to Gerald Kauffman (SEC Donohue) from Dave Moughton. Letter of Notice to proceed with Contract DACW-23-91-D-0001, Work order #0003.

04 December 1992 Contract Acknowledgment to Gerald Kauffman (SEC Donohue) from Dave Moughton. Letter of Notice to proceed with contract DACW23-91-D-0001, Work order # 0004.

04 December 1992 Fax to Tom Fogarty from Lynette Moughton CENCD
Subject: Conference Call, Location of Water Control Function in Reorganization.

07 December 1992 Conference call with District on Water Control for Tom Fogarty and Dave Moughton.

08 December 1992 Memorandum from CENCC-ED-HW to CENCD.
Subject: Lake Michigan Diversion Accounting Concerns.

18 December 1992 Letter to Donald R. Vonnahme (IDOT) from CENCC-ED-HW.
NCC and NCD reviewing the issues raised in letter of 24 November, 1992.

23 December 1992 Memorandum from CENCC-ED-HW for CENCD-PE-ED-WH.
Subject: Annual Report of Water Control Management Activities.

28 December 1992 Fax to Tom Fogarty from CECW-EH.
Subject: Principles and procedures for carrying out water control management functions.

05 January 1993 Fax to Dan Injerd from J. Jacobazzi.
Subject: Water Control Management Meeting Agenda & Presentation Notes.

12 January 1993 Memorandum from CENCC-ED-H to CENCR-CT-C.
Subject: Price changes in Contract DACW23-93-M-0214.

19 January 1993 Memorandum from CENCC-ED-P to ED Branch.
Subject: FY 93 O & M Cost Codes

22 January 1993 Fax to Tom Fogarty from John Vento (NCD).
Subject: District Water Control Cell Operations Division.

03 February 1993 Fax to Tom Fogarty from Lynette Moughton.
Subject: Corps User Exchange (CUE) 1993 Annual Meeting.

05 February 1993 Memorandum from Don Leonard NCD to Tom Fogarty.
Subject: Forwarded the latest proposal for the water control organization. Comments Requested.

08 February 1993 Debriefing Package for BG Fuhrman on water management reorganization meeting in Cincinnati on 3-4 February 1993.

08 February 1993 Memorandum from CENCD-PE-ED-WH to CENCC-ED-HW.
Subject: Lake Michigan Diversion Accounting Concerns.

08 February 1993 Fax to Tom Fogarty from Lynette Moughton.
Subject: Rock Island Comments on Draft Procedures for Water Management and Control Centers.

10 February 1993 to Tom Fogarty from Lynette Moughton.
Subject: Request for comments on staffing and activities for cells.

17 February 1993 Memorandum from CENCC-ED-HW to CENCD-ED
Subject: Lake Michigan Diversion Accounting Draft Report for Water Years 1986 thru 1989.

25 February 1993 Letter to Stephen Blanchard (USGS) from CENCC-ED-HW.
Comments on the draft report titled "Discharge and Regression Analysis For Acoustical Velocity Meter Data for the Chicago Sanitary and Ship Canal at Romeoville, IL".

08 March 1993 Letter to CENCC-ED-HW from USGS (Stephen Blanchard).
Forwarded one signed copy of agreement # A5212B-93004-001. Agreement request the USGS to analyze and update flows measured by the Romeoville AVM for WY 84 - WY 92 for period of Oct 1, 92 - Jul 31, 92.

11 March 1993 Letter to Gerry Wolkowicz from CENCC-ED-HW.
Request for construction drawings for the Crawford Ave. Main Drain from 146th Ave. to Cal Sag Channel.

12 March 1993 Letter to Stephen Blanchard (USGS) from CENCC-ED-HW.
Response to request concerning report on discharge comparison and analysis for AVM at Romeoville, IL.

12 March 1993 Letter to Donald Vonnahme (IDOT) from CENCC-ED-HW.
Response to letter of 24 Nov 1992, explaining that a modification of the Supreme Court Decree would be necessary in order to subtract excessive leakage from LMD.

16 March 1993 Letter to David Moughton from James A. Stewart (USGS).
Forwarded primary information for USGS Stations. Grand Cal River at Hammond and Gary, IN, and Indiana Harbor Canal at E. Chicago, IN.

17 March 1993 Memorandum from CENCD-PE-ED-WH to CENCC-ED-HW.
Subject: NCD H & H Personnel Meeting.

19 March 1993 Letter to Tom Fogarty from Kevin Oberg (USGS).
The report "Comparison, analysis and estimation of discharge data from two acoustic velocity meters on the Chicago Sanitary and Ship Canal at Romeoville, IL" by C.S. Melching and Kevin Oberg approved for publication.

29 March 1993 Memorandum from CECW-EH-W to CENCC-ED-HW
Subject: Corps gaging cost for FY 93.

30 March 1993 Memorandum from CENCC-ED-H to CENCC-OC
Subject: Chicago Flood Records.

30 March 1993 Memorandum from CENCC-ED-HW to CENCD-PE-ED-WH.
Subject: Lake Michigan Diversion Accounting Annual Report for FY 90-92.

31 March 1993 Letter to Tom Fogarty from Kevin Oberg (USGS).
Questions relating to February 24, 1993 presentation to the LMDA Review Committee.
01 April 1993 Engineer Technical Letter from CECW-EP.
Engineering and design requirements and procedures for referencing coastal navigation projects to Mean Lower Low Water Datum.

01 April 1993 Memorandum from CENCC-ED-HW to CENCD.
Subject: Lake Michigan Diversion Accounting of Maintenance Diversions.

02 April 1993 Memorandum from CENCD-PE-ED-WH to CENCC-ED-HW.
Subject: Lake Michigan Diversion Accounting Annual Report for FY 90-92.

04 April 1993. Fax from USGS to Dave Moughton Provided rating tables for Grand Cal at Hohman Ave at Hammond, IN.

05 April 1993 Letter to J. Jacobazzi from W. Espey.
Third Tech Committee reviewed material from first Lake Michigan Diversion Accounting Meeting and found material to be reasonable and consistent.

08 April 1993 Memorandum from CENCC-ED-HW to CENCD-PE-ED-WH.
Subject: Lake Michigan Diversion Accounting Annual Report for FY 90-92.

08 April 1993 Letter to Brigadier general Russell Fuhrman to CENCC-ED-HW from Donald Vonnahme (Illinois Department of Transportation). Lakefront Leakage through Corps Installations.

08 April 1993 Letter to Honorable William C. Bryson from CENCC-ED-HW.
Transmitting the Joint Annual Report for WY 90-92.

09 April 1993 CC Mail message from DD to Dave Moughton.
Subject: Refrain from issuing the diversion report.

15 April 1993 Memorandum from CENCD-PE-ED-WH to CENCC-ED-H.
Subject: Collection of funds for operation of hydraulic programs.

18 April 1993 Memorandum from CENCD to CENCC-ED-HW.
Subject: Lake Michigan Diversion Accounting of Maintenance Diversions.

20 April 1993 Letter to James Flannery (LTV Steel) from CENCC-ED-HW.
Requesting data for the period of Oct 91 to Oct 92 for development of flow model for Grand Cal River/Indiana Harbor.

20 April 1993 Letter to Robert Johnston (Inland Steel) from CENCC-ED-HW.
Requesting data for the period Oct 91 to Oct 92 for development of flow model for Grand Cal River/Indiana Harbor.

05 May 1993 Memorandum from CENCD to CENCC-ED-H
Subject: Submittal of Annual Report for Water Control Data Systems.

12 May 1993 Memorandum from CENCD-PE-ED-WH to CENCC-ED-H.
Subject: Corps of Engineers NEXRAD Data Acquisition Plan Update.

12 May 1993 Letter to Thomas O'Connor from CENCC-ED-HW.
Asking for MWRDGC cooperation in taking measurements at Lockport Powerhouse Turbine Flow Meter with dataloggers and verify measurements at Upper Des Plaines pump station with 2 different options.

14 May 1993 Memorandum from CENCC-ED-HW to CENCD-PE-ED-WH.
Subject: NEXRAD data acquisition.

14 May 1993 Memorandum from CENCC-ED-HW to CENCD-PE-ED-WH.
Subject: Water control Data System Annual Report.

18 May 1993 Letter to Lynn Pruitt (Stanley Consultants) from CENCC-ED-HW.
Forwarding two copies of the Lake Michigan Diversion Report per request.

27 May 1993 Memorandum from CENCC-ED-HW to CENCD-RM-FA.
Subject: Collection of project funds for operation of hydrologic programs.

30 June 1993 Memorandum from CENCC-ED-H to CENCC-IM.
Subject: Ordering of PC parts for water control section.

13 July 1993 Letter to Thomas O'Connor (MWRDGC) from CENCC-ED-HW.
Announcing series of flow measurements at CRCW. Requesting that all sluice gates be closed during measurements which were scheduled for 13 to 16 July from 2AM to 6AM.

09 August 1993 Memorandum from CENCC-ED-HW to CENCC-CO.
Subject: Request by Metropolitan Water Reclamation District of Greater Chicago concerning Lockage Data.

20 August 1993 Memorandum from CENCC-ED-HW to CENCD-PE.
Subject: Updated flows for Lake Michigan Diversion Accounting.

27 August 1993 Memorandum from CENCCED-HW TO CENCD.
Subject: Cooperative stream gaging costs for FY 94.

30 August 1993 Letter to Tom Fogarty from RUST Environmental & Infrastructure.
HSPF Hydrologic Modeling Grand Cal Basin Contract # DACW23-91-D-0001.

31 August 1993 Letter to Tom O'Connor (MWRDGC) from CENCC-ED-HW.
Requesting measured data from the turbine AVM's for the period Aug 92 thru present.

31 August 1993 Letter to Robert Buchman from CENCC-ED-HW.
Response to letter of Aug 20 93 about Lake Michigan Water Levels.

22 September 1993 Letter to Melcy Curth Pons (RUST Environmental & Infrastructure Inc.) Response to request of Aug 30 93 letter for additional funding of HSPF Model of Grand Cal River.

APPENDIX B

LAKE MICHIGAN DIVERSION ACCOUNTING

WATER YEAR 1990 REPORT

TABLE OF CONTENTS

SECTION	PAGE
List of Tables	iv
List of Figures	v
Executive Summary	vi
Introduction	1
Authority for Report	1
History of the Diversion	1
Background of Lake Michigan Diversion Accounting	4
Diversion Accounting Procedures	6
Revisions to the Lake Michigan Diversion Accounting Procedures	12
Accounting Results	13
Discussion of Results	15
Columns	15
Column 1: Chicago Sanitary and Ship Canal (CSSC) at Romeoville, USGS AVM Gage Record	15
Column 2: Diversion from the CSSC Above the Gage	15
Column 3: Total Flow through the CSSC	15
Column 4: Groundwater Discharged to the CSSC and Adjoining Channels	16
Column 5: Water Supply Pumpage from Indiana Reaching the Chicago Sanitary and Ship Canal	17
Column 6: Runoff from the Des Plaines River Watershed Reaching the Chicago Sanitary and Ship Canal	18
Column 7: Lake Michigan Pumpage by Federal Facilities Which Discharge to the CSSC	18
Column 8: Total Deduction from the CSSC Romeoville Gage Record	18
Column 9: Lake Michigan Pumpage not Discharged to the Canal	19
Column 10: Total Diversion	19
Column 11 Through 13: Lake Michigan Diversion Components	20
Budgets	21
Budgets 1 and 2: Water Supply Pumpage	21
Budget 1: Diverted Lake Michigan Water Supply	23
Budget 2: Groundwater Diverted to the Chicago Sanitary Ship Canal	23
Budgets 3 Through 6: Stream Gaging Stations	24
Budgets 7 Through 13: MWRDGC Water Reclamation Facilities	24
Budget 7: Northside Water Reclamation Facility	25
Budget 8: Upper Des Plaines Pump Station	25
Budget 9: Mainstream TARP Pumping Station	29
Budget 10: Stickney Water Reclamation Facility	31
Budget 11: Calumet TARP Pumping Station	34
Budget 12: Calumet Water Reclamation Facility	36
Budget 13: Lemont Water Reclamation Facility	38
Budget 14: Chicago Canal System Balance	38
Areas for Improvement in the Diversion Accounting Procedures	42
Canal System Balance	42
Tunnel and Reservoir Plan Models	43

Table of Contents (cont.)

SECTION	PAGE
Ungaged Watershed Modeling	44
MWRDGC Upper Des Plaines Pump Station	45
O'Hare and Egan Basin Flow Transfer	46
Grand Calumet River	47
Summary	47
References	49
Appendix A - Summary of Daily Diversion Flows	A-1

LIST OF TABLES

TABLE NO.	TITLE	PAGE
1	Description of Diversion Accounting Columns	8
2	Description of the Diversion Accounting Computational Budgets	9
3	Status of the State of Illinois' Diversion from Lake Michigan under the 1980 Modified U.S. Supreme Court Decree	13
4	Lake Michigan Diversion Accounting - WY 1990 Summary of Diversion Flows (cfs)	14
5	Breakdown of the Diversion by the State of Illinois Based on Columns 11 Through 13	21
6	Stream Gage Flow Separation	24
7	WY 1990 Summary of Simulation Statistics	26
8	Summary of Flow Components for Canal System Balance - WY 1990	41

LIST OF FIGURES

FIGURE NO.	TITLE	PAGE
1	Development of the Chicago Canal System	3
2	Component Breakdown of Illinois' Diversion	22
3	Budget 7 - Simulation of the MWRDGC Northside Water Reclamation Facility	27
4	Budget 8 - Simulation of the MWRDGC Upper Des Plaines Pump Station	28
5	Map of Mainstream and Calumet TARP	30
6	Budget 9 - Simulation of the MWRDGC Mainstream TARP Pumping Station	32
7	Budget 10 - Simulation of the MWRDGC Stickney Water Reclamation Facility	33
8	Budget 11 - Simulation of the MWRDGC Calumet TARP Pumping Station	35
9	Budget 12 - Simulation of the MWRDGC Calumet Water Reclamation Facility	37
10	Budget 13 - Simulation of the MWRDGC Lemont Water Reclamation Facility	39
11	Budget 14 - Canal System Balance	40

EXECUTIVE SUMMARY

In compliance with the modified 1980 U.S. Supreme Court decree (hereinafter the Decree), the WY90 diversion was computed using the best engineering technology available to date.

Given the complexity of the hydrologic cycle in the heavily urbanized Chicago metropolitan area, and given the number of human and other factors that cannot be adequately represented in numerical modeling procedures, the results of the simulations which compute diversion flows worked exceptionally well.

The WY90 diversion accountable to the State of Illinois is 3,531.2 cfs. This is 331.2 cfs greater than the 3,200 cfs average specified by the Decree. The 40 year running average, rounded to the nearest cfs, beginning with WY81 is 3,452 cfs and the cumulative deviation from the 3,200 cfs average is -2,520 cfs-years. The negative cumulative deviation indicates a water allocation deficit and the maximum allowable debt is 2,000 cfs-years.

INTRODUCTION

The diversion of water from the Lake Michigan watershed is of major importance to the Great Lake states and to the Canadian province of Ontario. The states and province that border the Great Lakes have concerns with both diversions during periods of low lake levels as well as the long term effects of diversion. To insure that the concerns of these interested parties are considered, the U. S. Army Corps of Engineers has been given the responsibility for the accounting of flow that is diverted from the Lake Michigan watershed.

The Corps of Engineers, Chicago District, is responsible for monitoring the measurements and the computation of the diversion of Lake Michigan water by the State of Illinois. The computations for Water Year 1983 (WY83), WY84 and WY85 (1 October 1984 through 30 September 1985) were completed by the Northeastern Illinois Planning Commission (NIPC) for the Illinois Department of Transportation (IDOT). Prior to the WY83 report, the calculations were made by the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) for IDOT. The Corps reviewed, modified, and updated the WY84 and WY85 diversion accounting completed by NIPC. The computations for WY86 were performed jointly by NIPC (under contract to the Corps of Engineers) and the Corps of Engineers. Beginning in WY87 the computations were performed solely by the Corps of Engineers. This report represents the final Lake Michigan diversion accounting for WY90.

AUTHORITY FOR REPORT

Under the provisions of the U.S. Supreme Court Decree in the Wisconsin, et al v. Illinois et al, 388 U.S. 426,87 S.Ct. 1774 (1967) as modified 449 U.S. 48, 101 S.Ct. 557 (1980), the Corps of Engineers is responsible for monitoring the measurement and computation of diversion of Lake Michigan water by the State of Illinois. The Water Resources Development Act of 1986 (Section 1142 of PL 99-662) gave the Corps total responsibility for the computation of diversion flows as formerly done by the State of Illinois. The Corps' new mission became effective 1 October 1987.

HISTORY OF THE DIVERSION

Water has been diverted from Lake Michigan at Chicago into the Mississippi River Basin since the completion of the Illinois and Michigan Canal in 1848. At that time, diversion averaged about 500 cubic feet per second (cfs). The Illinois and Michigan Canal was built primarily to serve transportation needs. The canal provided a connecting watercourse between the Great Lakes and the Mississippi River system.

With the development of the Chicago metropolitan area, sewer and drainage improvements led to severe sanitation problems in the mid to late 1800's. The newly constructed sewers moved water and wastes into the Chicago River, which until 1900 drained to Lake Michigan. The water quality of Lake Michigan deteriorated and contaminated the city's primary water supply.

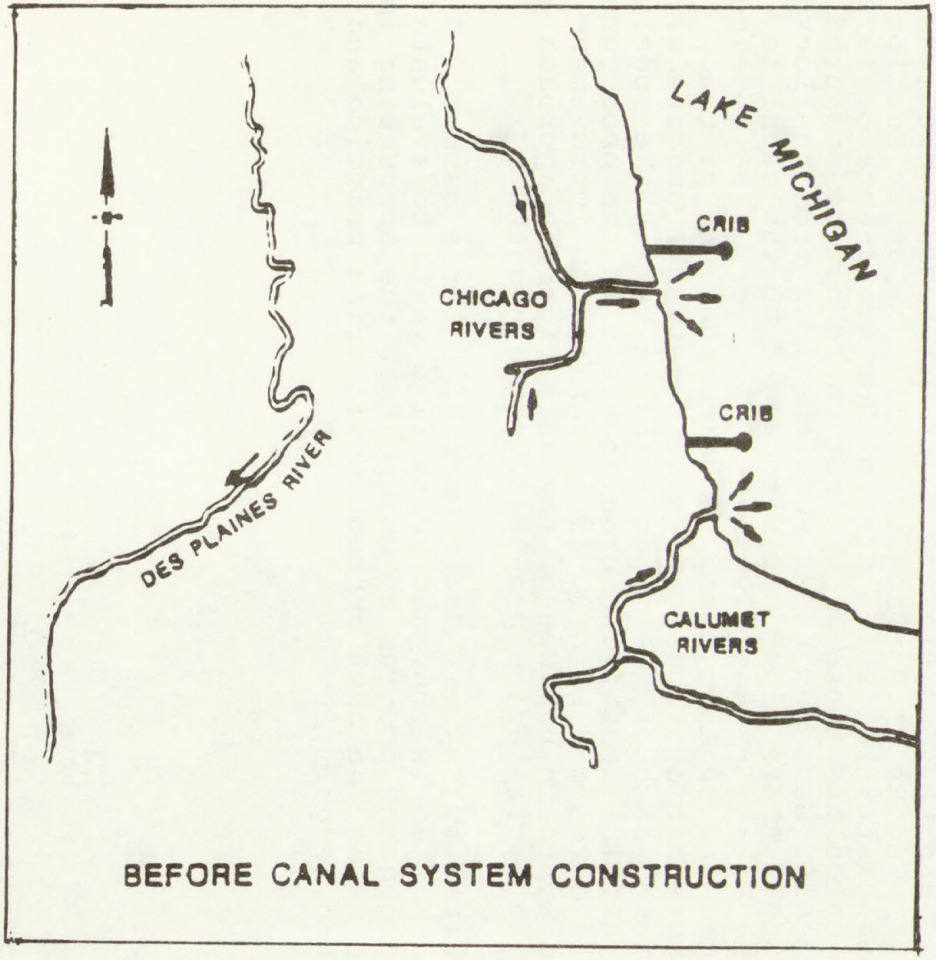
A second problem that occurred during this time period was an increase in the overbank flooding within the city. As more roads were built and buildings constructed the sewer system was correspondingly expanded. This increased the rate and volume of runoff and resulted in increased flooding.

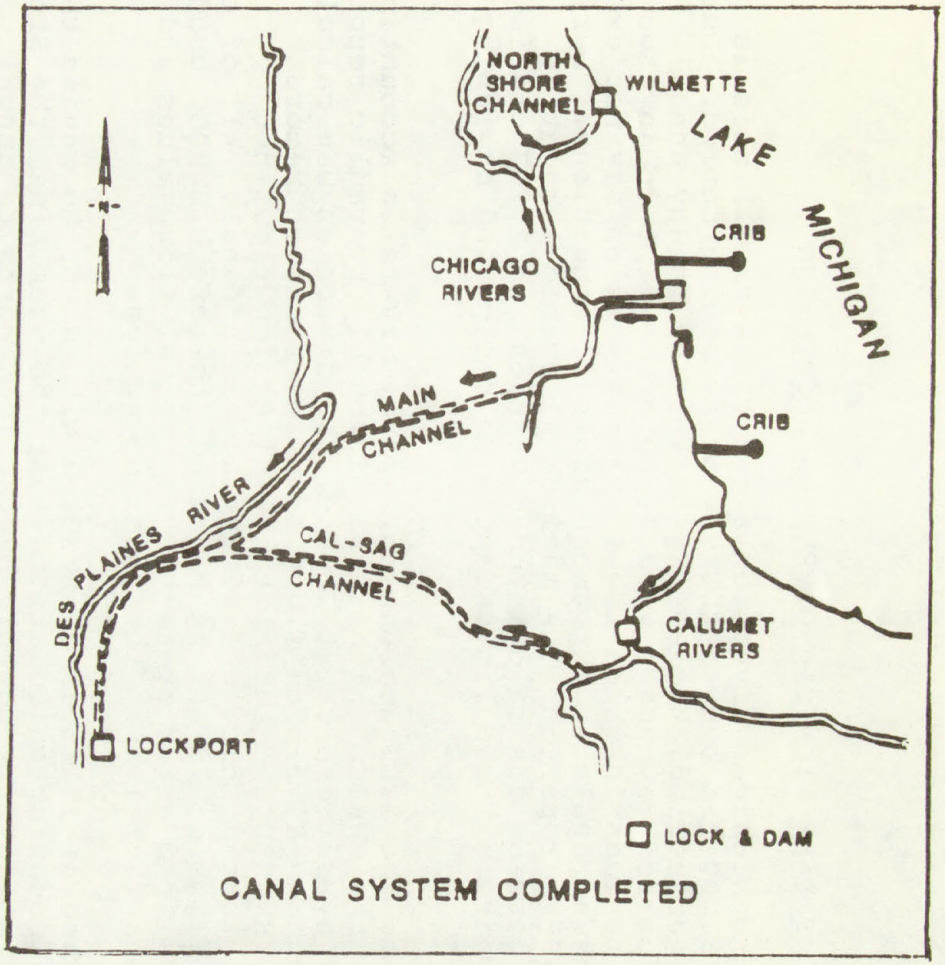
As a solution to the sanitation and flooding problems construction of the Chicago Sanitary and Ship Canal (CSSC) was undertaken. This construction allowed the flow direction of the Chicago River to be reversed (Figure 1). Construction of the Chicago Sanitary and Ship Canal was completed in 1900 by the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) (formerly Metropolitan Sanitary District of Greater Chicago, MSDGC). The Sanitary and Ship Canal followed the course of the older I and M Canal. This canal is much larger than the I and M canal and can handle the Chicago River flow as well as increased shipping. The Chicago River Controlling Works was constructed at the mouth of the Chicago River. The lock regulates the amount of Lake Michigan water allowed to pass into the river and restricts river flooding from entering Lake Michigan.

Between 1907 and 1910 the MWRDGC constructed a second sanitary canal called the North Shore Canal. It extended from Lake Michigan at Wilmette in a southerly direction 6.14 miles to the north branch of the Chicago River. The Wilmette Controlling Works regulates the amount of Lake Michigan flow allowed down the channel.

Construction of a third canal, the Calumet Sag Canal, was completed in 1922. The canal connects Lake Michigan through the Grand Calumet River, to the Sanitary and Ship Canal. This canal was constructed to carry sewage from South Chicago, Illinois and East Chicago, Indiana. The O'Brien Lock and Dam located on the Calumet River, regulates the flow of Lake Michigan waters down the canal.

Development of the Chicago Canal System





BACKGROUND OF LAKE MICHIGAN DIVERSION ACCOUNTING

The Lake Michigan diversion accountable to Illinois is limited to 3,200 cfs over a forty year averaging period. During the forty year period, the average diversion in any annual accounting period may not exceed 3,680 cfs except in any two accounting periods in which the average diversion may not exceed 3,840 cfs as a result of extreme hydrologic conditions. During the first 39 year period, the maximum allowable cumulative difference between the calculated diversion and 3,200 cfs is 2,000 cfs-years. These limits apply to the period beginning with WY81.

Prior to the 1983 accounting report, diversion accounting was done by the MWRDGC in the form of monthly hydraulic reports. As required by Supreme Court Decree, the diversion was calculated by deducting non-diversion flows from the Lockport record measured by MWRDGC and adding those diversion flows not discharging to the Chicago Sanitary and Ship canal. All of the deductible flows could not be measured, therefore MWRDGC used flow records from gaged areas to get typical flow values and then extrapolated to arrive at the total deduction.

The State of Illinois contracted with NIPC, to revise the diversion accounting calculations. At the same time, the State of Illinois moved from monthly hydraulic reports to annual accounting reports. NIPC adapted computer models of the diverted Lake Michigan and the Des Plaines River watersheds, previously developed for studies in Northeastern Illinois under Section 208 of the Federal Water Pollution Control Act Amendments of 1972 (PL 92-500), to calculate those flows that could not be measured. Like MWRDGC, NIPC deducted non-diversion flows from the Lockport record and added those flows not discharged to the canal to calculate the Lake Michigan diversion. However, NIPC modeled both the gaged and ungaged areas to calculate much of the deduction and addition flows. Then computational budgets were developed around each of the gaged areas to verify the models. The budgets aid in identifying problem areas in the procedure. The procedure developed by NIPC is a significant improvement over the previous approach because of the more rigorous approach and because of the verification provided by the budgets.

As required by Supreme Court Decree, a three member technical committee is convened every five years to evaluate the diversion accounting program to ensure that the accounting is accomplished using the best current engineering practice and scientific knowledge.

The first technical committee was convened during the period when diversion accounting was done by MWRDGC. The committee was primarily concerned with the rating of the various components at the Lockport facility, the primary diversion measurement location (Espey et al, 1981). In response to the Committee's concerns, the Corps' Waterways Experiment Station (WES) revised the ratings of the two sets of Lockport sluice gates (Hart and McGee, 1985).

In response to the Committee's concerns, the State of Illinois installed an acoustic velocity meter (AVM) at Romeoville five miles upstream of Lockport. The AVM is a highly accurate flow meter that proved to provide better flow measurements than the MWRDGC reported Lockport flows and the new Corps rating curves. The AVM became operational 12 June 1984. However, USGS did not publish the AVM flows until 1 October 1985. Because of significant equipment problems with the AVM, a replacement AVM was installed in November 1988.

To provide flows during periods of malfunction, various regression analyses were done to relate the MWRDGC reported Lockport flows to the AVM flows. Several sets of equations were proposed by the Corps of Engineers, the USGS, Harza Engineering Co., and the Second Technical Committee. The report, Chicago Sanitary and Ship Canal at Romeoville Acoustical Velocity Meter Backup System, was completed September 1989 (USACE, 1989). That report documents the many efforts taken by various parties to develop useful regression equations. The regression equations that were ultimately used to estimate missing AVM flows from WY86 through WY91 were developed by the USGS in a report titled "Comparison, Analysis, and Estimation of Discharge Data from the Two Acoustic Velocity Meters on the Chicago Sanitary and Ship Canal at Romeoville, Illinois." This report is contained in the Lake Michigan Diversion Accounting WY93 Annual Report.

The second technical committee reviewed the NIPC hydrologic and hydraulic computer models and agreed that the approach was consistent with what was required by the decree (Espey et al, 1987). However, the committee felt that some of the parameters used in the models were out of date and in need of revision. To address the committee's concerns, the Corps hired a consultant (C. B. Burke Engineering, Ltd.) in September of 1988 to review and update the modeling parameters. The final report (Burke, 1990) concerning the updating of modeling parameters was submitted to the Corps in October 1990.

The Water Resources Development Act of 1986 gave the Corps of Engineers the full responsibility for computation of the Illinois Lake Michigan diversion as of 1 October 1987. When the Corps' new responsibility became effective, the WY84 diversion accounting report, developed by NIPC, had not been certified. As a result, the Corps was responsible for the WY84 and all subsequent reports.

NIPC completed the WY84 diversion accounting report in April of 1987. It was subsequently reviewed by the Corps. The Corps found the report to be adequate with two exceptions. First, the 1984 accounting was completed with the modeling parameters questioned by the second technical committee. Second, MWRDGC reported Lockport flows, adjusted using the WES rating curves, were used rather than AVM flows. The Corps, knowing that the modeling parameters required updating and that AVM flows for the period prior to installation could be calculated accurately using regression equations, refrained from certifying the WY84 report until these issues were resolved.

NIPC completed the WY85 diversion accounting report in December of 1988 and the report was reviewed by the Corps. Like the WY84 report, the WY85 accounting was done with the modeling parameters questioned by the second technical committee. Additionally, NIPC used the AVM flows published by the U.S. Geological Survey (USGS) in their WY85 Water Resources Data for Illinois report. Since the publication of the WY85 USGS report, more reliable equations have been developed for calculating flows when the AVM was malfunctioning. These equations are periodically reviewed and updated as necessary.

Upon completion of the analysis of the modeling parameters by Christopher B. Burke Engineering, LTD, the WY84 and WY85 diversion flows were recalculated using the revised modeling parameters and the Romeoville AVM flows. The diversion flows were certified by the Corps of Engineers and transmitted to all interested parties in the Lake Michigan Diversion Accounting 1989 Annual Report (USACE, 1990).

The computation of Illinois' diversion from Lake Michigan for WY86 was undertaken as a joint effort between NIPC (under contract to the Corps of Engineers) and the Corps of Engineers. The computation of Illinois' diversion from Lake Michigan for WY87 through WY90 was performed solely by the Corps of Engineers. The WY86 through WY89 Diversion Accounting Reports are contained in the Lake Michigan Diversion Accounting Annual Report covering WY90 through WY92 (USACE, 1994).

DIVERSION ACCOUNTING PROCEDURES

The Lake Michigan diversion accountable to the State of Illinois is calculated by measuring the flow in the Chicago Sanitary and Ship Canal at Romeoville and deducting flows that do not constitute Lake Michigan diversion and are not accountable to the State of Illinois. Finally, additions are made to the Romeoville record for diversions that are not discharged to the canal. The deductions include groundwater water supply pumpage whose effluent is discharged to the canal, runoff from the Des

Plaines River watershed that is discharged to the canal, Lake Michigan water supply pumpage from Indiana that is discharged to the canal, and water supply pumpage from Lake Michigan used for Federal facilities that is discharged to the canal. The additions to the Romeoville record include flows diverted from the canal upstream of Romeoville, and Lake Michigan water supply whose effluent is not discharged to the canal. This procedure represents the accounting method required by the Supreme Court Decree.

The diversion accounting results are presented as a series of columns that are listed in Table 1. Column 1 through Column 3 compute the total flow in the Sanitary and Ship Canal. Column 4 through Column 7 presents the deductions from the canal system flows with the total deduction being presented in Column 8. Column 9 presents the additions to the canal system record. Column 10 is the computed Lake Michigan diversion accountable to Illinois and is equal to the canal system flow minus the deductions plus the additions. Columns 11 through 13 are independent flow estimates for the three sources of diversion: water supply pumpage from Lake Michigan, runoff from the diverted Lake Michigan Watershed, and direct diversion through the lakefront structures. Column 11 through Column 13 are not used in the diversion calculation but are included as another estimate of the diversion for verification of the accounting flows in Column 10. The sum of Column 11 through Column 13 should theoretically equal the flow in Column 10.

In addition to the diversion calculations presented in the 13 columns, 14 computational budgets are prepared as input to the diversion calculation and to verify the estimated flows that cannot be measured. A summary of these budgets is presented in Table 2. Budgets 1 and 2 do not compare simulated to measured flows but are summations of critical water supply pumpage data. Budget 3 through Budget 6 partition stream gage records into runoff and sanitary/industrial discharge components to estimate a portion of the runoff from the diverted watershed that is used as input to Column 12, "Runoff from the Diverted Lake Michigan Watershed." Budget 7 through Budget 13 compare simulated to measured flows at MWRDGC facilities. These budgets are for verification of the diversion accounting procedures and give an indication of the accuracy of the diversion accounting models. Budget 14 compares canal system inflows and outflows. It is used primarily as a verification of modeling results as well as an indicator of the accuracy and completeness of measured/reported flows.

Table 1

Description of the Diversion Accounting Columns

Column No.	Description
1	Chicago Sanitary and Ship Canal (CSSC) at Romeoville AVM Gage Record
2	Diversion from the CSSC above the Romeoville AVM Gage
3	Total Flow Through the CSSC
4	Groundwater Pumpage Discharged into the CSSC and Adjoining Channels
5	Water Supply Pumpage from Indiana Reaching the CSSC
6	Runoff from the Des Plaines River Watershed which Reaches the CSSC
7	Lake Michigan Pumpage by Federal Facilities which Discharge to the CSSC and Adjoining Channels
8	Total Deduction from the CSSC Romeoville AVM Gage Record
9	Lake Michigan Pumpage Which is not Discharged into the CSSC
10	Total Diversion Accountable to the State of Illinois
11	Pumpage from Lake Michigan Which is Accountable to State of Illinois
12	Runoff from the Diverted Lake Michigan Watershed
13	Direct Diversions Through Lake Front Control Structures Which is Accountable to the State of Illinois

Table 2

Description of the Diversion Accounting Computational Budgets

Budget No.	Title	Description
1	Diverted Lake Michigan Pumpage	This budget sums Lake Michigan water diverted by the State of Illinois in the form of municipal and industrial water supply. The results of this budget are used in Column 11.
2	Groundwater Discharged to the CSSC	This budget sums groundwater pumpages that are discharged to the CSSC. The results of this budget are used in Column 4.
3	North Branch Chicago River at Niles, IL	This budget performs a simple separation of stream flow into sanitary and runoff portions. The results this budget are used in Budget 14 and Column 12.
4	Little Calumet River at the IL-IN State Line	This budget performs a simple separation of stream flow into sanitary and runoff portions. The results this budget are used in Budget 14 and Column 12.
5	Thorn Creek at Thornton, IL	This budget performs a simple separation of stream flow into sanitary and runoff portions. The results this budget are used in Budget 14 and Column 12.
6	Little Calumet River at South Holland, IL	This budget performs a simple separation of stream flow into sanitary and runoff portions. The results this budget are used in Budget 14 and Column 12.
7	MWRDGC Northside Water Reclamation	This budget performs hydrologic and hydraulic simulation of the service basin tributary to the MWRDGC Northside Water Reclamation Facility. The simulations estimates the runoff from portions of the Lake Michigan and Des Plaines River watersheds within the Northside service basin that is diverted to the CSSC in the form of inflow-infiltration. The budget provides an internal verification of the accounting procedures. The results of this budget are used on Budget 14 and Columns 6 and 12.

Table 2 (cont)

Description of the Diversion Accounting Computational Budgets

Budget No.	Title	Description
8	MWRDGC Upper Des Plaines Pumping Station	This budget performs hydrologic and hydraulic simulation of the MWRDGC Upper Des Plaines Pumping Station. This budget provides a calibration point to verify models of the Des Plaines River watershed
9	MWRDGC Mainstream TARP Pumping Station	This budget performs hydrologic and hydraulic simulation of the MWRDGC Mainstream TARP Pumping Station. The results of this simulation are used in Budgets 10 and 14 and Columns 6 and 12. The budget also provides internal verification of the accounting procedures.
10	MWRDGC Stickney Water Reclamation Facility	This budget performs hydrologic and hydraulic simulation of the service basin tributary to the MWRDGC Stickney Water Reclamation Facility. The simulations estimates the runoff from portions of the Lake Michigan and Des Plaines River watersheds within the Stickney service basin that is diverted to the CSSC in the form of inflow-infiltration. The budget provides an internal verification of the accounting procedures. The results of this budget are used in Budget 14 and Columns 6 and 12.
11	MWRDGC Calumet TARP Pumping Station	This budget performs hydrologic and hydraulic simulation of the MWRDGC Calumet TARP Pumping Station. The results of this simulation are used in Budgets 12 and 14 and Columns 6 and 12. The budget also provides internal verification of the accounting procedures.

Table 2 (cont)

Description of the Diversion Accounting Computational Budgets

Budget No.	Title	Description
12	MWRDGC Calumet Water Reclamation Facility	This budget performs hydrologic and hydraulic simulation of the service basin tributary to the MWRDGC Calumet Water Reclamation Facility. The simulations estimates the runoff from portions of the Lake Michigan and Des Plaines River watersheds within the Calumet service basin that is diverted to the CSSC in the form of inflow-infiltration. The budget provides an internal verification of the accounting procedures. The results of this budget are used in Budget 14 and Columns 6 and 12.
13	MWRDGC Lemont Water Reclamation Facility	This budget performs hydrologic and hydraulic simulation of the service basin tributary to the MWRDGC Lemont Water Reclamation Facility. The simulations estimates the runoff from portions of the Des Plaines River watershed within the Lemont service basin that is diverted to the CSSC in the form of inflow-infiltration. The budget provides an internal verification of the accounting procedures. The results of this budget are used in Budget 14 and Column 6.
14	Chicago Canal System	This budget performs a water balance of the Chicago Canal System which includes the CSSC and adjoining channels. This budget provides a verification point for the accounting procedures.

REVISIONS TO THE LAKE MICHIGAN DIVERSION ACCOUNTING PROCEDURES

The primary revision implemented for the WY90 diversion accounting was the incorporation of the new 25-gage precipitation network into the runoff simulation models. The 25-gage precipitation network replaces the previous 13-gage network. The new precipitation network has solved many of the problems associated with the old network, such as poor exposure and distribution patterns. The Illinois State Water Survey, ISWS, installed and maintains the precipitation network for the Corps of Engineers. They also collect the data and adjust it if necessary. A description of the new 25-gage precipitation network can be found in the ISWS report titled "Installation and Operation of a Dense Raingage Network to Improve Precipitation Measurements for Lake Michigan Diversion Accounting: Water Year 1990" (ISWS, 1991). That report is contained in the Lake Michigan Diversion Accounting WY93 Annual Report.

In addition to the installation and use of the new 25-gage precipitation network was the subsequent modifications to the hydrologic runoff models and hydraulic sewer routing models. These models were revised in order to reflect the changes in the precipitation network. Many of the model changes were accomplished by Rust Environment and Infrastructure under contact with the Corps of Engineers. Their work culminated in a report titled "Diversion Accounting Update for the New 25-Gage Precipitation Network" (Rust,1993). That report is also contained in the Lake Michigan Diversion Accounting WY93 Annual Report.

Rust's work involved review and correction of map delineations of combined sewer special contributing areas, delineation of precipitation gage assigned areas for the 25-gage network, land-use/land-cover delineations, modifications to the hydraulic sewer routing model to reflect the revised precipitation network and land cover assignments, and an assessment of the model parameters used in the hydrologic runoff model, Hydrologic Simulation Program - Fortran (HSPF).

The Corps of Engineers modified the hydraulic sewer model, Special Contributing Area Loading Program (SCALP), in separate sewer areas in order to incorporate changes in the precipitation network. Since actual boundaries have not been mapped for those areas some assumptions as to the location of the separate sewer areas were made. This was necessary since effective areas have been applied for the separate sewer areas in the SCALP model. These assumptions will continue until a further study can be accomplished that will reflect actual boundaries for these separately sewered areas.

A study was also done by the Corps to improve the response of the HSPF hydrologic runoff models. Input on parameter improvements were received from NIPC and Rust. The study resulted in some minor parameter modifications to the HSPF runoff model to correct for past inconsistencies and improve parameter accuracy.

It will be shown later in the report that the effect of the new 25-gage precipitation network, improvements and updates in the land cover delineations, and modifications to the hydrologic and hydraulic models has resulted in improved water balances at the primary calibration points, the four MWRDGC water reclamation plants as well as the Upper Des Plaines pumping station.

ACCOUNTING RESULTS

The WY90 diversion accounting monthly summary is presented in Table 4. Table 4 shows the total WY90 Lake Michigan diversion accountable to the State of Illinois is 3,531.2 cfs (Column 10). This is 331.2 cfs greater than the 3,200 cfs average specified by the Decree. The 40 year running average (Table 3), rounded to the nearest cfs, beginning with WY81 is 3,452 cfs and the cumulative deviation from the 3,200 cfs average is -2,520 cfs-years. The negative cumulative deviation indicates a water allocation deficit. The maximum allowable deficit is 2,000 cfs-years. Tabular data on daily diversion flows is presented in Appendix A.

Table 3

Status of the State of Illinois' Diversion from Lake Michigan Under the 1980 Modified U.S. Supreme Court Decree

Accounting Year	Certified Flow, cfs	Running Average, cfs	Cumulative Deviation, cfs
1981	3,106	3,106	+ 94
1982	3,087	3,097	+ 207
1983	3,613	3,269	- 206
1984	3,432	3,309	- 438
1985	3,472	3,342	- 710
1986	3,751	3,410	- 1,261
1987	3,774	3,462	- 1,835
1988	3,376	3,451	- 2,011
1989	3,378	3,443	- 2,189
1990	3,531	3,452	- 2,520

Table 4

Lake Michigan Diversion Accounting – WY 1990

Summary of Diversion Flows (cfs)

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
MONTH	1	2	3	4	5	6	7	8	9	10	11	12	13
OCT 89	2856.5	0.5	2857.0	85.4	27.9	72.0	1.9	187.2	103.4	2773.2	1742.2	305.7	462.0
NOV 89	2654.6	0.6	2655.2	90.3	27.4	98.3	2.0	218.0	97.0	2534.3	1666.2	481.4	65.5
DEC 89	2231.2	0.6	2231.8	76.0	27.6	28.7	1.8	134.1	99.9	2197.6	1687.9	135.5	48.8
JAN 90	2883.4	0.7	2884.0	111.4	27.3	183.0	2.1	323.8	95.9	2656.2	1689.2	829.2	51.8
FEB 90	3254.2	0.7	3254.8	121.8	27.7	284.2	1.7	435.4	97.5	2916.9	1644.7	1241.8	44.5
MAR 90	3966.8	0.7	3967.5	112.4	27.5	302.9	1.8	444.5	97.9	3620.9	1636.3	1404.8	57.7
APR 90	2933.2	0.6	2933.9	88.7	27.6	141.3	1.7	259.3	97.3	2771.9	1661.6	626.9	87.7
MAY 90	5491.1	0.7	5491.7	116.8	28.2	371.4	2.0	518.3	106.0	5079.4	1686.6	2013.5	252.9
JUN 90	3882.3	1.1	3883.4	107.6	28.6	191.5	2.2	329.9	114.6	3668.1	1867.8	676.1	650.5
JUL 90	4657.7	1.2	4658.9	99.5	28.7	301.5	1.9	431.6	121.6	4348.9	1964.7	818.7	894.6
AUG 90	5224.7	1.0	5225.7	129.0	28.7	223.2	2.5	383.4	126.4	4968.7	1901.7	1526.9	943.7
SEP 90	4736.8	0.9	4737.7	86.5	28.2	75.6	2.1	192.4	110.9	4656.2	1906.6	300.0	1790.2
AVERAGES	3748.5	0.8	3749.3	102.3	27.9	191.6	2.0	323.8	105.7	3531.2	1754.7	872.9	449.6

COMPUTATIONS:

1. COLUMN 3 EQUALS THE SUM OF COLUMN 1 AND COLUMN 2.
2. COLUMN 8 EQUALS THE SUM OF COLUMN 4 THROUGH COLUMN 7.
3. COLUMN 10 EQUALS COLUMN 3 MINUS COLUMN 8 PLUS COLUMN 9.

DEDUCTIONS FROM THE ROMEOVILLE GAGE RECORD

ADDITIONS TO THE ROMEOVILLE GAGE RECORD

NOTES:

1. ALL VALUES ARE ROUNDED TO THE NEAREST TENTH.
2. MATHEMATICAL COMPUTATIONS BETWEEN COLUMNS UTILIZE UNROUNDED VALUES.
3. AVERAGE VALUES FOR WY90 WERE COMPUTED USING DAILY VALUES.

DISCUSSIONS OF RESULTS

The following is a discussion of the column functions and computational budgets. The discussion of the column functions describes the purpose of each column as well as some observations on the WY90 values in the columns. The discussion of the computational budgets presents the purpose of each budget and the results of the budget flow balances. The results of the computational budgets are used in the diversion calculations where seven budgets are used to verify the diversion simulation models. The columns are discussed first followed by the discussion of the budgets.

COLUMNS

The first ten columns display the components of the diversion calculation and include the Romeoville flow as well as the various deductions and additions to the Romeoville record. The final three columns display the three diversion components (Lake Michigan pumpage accountable to Illinois, runoff from the diverted watershed, and direct diversion through the lakefront control structures) and the sum of the three columns should theoretically equal the Romeoville based diversion calculation. A comparison of the sum of these three columns to the calculated diversion (Column 10) is presented in the discussion of Column 11 through Column 13.

COLUMN 1: CHICAGO SANITARY AND SHIP CANAL (CSSC) AT ROMEOVILLE, USGS AVM GAGE RECORD

The discharge at Romeoville for WY90 is 3,748.5 cfs. For days when the AVM was inoperable, the flow at the Romeoville site was calculated from regression equations.

COLUMN 2: DIVERSIONS FROM THE CSSC ABOVE THE GAGE

Argonne Laboratories and Uno-ven Corporation were the only diversions from the Chicago Sanitary and Ship Canal upstream of the Romeoville gage in WY90. The average withdrawal upstream of the AVM for WY90 is 0.8 cfs.

COLUMN 3: TOTAL FLOW THROUGH THE CSSC

Column 3 is the sum of Column 1 and Column 2 and represents the total flow entering the canal system. The average canal flow is 3,749.3 cfs for WY90.

COLUMN 4: GROUNDWATER DISCHARGED TO THE CSSC AND ADJOINING CHANNELS

Column 4 is groundwater water supply pumpage by communities, industrial users, and other private users as reported by the Illinois State Water Survey (ISWS) whose effluent is discharged to the CSSC. It also includes the groundwater seepage into the TARP system that is discharged to the canal. This quantity is determined by summing all reported groundwater pumpages tributary to the canal along with the estimated groundwater seepage into the Mainstream TARP (Budget 9) and Calumet TARP (Budget 11) systems. This total is then adjusted by subtracting the groundwater normally tributary to the canal that is contained in the combined sewer overflows that discharge to the Des Plaines River and other watercourses not tributary to the CSSC. This method prevents double accounting of the combined sewer overflow portion of the groundwater supply pumpage.

Using ISWS groundwater records, groundwater pumpages were assumed to reach the CSSC and adjoining channels if they were located in the diverted Lake Michigan watershed in Illinois or if they were located within MWRDGC Water Reclamation Plant (WRP) service boundaries in which their effluent was discharged into the CSSC and adjoining channels. Groundwater seepage into the Mainstream TARP and Calumet TARP systems was determined through simulation and is discussed in Budgets 9 and 11. The groundwater constituent of combined sewer overflows is determined entirely thorough simulation.

Groundwater pumpage from the Lake Michigan watershed whose effluent is discharged to the canal is a deduction except to the extent that the groundwater sources are recharged by Lake Michigan. Current piezometric levels indicate that groundwater is discharging to the lake. Therefore, groundwater pumpage from within the Lake Michigan Watershed that reaches the canal continues to be a deduction. Research literature will be reviewed periodically to verify this assumption.

Column 4 represents a deduction from the Romeoville record and averaged 102.3 cfs for WY90. This is an increase of 20.3 cfs from WY89. Groundwater pumpage tributary to the canal is composed of 18.9 cfs of groundwater pumpage from the Lake Michigan watershed, 25.0 cfs of groundwater pumpage from outside of the Lake Michigan watershed, 52.0 cfs of groundwater seepage into the Mainstream TARP system, and 6.6 cfs of groundwater seepage into the Calumet TARP system. The total of these components is 102.5 cfs. However, the deduction from the Romeoville gage record is 102.3 cfs since 0.2 cfs of this groundwater supply pumpage was determined, through simulation, to be discharged to the Des Plaines River and other watercourses not tributary to the CSSC in the form of combined sewer overflows.

COLUMN 5: WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CHICAGO SANITARY AND SHIP CANAL

Column 5 represents the computation of Indiana water supply reaching the canal through the Grand Calumet and the Little Calumet Rivers. In the case of the Little Calumet River, a drainage divide exists east of the confluence with Hart Ditch. Therefore, flows from Hart Ditch, including virtually all dry weather flows, normally flow westward into Illinois. Under high flow conditions, the drainage divide may shift westward and a portion of the Hart Ditch flows may be diverted eastward to Burns Ditch and ultimately to Lake Michigan. However, it is believed that the occurrence in the shift in the drainage divide is infrequent and the flow that is diverted eastward is insignificant. Therefore, it is assumed that all effluent discharged into Hart Ditch and the Little Calumet River west of the divide flow westward. For WY90, total flow in the Little Calumet River was 77.3 cfs, with 4.9 cfs of that flow being determined to be Indiana water supply.

The Grand Calumet River has a summit. On one side of the summit, the flow is toward Lake Michigan. On the other side of the side of the summit, the flow is toward the Calumet Sag Channel which flows into the CSSC. However, the location of the summit is variable and highly influenced by Lake Michigan levels (USGS, 1984). Thus the calculation of this deduction from the Romeoville record is influenced by Lake Michigan levels. In the absence of a stream gaging station on the Grand Calumet River to measure westward flow into Illinois, flow is computed based on a statistical relationship of which the principal variable is lake levels. Beginning with the WY91 accounting, Grand Calumet River flow will be measured by a gage that was installed in 1990 that began officially measuring flows on 1 October 1990.

Flow in the Grand Calumet River is estimated to be in excess of 90% sanitary effluent. Therefore, it is assumed that the portion of this flow that is attributable to domestic water supply is equal to the sum of the daily water supply pumpage for East Chicago, Whiting, and Hammond (whose pumpage includes water supply for Munster, Highland, and Griffith). If the total water supply pumpage for these communities is greater than the flow in the Grand Calumet River, it is assumed that the flow consists entirely of effluent that originates from water supply.

The total Grand Calumet flow reaching Illinois in WY90 was computed as 23.0 cfs. It was determined that all of the 23.0 cfs was water supply pumpage. Therefore, the total WY90 Indiana water supply deduction, including the flow from the Little Calumet and Grand Calumet Rivers is 27.9 cfs. This is virtually the same as the Indiana water supply deduction for WY89 which was 27.8 cfs.

COLUMN 6: RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CHICAGO SANITARY AND SHIP CANAL

The WY90 average discharge of Des Plaines River watershed runoff reaching the canal (Column 6) is 191.6 cfs. This deduction is determined almost entirely through simulation. The runoff is composed of two elements, surface runoff and subsurface runoff. Surface runoff that enters sewers is referred to as inflow while subsurface runoff is referred to as infiltration. The infiltration and inflow discharged to the water reclamation plants is 97.1 cfs, the infiltration and inflow reaching the canal through combined sewer overflows is 23.8 cfs, and the runoff from the Lower Des Plaines and Summit Conduit areas is 70.7 cfs. The deduction is also influenced by the O'Hare basin flow transfer that contributed 10.9 cfs of the 97.1 cfs runoff to the water reclamation facilities during WY90. The deductible Des Plaines River watershed runoff increased 56.7 cfs from WY89 to WY90 as a result of increased antecedent soil moisture conditions, slightly greater precipitation over the Des Plaines Watershed, and changes in storm characteristics during WY90. Increased runoff may also be partially due to the improvements in the rain gage network as well as the subsequent changes to the hydrologic and hydraulic models.

COLUMN 7: LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES WHICH DISCHARGE TO THE CSSC

Column 7 represents Lake Michigan diversions for Federal use, not chargeable to the State of Illinois, and is typically comprised of water supply pumpage used by federal facilities. Also included is emergency navigation makeup water used for federal purposes. Column 7 represents a deduction from the Romeoville record and the total amount of the WY90 deduction is 2.0 cfs.

COLUMN 8: TOTAL DEDUCTIONS FROM THE CSSC ROMEOVILLE GAGE RECORD

Column 8 is the sum of Columns 4, 5, 6, and 7 and represents the total deduction from the Romeoville record. The total deduction for WY90 is 323.8 cfs.

COLUMN 9: LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL

This column represents water supply pumpage from Lake Michigan that is not discharged to the canal. The water supply pumpage not discharged to the canal is composed of two components:

(1) Lake Michigan water supply used by communities serviced by water reclamation facilities that do not discharge to the CSSC (104.2 cfs). This is a decrease of 2.8 cfs from WY89.

(2) The sanitary portion of combined sewer overflows attributable to Lake Michigan domestic water supply that does not discharge to the CSSC (1.5 cfs).

The communities that make up the flow in the first component are suburbs whose treated effluent is discharged to the Des Plaines River and other watercourses not tributary to the CSSC. These communities include Elk Grove Village, Hoffman Estates, Mount Prospect, Schaumburg, Hanover Park, Rolling Meadows, Streamwood, Arlington Heights, Buffalo Grove, Palatine, Wheeling, Lincolnshire, Riverwoods, Libertyville, Illinois Beach State Park, Winthrop Harbor, Zion, Waukegan, 76 percent of North Chicago, and 38.2 percent of Des Plaines. It should also be noted that the Lake Michigan water supply component of the O'Hare flow transfer is subtracted from the total Lake Michigan water supply of the above communities since (1) the O'Hare flow transfer is treated at the Northside WRP which discharges sanitary effluent that is tributary to the CSSC and (2) the entire Lake Michigan water supply component of the O'Hare flow transfer is from communities contained in the above list. The Lake Michigan water supply for these communities is measured while the sanitary portion of the CSO's is derived through simulation. Column 9 represents an addition to the Romeoville record and the total WY90 addition is 105.7 cfs. This is a decrease of 2.5 cfs from WY89 to WY90.

COLUMN 10: TOTAL DIVERSION

Column 10 is equivalent to Column 3 with the deduction of Column 8 and the addition of Column 9. The total diversion for WY90 is 3,531.2 cfs. This amount is 331.2 cfs greater than Illinois's long term diversion allocation of 3,200 cfs. The 40-year running average diversion, rounded to the nearest cfs, beginning with WY81, is 3,452 cfs and the cumulative deviation from the 3,200 cfs allocation is -2,520 cfs. The negative deviation indicates that the cumulative diversion is greater than an average of 3,200 cfs for the period.

COLUMN 11 THROUGH COLUMN 13: LAKE MICHIGAN DIVERSION COMPONENTS

Column 11 through Column 13 represent the three Lake Michigan diversion components: Lake Michigan pumpage accountable to Illinois (1,754.7 cfs), runoff from the diverted Lake Michigan watershed (872.9 cfs), and direct diversion through the lakefront structures (449.6 cfs) which also accounts for the 1.3 cfs backflow during WY90. The sum of the columns (3,077.2 cfs) should theoretically equal the total diversion as shown in Column 10 (3,531.2 cfs) with one exception. The Romeoville record receives effluent that is assumed to contain only 90% of the water supply pumpage while Column 11, Lake Michigan water supply pumpage accountable to Illinois, does not account for consumptive use. This is based on a consumptive loss (water supply pumpage that is consumed or lost prior to reaching the water reclamation facilities) estimate of 10% of the water supply pumpage (International Great Lake Diversion Consumptive Use Study Board, 1981).

Because the diversion estimate from Columns 11 - 13 is based on simulation, suspect ratings of the lakefront structures, and simple flow separation techniques, the estimate is not expected to be as accurate as the AVM based calculations. Consequently, a difference between estimates of 454.0 cfs or 12.9% is a fair balance. However, this discrepancy becomes even greater when consumptive use is accounted for in Column 11. The discrepancy in these two estimates is related to the canal system balance in Budget 14, discussed in a subsequent section, and potential sources of the discrepancy are addressed in that budget discussion.

Using the figures from these three columns, 57.0% of the WY90 Illinois diversion is attributable to pumpage from Lake Michigan for domestic water supply. Runoff from the diverted Lake Michigan Watershed accounted for 28.4% of the diversion, and direct diversion through the lakefront structures accounted for 14.6% of the diversion. Water supply from Lake Michigan actually dropped 37.2 cfs from WY89 to WY90. This is most likely due to the slight increase in basin wide precipitation during WY90. Additionally, a larger percentage of rain fell during the summer months when demand tends to be slightly higher. Due to increased antecedent soil moisture and slightly higher precipitation during WY90 there was a 164.1 cfs increase in runoff from the Lake Michigan watershed that occurred between WY89 and WY90. A more detailed breakdown of these percentages is shown in Table 5 and Figure 2.

Table 5

Breakdown of the Diversion by the State of Illinois
Based on Columns 11 Through 13

Category	Flow	Percentage
Lake Michigan Pumpage by the State of Illinois	1,754.9 cfs	57.0 %
Runoff from the Diverted Lake Michigan Watershed	872.9 cfs	28.4 %
Direct Diversions		
Lockages	71.8 cfs	2.3 %
Leakages	28.3 cfs	0.9 %
Navigation Makeup Flow	46.1 cfs	1.5 %
Discretionary Flow	304.7 cfs	9.9 %

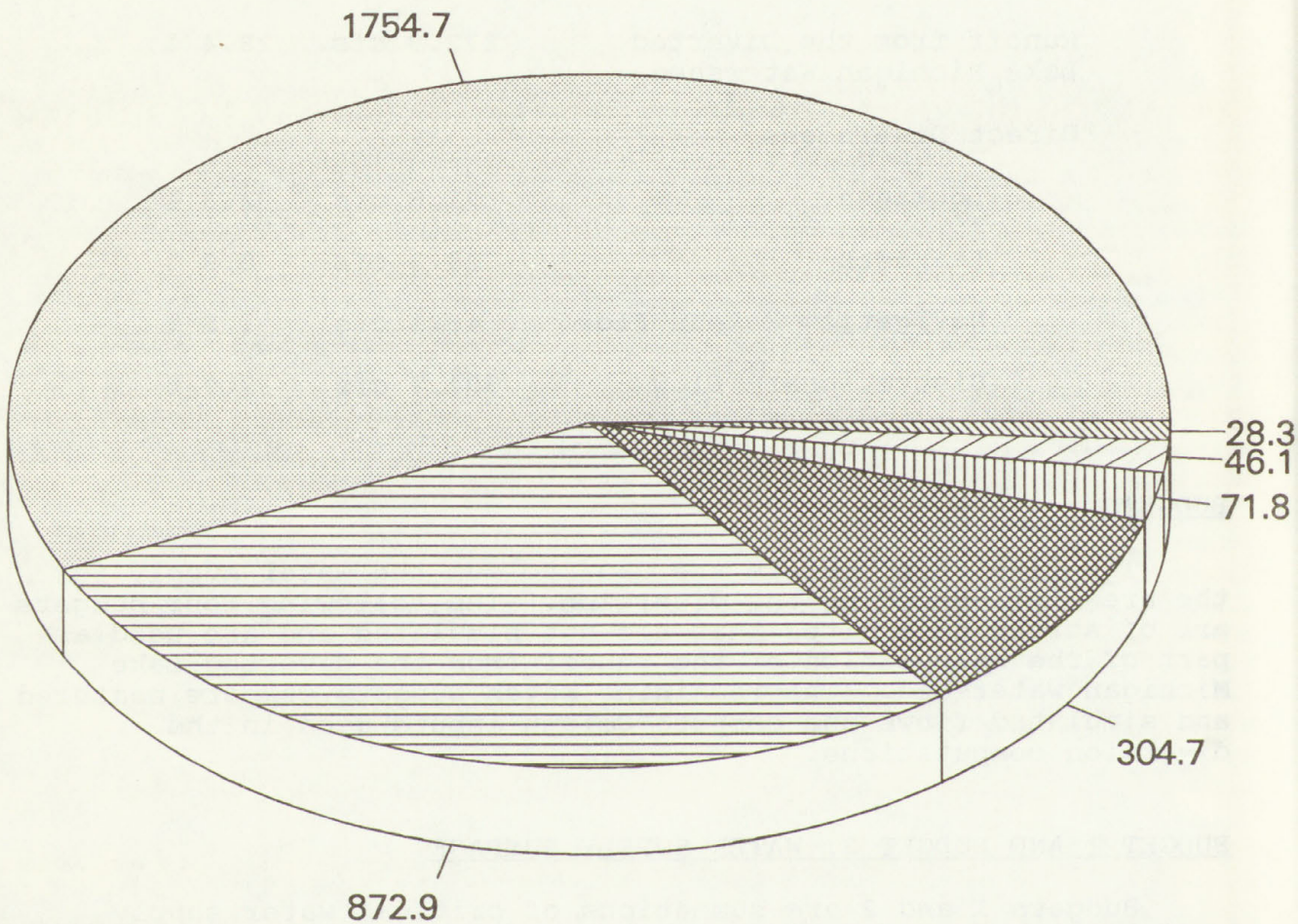
BUDGETS

The first two budgets are used to sum the water supply for the area influenced by the diversion. The following four budgets are of stream gage sites that are not simulated and are used as part of the calculation of the runoff from the diverted Lake Michigan watershed. The remaining seven budgets compare measured and simulated flows and compute Column inputs used in the diversion computations.

BUDGET 1 AND BUDGET 2: WATER SUPPLY PUMPAGE

Budgets 1 and 2 are summations of critical water supply pumpage data. Budget 1 sums Lake Michigan water supply diverted by the State of Illinois. The Lake Michigan water supply data is supplied by the state as daily values for primary users and monthly data for secondary users. Budget 2 sums groundwater pumpages in the Lake Michigan and Des Plaines River watersheds that are diverted to the Chicago Sanitary and Ship Canal. Groundwater pumpage data is recorded as a total annual withdrawal based on calendar years.

Figure 2
Component Breakdown of Illinois' Diversion
Based on Columns 11 through 13



DIVERSION COMPONENTS

WATER SUPPLY	57.0 %	RUNOFF	28.4 %
DISCRETIONARY	9.9 %	LOCKAGES	2.3 %
NAV MAKEUP	1.5 %	LEAKAGES	0.9 %

BUDGET 1: DIVERTED LAKE MICHIGAN WATER SUPPLY

Budget 1 represents the summation of Lake Michigan pumpage accountable to the State of Illinois. For WY90, the average annual Lake Michigan pumpage accountable to Illinois is 1,754.7 cfs. This is a decrease of 37.2 cfs from WY89. As stated previously, this is most likely due to the slight increase in precipitation, especially during the summer months when demand is usually higher.

BUDGET 2: GROUNDWATER DIVERTED TO THE CHICAGO SANITARY AND SHIP CANAL

Budget 2 is groundwater water supply pumpage by communities, industrial users, and other private users, as reported by the Illinois State Water Survey (ISWS), whose effluent is discharged to the canal. This quantity is determined by summing all reported groundwater sources in the area tributary to the canal less groundwater not discharged to the canal in the form of combined sewer overflows.

Using ISWS groundwater records, groundwater pumpages were assumed to reach the CSSC and adjoining channels if they were located in the diverted Lake Michigan watershed in Illinois or if they were located within MWRDGC service boundaries in which their effluent was discharged into the CSSC and adjoining channels.

The total groundwater pumpage by communities, industrial users, and other private users whose sanitary effluent is tributary to the canal is 43.9 cfs for WY90. It was determined through simulation that 0.2 cfs of this flow never reached the canal. Instead it was discharged to the Des Plaines River or other watercourses not tributary to the canal in the form of combined sewer overflows. The total groundwater pumpage reaching the canal represents an increase of 21.0 cfs from WY89 to WY90.

In addition to groundwater supply pumpage there was also a significant amount of groundwater infiltration into the two TARP systems that ultimately reached the canal. Mainstream TARP and Calumet TARP accounted for 52.0 cfs and 6.6 cfs, respectively, of groundwater discharged to the canal during WY90. Groundwater infiltration into the Calumet TARP system was significantly less than in previous years due to changes in the simulation model during the WY89 accounting.

BUDGETS 3 THROUGH BUDGET 6: STREAM GAGING STATIONS

The stream gage budgets are used to make estimates of runoff from portions of the diverted Lake Michigan watershed. Sanitary and other point source flows are subtracted from the stream gaging record to develop the runoff estimates. The runoff estimates are used in Column 12. The flows at the stream gaging sites is also part of Budget 14, the canal system budget. Table 6 presents the estimated runoff from these budgets. It should be noted that Budgets 4 through 6 are a composite calculation of the runoff above the Little Calumet River at South Holland gage. It should also be noted that the Little Calumet River is a losing stream, i.e. it recharges groundwater. The computations in deriving runoff account for this when recharge is significant (i.e., when groundwater recharge is computed).

Table 6
Stream Gage Flow Separation

Budget	Location	Flow cfs	Sanitary cfs	Runoff cfs
3	North Branch Chicago River at Niles, IL	125.8	18.8	107.0
4	Little Calumet River at IL-IN State Line	77.3	3.9	73.4
5	Thorn Creek at Thornton, IL	118.9	16.8	102.1
6	Little Calumet River at South Holland, IL_1	190.2	182.0	8.2

1 Incremental Runoff

BUDGETS 7 THROUGH BUDGET 13: MWRDGC WATER RECLAMATION FACILITIES

The budgets for the water reclamation plants compare the simulated flows to the measured inflows at the MWRDGC facilities and perform verifications of the diversion accounting program. The simulated flows were developed from an estimated sanitary flow with a daily, weekly, and monthly flow variation, from hydrologic precipitation-based runoff models, and from hydraulic sewer routing models. The estimated sanitary flow input to the hydraulic simulation models is based on the population estimates for each plant's service basin. Per capita sanitary flows are determined based on the service basin's water supply minus an assumed 10 percent consumptive loss. Simulated flows were compared with recorded inflows at each facility to assess the accuracy of the simulations.

The discussion of the budgets will concentrate on the results of each simulation as the development of these models have been discussed in previous reports. A summary of the simulation results is presented in Table 7. At all four water reclamation plants and the Upper Des Plaines Pump Station the simulation results were improved. This is the result of the new 25-gage precipitation network first utilized for the WY90 diversion accounting, improvements and updates in the land cover delineations, and modifications to the hydrologic and hydraulic models .

BUDGET 7: NORTHSIDE WATER RECLAMATION FACILITY

Budget 7 analyzes the water balance at the MWRDGC Northside Water Reclamation Facility (Figure 3). The balance for WY90 of the inflow to the Northside facility is very good. The simulated to adjusted recorded inflow ratio (S/R) for the Northside WRP is 0.94, indicating that the simulated inflow volume is slightly less than the adjusted observed inflow volume. The coefficient of correlation (R) of simulated to observed flow is 0.89, indicating that the model predicted the inflow hydrograph to the Northside facility very well.

BUDGET 8: UPPER DES PLAINES PUMP STATION

Budget 8 analyzes the water balance at Upper Des Plaines Pump Station (UDPPS) (Figure 4). The pump station budget is used to verify simulated flows. Although it has no direct impact on the diversion calculation, it is intended to be used as a primary calibration point for the models that simulate the deductible runoff from the Des Plaines watershed contained in Column 6. This will be possible only after the existing measurement problems at that site are resolved. This will be discussed later in the report.

The balance at UDPPS for WY90 was reasonable. The simulated to recorded flow ratio (S/R) for the UDPPS is 1.08, indicating that the simulated inflow volume to UDPPS is greater than the recorded inflow volume. However, the daily S/R ratio shows a high degree of variability, indicating that the trends within the recorded and simulated inflow may not correspond very well. The coefficient of correlation (R) of simulated to recorded flow is 0.64, indicating the time series trends in the simulated inflow compared fairly well with the time series trends of recorded inflow. This is a significant improvement over previous years. This may be the direct result of the revised raingage network and subsequent modifications to the hydrologic and hydraulic models.

TABLE 7

WY 1990 SUMMARY OF SIMULATION STATISTICS

Budget No.->	7	8	9	10	11	12	13	14
Description	Northside WRP (1)	Upper Des Plaines Pump Station (1),(3)	Mainstream TARP Pump Station (2)	Stickney WRP (1),(4)	Calumet TARP Pump Station (2)	Calumet WRP (1),(4)	Lemont WRP (1)	Chicago Canal System Balance (1)
Mean Recorded Flow, cfs	437.3	69.2	96.4	1054.9	39.0	385.6	2.2	3269.3
Max. Recorded Flow, cfs	792.0	111.9	262.9	2038.1	129.1	593.1	8.5	17360.0
Min. Recorded Flow, cfs	333.6	20.1	19.9	608.2	2.7	263.5	1.1	1513.8
Mean Simulated Flow, cfs	411.2	74.7	101.3	1125.0	28.6	388.0	1.9	3758.3
Max. Simulated Flow, cfs	695.0	188.3	254.0	2591.7	98.7	611.9	5.4	18098.0
Min. Simulated Flow, cfs	301.6	43.6	39.5	803.6	3.0	308.9	1.2	1772.1
Mean S/R	0.94	1.08	1.05	1.07	0.73	1.00	0.86	1.15
Max. S/R	1.15	2.96	6.19	1.75	4.85	1.35	1.65	2.18
Min. S/R	0.56	0.57	0.39	0.70	0.19	0.83	0.41	0.51
Correlation	0.89	0.64	0.49	0.86	0.51	0.89	0.92	0.84

(1) Based on daily values.
(2) Based on weekly values.
(3) Does not include days with missing records.
(4) Does not include pumpage from TARP.

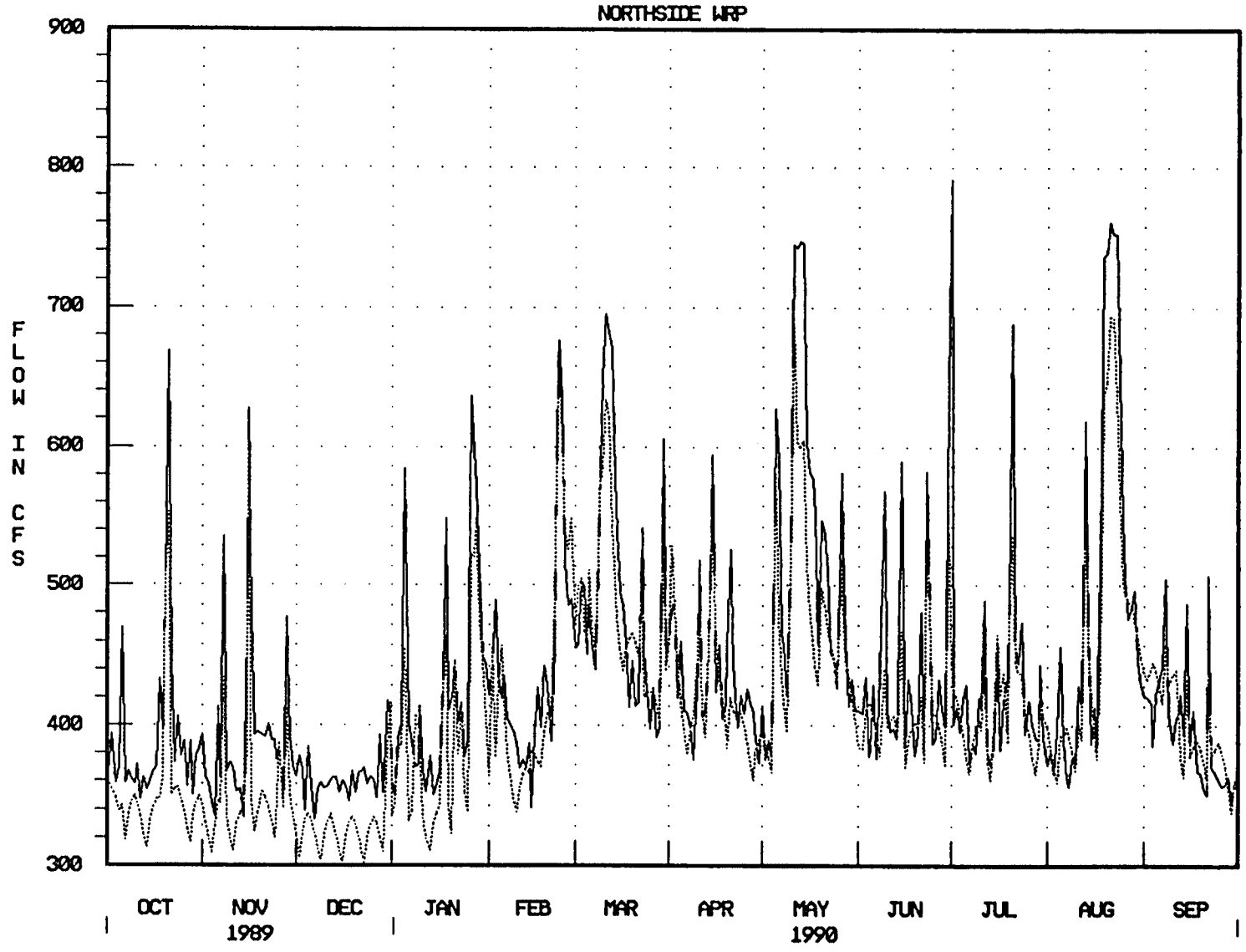


FIGURE 3

Budget 7 - Simulation of the MWRDGC Northside Water Reclamation Facility

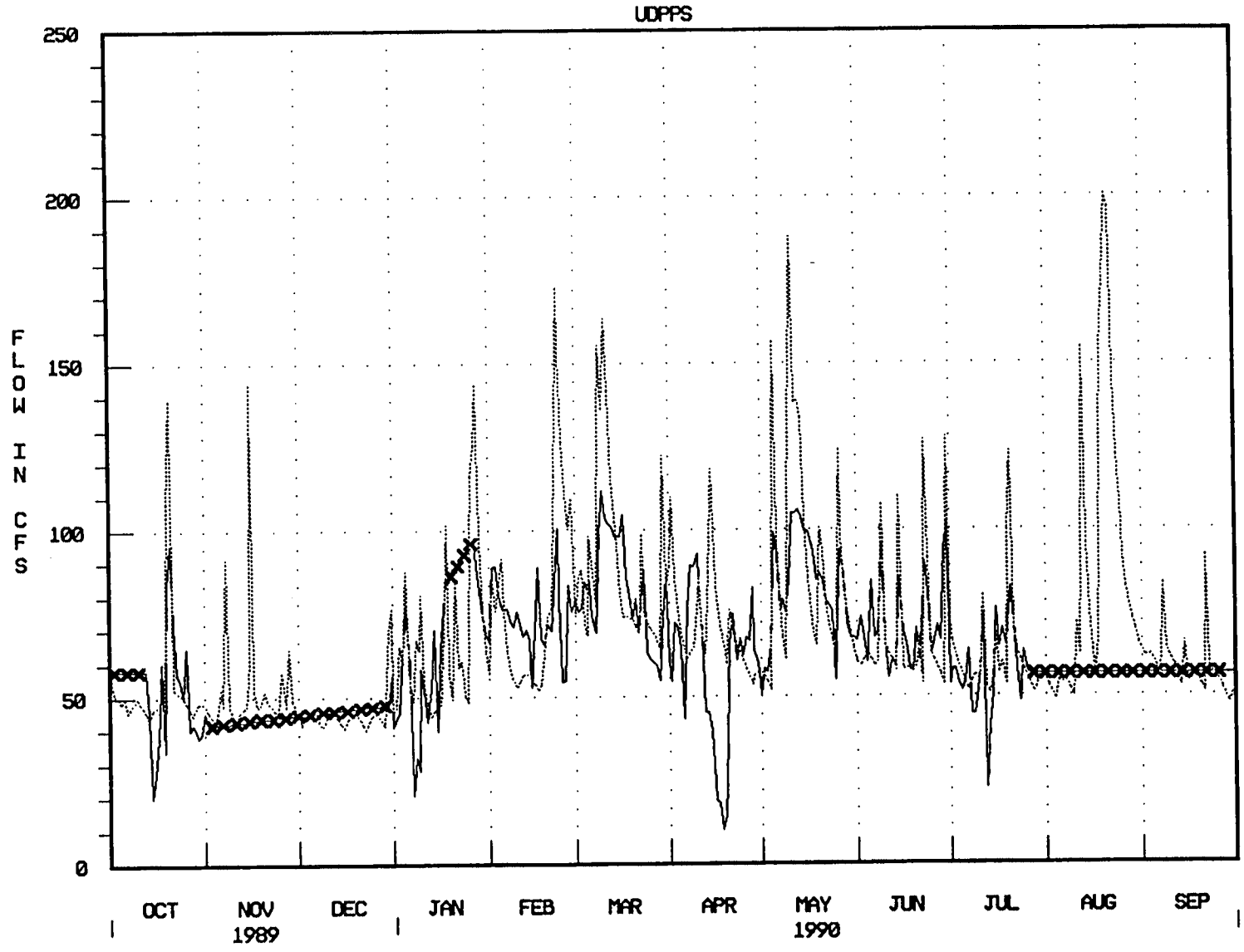


FIGURE 4

UPPER DES PLAINES PUMP STATION OBSERVED FLOW
UPPER DES PLAINES PUMP STATION SIMULATED FLOW

Budget 8 - Simulation of the MWRDGC Upper Des Plaines Pump Station

While the statistical results for WY90 at the Upper Des Plaines Pump Station have improved, this does not lead to the conclusion that flow measurement alternatives should not be investigated. This site has continued to experience its share of problems. During WY90, 145 days of records were unavailable that were attributable to meter malfunctions, problems with the recording charts which made data transformation undoable, and various other reasons. In view of the significant quantity of missing data (39.7 % missing data), the quantitative analyses of the simulation are of limited value. Second, the accuracy of the flow meters at the pump station is questionable and unmetered bypass flows are a frequent occurrence. Therefore, total flow may not be measured in storm events and the recycling of flow is possible. Further investigation of the accuracy of flow measurement at the pump station is required to verify and calibrate the simulation models that compute the deductible runoff from the Des Plaines watershed contained in Column 6.

BUDGET 9: MAINSTREAM TARP PUMPING STATION

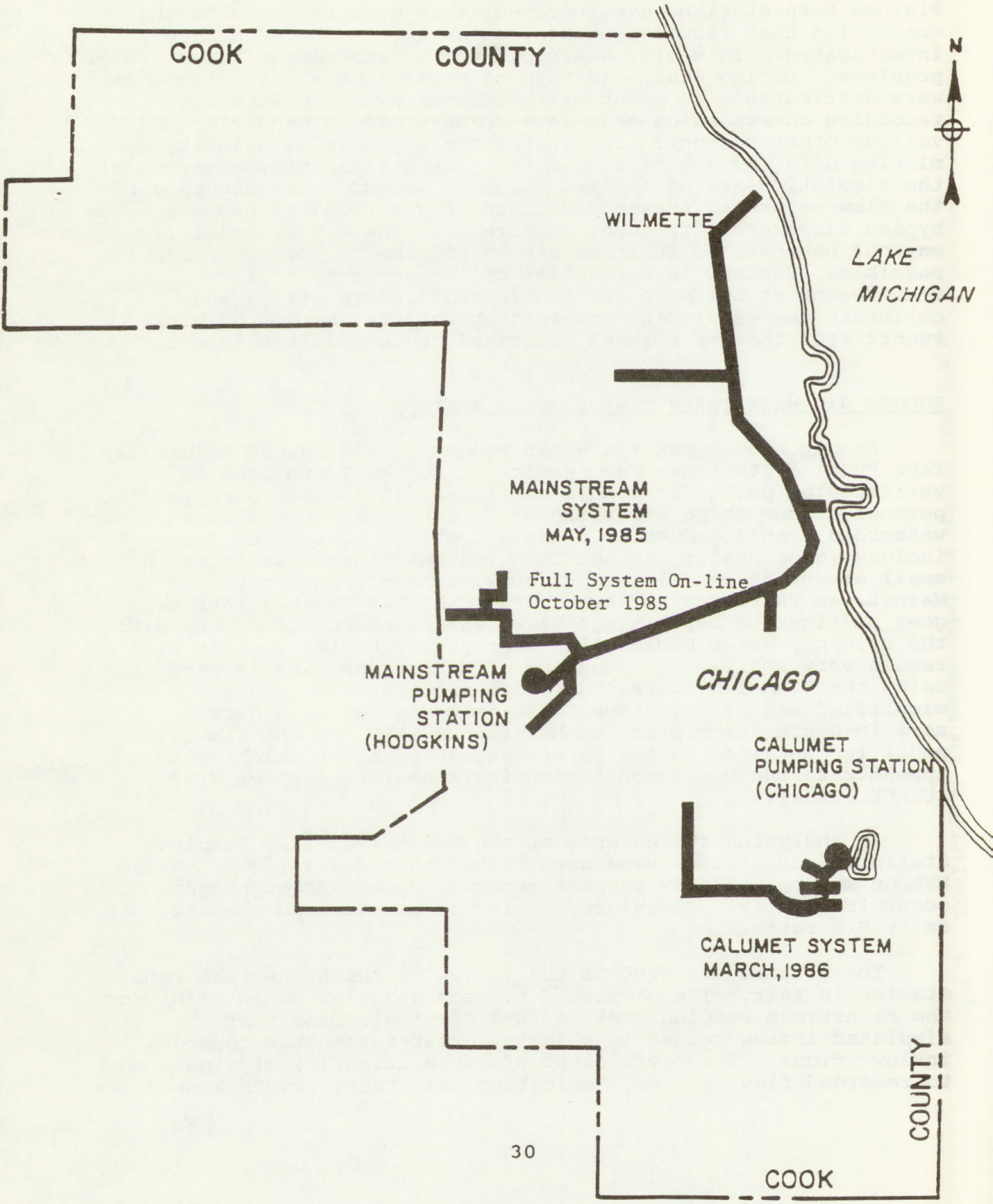
Budget 9 analyzes the water budget at the MWRDGC Mainstream TARP Pumping Station. The results of Budget 9 are used as a verification point for simulated flows. It is also used for the purpose of computing a portion of Column 6, the Des Plaines River watershed runoff deduction. The deductible portion of Budget 9 includes groundwater seepage into the TARP tunnel walls and a small amount of Des Plaines River watershed runoff captured by Mainstream TARP as overflows. Until the Des Plaines TARP segment goes on-line the Des Plaines River watershed runoff conveyed to the Stickney Water Reclamation Plant through TARP tunnels will remain very small. The modeling of Mainstream TARP is performed using the Tunnel Network (TNET) dynamic hydraulic model. A simplified map of Mainstream TARP is contained in Figure 5. A more in-depth description of Mainstream TARP and the simulation model is contained in the Water Year 1986 report which is an appendix to the Diversion Accounting Annual Report for WY90-92 (USACE, 1994).

In analyzing the balance at the Mainstream TARP Pumping Station, weekly flows were used rather than daily flows. While MWRDGC maintains daily pumpage records, days with no pumpage occur frequently. Therefore, it is not possible to compute a daily S/R ratio.

The balance for WY90 of the inflow to the Mainstream Pumping Station is fair. The simulated to recorded flow ratio (S/R) for the Mainstream Pumping Station is 1.05, indicating that the simulated inflow volume is slightly greater than the recorded inflow volume. The coefficient of correlation (R) of simulated to recorded flow is 0.49, indicating that there exists a need for

Figure 5

Map of Mainstream and Calumet TARP



improvement in the ability of the model to predict trends in the pump station flows.

From a review of the plot of the simulated versus recorded flow at the pump station (Figure 6), it appears that the model responds similarly to recorded pumpage record. However, the model is sometimes out of phase with the observed record. This could be the result of simulated pumpages occurring sooner and more frequently than actual pumpages. The model pumps normally turn on sooner and pump more frequently in order to maintain model stability during a run. Additionally, base flows appear to be overestimated in the simulation. This is probably due to overestimation of groundwater infiltration into the TARP tunnels.

In summary, it appears that the simulation of the Mainstream TARP system is reasonable. However, there is concern regarding the estimation of pumpage volume and the difference in simulated and recorded pumpage time series. A review of MWRDGC information regarding Mainstream TARP indicates that bypass flows are discharged to TARP, when available, via drop shaft 11 (DSN 11). Coordination with MWRDGC established that this is a frequent occurrence. This may account for the simulation of a pumpage volume that is less than the recorded pumpage volume. Records concerning the dates and pumpages back to TARP were not maintained for WY90. Therefore, data necessary to evaluate the impact of pumping back into TARP is not available. Therefore, it was decided that the model would not be adjusted so as to avoid double accounting of flows.

BUDGET 10: STICKNEY WATER RECLAMATION FACILITY

Budget 10 analyzes the water balance at the MWRDGC Stickney Water Reclamation Facility (Figure 7). Simulated Mainstream TARP pumpages from Budget 9 are no longer combined with simulated interceptor inflow to the Stickney Water Reclamation Facility to derive the total simulated inflow to the Stickney Facility. Instead, only simulated interceptor inflows are compared with recorded interceptor inflows to assess the accuracy of the simulation. The decision to not include TARP pumpages in the treatment plant budgets was based on the fact that the TARP systems are already analyzed in separate budgets. Including TARP pumpages in the treatment plant budgets is detrimental to the statistical results of the treatment plant budgets since the TARP models generally do not respond as well. When simulations of interceptor flows are treated separately, the response of the hydrologic runoff models (HSPF) and the hydraulic sewer routing models (SCALP) can be better isolated and not diluted by the TARP model results which are analyzed separately on their own merits and contained in their own budgets, Budgets 9 and 11.

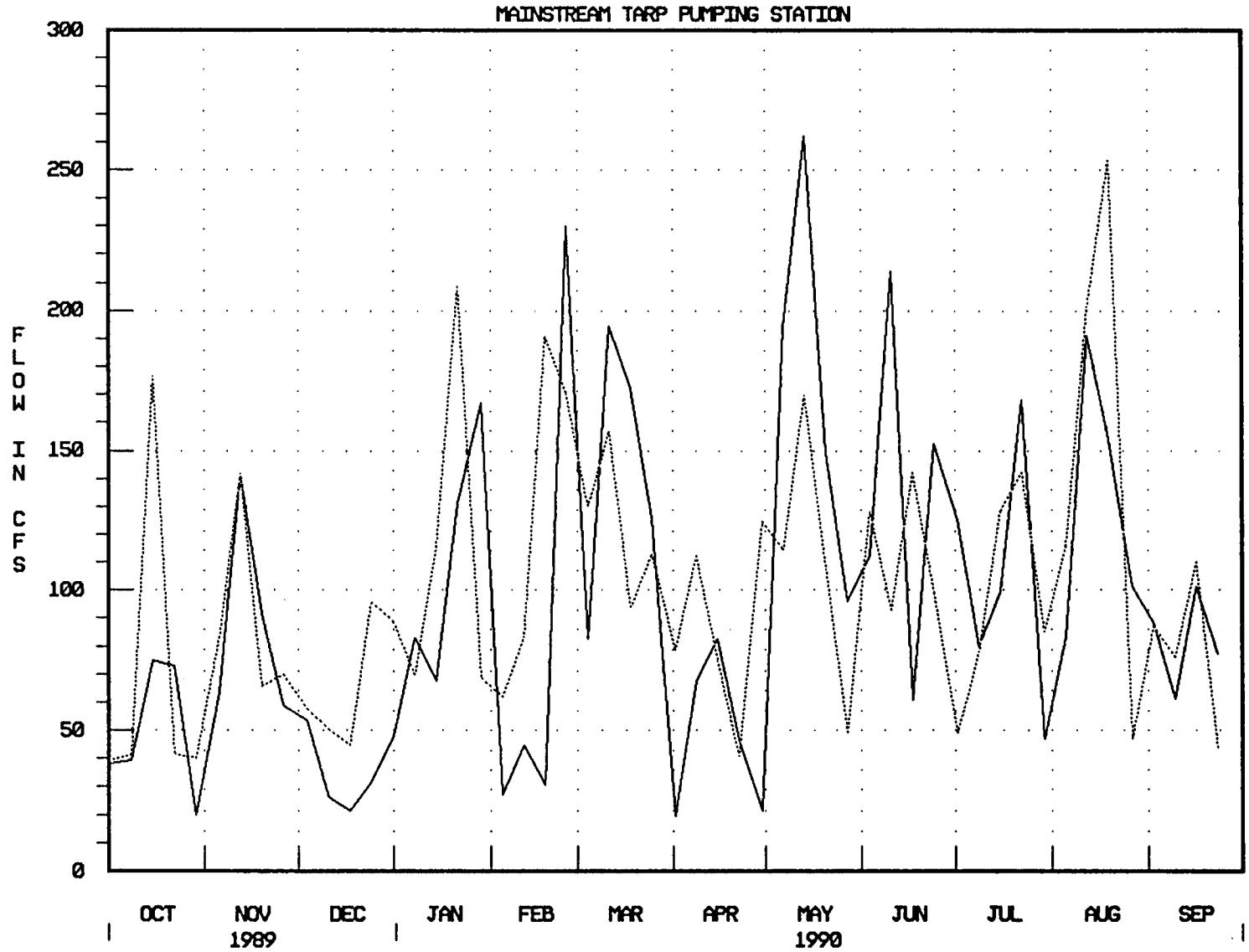


FIGURE 6

Budget 9 - Simulation of the MWRDGC Mainstream TARP Pumping Station

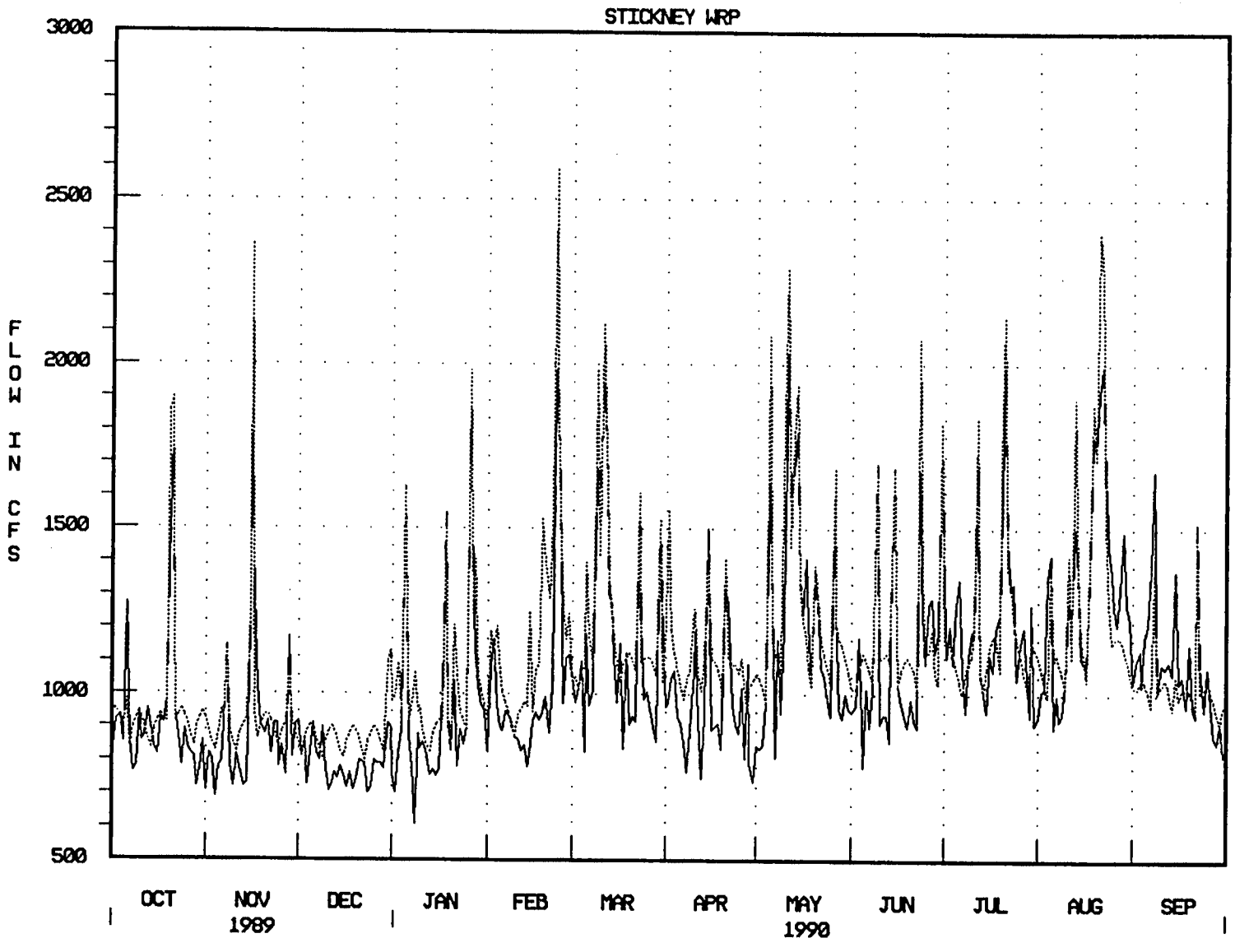


FIGURE 7

OBSERVED FLOW
SIMULATED FLOW

Budget 10 - Simulation of the MWRDGC Stickney Water Reclamation Facility

Overall, the balance for WY90 of the inflow to the Stickney facility is very good. The simulated to recorded flow ratio (S/R) for the Stickney is 1.07, indicating that the simulated interceptor inflow volume is slightly greater than the recorded interceptor inflow volume. The coefficient of correlation (R) of simulated to recorded flow is 0.86, indicating that the model predicted the interceptor inflow hydrograph to the Stickney facility very well.

BUDGET 11: CALUMET TARP PUMPING STATION

Budget 11 analyzes the water budget at the MWRDGC Calumet TARP Pumping Station (Figure 8). The results of Budget 11 are used as a verification point for simulated flows. The modeling of Calumet TARP is performed using the Tunnel Network (TNET) dynamic hydraulic model. A simplified map of Calumet TARP is contained in Figure 5. A more in-depth description of Calumet TARP and the simulation model is contained in the Water Year 1987 report contained in the Diversion Accounting Annual Report for WY90-92 (USACE, 1994).

In analyzing the balance at the Calumet TARP Pumping Station, weekly flows were used instead of daily flows. While MWRDGC maintain daily pumpage records, days with no pumpage occur frequently. Therefore, it is not possible to compute a daily S/R ratio.

The balance for WY90 of the inflow to the Calumet TARP Pumping Station is fair. The simulated to recorded flow ratio (S/R) for the Calumet TARP Pumping Station is 0.74 indicating that the simulated inflow volume is less than the recorded inflow volume. The coefficient of correlation (R) of simulated to recorded flow is 0.51, indicating that there was a need for improvement in the agreement between the trends of the simulated and observed Calumet TARP pumpages.

From a review of the plot of the simulated versus recorded flow at the pump station (Figure 8) it appears that the model responds similarly to the recorded pumpage record except that the recorded pumpage often lagged behind the simulated pumpages for WY90. During a review it was discovered that each of the four variable speed pumps were out of service during portions of WY90, often several at one time. Since these pumps are used during high flows (the two constant rate pumps shut off above a certain tunnel elevation) dewatering of the tunnel may have occurred over a longer time period. Consequently, simulated daily pumpages were lagged and compared to the recorded pumpages on a weekly basis in order to check if the model response would improve. It was determined that when simulated daily pumpages were lagged 3 days the coefficient of correlation improved from 0.51 to 0.65 indicating that the Calumet TARP model response was better than

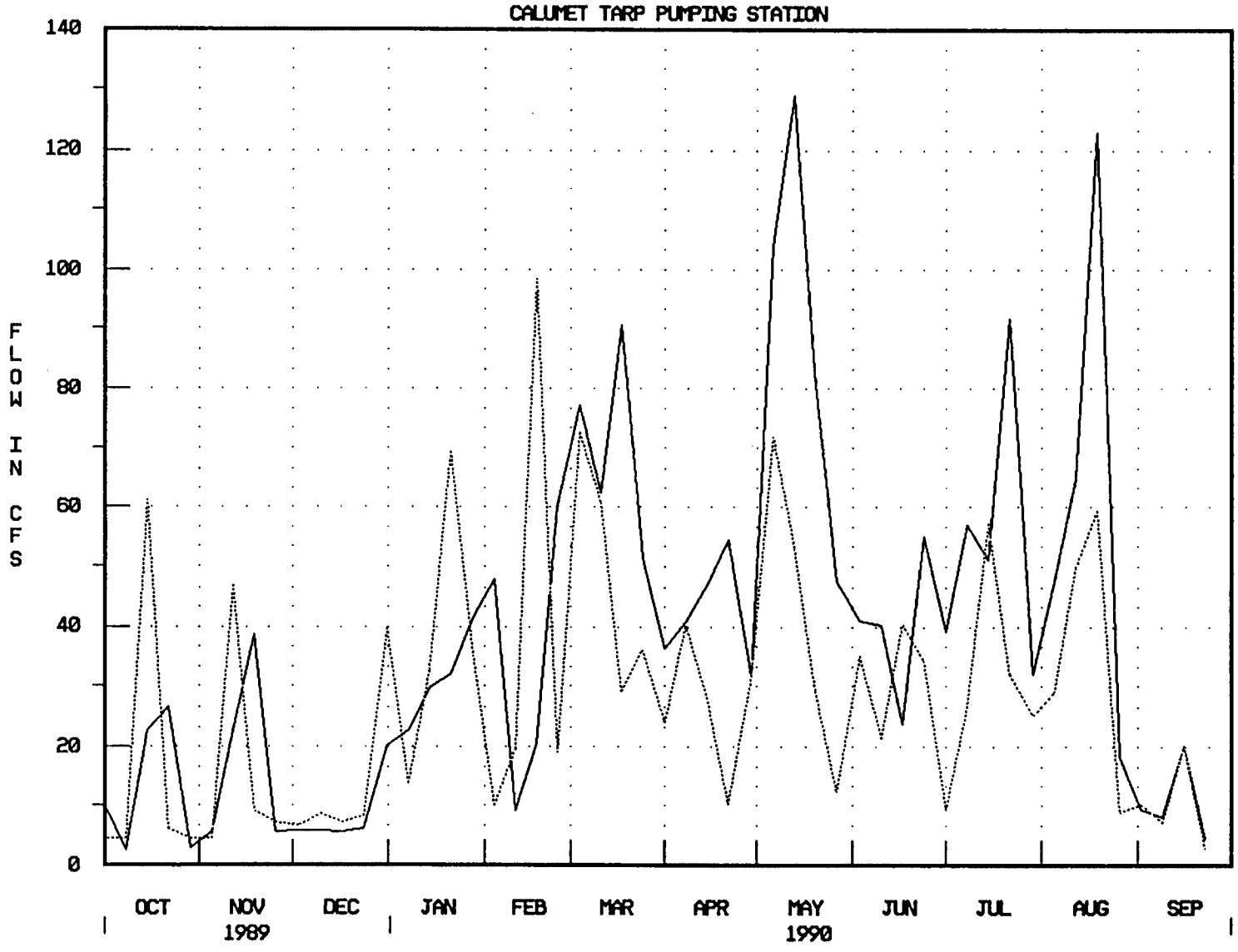


FIGURE 8

OBSERVED FLOW TO CALUMET WRP
SIMULATED FLOW TO CALUMET WRP

Budget 11 - Simulation of the MWRDGC Calumet TARP Pumping Station

originally thought. It is due to the dynamic nature of variables like this that the successful modeling of the TARP systems is a difficult task.

Volume matching between the simulated and recorded Calumet TARP pumpages also was more difficult for WY90 as evidenced by the 0.74 S/R ratio. Because of the instability of the TARP model as well as uncertainties in the Calumet TARP system it was difficult to improve on this ratio. However, as the system is presently modeled, this does not impact the computed diversion since all Des Plaines River watershed areas whose overflows are modeled as tributary to Calumet TARP are also modeled such that "non-captured" overflows overflow to rivers that are tributary to the CSSC. Therefore, whether or not these Des Plaines River watershed runoff flows enter the tunnel or not, they are presently included in the Des Plaines River watershed runoff deduction in Column 6. This assumption will remain until separately sewered areas are modeled such that actual areas are used instead of effective areas in the hydraulic models. This will be discussed in the subsequent section on areas for improvement in diversion accounting procedures.

BUDGET 12: CALUMET WATER RECLAMATION FACILITY

Budget 12 analyzes the water balance at the MWRDGC Calumet Water Reclamation Facility (Figure 9). Simulated Calumet TARP pumpages from Budget 11 are no longer combined with simulated interceptor inflows to the Calumet Water Reclamation Facility to derive the total simulated inflow to the Calumet Facility. Instead, only simulated interceptor inflows are compared with recorded inflows to assess the accuracy of the simulation. This was revised for the same reasons as outlined previously in the discussion for Budget 10.

The annual simulated to recorded flow ratio (S/R) and the coefficient of correlation for the Calumet Water Reclamation Facility are considered very good to excellent. The S/R ratio is 1.00 indicating that the simulated Calumet interceptor flow volume matched the recorded interceptor flow volume. The coefficient of correlation was 0.89 indicating a very good correlation between simulated and recorded interceptor flows.

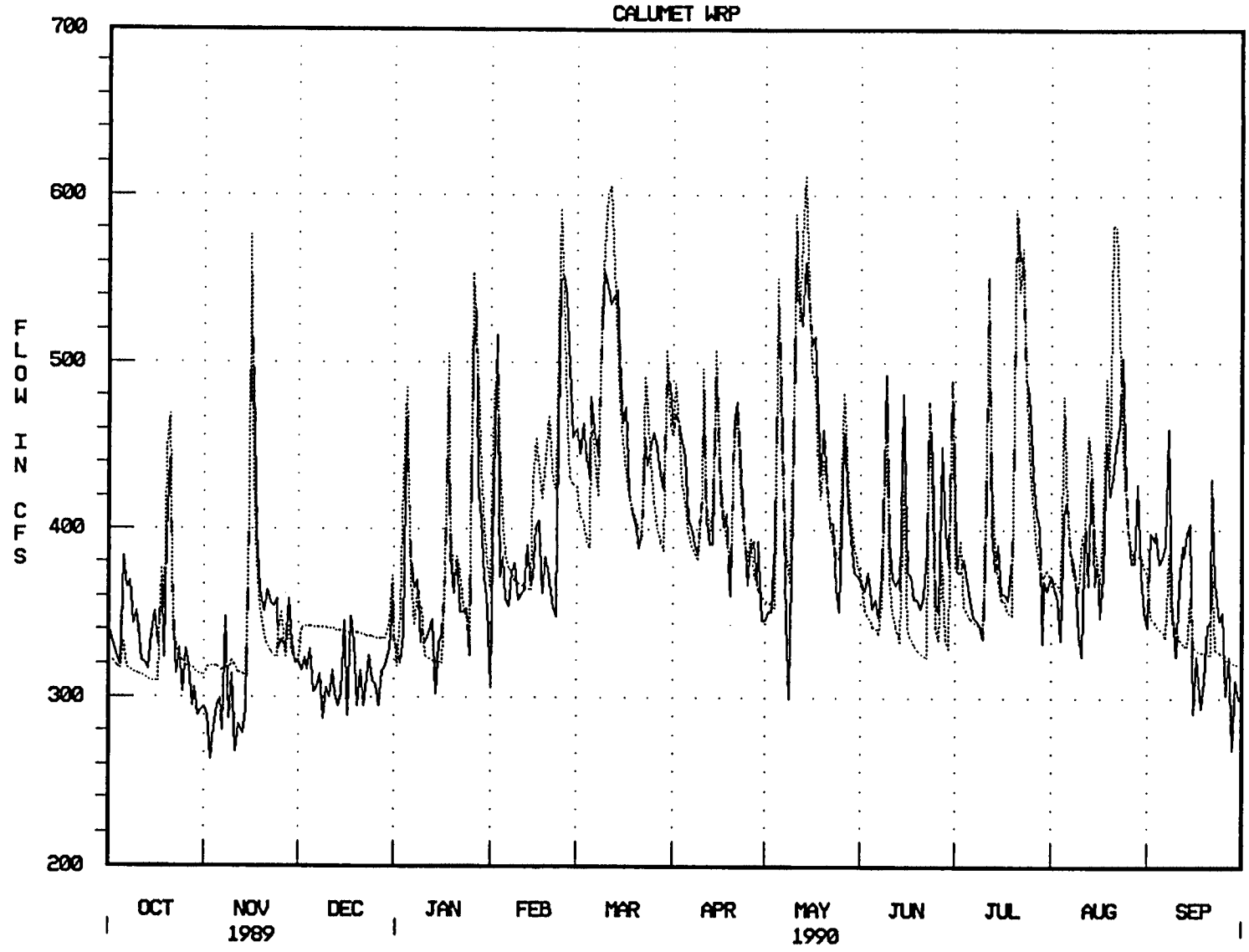


FIGURE 9

OBSERVED FLOW
SIMULATED FLOW

BUDGET 13: LEMONT WATER RECLAMATION FACILITY

Budget 13 analyzes the water balance at the MWRDGC Lemont Water Reclamation Facility (Figure 10). Overall, the balance for WY90 of the inflow to the Lemont facility is very good. The simulated to recorded flow ratio (S/R) for the Lemont is 0.86, indicating that the simulated inflow volume was somewhat less than the recorded inflow volume. The coefficient of correlation (R) of simulated to recorded flow is 0.92, indicating that the model predicted the inflow hydrograph to the Lemont facility quite well.

BUDGET 14: CHICAGO CANAL SYSTEM BALANCE

Budget 14 compares the inflows and outflows to the canal system (Figure 11). The inflow components include direct diversions through the lakefront structures, stormwater runoff discharged to the canal system, and domestic water supply whose effluent discharges to the canal system. The outflows from the canal system include the discharge past the Romeoville AVM, backflows through the lakefront structures, and withdrawals upstream of Romeoville by Argonne National labs and Uno-ven corporation. The individual components are presented in Table 8 for WY90.

Overall, the balance for WY90 between the inflows to the canal system and the outflows from the canal system is fair. The S/R (outflow/inflow) for the canal system is 1.15, indicating that the inflow to the canal system is less than the outflow from the canal system. The average measured/simulated inflow was 3269.3 cfs while the average measured/simulated outflow was 3758.3 cfs. This is a difference of 489.0 cfs (13.0%) for WY90 as compared to 480.2 cfs (13.6%) for the previous water year, WY89.

The coefficient of correlation (R) of inflow to outflow is 0.84, indicating that the time series trends of inflow to outflow are fairly well correlated. The coefficient of correlation is based on daily flows. Therefore, timing between inflows and measured outflows at Romeoville is a major issue, especially during changes in flow that occur at the beginning or end of a day. This is the result of travel time from inflow locations downstream to the Romeoville AVM site. Therefore, variability in the coefficient of correlation from year to year may be attributed to the variability in the timing of significant flow changes during a particular year.

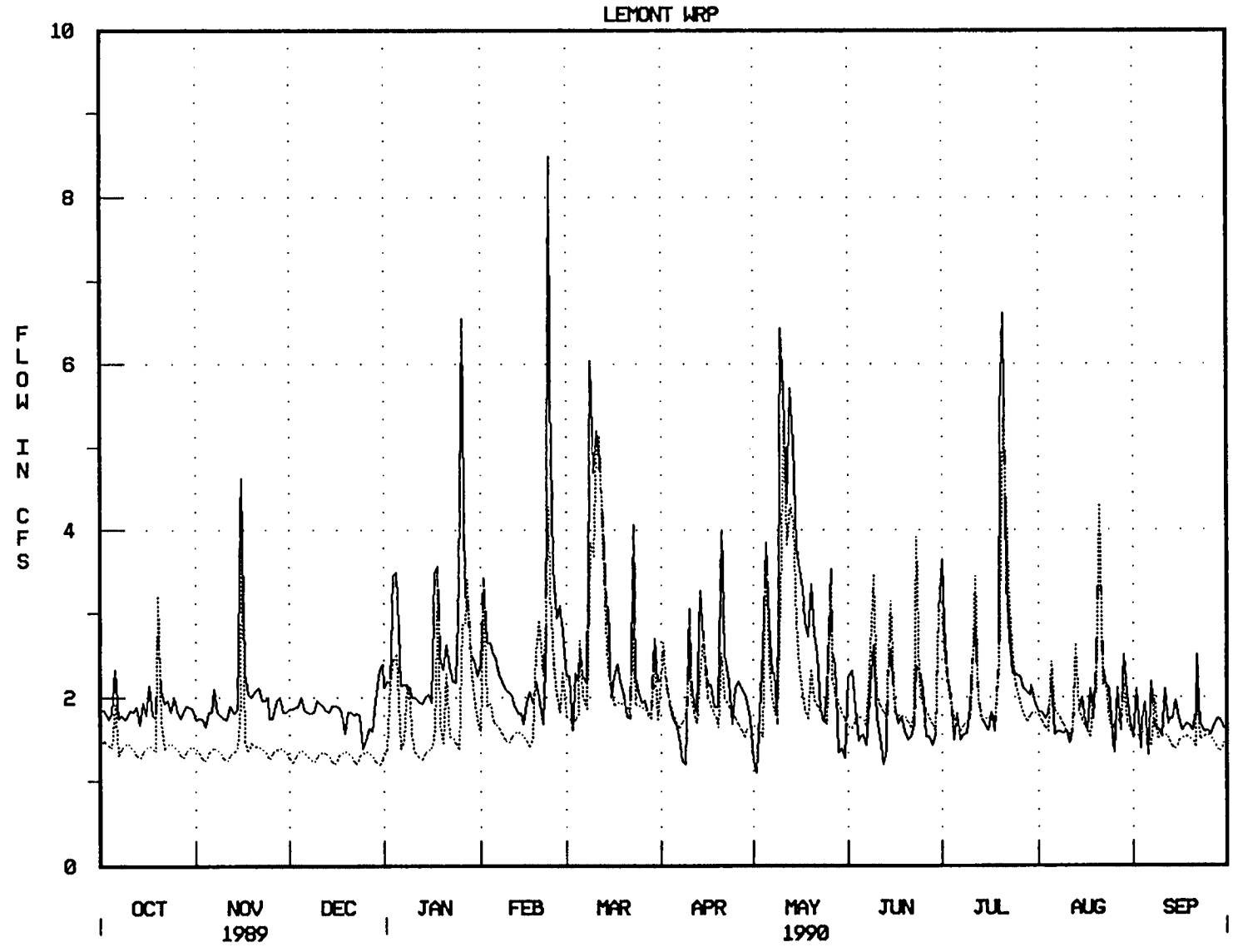


FIGURE 10

Budget 13 - Simulation of the MWRDGC Lemont Water Reclamation Facility

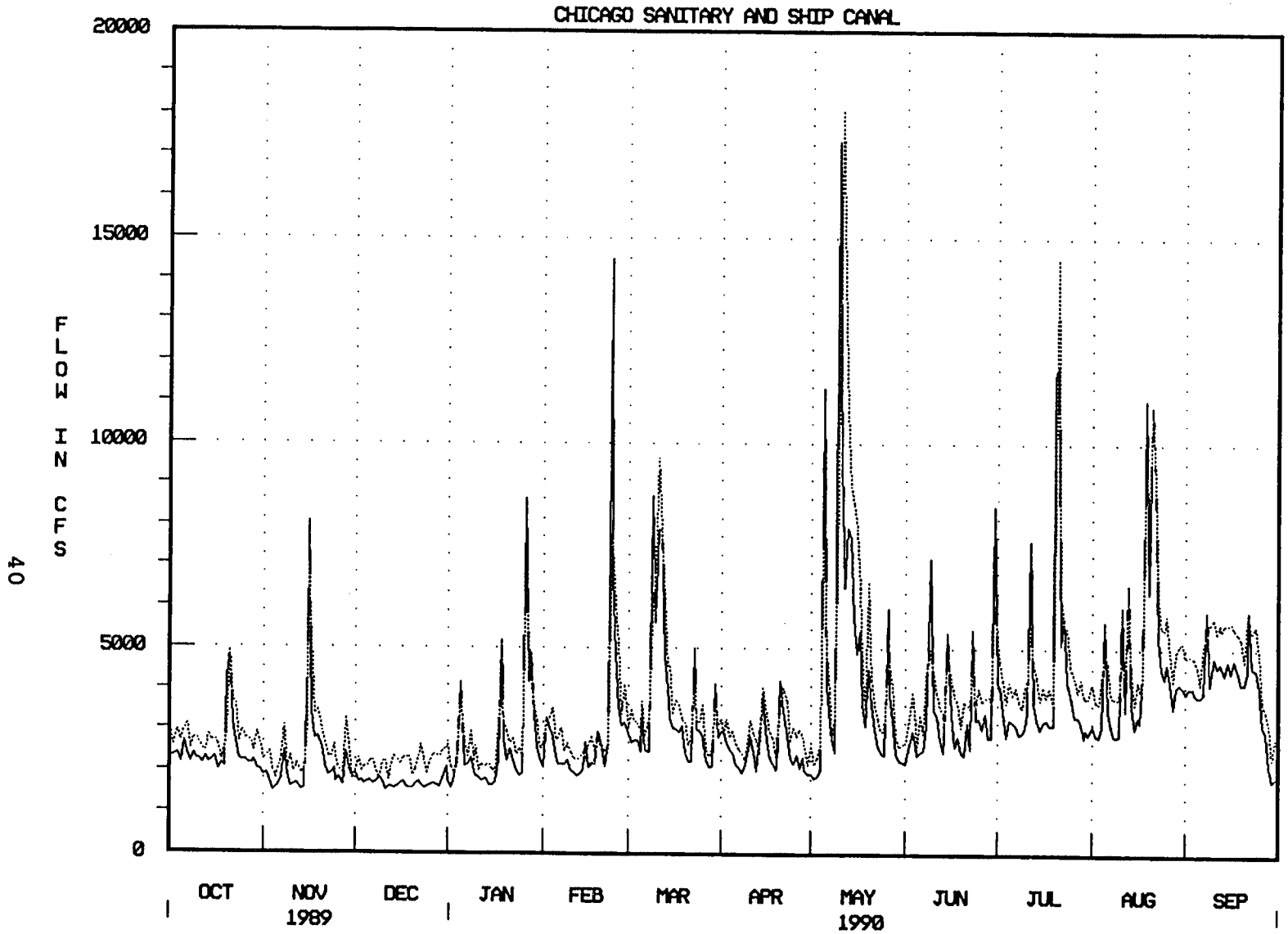


FIGURE 11

CANAL SYSTEM INFLOWS
CANAL SYSTEM OUTFLOWS

Budget 14 - Canal System Balance

TABLE 8

SUMMARY OF FLOW COMPONENTS FOR
CANAL SYSTEM BALANCE – WY1990

INFLOWS (cfs)	
Lake Controlling Structures (measured)	
– Wilmette Controlling Works	22.7
– Chicago River Controlling Works	224.7
– O'Brien Lock and Dam	203.4
Streamflows (measured)	
– North Branch Chicago River at Niles	125.8
– Little Calumet River at South Holland	190.2
Streamflow (estimated)	
– Grand Calumet River at Holman Ave.	23.0
MWRDGC Water Reclamation Facilities (measured)	
– Northside	437.3
– Stickney	1151.0
– Calumet	424.5
– Calumet TARP Pumpage to River	0.0
– Lemont	2.2
Other Point Sources (measured)	6.0
Summit Conduit (simulated)	8.4
Combined Sewer Overflows (simulated)	261.7
Direct Runoff to CSSC (simulated)	188.4
TOTAL INFLOWS (cfs)	3269.3

OUTFLOWS (cfs)	
Cal–Sag Flow Transferred to Calumet WRP as Steel Mill Blow–down	1.5
Lake Front Backflows	1.3
Argonne Laboratory	0.8
Uno–ven Corporation	6.2
USGS AVM Record	3748.5
TOTAL OUTFLOWS (cfs)	3758.3

DIFFERENCE (cfs)	–489.0

Based on the fact that the inflow is well correlated with the outflow, it appears that there is a moderately variable to constant underreported or unreported inflow. Possible sources of the canal system flow imbalance may include underreporting of the lakefront flows through the sluice gates and locks as well as unaccounted for flow sources. The underreporting of the lakefront flows could be the result of both inaccurate rating curves for the lakefront control structures and leakage through those structures. Flow meter measurements at the lakefront direct diversion points were done to assess if leakage is significant. This study (USGS, 1994) showed that lakefront leakage flows are greatly underreported. Unaccounted flows could also include unreported discharges to the canal.

AREAS FOR IMPROVEMENT IN THE DIVERSION ACCOUNTING PROCEDURES

As a result of reviewing and calculating the WY90 diversion accounting, and referencing the results of the WY84 through WY89 diversion accounting, a number of areas of potential improvement have become evident. The following paragraphs discuss those areas where improvement is needed.

CANAL SYSTEM BALANCE

As discussed previously, the canal system balance indicated that the total inflows were 13.0% less than the outflows. Flow meter measurements at the three lakefront direct diversion points were conducted between April and October 1993 by the USGS. The three lakefront diversion points that were measured are the Chicago River Controlling Works, O'Brien Lock and Dam, and the Wilmette Controlling Works. This study (USGS, 1994) included in the WY93 Annual Report indicated that measured leakage rates were significantly greater than the reported leakage. The underreported leakage accounts for a large portion of the flow imbalance determined in Budget 14. Additionally, sluice gate flows were also measured. Those measurements indicated that sluice gate flows were slightly underreported. Additional leakage and sluice gate measurements over a wide range of head conditions and gate settings will be required to further verify the WY93 results as well as to accurately estimate the actual discrepancy between reported and measured flows.

Another potential contributing factor in the canal system imbalance is the possibility of unreported discharges to the CSSC and adjoining waterways. Reconnaissance missions will be made in the future to determine the existence of any unreported discharges to the canal system.

It must be noted that the canal system imbalance does not directly affect the diversion computation. Its purpose is to provide a means of verification of modeled and measured flows and to indicate potential problems associated with these flows.

TUNNEL AND RESERVOIR PLAN MODELS

The TARP models should be revised such that they are more representative of actual operating conditions, if possible. Due to the model instability it was necessary to simplify or modify the gate closing and pump operation parameters. Improvements in model stability will first be required before the models can better represent operating procedures. Even after that is accomplished, representation of "actual" operating procedures may be difficult due to the TARP system operation plan not being adhered to at all times, tendencies to pump at night, down times for various pumps, changes in pump ratings, implementation of forecasting algorithms, etc.

It must be remembered that the primary purpose of the TARP models is to accurately estimate deductible components of the diversion such as the Des Plaines River watershed runoff and groundwater infiltration through tunnel walls. Currently, the Mainstream system deduction is composed almost entirely of groundwater infiltration. Until the Des Plaines tunnel goes on-line only a small percentage of the TARP pumpages at the Stickney WRP will be composed of Des Plaines River watershed runoff.

On the other hand, the Calumet TARP model requires modifications in separately sewered areas due to existing uncertainties in sanitary sewer connections as well as separate sewer area boundary delineations. These issues are being investigated and should be resolved within the next few years. As previously discussed in the Budget 11 section, all Calumet separate sewered areas within the Des Plaines River or Lake Michigan watershed are modeled such that all flows, sanitary plus runoff, are tributary to the CSSC either as interceptor or TARP flows that are conveyed to the Calumet WRP or as overflows that discharge to watercourses that are modeled as being tributary to the CSSC. Therefore, all Des Plaines River watershed runoff entering sanitary sewers within the Calumet WRP service basin is included as deductible runoff contained in Column 6, whether or not the overflows are captured by TARP or not.

It is important that low flows or dry weather flows are modeled accurately so that groundwater infiltration into the two TARP systems is properly modeled. This is important since these flows are substantial and since they constitute a deduction to the diversion. They are included in the flows of Column 4. In the Calumet TARP system there are also sanitary sewers that are connected to TARP. These sewers must be accounted for in the

modeling of groundwater infiltration since they contribute to the baseflow or dry weather flows of TARP. Currently, there is some uncertainty as to the connection of these separately sewered areas. This will be investigated in the near future.

There are several other areas where the TARP models can be improved. First, modeling of dry weather flow can be improved to more accurately simulate MWRDGC operational procedures which include less frequent pumping and pumping during the night. Second, the incorporation of a pseudo-forecasting algorithm would allow the model to simulate MWRDGC dewatering procedures prior to a storm. Third, dynamic constituent (I-I versus sanitary versus groundwater) tracking can be incorporated to allow more accurate determination of the deductible components of TARP flow. Fourth, the inclusion of an algorithm to operate gaged dropshafts based on average water surface elevation in a tunnel reach would provide better simulation of "actual" operations.

Additionally, better estimates of the simulated groundwater infiltration rates for the Mainstream TARP model are necessary to better match the simulated to the recorded dry-weather flows. Procedures for accomplishing this are similar to those used for improving the simulated groundwater infiltration rates for WY89 Calumet TARP as discussed in the WY89 Accounting Report that is contained in the Lake Michigan Diversion Accounting Annual Report for WY90-92 (USACE, 1994).

UNGAGED WATERSHED MODELING

The two ungaged watersheds, the Lower Des Plaines watershed and Calumet/Cal-sag basin of the Lake Michigan watershed, are areas where runoff is not accounted for in stream gage budgets or fully accounted for in water reclamation plant budgets. The runoff generated from these areas flows to the CSSC through direct surface runoff, through storm sewers that discharge to adjoining waterways that are tributary to the CSSC, or through effluent discharge from water reclamation plants. These two areas are modeled individually by multiplying areas of land cover types by unit runoff output generated by the HSPF hydrologic models. The totals of these areas equals the actual total of a particular ungaged watershed area less any combined sewer areas that are already accounted for in water reclamation plant budgets. There is however significant portions of both ungaged watersheds that contain separately sewered areas that flow to water reclamation plants, primarily to the Calumet WRP and to a lesser extent to the Stickney and Lemont WRPs. SCALP models these sanitary sewers as collecting 5% of the impervious runoff ,inflow, and 100% of the subsurface runoff, infiltration, as suggested by Christopher B. Burke Engineering (Burke, 1990). These sanitary sewers either convey this flow to the treatment facilities or they overflow to TARP or to adjoining rivers. The

problem arises from the fact that the areas used in SCALP for separately sewered areas are effective and not actual areas. To complicate matters further, a small portion of the separately sewered areas appears to fall in neither of the two ungaged watersheds, but instead within a separate watershed that is not tributary to the canal. Because of the unavailability of separate sewer boundary delineations it is impossible to know with any certainty the watershed from which those sanitary sewer inflows and infiltration flows originated. Consequently, modeling of the two ungaged watersheds (one that is part of the Des Plaines River watershed and one that is part of the diverted Lake Michigan watershed) includes those separately sewered areas that are already accounted for in the modeling of the WRPs. Since a portion of the simulated runoff for the ungaged watersheds is modeled as entering the sanitary sewers, a double accounting of flows exists. This results in a slight overestimation of the deductible Des Plaines River watershed runoff of Column 6, the estimated Lake Michigan watershed runoff of Column 12, and the inflows to the canal system balance of Budget 14.

In order to correct for the double accounting of runoff in the ungaged watersheds, a separate study is necessary to isolate those separately sewered areas that are presently modeled in SCALP. Through this study sewer boundaries would be developed for those areas and effective areas would be replaced with actual areas in the model. Then a revision of the estimated percent of HSPF unit runoffs from impervious and grassland areas that enter the sanitary sewers would need to be revised in the Lands block of the SCALP input decks. Finally, the actual separately sewered areas would be removed from the models that simulate runoff to the CSSC from the two ungaged watersheds.

MWRDGC UPPER DES PLAINES PUMP STATION

A review of the Upper Des Plaines pump station and its flow record indicates that the flow at the pump station is suspect and subject to operator error. Better flow measurement is needed at the pump station. With better flow measurement, this will become the most important balances for calibrating and verifying the simulation models of the Des Plaines watershed. In the diversion calculation, the primary use of the models is to calculate the deduction for runoff from the Des Plaines watershed discharged to the canal. Runoff from the Des Plaines watershed is deductible. The Upper Des Plaines Pump Station is the only point at which a model of inflow-infiltration can be calibrated and extrapolated to the remaining Des Plaines River watershed. Installation of better flow measurement equipment at the pump station would facilitate model calibration. Plans are currently underway with the USGS for implementation of a new measurement system during WY95.

O'HARE AND EGAN BASIN FLOW TRANSFER

A portion of the flows originating in the O'Hare and Egan Water Reclamation Plants' (WRP) service basins are transferred east to the Northside WRP. The extent of the O'Hare service area being diverted is not known and the diverted flow is not measured. Thus an estimate of the annual basin transfer is provided by MWRDGC. The total O'Hare-Egan flow transfer for WY90 was estimated by the MWRDGC to be 31.0 cfs (20 MGD).

This transfer is significant to diversion accounting in light of the fact that the O'Hare and Egan facilities discharge outside of the CSSC while the Northside facility discharges flows that reach the CSSC. Therefore, this transfer contains two components that are deductions to the flow measured in the CSSC. The two deductible components are groundwater pumpage contained in the sanitary portion of the transfer, and diverted Des Plaines River watershed runoff. These two deductible components are contained in Columns 4 and 6, respectively.

To determine the two deductible components requires an estimate of the sanitary and runoff portions of the flow transfer. Presently the sanitary and runoff portions of the flow transfer are estimated using the same constituent (sanitary, inflow, and infiltration) proportions simulated for the Upper Des Plaines Pump Station by SCALP (Special Contributing Area Program). Additionally, estimates must be made of the groundwater and Lake Michigan water components contained in the sanitary portion of the transfer. For WY90 it was estimated that the water supply for the O'Hare and Egan service basins was composed of 7.8 percent groundwater (1.6 cfs) and 92.2 percent Lake Michigan Water (18.5 cfs). The diverted Des Plaines River watershed runoff was estimated at 10.9 cfs.

For future accounting, simply measuring the basin transfer will not provide any information on the component makeup of the transfer. Thus, a review of the complex hydraulics and hydrology is necessary to determine the best procedure for estimating these flows. Several alternatives, including flow measurement and modeling are under consideration at this time. A more detailed discussion of the O'Hare and Egan basin transfer can be found in the Lake Michigan Diversion Accounting WY86 Report that is contained in the Lake Michigan Diversion Accounting WY90-92 Annual Report (USACE, 1994).

GRAND CALUMET RIVER

The flow in the Grand Calumet River drains both to the Lake Michigan via Indiana Harbor and to the Calumet Sag Channel which is tributary to the CSSC. When lake levels are high a larger portion of the flow drains to the Calumet Sag Channel. The Grand Calumet River flow calculation is currently based on a regression equation relating Lake Michigan stages and measured flows in Hart Ditch to the Grand Calumet River flow. Through current meter measurements by MWRDGC and other agencies it was determined that the accuracy of these regression equations are in question.

The Grand Calumet River flow to Illinois is important to diversion accounting because the majority of the flow in the Grand Calumet River is water supply effluent. This is a deduction to the AVM gage record and is contained in Column 5, Water Supply Pumpage from Indiana Reaching the CSSC. The Indiana water supply deduction is equal to the total water supply pumpage discharged to the Grand Calumet River if the pumpage rate is less than the calculated river flow. The deduction is equal to the river flow if the pumpage rate is greater than the river flow since under these conditions it is assumed that the river flow is composed entirely of sanitary effluent.

This procedure is the only method currently available to calculate the Indiana deduction. A stream gage has been installed for WY91 in the West Branch of the Grand Calumet River to measure flow into Illinois. This should increase the accuracy of this computation significantly. The same computational procedure for separating stream flow into sanitary and runoff will be used with the Grand Calumet stream gage record.

SUMMARY

In compliance with the modified 1980 U.S. Supreme Court decree, the WY90 diversion was computed using the best engineering technology available to date.

Overall, the simulations that comprise a significant portion of the diversion accounting computations worked well. The two most significant budgets to the diversion accounting computations, Budget 7, Northside Water Reclamation Facility, and Budget 10, Stickney Water Reclamation Facility, performed very well. Together, Budgets 7 and 10 compute the majority of the deductible Des Plaines River watershed runoff. These budgets have simulated to recorded ratios of 0.94 and 1.07 and correlations of 0.89 and 0.86, respectively. Given the complexity of the hydrologic cycle in the heavily urbanized Chicago metropolitan area, and given the number of human and other factors that cannot be adequately represented in numerical

modeling procedures, the results of these two budgets are very good. Additionally, results for Budget 12, the Calumet WRP, were also very good. This budget also models a portion of the deductible Des Plaines River watershed runoff. The S/R ratio was 1.00 while the coefficient of correlation was 0.89. Areas of improvement previously outlined will be considered in order to improve the accuracy of the diversion computation.

The WY90 diversion accountable to the State of Illinois is 3,531.2 cfs. This is 331.2 cfs greater than the 3,200 cfs average specified by the Decree. The 40 year running average beginning with WY81 and rounded to the nearest cfs is 3,452 cfs, and the cumulative deviation from the 3,200 cfs average is -2,520 cfs-years. The negative cumulative deviation indicates a water allocation deficit and the maximum allowable deficit is 2,000 cfs-years.

REFERENCES

1. Barkau, Robert L. 1991. Modeling of the Chicago Tunnel and Canal System. Prepared for Christopher B. Burke Engineering LTD as part of reference 2.

2. Burke, Christopher B. 1991. Data Collection and Model Revisions. Prepared for the U.S. Army Corps of Engineers, Chicago.

3. Burke, Christopher B. 1990. Infiltration and Inflow Study and Diversion Accounting Model Modification. Prepared for the U.S. Army Corps of Engineers, Chicago.

4. Espey, William H., Harry H. Barnes, and Svein Vigander. 1981. Lake Michigan Diversion Findings of the Technical Committee for Review of Diversion Flow Measurements and Accounting Procedures. Prepared for the U.S. Army Corps of Engineers, Chicago.

5. Espey, William H., Harry H. Barnes, and David Westfall. 1987. Lake Michigan Diversion Findings of the Second Technical Committee for Review of Diversion Flow Measurements and Accounting Procedures. Prepared for the U.S. Army Corps of Engineers, Chicago.

6. Hart, Dale E., and Richard G. McGee. 1985. Final Report - Lockport Power Plant Sluice Gate and Control Works Discharge Evaluation. Waterways Experiment Station. Vicksburg, MS.

7. Illinois State Water Survey. 1991, Installation and Operation of a Dense Raingage Network to Improve Precipitation Measurements for Lake Michigan Diversion Accounting: Water Year 1990.

8. International Joint Commission. 1981. Great Lakes Diversions and Consumptive Uses, Annex F, Consumptive Use.

9. Keifer Engineering. 1982. Input Data CRSM for Existing Conditions - Mainstream System. Prepared for the U.S. Army Corps of Engineers, Chicago.

10. Kleinbaum, David G., and Lawrence L Kupper. 1978. Applied Regression Analysis and Other Multivariable Methods. Wadsworth Publishing Company.

11. Metropolitan Water Reclamation District of Greater Chicago. 1984. 1984 Facility Planning Study - MSDGC Update Supplement and Summary.

12. Metropolitan Water Reclamation District of Greater Chicago. 1989. 1988 Annual Report of the Maintenance and Operations Department.

13. Neubauer, Ronald A. 1990. Request for TARP Information from the Army Corps of Engineers. Memorandum to Mr. William Eyre, Supervising Civil Engineer, Metropolitan Water Reclamation District of Greater Chicago.

14. Northeastern Illinois Planning Commission. 1985. Lake Michigan Diversion Accounting Manual of Procedures.

15. Kleinbaum, David G., and Lawrence L Kupper. 1978. Applied Regression Analysis and Other Multivariable Methods. Wadsworth Publishing Company.

16. Rust Environment & Infrastructure. 1993. Diversion Accounting Update for the New 25-Gage Precipitation Network.

17. Steel, Robert G. D., and James H. Torrie. 1980. Principles and Procedures of Statistics - A Biometrical Approach. McGraw-Hill, Inc.

18. U.S. Army Corps of Engineers. 1990. Lake Michigan Diversion Accounting 1989 Annual Report Including WY84 and WY85 Accounting.

19. U.S. Army Corps of Engineers. 1990. Current Meter Measurements of Discretionary and Leakage Flows at the Chicago River Controlling Works, O'Brien Lock and Dam, and the Wilmette Controlling Works.

20. U.S. Army Corps of Engineers. 1994. Lake Michigan Diversion Accounting Annual Report, Water Years 1990-92.

21. U.S. Army Corps of Engineers. 1994. Lake Michigan Diversion Accounting Water Year 1986 Report.

22. U.S. Army Corps of Engineers. 1994. Lake Michigan Diversion Accounting Water Year 1987 Report.

23. U.S. Army Corps of Engineers. 1994. Lake Michigan Diversion Accounting Water Year 1988 Report.

24. U.S. Army Corps of Engineers. 1994. Lake Michigan Diversion Accounting Water Year 1989 Report.

25. U.S. Geological Survey. 1984. Streamflow and Water Quality of the Grand Calumet River, Lake County, Indiana, and Cook County, Illinois.

26. U.S. Geological Survey. 1991. Water Resources Data, Illinois, Water Year 1990, Volume 2, Illinois River Basin.

27. U.S. Geological Survey. 1994. Comparison, Analysis, and Estimation of Discharge Data from Two Acoustic Velocity Meters on the Chicago Sanitary Ship Canal at Romeoville, Illinois.

28. U.S. Geological Survey. 1994. Measurements of Leakage through Chicago River Controlling Works and Other Control Structures Near Chicago Illinois.

29. Wisconsin et al., v. Illinois et al., Michigan v. Illinois et al. New York v. Illinois et al. U.S. 2, 3, and 4, Original 1 - 18, 1980.

Lake Michigan Diversion Accounting

Water Year 1990 Report

Appendix A

Summary of Daily Diversion Flows

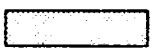
COLUMN COMPUTATIONS:

1. COLUMN 3 = COLUMN 1 + COLUMN 2
2. COLUMN 8 = COLUMN 4 + COLUMN 5 + COLUMN 6 + COLUMN 7
3. COLUMN 10 = COLUMN 3 – COLUMN 8 + COLUMN 9

NOTES:

1. ALL VALUES ARE ROUNDED TO THE NEAREST TENTH.
2. MATHEMATICAL COMPUTATIONS BETWEEN COLUMNS UTILIZE UNROUNDED VALUES.

LEGEND:

 DEDUCTIONS FROM THE ROMEOVILLE GAGE RECORD

 ADDITIONS TO THE ROMEOVILLE GAGE RECORD

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Oct-89	2976	0.6	2976.6	46.4	28.1	51.3	1.9	127.7	104.5	2953.3	1667.5	111.5	648.0
02-Oct-89	2600	0.5	2600.5	43.9	27.9	36.5	1.9	110.2	106.1	2596.4	1845.4	87.0	581.0
03-Oct-89	3003	0.5	3003.5	43.9	27.8	35.3	1.9	108.9	103.1	2997.7	1769.1	85.4	532.0
04-Oct-89	2725	0.4	2725.4	106.7	27.9	37.1	1.9	173.6	104.8	2656.4	1772.2	137.0	530.0
05-Oct-89	3007	0.4	3007.4	46.1	28.3	74.4	2.0	150.8	103.8	2960.1	1745.1	267.9	371.0
06-Oct-89	3161	0.3	3161.3	126.0	28.1	36.8	2.0	192.9	103.1	3071.6	1718.3	172.6	555.0
07-Oct-89	2508	0.4	2508.4	43.9	28.2	32.9	1.9	106.9	102.8	2504.2	1682.6	83.9	577.0
08-Oct-89	2812	0.5	2812.5	43.9	27.9	31.8	1.9	105.5	99.8	2806.7	1647.5	79.6	618.0
09-Oct-89	2799	0.4	2799.4	89.2	27.8	33.3	1.9	152.1	101.9	2749.2	1721.0	109.3	514.0
10-Oct-89	2712	0.5	2712.5	79.3	27.9	31.1	1.9	140.1	101.6	2674.0	1743.8	109.5	575.0
11-Oct-89	2534	0.5	2534.5	46.5	27.8	30.3	1.9	106.4	103.2	2531.3	1785.2	69.6	473.0
12-Oct-89	2456	0.6	2456.6	79.4	27.9	30.3	1.9	139.5	103.9	2421.0	1828.5	87.2	514.0
13-Oct-89	2866	0.5	2866.5	82.3	28.0	26.7	1.9	140.8	102.2	2828.0	1828.7	85.8	548.0
14-Oct-89	2720	0.5	2720.5	43.9	28.0	26.8	1.9	100.6	101.8	2721.4	1832.7	55.9	614.0
15-Oct-89	2741	0.5	2741.5	90.8	28.1	29.6	1.9	149.3	100.2	2692.4	1788.3	94.5	577.0
16-Oct-89	2639	0.9	2639.9	82.2	27.8	27.7	1.9	139.6	103.9	2604.2	1746.8	88.1	255.0
17-Oct-89	2249	0.4	2249.4	51.3	27.8	49.9	1.9	130.9	105.0	2223.6	1703.2	333.5	99.0
18-Oct-89	2570	0.3	2570.3	155.9	27.8	62.4	1.9	248.0	102.2	2424.5	1697.8	386.2	84.0
19-Oct-89	2955	0.3	2955.3	92.4	27.8	581.1	1.9	703.1	114.2	2366.4	1699.1	2401.2	76.0
20-Oct-89	4923	0.4	4923.4	280.9	28.0	544.5	1.9	855.3	112.8	4180.8	1683.6	2643.7	314.0
21-Oct-89	3685	0.6	3685.6	289.5	27.8	60.9	1.9	380.1	100.4	3405.9	1648.8	555.8	585.0
22-Oct-89	3613	0.5	3613.5	46.6	27.7	45.8	1.9	122.0	100.3	3591.9	1641.7	223.5	474.0
23-Oct-89	2720	0.7	2720.7	43.9	27.8	40.2	1.9	113.8	103.8	2710.7	1717.6	170.2	492.0
24-Oct-89	2975	0.9	2975.9	102.7	27.9	40.3	1.9	172.8	103.5	2906.5	1766.1	192.3	506.0
25-Oct-89	2800	0.9	2800.9	61.0	27.8	36.0	1.9	126.7	103.4	2777.6	1754.8	144.4	511.0
26-Oct-89	2740	0.7	2740.7	43.9	27.8	34.0	1.9	107.6	101.6	2734.7	1734.8	112.3	527.0
27-Oct-89	2665	0.6	2665.6	46.5	27.8	33.9	1.9	110.1	102.2	2657.8	1748.4	99.6	507.0
28-Oct-89	2490	0.7	2490.7	124.7	28.0	35.0	1.9	189.6	102.6	2403.7	1732.1	159.7	495.0
29-Oct-89	2934	0.6	2934.6	43.9	27.9	30.5	1.9	104.2	101.5	2932.0	1739.1	89.6	475.0
30-Oct-89	2677	0.8	2677.8	89.4	27.9	32.1	1.9	151.3	103.1	2629.6	1725.2	121.4	346.0
31-Oct-89	2296	0.5	2296.5	81.7	27.7	31.1	1.9	142.3	102.6	2256.7	1694.0	117.4	349.0
OCT 89	2856.5	0.5	2857.0	85.4	27.9	72.0	1.9	187.2	109.4	2773.2	1742.2	305.7	462.0

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Nov-89	2359	0.6	2359.6	43.9	27.4	26.3	1.9	101.4	99.7	2357.9	1674.7	106.0	282.0
02-Nov-89	2424	0.6	2424.6	92.6	27.3	29.2	2.0	151.0	97.9	2371.5	1685.9	130.1	65.0
03-Nov-89	2001	0.6	2001.6	62.5	27.5	27.7	2.0	119.6	97.7	1979.8	1667.7	86.3	70.0
04-Nov-89	1812	0.4	1812.4	46.4	27.4	27.7	2.0	103.4	95.7	1804.8	1637.1	69.5	66.0
05-Nov-89	2079	0.4	2079.4	95.7	27.4	35.1	2.0	160.2	95.9	2015.2	1639.9	170.8	68.0
06-Nov-89	2332	0.6	2332.8	79.8	27.4	26.9	2.0	136.0	97.8	2294.6	1689.0	135.1	57.0
07-Nov-89	3105	0.4	3105.4	190.7	27.4	108.4	2.0	318.3	99.7	2886.7	1700.1	599.9	68.0
08-Nov-89	2064	0.9	2064.9	46.3	27.4	28.9	2.0	104.5	97.0	2057.4	1678.3	140.9	64.0
09-Nov-89	2350	0.6	2350.6	46.2	27.4	27.4	1.9	102.9	96.2	2344.1	1660.7	114.4	56.0
10-Nov-89	1968	0.7	1968.7	61.7	27.4	25.9	2.0	117.0	96.8	1950.5	1684.4	100.6	59.0
11-Nov-89	2158	0.6	2158.6	102.9	27.4	26.5	2.0	158.7	95.0	2094.9	1644.0	119.0	52.0
12-Nov-89	1916	0.6	1916.6	46.3	27.4	25.2	2.0	101.0	94.2	1909.9	1621.8	75.2	68.0
13-Nov-89	1991	0.7	1991.7	125.1	27.3	26.1	2.0	180.5	97.9	1909.2	1708.0	131.4	60.0
14-Nov-89	3068	0.8	3068.8	57.2	27.4	735.2	1.9	821.7	102.6	2349.7	1677.2	2113.5	70.0
15-Nov-89	7064	0.7	7064.7	165.7	27.3	1072.2	2.0	1267.1	109.7	5906.3	1675.7	5113.8	68.0
16-Nov-89	5160	0.5	5160.5	390.2	27.4	119.2	2.0	538.7	97.0	4718.8	1681.3	1410.9	42.0
17-Nov-89	3400	0.8	3400.7	64.0	27.4	68.2	2.0	161.5	96.9	3336.1	1656.4	655.4	44.0
18-Nov-89	3447	0.9	3447.9	46.5	27.4	53.2	2.0	128.9	94.8	3413.5	1650.7	375.2	56.0
19-Nov-89	3002	0.5	3002.5	62.3	27.3	54.6	2.0	146.1	94.8	2951.3	1647.7	303.1	42.0
20-Nov-89	2660	0.6	2660.6	101.1	27.5	38.8	2.0	169.4	98.2	2587.5	1685.3	270.5	52.0
21-Nov-89	2350	0.6	2350.6	46.5	27.4	34.5	2.0	110.3	96.5	2336.9	1680.2	210.2	56.0
22-Nov-89	2340	0.6	2340.6	90.2	27.4	33.0	1.9	152.6	95.9	2283.8	1668.0	201.5	55.0
23-Nov-89	2626	0.5	2626.5	83.7	27.6	32.6	2.0	146.1	92.3	2572.8	1624.4	191.8	45.0
24-Nov-89	2328	0.6	2328.6	46.5	27.2	39.1	2.0	114.8	91.0	2304.8	1597.1	203.8	42.0
25-Nov-89	2031	0.5	2031.5	92.9	27.4	35.9	1.9	158.1	93.9	1967.3	1592.8	277.0	45.0
26-Nov-89	1928	0.6	1928.6	83.1	27.4	31.1	1.9	143.6	92.6	1877.6	1619.8	177.0	52.0
27-Nov-89	3276	0.4	3276.4	130.6	27.5	67.9	2.0	227.9	98.2	3146.6	1653.5	617.2	51.0
28-Nov-89	2625	0.6	2625.6	141.2	27.3	35.4	2.0	205.9	95.2	2514.9	1673.5	253.2	64.0
29-Nov-89	1803	0.9	1803.9	43.9	27.3	29.5	2.0	102.6	94.0	1795.2	1676.5	145.6	40.0
30-Nov-89	1963	0.8	1963.8	77.3	27.4	29.3	1.9	136.0	95.6	1923.4	1669.9	154.9	27.0
NOV 89	2654.3	0.6	2655.0	91.8	27.4	98.4	2.0	219.5	96.7	2532.1	1661.4	488.5	62.9

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Dec-89	2312	0.7	2312.7	85.4	27.5	29.9	1.9	144.7	99.9	2267.9	1657.8	159.4	40.0
02-Dec-89	1972	0.5	1972.5	46.1	27.6	26.0	1.8	103.5	99.0	1968.1	1639.2	126.1	57.0
03-Dec-89	2070	0.4	2070.4	132.0	27.6	30.1	1.8	191.5	98.2	1975.1	1622.5	176.7	46.0
04-Dec-89	2098	0.6	2098.6	46.0	27.6	27.1	1.8	102.6	99.9	2095.9	1675.5	109.2	38.0
05-Dec-89	2259	0.9	2259.9	46.1	27.5	26.8	1.9	102.2	99.7	2257.4	1675.8	107.2	48.0
06-Dec-89	2286	0.7	2286.7	79.0	27.5	27.5	1.8	135.8	101.0	2251.9	1663.7	127.5	65.0
07-Dec-89	1977	0.7	1977.7	84.6	27.6	27.4	1.8	141.4	99.8	1936.1	1641.9	125.6	49.0
08-Dec-89	1829	0.6	1829.6	43.9	27.6	24.8	1.8	98.2	99.2	1830.7	1644.6	88.8	44.0
09-Dec-89	2208	0.5	2208.5	43.9	27.5	24.5	1.8	97.8	98.6	2209.3	1636.9	86.2	30.0
10-Dec-89	2244	0.5	2244.5	134.3	27.5	28.1	1.8	191.7	98.2	2151.0	1607.1	162.1	40.0
11-Dec-89	1762	0.8	1762.8	43.9	27.5	24.0	1.8	97.2	99.6	1765.2	1648.5	87.0	47.0
12-Dec-89	2052	0.6	2052.6	46.3	27.7	24.8	1.8	100.6	99.7	2048.7	1665.7	91.2	42.0
13-Dec-89	2367	0.5	2367.5	122.4	27.5	26.7	1.8	178.5	101.1	2290.2	1664.7	136.3	52.0
14-Dec-89	2295	0.8	2295.7	46.1	27.5	24.2	1.8	99.6	98.4	2294.6	1656.9	77.5	52.0
15-Dec-89	2168	0.8	2168.8	46.1	27.8	23.9	1.8	99.7	101.0	2170.1	1669.7	74.0	70.0
16-Dec-89	2338	0.7	2338.6	79.4	27.6	24.8	1.8	133.6	99.6	2304.6	1655.3	102.9	45.0
17-Dec-89	2342	0.6	2342.6	81.7	27.5	23.5	1.9	134.5	97.8	2305.8	1667.4	98.0	36.0
18-Dec-89	2349	0.8	2349.8	46.5	27.5	23.3	1.8	99.1	101.6	2352.5	1699.7	70.7	49.0
19-Dec-89	1902	0.9	1902.8	92.6	27.6	24.2	1.8	146.3	101.7	1858.3	1682.2	104.9	52.0
20-Dec-89	2053	0.7	2053.7	83.6	27.6	23.6	1.9	136.7	100.6	2017.7	1703.3	95.1	43.0
21-Dec-89	2323	0.6	2323.6	46.2	27.5	22.3	1.8	97.8	100.8	2326.7	1716.5	64.9	44.0
22-Dec-89	2653	0.5	2653.5	64.7	27.5	22.6	1.8	116.7	100.1	2636.9	1750.7	78.2	67.0
23-Dec-89	2300	0.5	2300.5	104.2	27.7	22.5	1.8	156.3	99.8	2243.9	1720.1	105.8	62.0
24-Dec-89	1935	0.5	1935.5	46.5	27.5	21.7	1.8	97.5	97.9	1935.8	1715.9	61.8	43.0
25-Dec-89	2231	0.5	2231.5	105.1	27.6	23.0	1.8	157.5	99.1	2173.1	1715.4	106.1	43.0
26-Dec-89	2403	0.6	2403.6	63.7	27.7	21.5	1.8	114.8	102.8	2391.5	1771.9	71.0	56.0
27-Dec-89	2393	0.5	2393.5	48.2	27.6	20.8	1.9	98.4	99.4	2396.5	1783.5	55.5	50.0
28-Dec-89	2356	0.6	2356.6	64.3	27.7	21.3	1.8	115.1	101.0	2342.6	1780.7	70.6	49.0
29-Dec-89	2531	0.5	2531.5	151.6	27.6	87.5	1.8	268.5	104.4	2367.3	1761.3	544.7	50.0
30-Dec-89	2529	0.5	2529.5	155.1	27.7	56.1	1.8	240.8	101.8	2390.5	1731.1	645.2	55.0
31-Dec-89	2631	0.5	2631.5	79.9	27.6	52.3	1.8	161.9	98.8	2568.4	1718.2	186.9	48.0
DEC 89	2231.2	0.6	2231.8	76.0	27.6	26.7	1.8	134.1	99.9	2197.6	1687.9	135.5	48.8

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Jan-90	2077	0.7	2077.7	46.5	27.3	42.1	2.1	118.0	92.5	2052.2	1697.2	146.1	41.0
02-Jan-90	1973	0.6	1973.6	141.1	27.5	172.9	2.1	343.6	99.0	1729.0	1756.9	420.2	38.0
03-Jan-90	2172	0.7	2172.6	66.0	27.3	178.0	2.1	273.3	96.3	1995.6	1761.0	771.0	40.0
04-Jan-90	3861	0.9	3861.9	236.6	27.3	444.6	2.1	710.6	101.4	3252.7	1765.1	2113.2	46.0
05-Jan-90	2686	0.9	2686.9	106.9	27.4	93.1	2.1	231.5	93.9	2549.3	1751.3	491.5	46.0
06-Jan-90	2322	0.8	2322.7	46.1	27.3	70.9	2.1	146.4	87.4	2263.8	1693.8	351.3	39.0
07-Jan-90	2954	0.9	2954.9	133.9	27.3	171.4	2.1	334.6	98.5	2716.8	1686.6	544.8	40.0
08-Jan-90	2253	0.5	2253.5	63.0	27.3	87.9	2.1	180.3	95.6	2166.2	1738.3	314.5	43.0
09-Jan-90	2642	0.7	2642.7	50.9	27.4	84.6	2.1	165.1	89.0	2576.6	1723.6	304.1	51.0
10-Jan-90	2012	0.5	2012.5	126.0	27.2	40.3	2.1	195.6	96.5	1913.5	1718.8	255.7	52.0
11-Jan-90	2182	0.5	2182.5	46.3	27.5	31.0	2.1	106.9	93.2	2170.9	1725.7	171.1	35.0
12-Jan-90	2143	0.7	2143.7	78.5	27.5	27.7	2.1	135.8	95.0	2102.9	1702.1	170.4	55.0
13-Jan-90	2159	0.5	2159.5	82.6	27.3	26.2	2.1	138.2	92.6	2113.9	1683.2	149.2	52.0
14-Jan-90	2067	0.6	2067.6	46.5	27.3	25.5	2.1	101.4	93.0	2059.2	1660.7	116.3	51.0
15-Jan-90	1938	1.0	1939.0	115.7	27.3	27.1	2.1	172.2	97.0	1863.7	1718.3	163.5	54.0
16-Jan-90	3148	0.6	3148.6	65.6	27.3	40.8	2.1	135.9	95.9	3108.6	1711.4	416.4	49.0
17-Jan-90	3939	0.9	3939.9	207.7	27.2	815.2	2.1	852.2	96.6	3165.7	1659.2	2552.8	239.0
18-Jan-90	3180	0.6	3180.6	142.5	27.3	93.3	2.1	265.2	96.1	3011.5	1686.8	826.6	45.0
19-Jan-90	2946	0.7	2946.7	48.4	27.4	61.1	2.2	139.1	94.3	2902.0	1677.8	458.2	42.0
20-Jan-90	2684	0.6	2684.6	150.1	27.2	231.4	2.1	410.8	95.8	2369.6	1654.1	876.2	54.0
21-Jan-90	2784	0.7	2784.7	45.7	27.3	89.1	1.9	164.0	93.7	2714.4	1644.0	514.5	48.0
22-Jan-90	2394	0.8	2394.8	46.1	27.4	47.5	2.1	123.1	94.5	2366.3	1672.1	367.5	40.0
23-Jan-90	2538	0.8	2538.8	129.0	27.4	44.1	2.1	202.5	95.3	2431.5	1665.5	334.2	43.0
24-Jan-90	2367	0.8	2367.8	46.1	27.2	37.4	2.1	112.9	94.5	2369.4	1663.9	217.1	60.0
25-Jan-90	6478	0.6	6478.6	200.5	27.3	914.4	2.1	1144.4	107.5	5441.8	1654.2	5734.1	46.0
26-Jan-90	5150	0.7	5150.7	365.7	27.2	433.3	2.1	828.4	97.2	4419.5	1663.7	2276.4	43.0
27-Jan-90	4322	0.7	4322.7	217.3	27.3	868.7	2.1	1135.4	101.4	3288.6	1644.2	2226.6	37.0
28-Jan-90	3537	0.5	3537.4	172.6	27.3	272.4	2.1	474.6	93.4	3156.3	1623.3	960.9	41.0
29-Jan-90	3227	0.9	3227.9	48.9	27.3	171.1	2.1	249.4	96.9	3075.4	1645.3	600.8	39.0
30-Jan-90	2536	0.7	2536.7	46.5	27.3	116.5	2.1	196.4	94.5	2434.8	1658.8	448.8	48.0
31-Jan-90	2693	0.8	2693.7	129.0	27.3	90.5	2.1	248.9	94.8	2539.6	1658.5	410.1	44.0
JAN 90	2883.4	0.7	2884.0	111.4	27.3	183.0	2.1	323.8	95.9	2656.2	1689.2	829.2	51.6

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Feb-90	3176	0.7	3176.7	67.7	27.7	377.6	1.9	474.9	96.8	2900.7	1649.4	1432.6	52.0
02-Feb-90	3292	0.7	3292.7	216.2	27.6	181.5	1.4	426.8	95.0	2960.9	1652.7	1272.0	59.0
03-Feb-90	3525	0.6	3525.6	49.6	27.8	233.7	1.4	312.6	94.9	3307.9	1622.2	972.3	49.0
04-Feb-90	2737	0.5	2737.5	98.3	27.6	128.7	1.4	255.9	94.5	2576.1	1610.8	741.8	46.0
05-Feb-90	2795	0.9	2795.9	48.5	27.7	94.3	1.4	171.9	95.3	2719.3	1662.6	486.5	36.0
06-Feb-90	3065	0.6	3065.6	125.2	27.6	82.4	1.4	236.7	95.5	2924.4	1663.3	457.7	46.0
07-Feb-90	2497	0.8	2497.8	45.8	27.7	72.8	1.4	147.7	94.9	2445.0	1652.5	365.6	52.0
08-Feb-90	2658	0.8	2658.8	45.9	27.7	69.1	1.4	144.2	94.9	2609.5	1655.5	336.8	46.0
09-Feb-90	2436	1.1	2437.1	137.6	27.9	69.9	1.4	236.8	96.6	2297.2	1640.1	397.1	46.0
10-Feb-90	2310	0.6	2310.6	45.8	27.7	65.7	1.4	140.7	93.9	2263.8	1623.2	305.9	47.0
11-Feb-90	2277	0.5	2277.5	45.6	27.6	64.8	1.4	139.9	92.8	2230.5	1596.3	280.3	44.0
12-Feb-90	2347	0.6	2347.6	60.5	27.6	64.4	1.4	154.0	94.2	2287.6	1670.3	270.7	40.0
13-Feb-90	2430	0.8	2430.8	98.8	27.7	64.7	1.9	193.2	96.1	2333.7	1641.5	290.8	68.0
14-Feb-90	2308	0.7	2308.7	101.0	27.6	100.2	1.9	230.7	95.3	2173.3	1628.3	1388.4	64.0
15-Feb-90	2610	0.8	2610.8	157.6	27.8	79.3	1.9	266.5	94.9	2439.2	1609.7	615.2	53.0
16-Feb-90	2630	0.6	2630.6	45.9	27.6	70.1	1.9	145.5	95.4	2780.5	1627.1	634.4	45.0
17-Feb-90	2488	0.6	2488.6	115.3	27.7	69.4	1.9	214.3	93.3	2367.5	1614.3	581.1	46.0
18-Feb-90	2771	0.6	2771.6	119.4	27.5	432.8	1.9	581.6	95.6	2285.5	1577.7	1288.2	36.0
19-Feb-90	2633	0.5	2633.5	157.1	27.7	323.1	1.9	509.8	96.9	2220.7	1646.3	1190.3	42.0
20-Feb-90	2495	0.7	2495.7	124.7	27.8	318.2	1.9	472.5	95.1	2122.4	1653.3	725.2	37.0
21-Feb-90	2534	0.7	2534.7	172.6	27.6	591.4	1.9	793.7	104.4	1845.4	1649.6	1252.8	35.0
22-Feb-90	7765	0.8	7765.8	160.0	27.7	2244.5	1.9	2434.0	117.9	5449.6	1627.0	9948.5	46.0
23-Feb-90	7516	0.6	7516.6	144.4	27.7	1252.3	2.0	1426.3	103.0	6193.2	1632.9	4315.0	24.0
24-Feb-90	6063	0.4	6063.4	182.7	27.7	549.4	1.9	761.7	96.7	5398.5	1605.8	2666.9	32.0
25-Feb-90	5085	0.7	5085.7	220.1	27.7	337.1	1.9	586.8	94.3	4593.3	1628.7	1496.8	36.0
26-Feb-90	3393	0.5	3393.5	360.7	27.7	240.5	1.9	630.7	98.1	2858.9	1641.9	1274.4	28.0
27-Feb-90	4086	0.7	4086.7	134.3	27.7	276.1	1.9	440.0	102.2	3749.1	1656.1	1240.6	33.0
28-Feb-90	3066	0.7	3066.7	108.7	27.7	159.0	1.9	297.3	95.3	2864.8	1635.8	892.2	39.0
FEB 90	3328.1	0.7	3328.8	121.1	27.7	307.6	1.7	458.1	97.1	2967.8	1634.8	1325.7	43.8

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Mar-90	3494	0.6	3494.6	91.3	27.5	133.0	1.8	253.6	96.3	3339.3	1658.4	733.3	35.0
02-Mar-90	3232	0.7	3232.7	93.0	27.5	125.8	1.8	248.1	99.1	3082.8	1659.3	776.7	33.0
03-Mar-90	3204	0.7	3204.7	48.5	27.5	113.7	1.8	191.5	95.0	3108.2	1637.1	718.6	44.0
04-Mar-90	2949	0.4	2949.4	144.2	27.3	105.9	1.8	279.3	94.4	2764.6	1601.0	676.7	43.0
05-Mar-90	3750	0.5	3750.5	95.1	27.6	336.6	1.8	461.1	102.1	3391.5	1633.6	1353.4	48.0
06-Mar-90	2873	0.8	2873.7	172.2	27.5	190.0	1.8	391.5	97.3	2579.5	1654.0	786.4	50.0
07-Mar-90	3266	0.5	3266.5	47.3	27.5	145.0	1.8	221.6	96.5	3141.4	1670.1	555.6	60.0
08-Mar-90	5705	0.8	5705.7	105.5	27.5	1268.9	1.8	1403.6	113.6	4415.7	1642.4	5327.2	46.0
09-Mar-90	7041	0.9	7041.9	186.6	27.5	566.9	1.8	782.7	99.1	6357.3	1659.2	3309.6	39.0
10-Mar-90	9647	0.5	9647.5	104.9	27.5	1069.3	1.8	1203.6	104.4	8548.3	1616.6	4945.4	128.0
11-Mar-90	7858	0.8	7858.8	136.2	27.6	922.9	1.3	1087.9	99.2	6869.0	1604.6	4817.0	216.0
12-Mar-90	6209	1.0	6210.0	198.2	27.5	457.2	2.2	685.0	97.6	5622.6	1694.4	2517.4	44.0
13-Mar-90	4565	1.2	4566.1	234.2	27.5	325.3	1.8	588.8	97.3	4074.7	1668.2	1791.9	59.0
14-Mar-90	4572	0.8	4572.8	213.1	27.5	249.3	1.8	491.7	97.1	4178.2	1651.4	1398.7	50.0
15-Mar-90	3439	0.7	3439.7	48.5	27.5	195.8	1.8	273.6	96.9	3263.0	1642.6	1017.3	52.0
16-Mar-90	3737	0.9	3737.9	49.1	27.4	170.3	1.8	248.6	96.0	3585.3	1654.3	820.3	40.0
17-Mar-90	3607	0.6	3607.6	47.3	27.5	153.2	1.8	229.9	95.4	3473.0	1630.0	697.8	45.0
18-Mar-90	3278	0.6	3278.6	126.0	27.4	145.0	1.8	300.2	94.3	3072.7	1597.8	673.3	43.0
19-Mar-90	3213	0.6	3213.6	45.4	27.4	133.5	1.9	208.2	96.1	3101.4	1639.6	525.0	53.0
20-Mar-90	2645	0.9	2645.9	47.6	27.5	126.2	1.8	205.3	97.1	2537.6	1651.6	463.7	47.0
21-Mar-90	2865	0.9	2865.9	135.4	27.3	125.3	1.8	289.8	97.4	2673.5	1651.4	488.6	49.0
22-Mar-90	4170	0.6	4170.6	120.7	27.5	626.5	1.8	776.6	102.8	3496.4	1629.3	2418.2	53.0
23-Mar-90	3243	0.7	3243.7	245.3	27.5	226.2	1.9	502.8	95.6	2836.5	1613.5	1046.2	55.0
24-Mar-90	3200	0.6	3200.6	48.9	27.4	168.9	1.8	247.0	95.0	3048.6	1621.7	616.0	56.0
25-Mar-90	3617	0.5	3617.5	47.4	27.6	139.3	1.8	216.1	95.4	3496.8	1601.0	472.9	44.0
26-Mar-90	2500	0.8	2500.8	45.5	27.3	121.8	1.8	196.4	96.1	2400.5	1630.8	393.1	55.0
27-Mar-90	2383	0.7	2383.7	127.5	27.5	115.2	1.8	272.0	96.1	2207.8	1634.9	403.4	68.0
28-Mar-90	2565	0.7	2565.7	43.6	27.5	104.9	1.9	179.8	97.3	2483.3	1635.9	319.6	58.0
29-Mar-90	3682	0.6	3682.6	132.5	27.5	425.3	1.8	587.1	104.5	3200.0	1617.4	1669.8	60.0
30-Mar-90	3141	0.6	3141.6	233.2	27.5	169.3	1.8	431.7	95.7	2805.6	1618.5	867.6	61.0
31-Mar-90	3320	0.6	3320.6	68.4	27.5	229.0	1.8	324.7	96.1	3092.0	1605.9	905.7	56.0
MAR 90	3966.8	0.7	3967.5	112.4	27.5	302.9	1.8	444.5	97.9	3620.9	1636.3	1404.8	57.7

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Apr-90	2912	0.7	2912.7	132.4	27.6	384.0	1.7	545.6	97.2	2464.3	1512.1	1499.9	66.0
02-Apr-90	3320	0.7	3320.7	253.9	27.6	193.7	1.7	476.9	94.1	2937.9	1606.3	894.2	63.0
03-Apr-90	2870	0.6	2870.6	47.2	27.5	140.9	1.7	217.2	98.8	2750.2	1644.1	535.0	77.0
04-Apr-90	2980	0.6	2980.6	45.4	27.6	116.1	1.7	190.7	96.0	2885.9	1626.0	476.0	69.0
05-Apr-90	2696	0.7	2696.7	47.6	27.5	104.0	1.7	181.0	96.7	2614.4	1637.7	407.8	67.0
06-Apr-90	2530	0.5	2530.5	126.9	27.5	97.6	1.7	253.7	97.2	2374.2	1632.4	423.0	60.0
07-Apr-90	2462	0.3	2462.3	48.1	27.5	91.1	1.7	168.4	95.0	2388.9	1597.7	324.3	94.0
08-Apr-90	2275	0.6	2275.6	45.7	27.5	86.0	1.7	160.9	94.8	2209.5	1585.1	290.5	90.0
09-Apr-90	2966	0.6	2966.6	96.7	27.5	233.2	1.7	359.1	97.0	2724.6	1626.3	780.3	62.0
10-Apr-90	3286	0.6	3286.6	185.8	27.5	192.4	1.7	407.4	97.9	2977.2	1633.8	971.1	65.0
11-Apr-90	2919	0.6	2919.6	45.4	27.5	108.6	1.7	183.1	98.8	2833.3	1632.5	461.9	63.0
12-Apr-90	2754	0.6	2754.6	47.6	27.5	94.2	1.7	171.2	96.0	2681.3	1640.0	390.3	72.0
13-Apr-90	2443	0.5	2443.5	46.5	27.4	318.5	1.6	396.1	100.8	2148.2	1598.8	1009.8	82.0
14-Apr-90	4054	0.6	4054.6	244.5	27.6	342.5	1.7	616.3	97.7	3536.0	1585.6	1586.5	83.0
15-Apr-90	3398	0.7	3398.7	109.8	27.6	148.1	1.7	287.2	94.8	3206.2	1575.2	753.2	90.0
16-Apr-90	2935	0.6	2935.6	45.4	27.5	110.2	1.7	184.8	96.4	2847.1	1616.5	514.8	71.0
17-Apr-90	2831	0.6	2831.6	45.7	27.5	92.7	1.7	167.6	96.6	2762.6	1633.1	423.0	59.0
18-Apr-90	2351	0.6	2351.6	115.3	27.5	84.6	1.7	229.3	96.9	2219.3	1645.6	412.1	75.0
19-Apr-90	2723	0.6	2723.6	46.1	27.6	76.5	1.7	151.9	96.6	2666.5	1652.3	320.0	71.0
20-Apr-90	3493	0.6	3493.6	75.0	27.6	422.7	1.7	527.0	99.6	3066.5	1633.5	1738.2	81.0
21-Apr-90	4058	0.4	4058.4	245.6	27.5	169.6	1.7	444.6	95.8	3709.7	1626.5	1205.4	132.0
22-Apr-90	3698	0.8	3698.8	47.2	27.6	110.4	1.9	187.1	95.7	3607.4	1706.9	855.2	137.0
23-Apr-90	2741	0.6	2741.6	45.6	27.7	88.1	1.7	163.0	97.4	2676.0	1782.4	478.8	98.0
24-Apr-90	3055	0.6	3055.6	45.8	27.8	78.0	1.7	151.4	99.8	3004.1	1842.8	378.2	130.0
25-Apr-90	2855	1.2	2856.2	114.6	27.7	71.6	1.7	215.6	99.2	2739.8	1829.1	549.4	100.0
26-Apr-90	3030	1.2	3031.2	45.8	27.8	63.6	1.7	138.9	100.1	2992.4	1835.0	318.0	103.0
27-Apr-90	2856	1.1	2857.1	75.0	27.8	60.3	1.6	164.8	99.4	2791.7	1738.7	302.3	110.0
28-Apr-90	2692	0.5	2692.4	97.8	27.7	57.6	1.7	185.0	97.5	2605.0	1696.0	284.5	134.0
29-Apr-90	2182	0.6	2182.6	45.9	27.9	53.6	1.7	129.1	94.4	2147.8	1727.3	220.5	143.0
30-Apr-90	2611	0.7	2611.7	43.9	27.7	50.3	1.7	123.6	98.6	2586.6	1748.5	204.2	85.0
APR 90	2933.2	0.6	2933.9	88.7	27.6	141.3	1.7	259.3	97.3	2771.9	1661.6	626.9	87.7

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-May-90	2206	0.6	2206.6	116.3	28.2	51.3	1.8	197.6	108.5	2115.5	1732.6	245.4	103.0
02-May-90	2325	0.5	2325.5	46.2	28.2	47.0	2.1	123.5	105.0	2307.0	1753.2	172.7	104.0
03-May-90	2649	0.6	2649.6	46.2	28.1	45.1	1.8	121.3	104.0	2632.3	1691.5	185.6	94.0
04-May-90	8411	0.5	8411.5	143.6	28.1	1498.5	2.0	1672.5	115.4	6854.4	1634.4	7593.0	95.0
05-May-90	4882	0.3	4882.3	273.9	28.1	258.1	2.0	562.2	101.6	4421.7	1640.2	1730.9	125.0
06-May-90	3920	0.5	3920.5	138.7	28.1	147.6	1.8	316.3	101.3	3705.5	1637.1	909.2	130.0
07-May-90	2831	0.7	2831.7	48.5	28.2	98.2	2.1	177.1	105.2	2759.8	1750.8	521.4	122.0
08-May-90	3262	0.8	3262.8	45.9	27.9	71.8	1.9	147.6	105.6	3220.7	1801.5	364.6	124.0
09-May-90	8755	0.5	8755.5	127.4	28.4	2217.8	1.9	2375.4	121.9	6502.0	1731.4	11658.0	329.4
10-May-90	17870	0.6	17871.0	207.6	28.5	1148.9	1.7	1386.6	117.2	16601.0	1661.3	6721.6	-17.6
11-May-90	14083	0.6	14084.0	138.3	27.9	526.7	2.0	694.8	106.0	13495.0	1675.0	3734.3	42.0
12-May-90	10787	0.4	10787.0	195.9	28.1	786.0	1.8	1011.9	106.9	9882.4	1615.3	3938.9	1272.0
13-May-90	8993	0.5	8993.5	125.4	28.2	770.8	2.0	926.4	104.7	8171.8	1618.3	4788.0	112.0
14-May-90	8539	0.9	8539.9	193.9	28.0	356.8	2.2	580.9	105.3	8064.3	1678.5	2981.4	115.0
15-May-90	7955	0.7	7955.6	245.6	28.3	253.2	1.9	529.2	103.2	7529.6	1657.5	1869.8	307.0
16-May-90	6467	0.7	6467.7	143.5	28.0	205.7	1.9	379.2	102.3	6190.8	1680.0	1457.9	1442.0
17-May-90	4710	1.0	4711.0	46.7	28.1	152.0	2.0	228.8	103.3	4585.5	1689.0	1168.5	101.0
18-May-90	3527	0.7	3527.7	45.4	28.1	132.8	2.0	208.3	104.4	3423.8	1688.2	914.6	137.0
19-May-90	6591	1.1	6592.1	124.5	28.1	372.0	2.0	526.5	107.3	6172.8	1642.7	1438.8	926.0
20-May-90	4235	0.4	4235.4	205.1	28.1	230.2	2.0	465.3	101.7	3871.8	1592.4	1202.3	208.0
21-May-90	3619	0.9	3619.9	45.5	28.1	155.6	2.1	231.3	103.9	3492.5	1640.6	714.4	133.0
22-May-90	3254	0.7	3254.7	48.2	28.1	131.8	2.3	210.3	104.3	3148.7	1689.4	582.1	158.0
23-May-90	3006	0.9	3006.9	45.7	28.1	115.9	2.2	191.8	104.5	2919.6	1739.2	485.9	148.0
24-May-90	2941	0.9	2941.9	133.9	28.3	110.5	2.1	274.8	105.0	2772.1	1731.8	489.6	164.0
25-May-90	4844	0.7	4844.7	88.8	28.1	703.5	2.0	822.4	114.4	4136.7	1688.1	3019.5	422.0
26-May-90	4617	0.7	4617.6	249.9	28.2	274.9	1.8	554.8	101.9	4164.8	1639.5	1252.1	179.0
27-May-90	4101	0.6	4101.6	72.9	28.1	186.9	2.0	289.9	102.5	3914.3	1619.6	710.3	183.0
28-May-90	2864	0.6	2864.6	47.7	28.2	143.2	2.2	221.2	102.5	2745.9	1658.3	502.0	175.0
29-May-90	2621	0.7	2621.7	47.9	28.2	119.2	2.0	197.4	106.0	2530.4	1731.2	398.3	120.0
30-May-90	2695	0.7	2695.7	134.6	28.6	106.3	2.1	271.5	106.0	2530.2	1765.0	395.0	144.0
31-May-90	2663	1.0	2664.0	45.7	29.0	94.1	2.0	170.8	105.8	2599.0	1812.1	271.1	143.0
MAY 90	5491.1	0.7	5491.7	116.6	28.2	371.4	2.0	518.3	106.0	5079.4	1686.6	2013.5	252.9

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Jun-90	3317	1.0	3318.0	46.2	28.9	87.3	2.3	164.7	110.2	3263.5	1857.1	241.0	670.0
02-Jun-90	3918	1.2	3919.2	122.9	28.9	84.1	2.3	238.1	110.4	3791.5	1765.0	313.1	736.0
03-Jun-90	3174	0.4	3174.4	60.2	28.7	77.0	2.0	167.8	109.0	3115.6	1695.9	227.3	563.0
04-Jun-90	2540	0.9	2540.9	46.1	28.6	70.3	2.2	147.2	110.3	2504.1	1780.1	193.2	654.0
05-Jun-90	3393	0.7	3393.7	46.0	28.7	65.8	2.1	142.5	110.2	3361.4	1743.4	181.2	488.0
06-Jun-90	2924	0.8	2924.8	134.6	28.6	65.5	2.2	230.9	112.3	2806.2	1779.2	252.8	645.0
07-Jun-90	4378	1.1	4379.1	51.3	28.7	89.5	2.2	171.6	111.4	4316.9	1832.1	308.1	889.0
08-Jun-90	4981	0.8	4981.7	187.4	28.7	950.4	2.2	1168.6	119.2	3932.4	1738.7	3313.9	658.0
09-Jun-90	4367	1.0	4368.0	301.4	28.6	124.3	2.3	456.5	110.9	4022.4	1802.9	664.9	647.0
10-Jun-90	3939	0.6	3939.6	59.6	28.6	86.6	2.1	177.1	113.3	3875.9	1795.3	302.0	686.0
11-Jun-90	3525	1.1	3526.1	46.0	28.7	70.5	2.1	147.4	114.8	3493.5	1901.4	217.2	691.0
12-Jun-90	3470	1.2	3471.2	46.1	28.7	61.4	2.3	138.5	115.8	3448.5	2011.0	181.4	637.0
13-Jun-90	3178	1.3	3179.3	125.3	28.6	169.0	2.3	325.4	118.1	2972.0	2202.3	573.6	462.0
14-Jun-90	4735	0.7	4735.7	208.4	28.9	498.4	2.1	735.8	122.7	4122.5	1937.3	1919.0	1032.0
15-Jun-90	4258	1.5	4259.5	161.6	28.5	93.7	2.3	286.3	115.6	4088.9	1913.4	356.2	715.0
16-Jun-90	3869	0.8	3869.8	46.1	28.6	70.7	2.0	147.4	112.3	3834.7	1853.3	181.7	438.0
17-Jun-90	3661	1.2	3662.2	46.2	28.7	60.1	2.1	137.0	112.2	3637.4	1933.6	166.1	989.0
18-Jun-90	3054	1.3	3055.3	141.9	28.6	55.4	2.2	228.2	116.2	2943.3	2032.4	205.3	684.0
19-Jun-90	3691	1.3	3692.3	46.4	28.6	51.1	2.1	128.4	116.8	3680.7	1891.1	138.4	511.0
20-Jun-90	3673	1.1	3674.1	135.5	28.7	63.3	2.2	229.7	116.2	3560.7	1807.7	331.5	1126.0
21-Jun-90	3556	1.1	3557.1	46.4	28.6	45.9	2.2	123.1	112.9	3547.0	1894.9	132.4	527.0
22-Jun-90	4675	1.2	4676.2	104.7	28.5	924.9	2.1	1060.2	125.3	3741.3	1808.7	2712.4	317.0
23-Jun-90	3593	0.8	3593.8	302.3	28.7	228.1	2.2	561.2	114.1	3146.7	1712.8	991.9	280.0
24-Jun-90	4058	1.1	4059.1	107.4	28.2	94.7	2.2	232.5	112.1	3938.7	1683.8	289.3	724.0
25-Jun-90	3680	1.3	3681.3	46.1	28.6	71.2	2.4	148.3	113.8	3648.8	1950.2	178.6	721.0
26-Jun-90	3782	1.6	3783.6	63.4	28.7	66.1	2.3	162.5	114.0	3735.1	1970.2	481.4	544.0
27-Jun-90	3847	1.4	3848.4	114.8	28.6	55.0	2.2	200.5	117.8	3765.7	1997.3	199.6	618.0
28-Jun-90	3689	1.7	3690.7	43.9	28.7	91.2	2.1	166.0	114.4	3639.2	1910.6	127.0	578.0
29-Jun-90	6051	1.1	6052.1	169.4	28.7	1187.6	2.2	1387.8	119.1	4783.4	1819.1	4671.1	375.0
30-Jun-90	5219	1.6	5220.6	234.7	28.4	196.6	2.0	461.6	114.0	4873.0	1998.2	727.3	547.0
JUN 90	3873.2	1.1	3874.3	109.7	28.6	195.3	2.2	335.8	114.5	3653.0	1867.2	692.6	638.4

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Jul-90	4694	1.4	4695.4	45.8	28.9	115.5	1.9	192.2	119.7	4622.0	1802.6	358.8	1128.0
02-Jul-90	4267	1.2	4268.2	45.9	28.8	80.9	1.9	157.5	118.0	4228.7	2000.4	226.8	1095.0
03-Jul-90	3659	1.6	3660.6	46.0	28.9	63.2	1.9	140.0	120.7	3641.4	2159.4	173.2	737.0
04-Jul-90	4215	1.5	4216.5	126.9	28.6	55.9	2.0	215.3	124.8	4125.5	2443.2	203.2	958.0
05-Jul-90	3869	1.2	3870.2	46.0	29.0	47.5	1.9	124.5	124.1	3869.8	2098.5	124.6	899.0
06-Jul-90	4060	1.1	4061.1	46.1	28.8	43.8	2.0	120.7	120.1	4060.5	1956.8	108.1	1068.0
07-Jul-90	3733	1.3	3734.2	141.2	28.8	44.1	1.9	216.1	120.0	3638.2	1937.9	166.9	1087.0
08-Jul-90	3526	1.7	3527.7	43.9	29.1	38.0	1.9	112.9	122.8	3537.5	2384.2	75.1	969.0
09-Jul-90	4096	1.8	4099.8	43.9	29.1	36.1	1.9	111.0	125.6	4114.3	2457.2	76.2	1003.0
10-Jul-90	3972	1.1	3973.1	67.9	29.0	185.1	1.9	284.0	117.2	3806.3	1993.3	642.1	718.0
11-Jul-90	5553	0.8	5553.8	184.5	28.4	874.8	1.9	1089.7	123.4	4587.5	1815.1	3477.4	476.0
12-Jul-90	4644	0.9	4644.9	109.5	28.5	97.9	1.9	237.9	119.9	4527.0	1849.0	452.0	1030.0
13-Jul-90	4288	1.1	4289.1	45.7	28.5	68.2	1.9	144.4	120.6	4265.3	1771.3	214.6	1066.0
14-Jul-90	3752	0.7	3752.7	69.9	28.5	57.0	1.9	157.4	120.2	3715.5	1752.4	235.7	990.0
15-Jul-90	4063	0.8	4063.8	109.8	28.5	68.1	1.9	208.2	121.7	3977.2	1709.6	389.8	946.0
16-Jul-90	3830	0.8	3830.8	146.0	28.4	141.8	1.9	320.2	123.0	3633.6	1820.0	402.7	963.0
17-Jul-90	4044	1.5	4045.5	199.1	28.6	86.1	1.9	305.7	122.1	3861.9	2005.0	385.2	937.0
18-Jul-90	3616	1.2	3617.2	45.9	28.5	78.6	1.9	154.9	125.6	3588.1	2093.0	238.6	762.0
19-Jul-90	5887	1.8	5888.8	59.4	28.6	2830.4	1.9	2920.3	134.0	3102.5	1993.4	5674.1	554.0
20-Jul-90	14477	0.9	14478.0	117.2	28.6	1761.6	1.9	1909.3	129.3	12696.0	1824.3	5737.0	138.0
21-Jul-90	6340	0.9	6340.9	183.1	28.6	892.5	1.9	1106.1	119.5	5354.2	1763.4	1409.7	465.0
22-Jul-90	5554	1.2	5555.1	197.6	28.5	592.1	1.9	820.1	118.4	4853.5	1708.3	1368.8	1248.0
23-Jul-90	5530	0.9	5530.9	240.2	28.5	207.7	1.9	478.2	119.7	5172.4	1845.3	816.8	914.0
24-Jul-90	4601	1.1	4602.1	232.6	28.5	140.3	1.9	403.3	121.7	4320.5	1865.0	575.8	1160.0
25-Jul-90	4324	1.0	4325.0	45.4	28.6	101.4	1.9	177.3	121.6	4289.3	1941.4	320.8	1024.0
26-Jul-90	3927	1.3	3928.3	47.9	28.6	85.6	1.9	164.1	123.2	3867.4	2019.7	237.1	1055.0
27-Jul-90	4280	1.4	4281.4	45.7	28.7	74.5	1.9	150.9	124.6	4255.1	2041.2	184.9	984.0
28-Jul-90	3807	1.3	3808.3	135.8	28.7	72.6	1.9	239.0	117.3	3688.6	2058.9	235.0	958.0
29-Jul-90	3831	1.1	3832.1	84.3	28.9	225.2	1.9	340.3	118.1	3609.9	1989.8	411.6	483.0
30-Jul-90	3792	1.9	3793.9	92.9	28.5	100.6	1.9	223.9	116.9	3688.9	1901.2	277.9	904.0
31-Jul-90	4156	1.0	4157.0	45.8	28.6	78.9	1.9	155.2	118.4	4120.2	1884.1	179.4	1013.0
JUL 90	4657.7	1.2	4658.9	99.5	28.7	301.5	1.9	431.6	121.6	4348.9	1964.7	818.7	894.6

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Aug-90	3715	0.7	3715.7	45.7	28.9	67.8	2.5	144.9	122.0	3692.8	1969.9	136.9	985.0
02-Aug-90	3649	1.1	3650.1	113.0	28.7	63.2	2.5	207.4	122.0	3564.8	2078.5	172.9	948.0
03-Aug-90	4195	1.2	4196.2	45.9	28.5	56.7	2.4	133.6	128.3	4188.9	2090.0	105.8	868.0
04-Aug-90	4885	1.0	4886.0	179.6	28.6	409.8	2.6	620.6	120.7	4386.2	1864.1	1879.7	965.0
05-Aug-90	4624	0.9	4624.9	94.0	28.4	92.1	2.4	216.9	121.5	4529.5	1773.5	421.6	1074.0
06-Aug-90	3844	1.0	3845.0	45.6	28.6	70.6	2.5	147.2	121.6	3819.4	1853.0	218.2	1059.0
07-Aug-90	3830	0.9	3830.9	45.8	28.6	59.8	2.6	136.7	121.4	3815.5	1971.2	158.5	1044.0
08-Aug-90	3754	1.1	3755.1	115.9	28.5	55.4	2.5	202.3	124.8	3677.6	2062.2	179.7	1000.0
09-Aug-90	3777	0.9	3777.9	45.8	28.5	49.2	2.6	126.2	123.3	3775.1	2116.6	108.1	999.0
10-Aug-90	4889	0.9	4889.9	75.2	28.6	100.7	2.4	207.0	132.0	4814.9	2048.5	2663.8	755.0
11-Aug-90	4247	0.9	4247.9	311.6	28.6	73.2	2.3	415.6	121.5	3953.8	1815.8	500.3	665.0
12-Aug-90	6117	0.8	6117.8	157.5	28.8	635.2	2.3	823.7	136.0	5432.1	1733.6	2607.5	1467.0
13-Aug-90	4352	0.8	4352.8	183.7	28.6	135.6	2.4	350.3	122.2	4124.7	1820.2	881.9	708.0
14-Aug-90	3668	0.9	3668.9	270.2	28.8	90.1	2.4	391.5	124.1	3401.5	1849.4	457.9	660.0
15-Aug-90	4252	1.3	4253.3	140.4	28.6	67.1	2.5	238.5	125.1	4139.8	1954.5	259.2	969.0
16-Aug-90	4005	1.0	4006.0	46.5	28.9	54.1	2.4	131.9	127.6	4001.7	2015.0	138.4	969.0
17-Aug-90	4198	1.0	4199.0	107.3	28.5	82.2	2.4	220.5	128.6	4105.1	1881.2	1676.5	688.7
18-Aug-90	9357	1.7	9358.7	169.5	28.7	545.9	2.5	746.5	133.6	8745.8	1900.1	6597.6	223.7
19-Aug-90	8415	0.8	8415.8	199.6	29.0	415.0	2.4	646.0	132.9	7902.7	1823.6	3271.7	898.0
20-Aug-90	10676	1.1	10677.0	246.3	28.8	1073.6	2.4	1351.3	140.7	9466.5	1825.8	6562.4	1336.0
21-Aug-90	9245	0.8	9245.6	142.1	28.7	1225.2	2.3	1398.3	139.4	7966.7	1822.7	6064.3	259.0
22-Aug-90	6206	0.9	6206.9	112.0	28.9	390.0	2.4	533.2	124.0	5797.7	189.3	2701.6	140.0
23-Aug-90	5518	1.0	5519.0	213.0	28.7	249.4	2.5	493.5	123.3	5148.8	1859.9	1873.0	322.0
24-Aug-90	5461	1.0	5462.0	273.7	28.6	185.4	2.5	490.2	123.8	5095.7	1877.1	1425.9	754.0
25-Aug-90	5841	1.0	5842.0	245.7	28.8	139.3	2.5	416.3	122.2	5547.9	1869.5	1080.4	1037.0
26-Aug-90	4950	1.3	4951.3	46.1	28.9	110.3	2.5	189.8	126.2	4887.8	1924.7	745.9	952.0
27-Aug-90	4252	1.6	4253.6	45.7	28.7	95.5	2.6	172.4	124.8	4206.0	2315.8	608.4	905.0
28-Aug-90	4942	1.4	4943.4	45.6	28.5	90.9	2.5	167.4	127.8	4903.9	2336.9	517.2	1199.0
29-Aug-90	5109	1.3	5110.3	140.8	28.7	85.3	2.6	257.4	125.8	4978.7	2152.7	504.7	1691.0
30-Aug-90	5189	1.0	5190.0	45.9	29.0	77.6	2.6	155.1	129.7	5160.5	2086.1	337.0	1826.0
31-Aug-90	4803	0.9	4803.9	46.5	28.7	74.1	2.6	151.9	126.8	4778.8	2071.9	277.2	1867.0
AUG 90	5224.7	1.0	5225.7	129.0	28.7	223.2	2.5	383.4	126.4	4968.7	1901.7	1526.9	943.7

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1990	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
DATE	1	2	3	4	5	6	7	8	9	10	11	12	13
01-Sep-90	4869	1.1	4870.1	70.5	28.2	70.5	2.2	171.4	113.0	4811.7	2190.4	246.1	1859.0
02-Sep-90	4809	0.8	4809.8	93.4	28.2	68.2	2.1	192.0	114.6	4732.4	1996.5	251.3	1906.0
03-Sep-90	4783	0.8	4783.8	43.9	28.2	64.1	2.1	138.3	113.9	4759.4	1968.2	197.1	1957.0
04-Sep-90	4577	1.6	4578.6	46.5	28.2	62.7	2.2	139.5	115.9	4555.0	2267.7	187.3	1722.0
05-Sep-90	4166	1.3	4167.3	114.7	28.2	62.1	2.1	207.1	114.5	4074.7	2243.0	230.7	1640.0
06-Sep-90	5063	1.6	5064.6	46.1	28.2	57.6	2.3	134.2	116.1	5046.5	2353.4	165.9	1476.0
07-Sep-90	5304	0.7	5304.7	244.2	28.2	260.4	2.2	535.0	117.6	4887.6	2018.0	2163.7	1560.0
08-Sep-90	5604	1.0	5605.0	97.8	28.2	71.2	2.1	199.3	109.6	5515.3	1908.1	283.5	1686.0
09-Sep-90	5757	0.9	5757.9	43.9	28.2	59.7	2.2	133.9	111.6	5735.6	1994.7	172.0	2720.0
10-Sep-90	5414	1.3	5415.3	71.9	28.2	56.3	2.2	158.5	115.1	5371.9	2060.6	167.1	2561.0
11-Sep-90	5639	1.1	5640.1	95.6	28.2	53.4	2.2	179.4	110.9	5571.6	2053.4	157.3	2612.0
12-Sep-90	5468	1.3	5469.2	46.3	28.2	49.5	2.2	126.1	115.1	5458.2	2083.1	114.3	2475.0
13-Sep-90	5625	1.1	5626.1	73.2	28.2	47.1	2.1	150.6	118.6	5592.1	2095.0	125.2	2404.0
14-Sep-90	5582	1.1	5583.1	215.6	28.2	98.7	2.1	344.6	114.3	5352.8	1906.3	672.1	2077.0
15-Sep-90	5645	0.8	5645.8	46.1	28.2	46.4	2.2	122.9	107.0	5629.9	1796.8	170.2	2323.0
16-Sep-90	5496	0.6	5496.6	46.3	28.2	44.4	1.9	120.8	109.1	5484.9	1734.9	141.3	2805.0
17-Sep-90	5372	0.8	5372.8	69.9	28.2	42.4	2.1	142.6	111.9	5342.1	1766.9	131.7	2660.0
18-Sep-90	5220	0.6	5220.6	96.7	28.2	42.9	1.9	169.7	105.8	5156.6	1771.7	148.0	2201.0
19-Sep-90	4703	0.9	4703.9	46.2	28.2	40.0	2.4	116.7	107.6	4694.8	1777.0	86.3	2337.0
20-Sep-90	5755	0.6	5755.6	43.9	28.2	37.7	2.1	111.8	110.9	5754.7	1787.5	82.2	2639.0
21-Sep-90	5663	0.7	5663.7	251.2	28.2	637.8	2.1	919.4	113.6	4857.9	1746.5	2083.9	1652.0
22-Sep-90	5442	0.9	5442.9	181.2	28.2	62.2	1.9	273.4	107.0	5276.6	1694.4	350.6	1795.0
23-Sep-90	5599	0.3	5599.3	43.9	28.2	46.6	2.2	120.8	108.0	5586.5	1673.9	145.0	2271.0
24-Sep-90	4921	0.8	4921.8	43.9	28.2	40.6	2.2	114.8	109.0	4916.0	1734.4	102.5	2134.0
25-Sep-90	3745	1.1	3746.1	135.4	28.2	39.6	2.1	205.2	110.1	3651.0	1798.6	162.5	1388.0
26-Sep-90	3505	0.6	3505.6	43.9	28.2	34.0	2.2	108.3	109.4	3508.7	1856.4	80.4	1295.0
27-Sep-90	3227	1.4	3228.4	46.2	28.2	32.1	2.1	108.6	108.7	3226.5	1859.2	78.3	575.0
28-Sep-90	2315	0.5	2315.5	77.5	28.2	30.2	2.1	137.9	108.9	2286.5	1808.2	102.1	121.0
29-Sep-90	2824	0.5	2824.5	79.4	28.2	28.3	2.1	138.0	109.8	2792.8	1745.5	110.9	152.0
30-Sep-90	2453	0.6	2453.6	43.9	28.2	26.5	2.0	100.6	105.3	2458.3	1720.2	70.3	144.0
SEP 90	4818.2	0.9	4819.1	86.6	28.2	77.1	2.1	194.0	111.2	4736.2	1913.7	306.1	1838.2

APPENDIX C

INSTALLATION AND OPERATION OF A DENSE RAINGAGE NETWORK TO IMPROVE PRECIPITATION MEASUREMENTS FOR LAKE MICHIGAN DIVERSION ACCOUNTING: WATER YEAR 1990

INSTALLATION AND OPERATION OF A DENSE RAINGAGE NETWORK TO IMPROVE PRECIPITATION MEASUREMENTS FOR LAKE MICHIGAN DIVERSION ACCOUNTING: WATER YEAR 1990

By

Randy A. Peppler

FINAL REPORT

to

U.S. Army Corps of Engineers, Chicago District

on Contract

DACW 23-89-C-0005

F. A. Huff, D. M. A. Jones and R. A. Peppler
Principal Investigators

Division of Atmospheric Sciences
Office of Cloud and Precipitation Research
Office of Applied Climatology
Illinois State Water Survey
2204 Griffith Drive
Champaign, Illinois 61820

September 1991

CONTENTS

	PAGE
1. INTRODUCTION	1
2. NETWORK DESIGN AND INSTALLATION	7
3. NETWORK OPERATION AND MAINTENANCE . . .	14
4. DATA REDUCTION	15
5. DATA ANALYSIS	20
6. EVALUATION OF THE NETWORK TO DATE	48
7. SUMMARY	54
8. ACKNOWLEDGEMENTS	55
9. REFERENCES	56
APPENDIX I	58
APPENDIX II	72
APPENDIX III	75
APPENDIX IV	85

INSTALLATION AND OPERATION OF A DENSE RAINGAGE NETWORK TO IMPROVE PRECIPITATION MEASUREMENTS FOR LAKE MICHIGAN DIVERSION ACCOUNTING: WATER YEAR 1990

1. INTRODUCTION

The volume of water diverted from Lake Michigan into the state of Illinois is monitored to ensure that the diversion does not exceed a long-term average of 3,200 cubic feet per second (cfs) as imposed by the U.S. Supreme Court Order of 1967, which was updated in 1980. This diversion has a long history, dating back to the mid-1800s with the completion of the Illinois and Michigan Canal. It has been affected over the years by such events as the reversal of the flow of the Chicago River and completion of the Chicago Sanitary and Ship Canal in 1901, and has weathered various legal proceedings this century that attempted to ensure the diversion could be monitored and did not exceed certain limits. One of the key components of the monitoring procedure, administered by the U.S. Army Corps of Engineers (COE), Chicago District, is the accurate representation of the precipitation that falls over portions of the Cook County, Illinois region.

The primary components of Illinois' diversion from Lake Michigan are as follows: (1) water is pumped directly from Lake Michigan as the source of potable water supply and discharged into the river and canal system in the greater Chicago area as treated sewage; (2) storm runoff is discharged from the diverted watershed area of Lake Michigan, draining to the river and canal system; and (3) water enters into the river and canal system directly from Lake Michigan.

The storm runoff from the Lake Michigan watershed basin enters the combined sewer systems and watercourses. The combined sewers mix sanitary systems with the runoff, and this water then goes to the treatment plants or, during major flood events, becomes surcharge into the watercourses. When large storm events are predicted (and greater than normal storm runoff is anticipated), the canal system is drawn down prior to the event to prevent flooding. If the event fails to materialize, canal system levels are restored using a direct diversion from Lake Michigan through one of three facilities located along the shoreline: Chicago River Controlling Works, O'Brien Lock and Dam, and the Wilmette Controlling Works.

There are two methods by which diversion is computed. The first involves the direct measurement of diversion flow at Romeoville, Illinois, as measured by an acoustic velocity meter. Flow at Romeoville consists of both diversion and nondiversion flows (deductions). The theory behind diversion accounting is to use the flow at Romeoville and deduct from it flows not attributable to diversion. Diversion flows that bypass Romeoville are added to the resultant flow, yielding a net computed diversion of water from Lake Michigan. The deductions to the Romeoville record include runoff from 217 square miles of the Des Plaines River watershed that is discharged into the canal, groundwater supply whose effluent is discharged into the canal, and Indiana water supply that is discharged to the canal via the Calumet River system and the Calumet Sag Channel.

The second method estimates diversion by adding the Lake Michigan water supply pumpage, direct diversions from Lake Michigan, and runoff from 673 square miles of diverted Lake Michigan watershed. This computation is performed as a means of cross-checking the first method.

In both of these procedures, it is necessary to estimate runoff from the Des Plaines River and the Lake Michigan watersheds. Since a significant portion of this area is not gaged with respect to water flows, runoff is estimated through hydrologic simulation. Inputs into the simulation model consist of land-use and climatological data. Of the latter, the most significant is precipitation data.

Accurate precipitation data, thus, are essential to properly simulate the runoff process. Runoff can be a substantial portion of the diversion. For example, in Water Year 1984 (a water year extends from 1 October through 30 September of the following calendar year), runoff from the Des Plaines River watershed constituted a 196.5 cfs (5 percent) deduction from the Romeoville measurement record in the diversion computations. In the verification computations, the Lake Michigan watershed runoff constituted an 829.0 cfs (27 percent) deduction from the total diversion.

However, the precipitation data available for use in the accounting procedure in recent water years (1984-1989) have displayed patterns inconsistent with known, long-term Chicago-area patterns (e.g., Changnon, 1961, 1968; Huff and Changnon, 1973; Vogel, 1988, 1989, 1991; Peppler, 1990, 1991). These patterns also diverge from the known urban effects found within the isohyetal patterns for the Cook County region for heavier rainfall distributions from 1949-1974 (Huff and Vogel, 1976), particularly towards the south, and within patterns observed during the operation of a dense raingage network and a radar system in the Chicago area during the late 1970s (Changnon, 1980, 1984). The recent unusual patterns have been caused by abnormally low precipitation totals at a select number of the 13 sites that the accounting procedure used (Figure 1). Inspection of these sites (Vogel, 1988), which are irregularly distributed over the region, revealed that the low totals

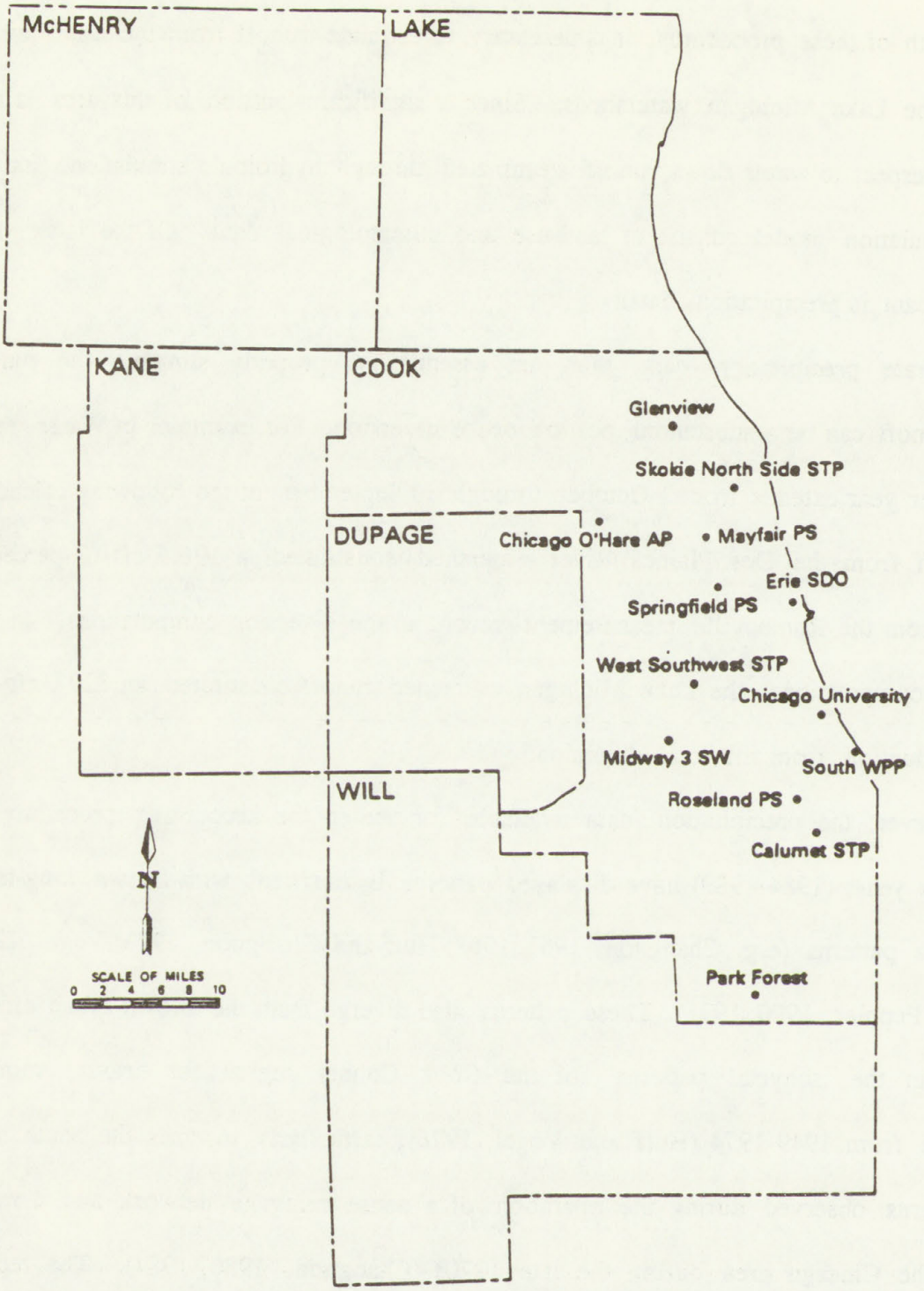


Figure 1. For Lake Michigan diversion accounting purposes, 13 raingage sites were used prior to Water Year 1990. Chicago O'Hare AP, Midway 3 SW, Chicago University, and Park Forest are National Weather Service sites; Mayfair PS, Springfield PS, South WPP, and Roseland PS are City of Chicago sites; Glenview, Skokie North Side STP, Erie SDO, West Southwest STP, and Calumet STP are Metropolitan Water Reclamation District of Greater Chicago sites. Abbreviations are as follows: AP = Airport; SDO = Sanitary District Office; SW = Southwest; WPP = Water Purification Plant; PS = Pumping Station; and STP = Sewage Treatment Plant.

were caused by 1) inadequate raingage exposure (e.g.,gages situated on rooftops or too near natural or man-made, flow-restricting obstructions) and 2) different observing, data reduction, and quality control practices used by the individual groups responsible for raingage operation and data collection (National Weather Service - NWS, Metropolitan Water Reclamation District of Greater Chicago - MWRDGC, and City of Chicago - CC). Vogel (1988) established that the unusual precipitation patterns began occurring in the late 1960s when some changes were made in data collection and reduction.

Vogel (1988) devised a procedure to adjust the questionable values, thus making the data suitable for use in the accounting procedure. This procedure, however, is tedious to implement, and the adjusted precipitation values may not completely capture the actual precipitation regime, although the data produced are an improvement on the original values. This procedure also illuminated difficulties experienced when trying to merge data observations from several different observing platforms into one data set. Vogel (1988) gave the following recommendation at the end of his report on the reduction and adjustment of the Water Year 1984 data and on field evaluations of the NWS, MWRDGC, and CC sites:

> "With these types of differences it will always be hard to maintain a consistent set of high-quality precipitation observations for the Chicago urban region. A precipitation network which must produce a set of high-quality observations should have a consistent set of gages; should be managed by one group with fixed quality control procedures, exposure criteria, and a set operating procedure. Management by one group would allow for consistent 1) observations, 2) quality control, and 3) spatial and temporal precipitation patterns.
>
> "To achieve this, it is recommended that a raingage network be established to monitor the precipitation over northeast Illinois relevant to the diversion of Lake Michigan waters. This network should consist of 10 to 15 weighing-bucket recording raingages. The raingages should be reasonably spaced across

> the affected area. The network should be managed by one group to ensure that the best possible exposures are obtained initially, and that these exposure are inspected at least annually. The data from such a network should all be quality-controlled in a consistent manner. Weighing-bucket raingages with daily charts would be capable of obtaining hourly or smaller time increments if daily charts are used. To reduce costs and to increase security, it is recommended that these raingages be located on private property, and that the observers be given a modest annual stipend. The charts from the observers should be mailed to a central location for data processing, quality control, and extraction of hourly precipitation totals. Raingages should be evenly spaced, as much as possible, and sites would be found after consulting with the agencies involved."

Thus, using this recommendation as a model, the Water Survey and the Corp Engineers jointly decided in late 1988 to devise, install, and operate a new raingage netw funded by the Corps of Engineers, that would produce consistent, accurate data for diversion accounting, free of the need for adjustment. The implementation and operat of such a network would have to be justified on the grounds of both long-term cost savi and greater accuracy. This report describes the design, installation, maintenance, operation of the network, along with the data reduction and analysis techniques employ and presents the data and analyses for Water Year 1990, to evaluate the netwo effectiveness at collecting precipitation.

2. NETWORK DESIGN AND INSTALLATION

The Water Survey has operated dense raingage networks in the past (e.g.,Huff, 1970, 1979), which tested gridded raingage spacings of 6 feet to 6 miles. Adequate sampling of convective-type precipitation (spring and summer) required nearly twice as many gages as required by more widespread, continuous precipitation (fall and winter). With that in mind, and opting for an optimum grid spacing, an initial attempt at creating a grid resulted in an array of 40 raingages located in the Cook County region encompassed by the Lake Michigan and Des Plaines River watersheds within the MWRDGC North, Central, South, and Lemont basins. However, due to cost considerations, some catchment ability was sacrificed, especially in the warmer seasons, and a 25-site grid was constructed using a 6-mile grid spacing between gages (Figure 2). Also due to cost considerations, a further step of placing raingages outside the watershed boundaries to better define isohyetal patterns at those boundaries was not pursued. Twenty-five raingages, more than Vogel had originally envisioned (10-15), are necessary to provide adequate coverage for precipitation catchment. This number, though, is consistent with the "best current engineering practice" as specified in the Supreme Court decree. Also, to have a consistent set of gages operated by one group (the Water Survey), an initial plan to incorporate the three NWS recording gages at Chicago O'Hare AP, Mid·vay 3 SW, and Chicago University into the network was scuttled.

Using the idealized grid, topographic maps of the Cook County region were used to approximate the location of each of the 25 sites and fine-tune their placement to best position the sites with respect to residential areas, industrial facilities, or municipal grounds. Since terrain effects are fairly minimal in northeastern Illinois, gridding was possible.

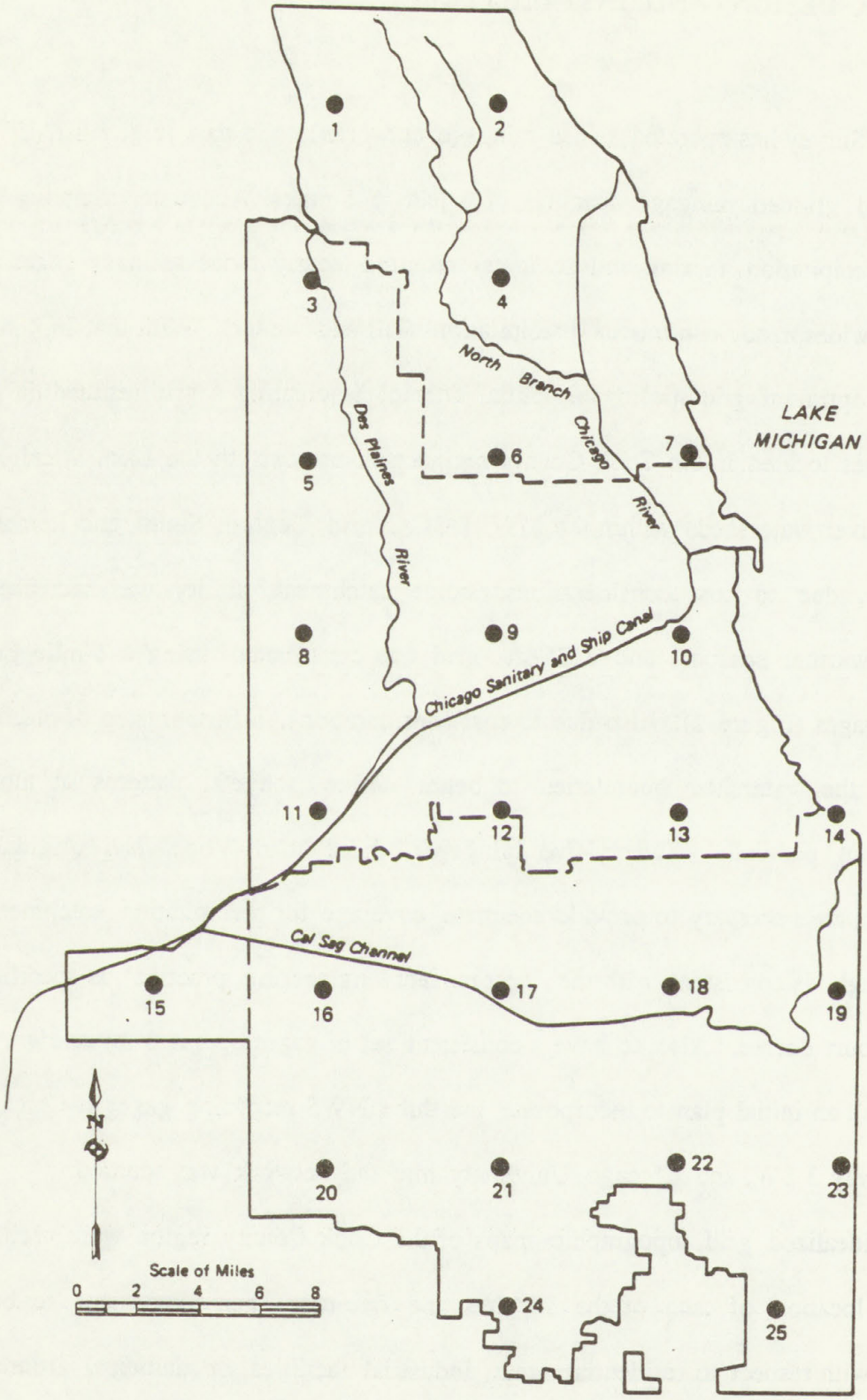


Figure 2. Original 25-site grid of the proposed raingage network.

Gridding also allows the use of simple arithmetic averaging to compute areal depths instead of other labor-intensive methods such as the Thiessen polygonal method. Once candidate locations were found, several field trips were made and letters were written in the summer of 1989 seeking permission to use the locations as raingage sites. This was sometimes a frustrating venture due to the urbanization of the region, making the identification of good catchment areas free of barriers for ground-level placement difficult. When selecting sites, highest priority was given to those at ground level in relatively open areas that were secure, since obstructions and local wind eddies produced by flow barriers present the largest sources of error in collecting precipitation data. Placing the collector at ground level mitigates wind effects on catchment and represents the ideal exposure (Legates and Willmott, 1990), but it is not practical in wintertime when snow is measured. Thus, as has been standard Water Survey practice, each raingage was placed with its base approximately 8 inches above ground level on stakes and the top of its orifice at about 4 feet. When asked for permission to site on their property, most individuals, businesses, and municipalities, were extremely receptive. In fact, only three of the sites have been relocated since the network began operation in October 1989: one landowner no longer wanted a raingage on-site, another was due to an impending move, and a third was due to the closing of a Chicago Park District facility.

In late September and early October 1989, the entire 25-gage network was installed. The Belfort weighing-bucket raingage was used throughout the network, and each fitted each with a battery-powered electric chart drive for more consistent and reliable operation. The Water Survey donated all raingages needed from its inventory. Figure 3 shows the current configuration of the network (as of July 1991), while Table 1 gives further information for

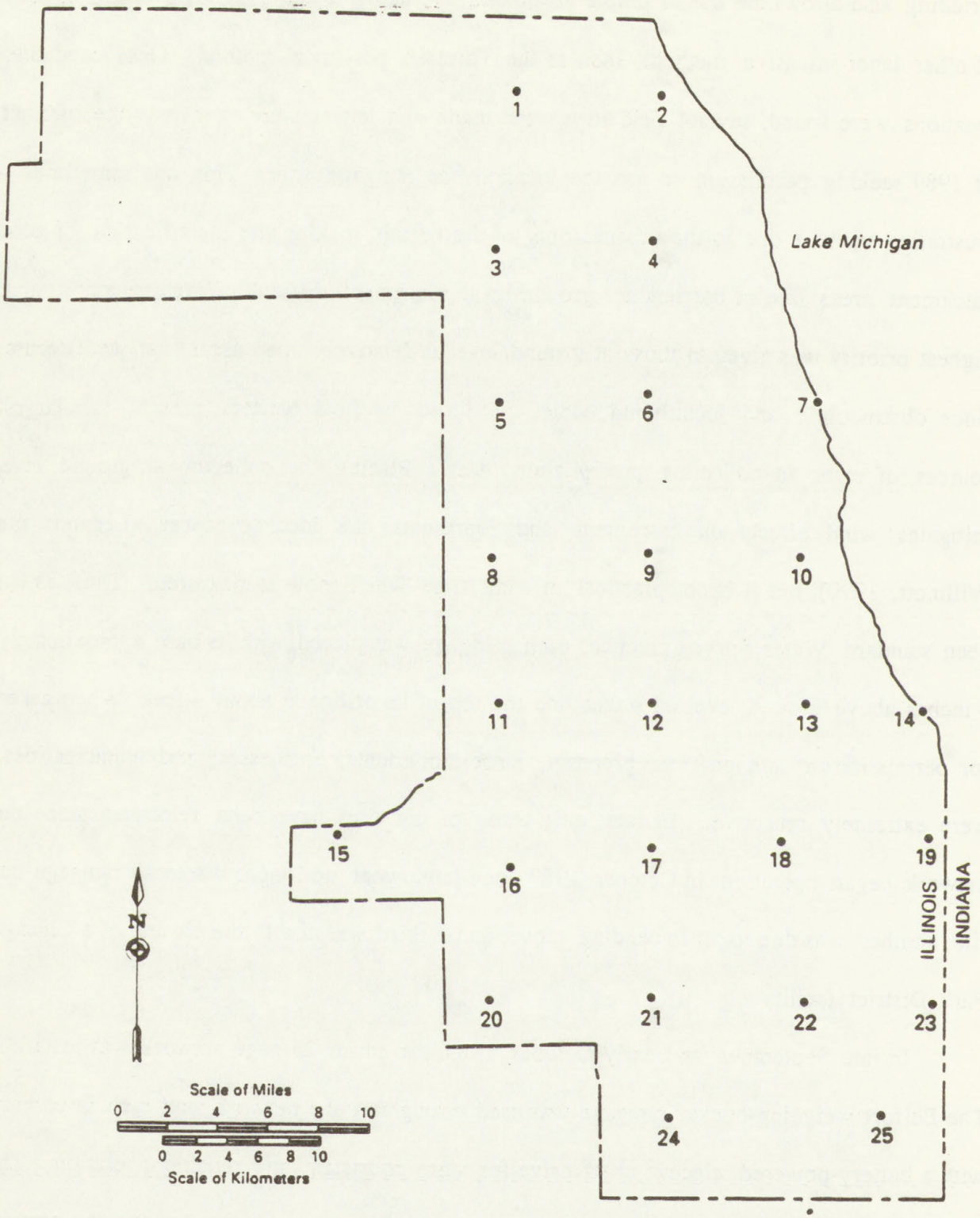


Figure 3. Current configuration of the 25-site raingage network.

Table 1. Raingage Network Site Information.

Site Number	Name	Address
1	Mission Brook Sanitary District	P.O. Box 2362 Northbrook, IL 60065
2	Winnetka Park District	510 Green Bay Rd. Winnetka, IL 60093
3	Private Residence	1885 Riverview Dr. Des Plaines, IL 60018
4	Village of Skokie	5127 Oakton St. Skokie, IL 60077
5	Private Residence	2925 N. Sarah Dr. Franklin Park, IL 60131
6	Private Residence	5340 W. Fletcher St. Chicago, IL 60641
7	Chicago Park District	425 McFetridge Dr. Chicago, IL 60605
8	Cook County Forest Preserve District	10400 Windsor Dr. Westchester, IL 60154
9	Mary Queen of Heaven Parish	5314 W. 24th Place Cicero, IL 60650
10	Rental Residence	527 W. 26th Street Chicago, IL 60616
11	Private Residence	10180 5th Ave. Cutoff LaGrange, IL 60525
12	Boyle Midway	5151 W. 73rd St. Bedford Park, IL 60638
13	Private Residence	7409 Eggleston St. Chicago, IL 60621
14	City of Chicago South Filtration Plant	3300 E. Chiltenham Place Chicago, IL 60649
15	Metropolitan Water Reclamation District	13 Stephen Street Lemont, IL 60439

Table 1. Concluded.

Site Number	Name	Address
16	Private Residence	240 Timber Edge Lane Palos Park, IL 60464
17	Sardee Industries	11900 S. Laramie St. Alsip, IL 60658
18	Ingersoll Products Company	1000 W. 120th St. Chicago, IL 60643
19	Graycor Industries	12233 Avenue O Chicago, IL 60603
20	Private Residence	10595 W. 167th St. Orland Park, IL 60462
21	Private Residence	16710 Lockwood Rd. Tinley Park, IL 60477
22	U. S. Army Reserve Center	400 E. 167th Street Harvey, IL 60426
23	City of Lansing Public Works	3300 E. 171st St. Lansing, IL 60438
24	Village of Matteson	3625 W. 215th St. Matteson, IL 60443
25	Big John's Farm Stand	1754 E. Joe Orr Rd. Chicago Heights, IL 60411

each site. Appendix I contains complete site descriptions as of July 1991 for each network location.

Weighing-bucket raingages were used. These recording gages are as reliable as any others available. All raingages are subject to catchment errors due to winds , wetting losses, evaporation, splashing into or out of the gage, and blowing snow (Legates and Willmott, 1990). Koschmieder (1934) noted that as wind speed increases, gage catch decreases. Legates and Willmott (1990) found that raingage errors "tend to be proportional to total precipitation and amount to nearly 11 percent of the global catch." Undercatch is generally much less in tropical areas than near the poles. To prevent loss due to blowing snow during winter, the Nipher shield and the shield used by Lindroth (1991) are helpful, but were not considered for the new network. Most likely they would have been subject to vandalism.

Jones (1969) compared the effectiveness of several types of raingages in a controlled setting. A weighing-bucket raingage fitted with a standard 8-inch diameter orifice with sloping shoulders caught 2.5 to 6.0 percent less precipitation than a standard 8-inch nonrecording device and 2.5 percent less than a weighing-bucket raingage fitted with a 12-inch diameter orifice (a right-cylinder). However, the maximum amount of precipitation that the latter gage can record compared to one with the standard orifice is reduced from 12.0 to 4.8 inches. Thus, the 8-inch diameter orifice was chosen for the new network since occasionally more than 4.8 inches of precipitation will fall into a gage between the weekly visits by the observer.

3. NETWORK OPERATION AND MAINTENANCE

Each raingage in the network was fitted with 24-hour chart drive and chart cylinder gears that rotate the chart cylinder once every 24 hours. The 24-hour chart allows adequate resolution down to 15-minute periods. Because a chart can measure up to 12 inches of precipitation, each gage is fitted with a galvanized bucket capable of holding that amount. An upward pen traverse on a chart measures the first 6 inches the bucket catches, and a reversed, downward pen traverse measures inches 7-12. The latter traverse, often unnecessary, is vital whenever more than 6 inches of precipitation occurs between chart periods, or during winter when the antifreeze-charged buckets are allowed to accumulate precipitation without dumping for long periods of time.

A single team of observers, living in Cook County, services each gage every 6-8 days, which means that 6-8 traces are drawn on each chart. Service includes removing and replacing the current chart, re-inking the pen, dumping the bucket from April-October (the warm season of the year), and noting any problems, including chart-drive malfunction, gage imbalance or instability, vandalism, unauthorized movement of the gage, etc. During the warm portion of the year, evaporation shields are fitted into the collection orifice just above the galvanized bucket to mitigate evaporation. During the cool portion of the year (November-March), these shields are removed and a 1-quart charge of antifreeze is added to each bucket. This allows snow and sleet to melt in the bucket as they are caught and allow the weighing mechanism to give a proper reading. Refer to Appendix II for a complete listing of service instructions used by the raingage observers.

A complete set of 25 charts collected by the observers is mailed to the Water Survey,

along with notations about problems. The next section on data reduction explains what happens to the data collected as ink traces on the charts.

Approximately once a month, the project leader at the Water Survey visits the network to perform maintenance and repairs for which the observers do not have adequate expertise. This includes a site assessment of a previously noted problem and the determination of a solution. Because most problems pertain to the chart drives, the solution is often to replace the drive. The old drive is cleaned and readied for re-use at the Water Survey. Two spare chart drives allow for this. The other frequent problems (mentioned above) can be solved on these trips as well. A complete maintenance history, including site relocations, is given for each of the 25 raingages in Appendix III, which more fully describes the kinds of repairs made. The information is current through June 1991 (i.e., includes first nine months of Water Year 1991).

4. DATA REDUCTION

When a set of charts arrives at the Water Survey, it is edited to identify the various traces on the charts and to number sequentially by date those with precipitation. This is perhaps the most important step in the reduction procedure. A running inventory of "on" and "off" chart times is maintained to ensure that the on-times on the newly received chart set matched the off-times on the most recent chart set previously analyzed. Occasionally, the observers will make inadvertent errors in the on-/off-time designations, particularly when Illinois time zones change in October and March (charts are always on Central

Standard Time). The on- and off-times are marked on the charts: the first revolution is designated "1",and the last designated as appropriate. Then, the various rain periods (storms) are identified and numbered based on their sequence in relation to the first and last revolutions. This is sometimes difficult, but with 25 charts to study, often with different starting days, all rain traces can be identified. This editing procedure also acts as a good "trouble-shooting" exercise to identify most chart-drive problems (running too slow, too fast, or stopped altogether). Raingage instability problems can also be identified by a shaky trace. Skipping or unusually heavy traces point to problems with the pen tip. Calibration problems can be noted if a trace reverses before the 6-inch line is reached. Finally, the editing stage permits the identification of missing periods of data on the charts, and these are appropriately marked. After all charts have been logged in with respect to on- and off-times, and all rain traces and missing periods have been identified, the charts are ready to be digitized.

A Summagraphics Microgrid II digitizer is used to digitize the charts. All values are fed into a Zenith Z-386 personal computer. Each chart is handled and logged into the computer separately. The four corners of a chart are digitized to set the grid, then the on- and off-times are entered, and their locations are digitized. The number of revolutions on the chart is noted. Each trace with precipitation is digitized by "clicking-in" each breakpoint along the respective trace. Once a chart is digitized, computer output gives details on the precipitation that was measured on the chart, in storm format, with appropriate beginning and ending times. Also included is an analysis of whether the chart drive is running slow or fast by comparing the on- and off-times with the chart's beginning and end points. This helps assess whether a chart drive needs to be replaced. Errors made during the editing

stage can also be caught during digitization. If a chart drive stopped during a collection period, the beginning and ending points of the missing information are digitized and appropriately stored in the computer.

Once a calendar month of data is logged into the computer, a C-language program, written at the Water Survey, calculates hourly precipitation values at all 25 sites for each hour of the month in question. These calculations are based on a linear interpolation between the digitized breakpoints on the various traces. The newly computed hourly values are compared to the digitized storm values (during program execution) to ensure consistent precipitation amounts. A printout of the entire monthly data array contains stations across the top as columns and data down the array as rows in chronological, hourly order. Monthly totals appear at the bottom. Missing values are denoted as dashes.

This data array is then used to check for time and space consistency, to divide the data into storm periods, and to fill in missing values with interpolated information, which is discussed further. A storm is defined as a precipitation period separated from preceding and succeeding precipitation periods by approximately 6 hours at all stations in the network. This definition was used by Huff (1967) for an area of similar dimensions in central Illinois, by Vogel (1986) to define extreme storm events in the Chicago area, and by Vogel (1988, 1989, 1991), and Peppler (1990, 1991) to define storms for Water Years 1984-1989. For each storm, values are summed and plotted on maps using all available data and stations, and isohyetal patterns are drawn.

After a generalized precipitation pattern is obtained for each storm, interpolated storm totals are estimated from the pattern for each site having missing information during that storm. Wind information, if available (usually the resultant direction and speed at

Chicago O'Hare Airport), and known urban effects in the Chicago area (Huff and Vogel, 1976; Changnon, 1980, 1984) are taken into account when drawing isolines and interpolating values. The newly found storm values are then redistributed into hourly values using the existing hourly percentage breakdown at "neighboring" nonmissing sites and the weighting factors are assigned to them. For each site in the network, neighbors identified for this purpose are weighted according to both distance and direction from the gage. See Table 2 for the designated neighbors and alternates (in case a neighbor is also unavailable) for each site and their associated weights. The weight given an alternate may vary from what is listed in Table 2, depending upon who is unavailable. For example, for Site #1, if neighbor #1 (Site #2) is missing, then Neighbor #2 (Site #3) is weighted 0.667 instead of 0.333, and the alternate (Site #4) is used and weighted 0.333.

Filling in missing values is the most labor-intensive data-reduction step. Once the missing hourlies are computed, they must be hand-entered into the computer. An automated process for this data-reduction step is being sought and will allow much quicker data reduction in general, perhaps within a week after all the information has been collected for a calendar month.

Table 2. Neighboring Sites Used for Missing Value Fill-in Procedure (Weighting Factors in Parentheses)

Site Number	Neighbor #1	Neighbor #2	Neighbor #3	Alternate
1	2 (0.667)	3 (0.333)		4 (0.333)
2	1 (0.667)	4 (0.333)		3 (0.333)
3	4 (0.500)	1 (0.250)	5 (0.250)	2 (0.250)
4	3 (0.500)	2 (0.250)	6 (0.250)	5 (0.250)
5	6 (0.500)	4 (0.250)	9 (0.250)	8 (0.250)
6	5 (0.500)	7 (0.500)		4 (0.500)
7	6 (0.500)	4 (0.250)	9 (0.250)	10 (0.250)
8	9 (0.500)	6 (0.250)	12 (0.250)	11 (0.250)
9	8 (0.500)	10 (0.500)		7 (0.333)
10	9 (0.500)	6 (0.250)	12 (0.250)	7 (0.250)
11	12 (0.667)	8 (0.333)		9 (0.333)
12	11 (0.500)	13 (0.500)		10 (0.333)
13	12 (0.500)	14 (0.500)		10 (0.500)
14	13 (0.500)	10 (0.250)	18 (0.250)	19 (0.250)
15	16 (0.500)	11 (0.250)	20 (0.250)	12 (0.250)
16	15 (0.500)	17 (0.500)		12 (0.333)
17	16 (0.500)	18 (0.500)		13 (0.333)
18	17 (0.500)	19 (0.500)		14 (0.333)
19	18 (0.500)	14 (0.250)	23 (0.250)	22 (0.250)
20	21 (0.500)	16 (0.250)	24 (0.250)	17 (0.250)
21	20 (0.500)	22 (0.500)		18 (0.333)
22	21 (0.500)	23 (0.500)		24 (0.500)
23	22 (0.500)	25 (0.500)		19 (0.500)
24	21 (0.500)	22 (0.500)		25 (0.333)
25	22 (0.500)	24 (0.500)		23 (0.500)

5. DATA ANALYSIS

Using the final, corrected, filled-in data array, several analyses were performed on the Water Year 1990 data. These include: (1) a water year total plot and monthly plots for the entire region as documentation of the data collected from the network (Figures 4-10), (2) monthly and water year totals at all sites (Table 3), (3) spatial correlation pattern analyses of the region (Figures 11-14), (4) statistical techniques assessing the validity of the correlation patterns (Figures 15-16), and (5) comparisons to data collected by NWS and CC raingages during this water year and past ones (Figures 17-19). The correlation patterns examine the degree of interrelation between the respective sites, which may be useful for identifying regions of preferred storm movement and categorizing the sites as well as for identifying and eliminating redundant sites. These analyses are done on a water year basis only.

Figure 4 contains the Water Year 1990 analysis. Isopleths are drawn to every inch. The figure reveals precipitation highs in the north central, east central, and extreme southwestern portions of Cook County encompassed by the network, centered on Sites #6, #13, and #24, respectively. Areas of lower precipitation were located in the extreme northern, northeastern, west central, southwestern, and southeastern portions of the region, centered on Sites #4, #7, #11, #17, and #25, respectively. The maximum at Site #6 seems to coincide with an urban high noted by Huff and Vogel (1976) from data for 1949-1974 ("north central high") and from data for other water years in the 1980s (e.g., Peppler, 1991). The minima in the extreme north and northeast may be associated with the stabilizing effect exerted by Lake Michigan on convective rainfall, as noted in data by Huff and Vogel (1976).

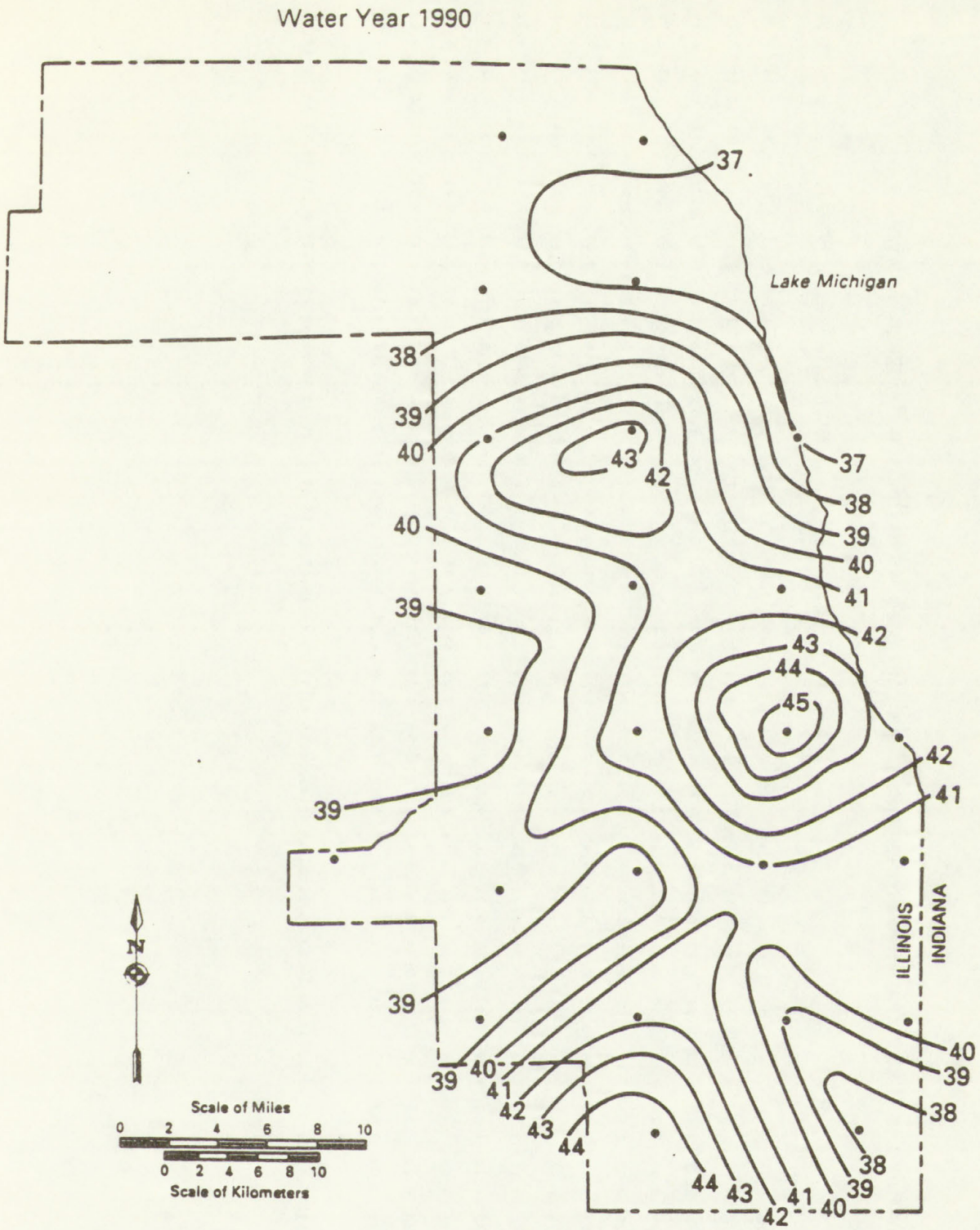


Figure 4. Precipitation pattern (inches) for Water Year 1990. Dots indicate network sites.

Table 3. Monthly and Water Year 1990 Precipitation Totals for All Sites in the New Raingage Network.

Month	G 1	G 2	G 3	G 4	G 5	G 6	G 7	G 8	G 9	G 10	G 11	G 12	G 13	G 14	G 15	G 16	G 17	G 18	G 19	G 20	G 21	G 22	G 23	G 24	G 25
Oct 1989	1.69	1.70	1.51	1.67	1.74	1.80	1.87	1.89	1.75	1.89	1.52	1.66	1.97	1.50	1.44	1.61	1.72	1.91	2.08	1.63	1.94	1.92	1.76	2.15	1.82
Nov 1989	1.96	2.48	2.11	2.05	2.15	2.42	2.99	2.25	2.48	3.17	2.41	2.88	2.44	2.55	2.11	2.61	2.76	2.74	2.91	2.57	2.45	2.12	2.94	3.15	2.83
Dec 1989	0.48	0.57	0.62	0.40	0.49	0.43	0.72	0.51	0.68	0.64	0.59	0.44	0.73	0.43	0.54	0.56	0.33	0.52	0.63	0.48	0.73	0.54	0.49	0.44	0.56
Jan 1990	2.09	2.06	2.35	2.21	2.41	2.48	1.82	2.14	2.59	2.57	2.48	2.43	2.55	2.28	2.60	2.66	2.41	2.34	2.03	2.53	2.67	2.37	2.07	2.59	2.41
Feb 1990	2.57	3.29	2.42	2.62	3.57	4.11	2.92	3.55	4.14	3.93	2.97	2.74	3.61	2.84	2.79	2.99	2.45	3.00	3.66	2.91	3.57	3.30	3.57	3.60	2.77
Mar 1990	2.78	2.97	2.86	2.62	3.25	3.33	2.97	3.28	3.43	3.53	3.32	3.29	3.66	3.54	3.45	3.62	3.71	3.46	3.65	4.03	3.70	3.84	3.57	4.37	3.04
Apr 1990	1.57	1.79	1.69	1.54	1.55	1.69	1.59	1.72	1.75	1.80	1.71	2.05	2.54	2.86	2.23	2.17	2.18	2.22	2.14	2.12	2.31	1.79	1.97	2.95	2.04
May 1990	6.16	6.51	7.11	7.21	7.94	7.96	7.03	8.22	7.92	7.18	7.56	6.66	7.51	6.68	7.19	6.78	6.68	6.85	6.91	6.54	7.55	6.88	6.45	9.32	6.18
Jun 1990	4.78	5.32	4.88	4.30	4.58	3.97	4.19	5.33	4.83	4.59	5.68	5.06	5.08	5.02	6.50	5.22	4.42	5.30	4.15	4.32	4.49	3.97	4.02	4.91	3.78
Jul 1990	4.66	2.91	2.57	2.43	2.84	3.48	3.23	3.45	3.59	4.01	5.15	7.25	6.45	7.84	5.38	7.00	6.66	5.85	6.57	5.11	4.81	4.52	6.01	4.08	3.76
Aug 1990	7.12	6.60	8.27	7.82	8.07	9.35	5.74	4.90	5.81	5.27	3.59	4.99	7.85	5.57	3.61	3.23	4.06	5.78	4.87	5.17	5.24	5.77	6.09	4.82	6.81
Sep 1990	1.25	1.31	1.09	1.37	2.52	2.24	1.46	1.96	2.02	2.57	1.58	1.70	1.50	1.78	1.60	1.52	0.96	1.02	0.66	1.51	2.18	2.07	1.44	1.96	1.49
W-Y 1990	37.11	37.51	37.48	36.24	41.11	43.26	36.53	39.20	40.99	41.15	38.56	41.15	45.89	42.89	39.44	39.97	38.34	40.99	40.26	38.92	41.64	39.09	40.38	44.34	37.49

The minimum in the west central region appears similar to adjusted ones found in Water Years 1988 and 1989 (Peppler, 1991), while the maximum in the east central region extending toward the Indiana border also resembles those 1988 and 1989 patterns (Peppler, 1991). Thus, the Water Year 1990 pattern, as collected by the raingages in the new network, displays characteristics found in previous established data. It generally disagrees with the unusual anomalies associated with the collection of raingages from the NWS, MWRDGC, and CC used to generate precipitation information in Water Years 1984-1989. Therefore, the new network can be considered to have improved collection properties.

Figures 5-10 contain monthly analyses for the water year period. Generally, light precipitation occurred in October, December, which had the lightest precipitation of the water year, April, and September (Figures 5, 6, 8, and 10, respectively). Heavier amounts occurred in May and over parts of the region in July and August (Figures 8-10, respectively). Looking at some of the more interesting cases, December's monthly amounts (Figure 6) ranged from just 0.33 inches at Site #17 to 0.73 inches at both Sites #7 and #21. February particularly showed the increased precipitation region in the north central portion of the area (Figure 7), as did August (Figure 10). Site #6 in August had the highest monthly total recorded during the water year at any site, 9.35 inches. May had high values throughout the entire network (Figure 8), with a maximum of 9.32 inches at Site #24 in the extreme southwestern portion of the grid. Both July and August had the greatest ranges of amounts over the network (Figures 9-10, respectively). In July, monthly totals ranged from only 2.43 inches at Site #4 in the north to 7.84 inches in the southeast at Site #14. A band in the 7-inch range spread from Site #16 in the southwest to Site #14, while values were below 3 inches north of Site #5. In August (Figure 10), values ranged from 3.23 inches at Site #16

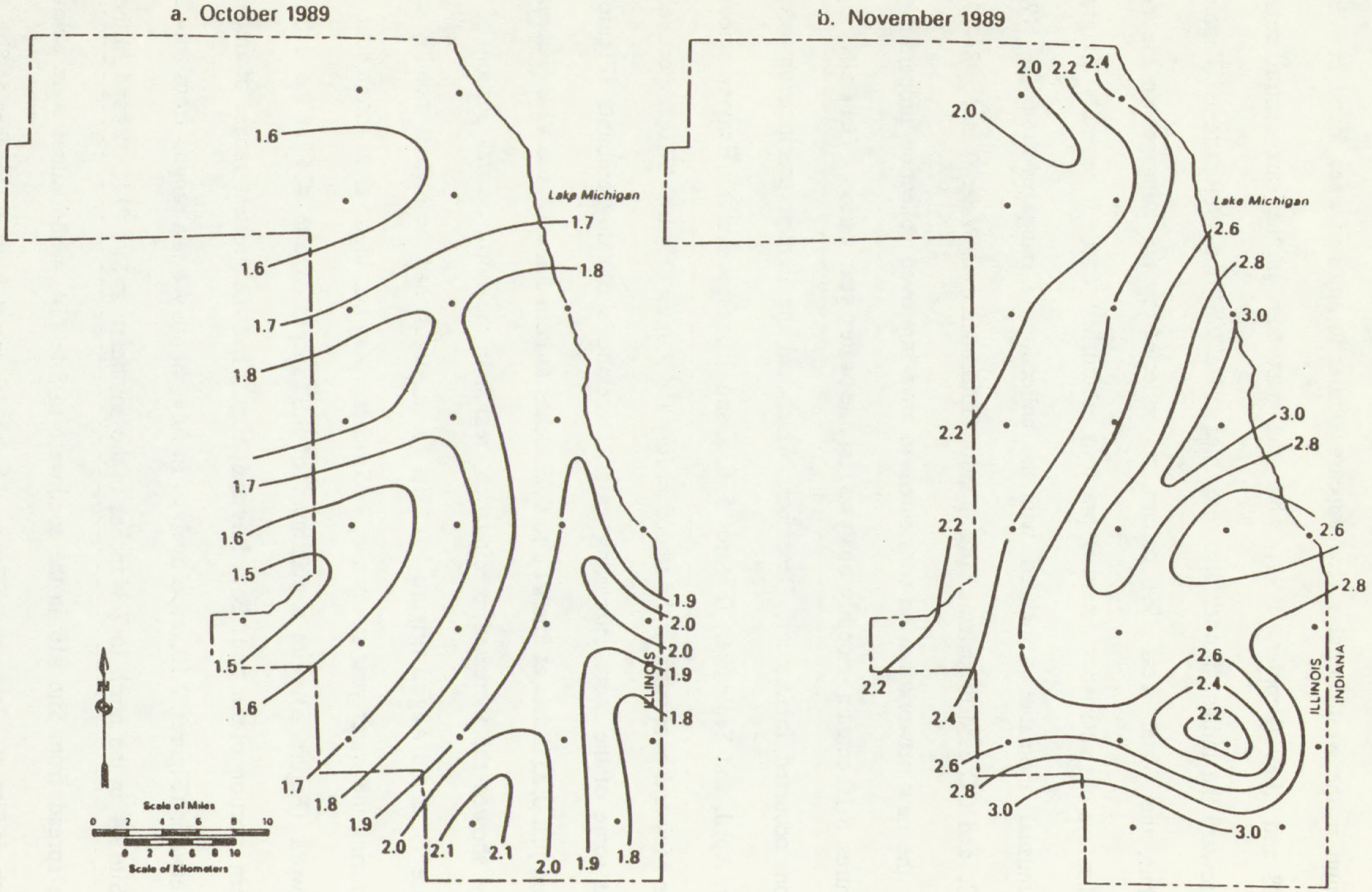


Figure 5. Precipitation patterns (inches) for October 1989 (Panel a) and November 1989 (Panel b). Dots indicate network sites.

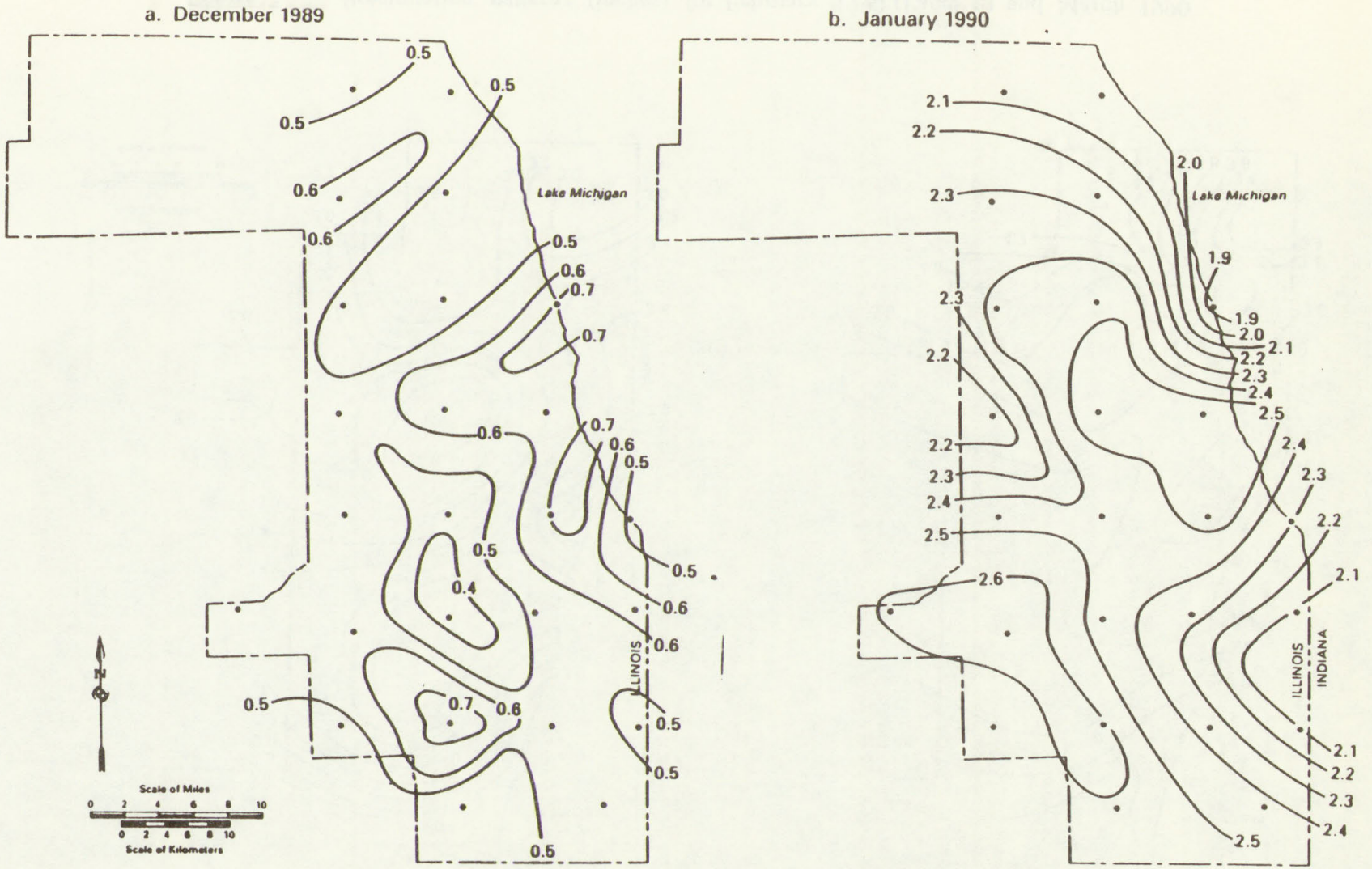


Figure 6. Precipitation patterns (inches) for December 1989 (Panel a) and January 1990 (Panel b). Dots indicate network sites.

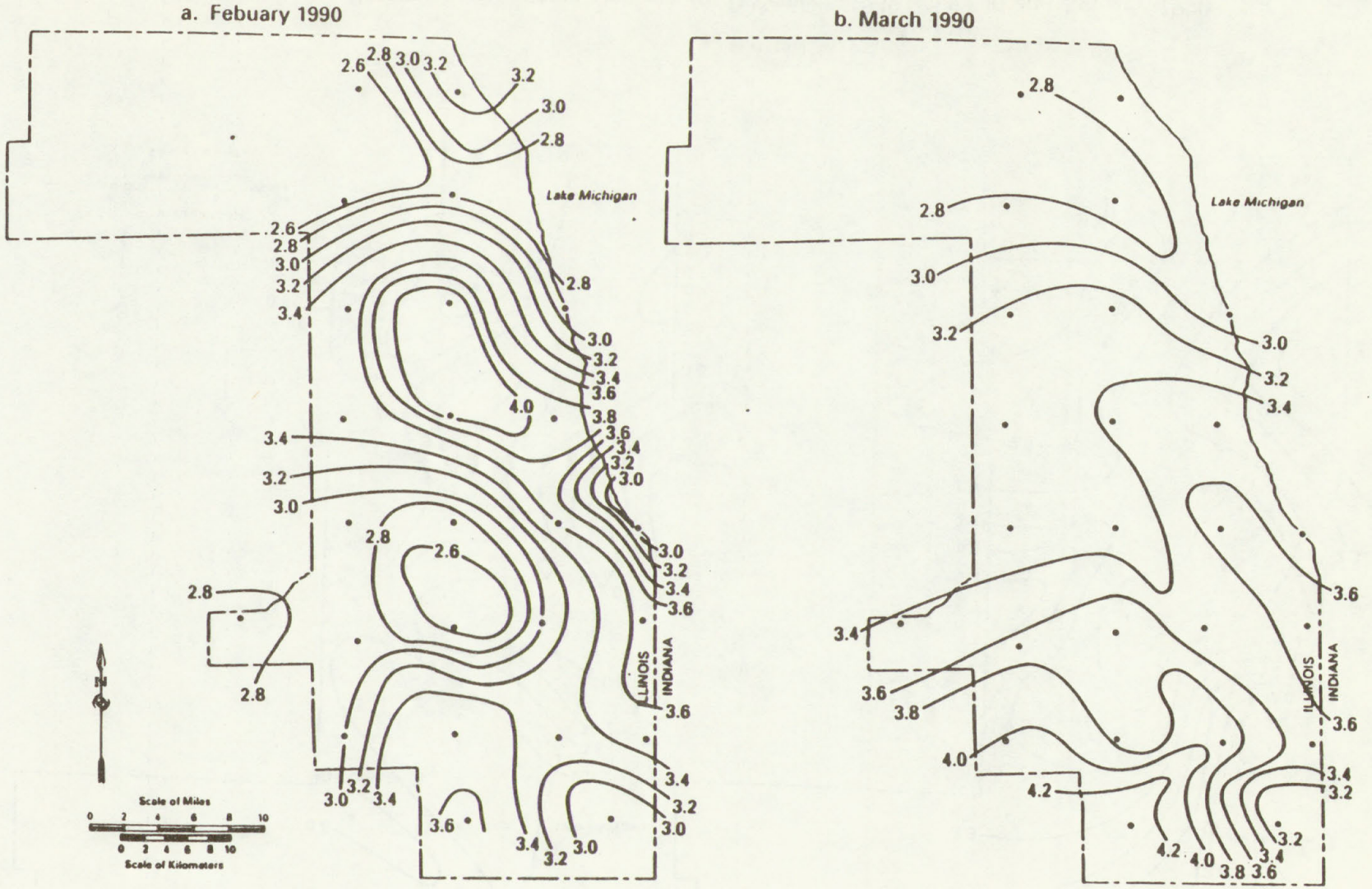


Figure 7. Precipitation patterns (inches) for February 1990 (Panel a) and March 1990 (Panel b). Dots indicate network sites.

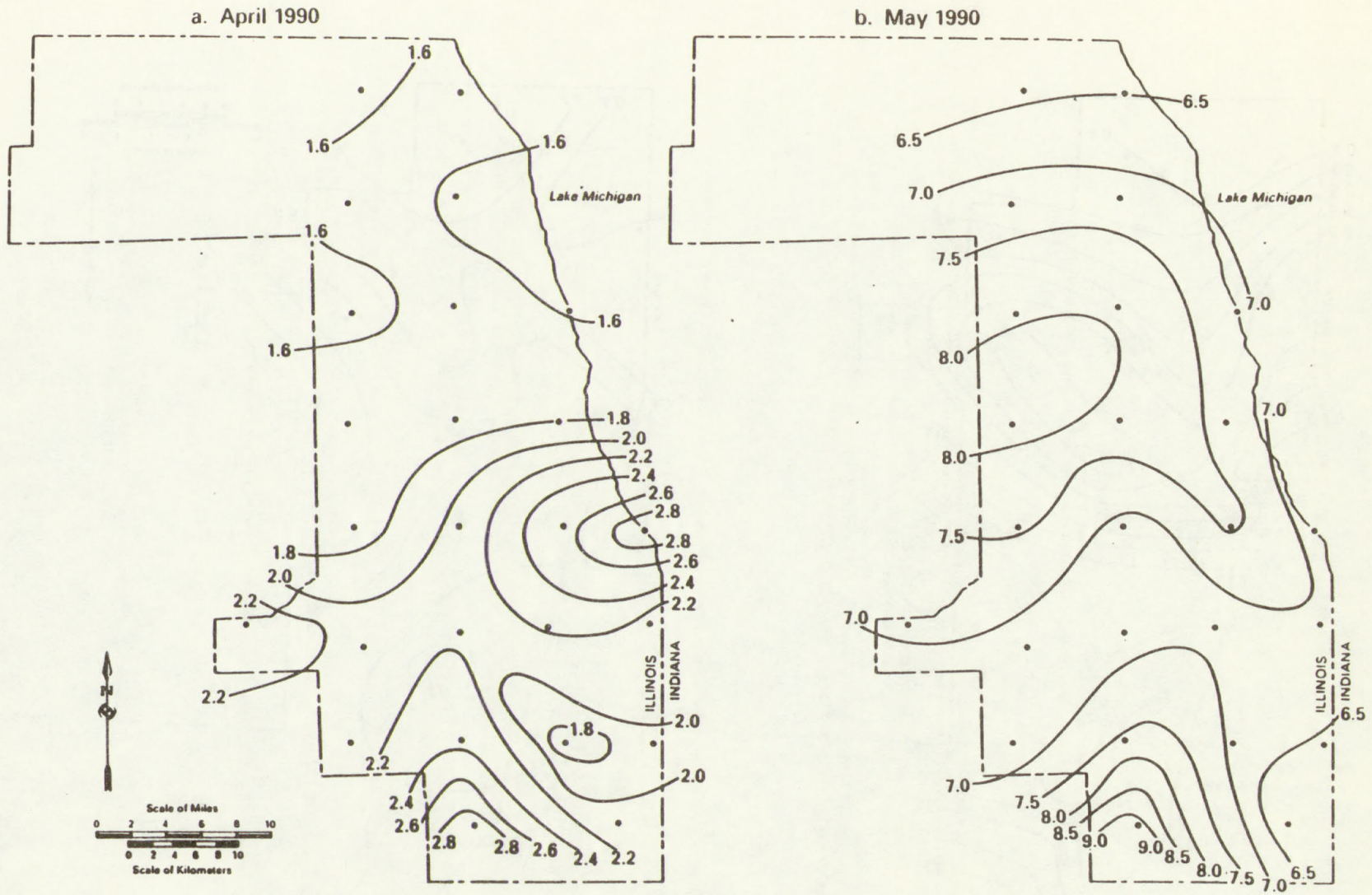


Figure 8. Precipitation patterns (inches) for April 1990 (Panel a) and May 1990 (Panel b). Dots indicate network sites.

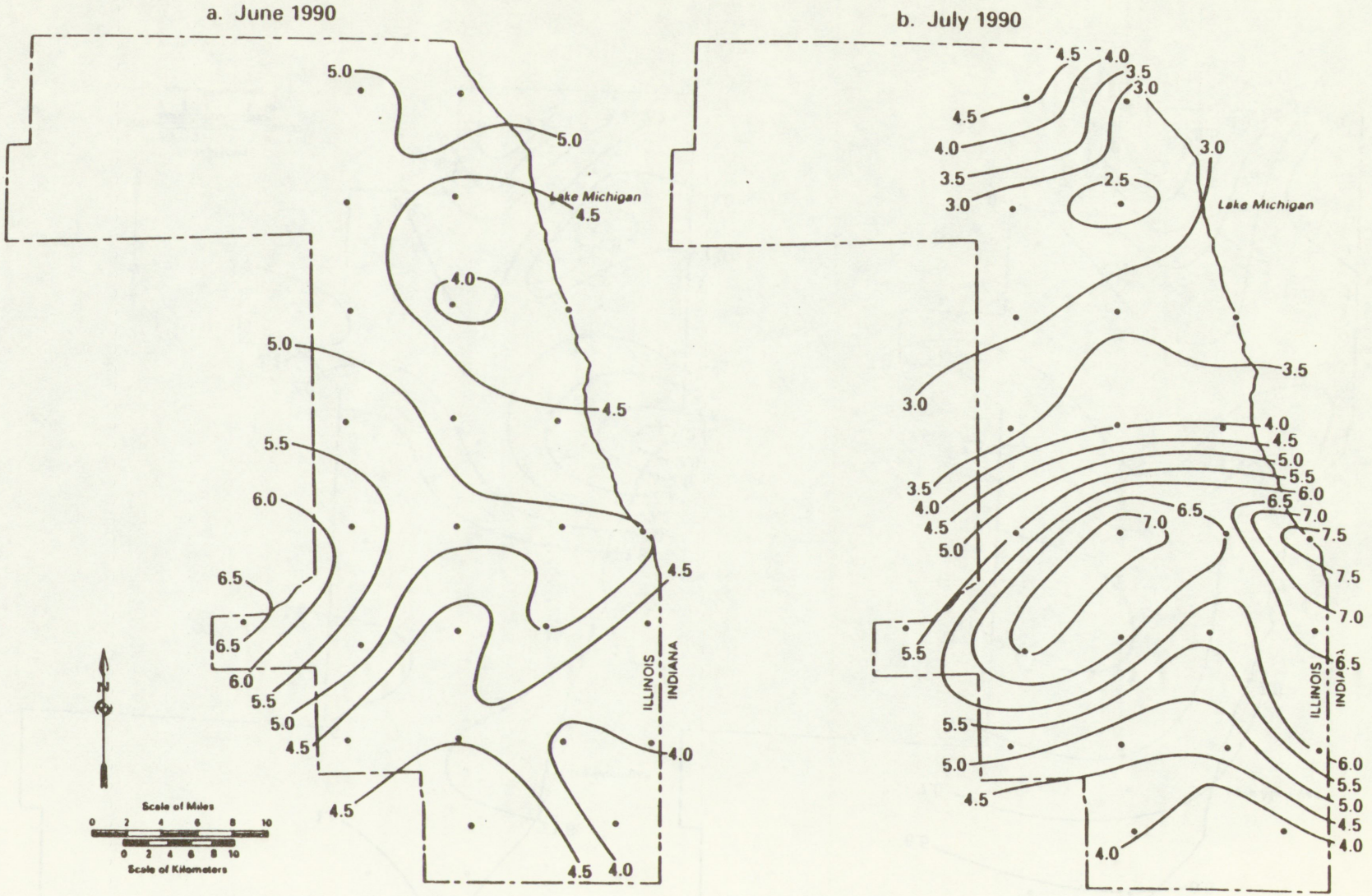


Figure 9. Precipitation patterns (inches) for June 1990 (Panel a) and July 1990 (Panel b). Dots indicate network sites.

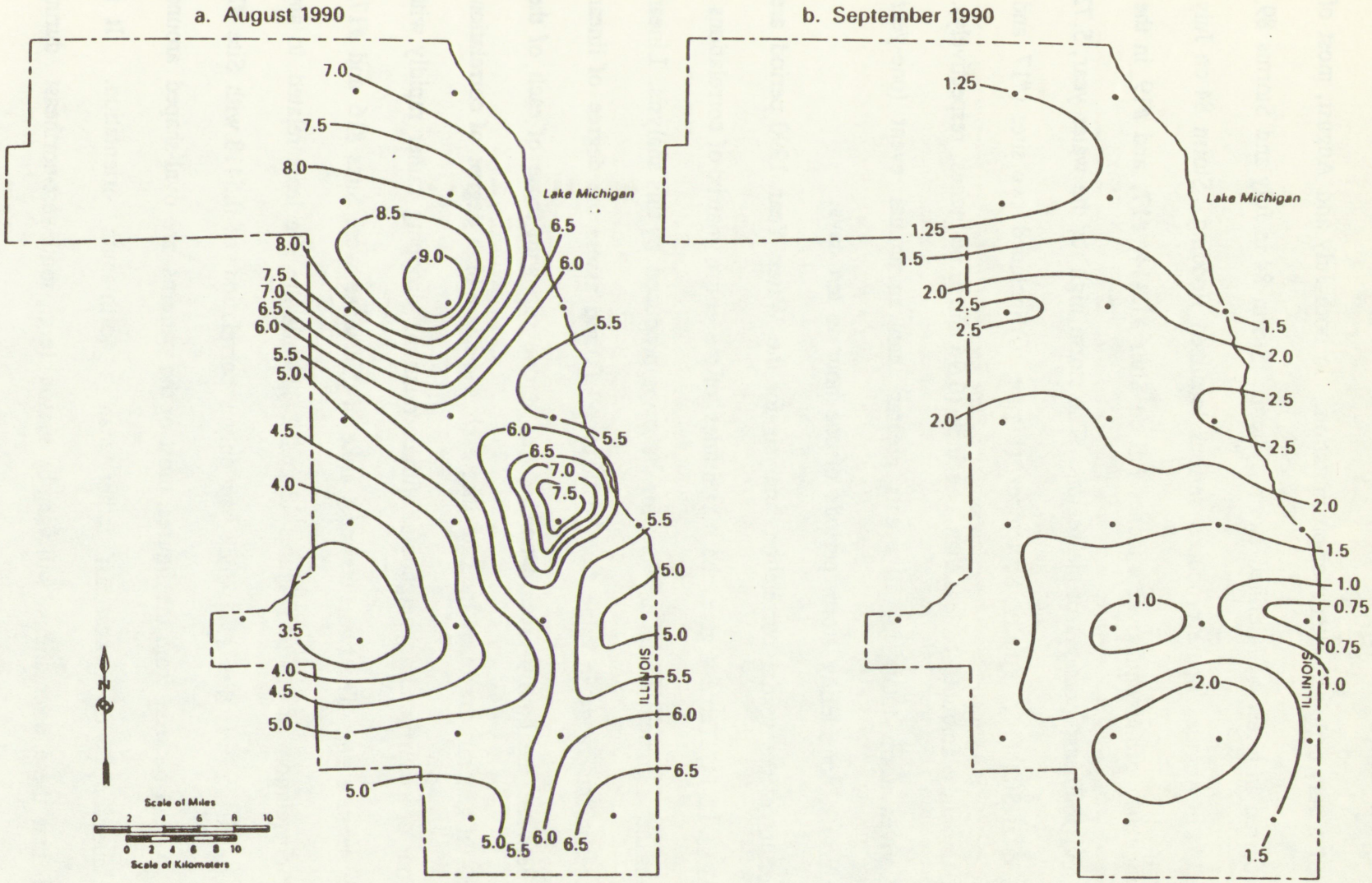


Figure 10. Precipitation patterns (inches) for August 1990 (Panel a) and September 1990 (Panel b). Dots indicate network sites.

to the previously mentioned water year high of 9.35 inches at Site #6 to the north. This month displayed a very cellular pattern of precipitation. In both July and August, most of the rainfall occurred in a few well-defined storm systems: Storm 84 in July and Storms 89, 90, 91, 92, and 94 in August. Some produced serious localized flooding. Storm 84 on July 19-20 was particularly prominent; over 4 inches fell on Sites #12-14, #17, and #19 in the south central and southeast portions of the region. The storm high for the water year, 5.72 inches, occurred at Site #14. Finally, September (Figure 10) included two sites (#17 and #19) that had monthly amounts of less than one inch (0.96 and 0.66 inches, respectively). Appendix IV gives storm totals for all events greater than an annual event (one-year recurrence interval), considering storm periods of one hour to ten days.

The results of the spatial correlation analysis for the Water Year 1990 period are shown in Figures 12-14. Table 4 gives the magnitudes of the entire matrix of correlations. Figure 11 gives the alignment of the network for the maps produced by this analysis. Linear correlation coefficients range in value from -1.00 to +1.00 and assess the degree of linear relationship between the hourly precipitation values at one site with those of each of the other 24 sites. In general, the maps indicate that each site has a high degree of correlation with one to four of its surrounding neighbors, with correlations dropping rather rapidly with distance away from a site. The two sites most alike in this sense were Sites #16 and #17, with a linear correlation of 0.8766 (Figures 13 and 14), while the site least related to any others in the network was Site #1, which had only a correlation of 0.5418 with Site #2 (Figure 12). As can be seen from the figures, most of the patterns are oval-shaped around the site in question, with a west-east rather than a north-south orientation. It is hypothesized that these orientations will vary by season (e.g., southwest-northeast during

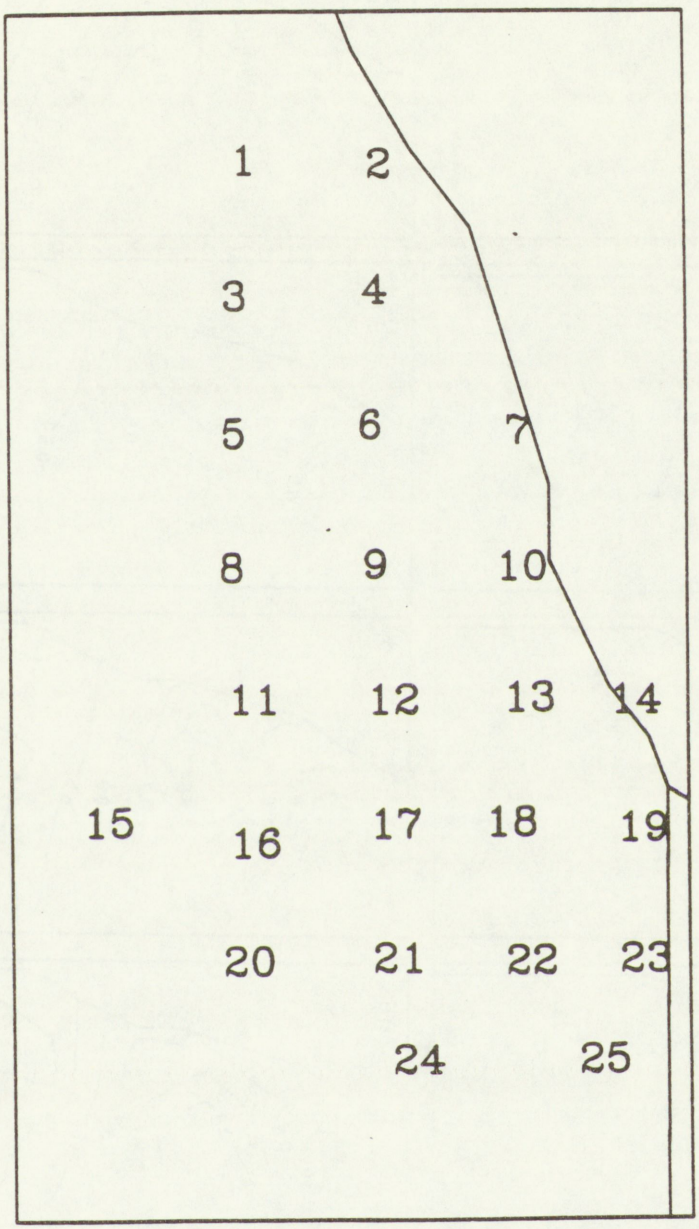


Figure 11. Computer-drawn configuration of current network layout, compatible with Figures 12-14 and Figure 16.

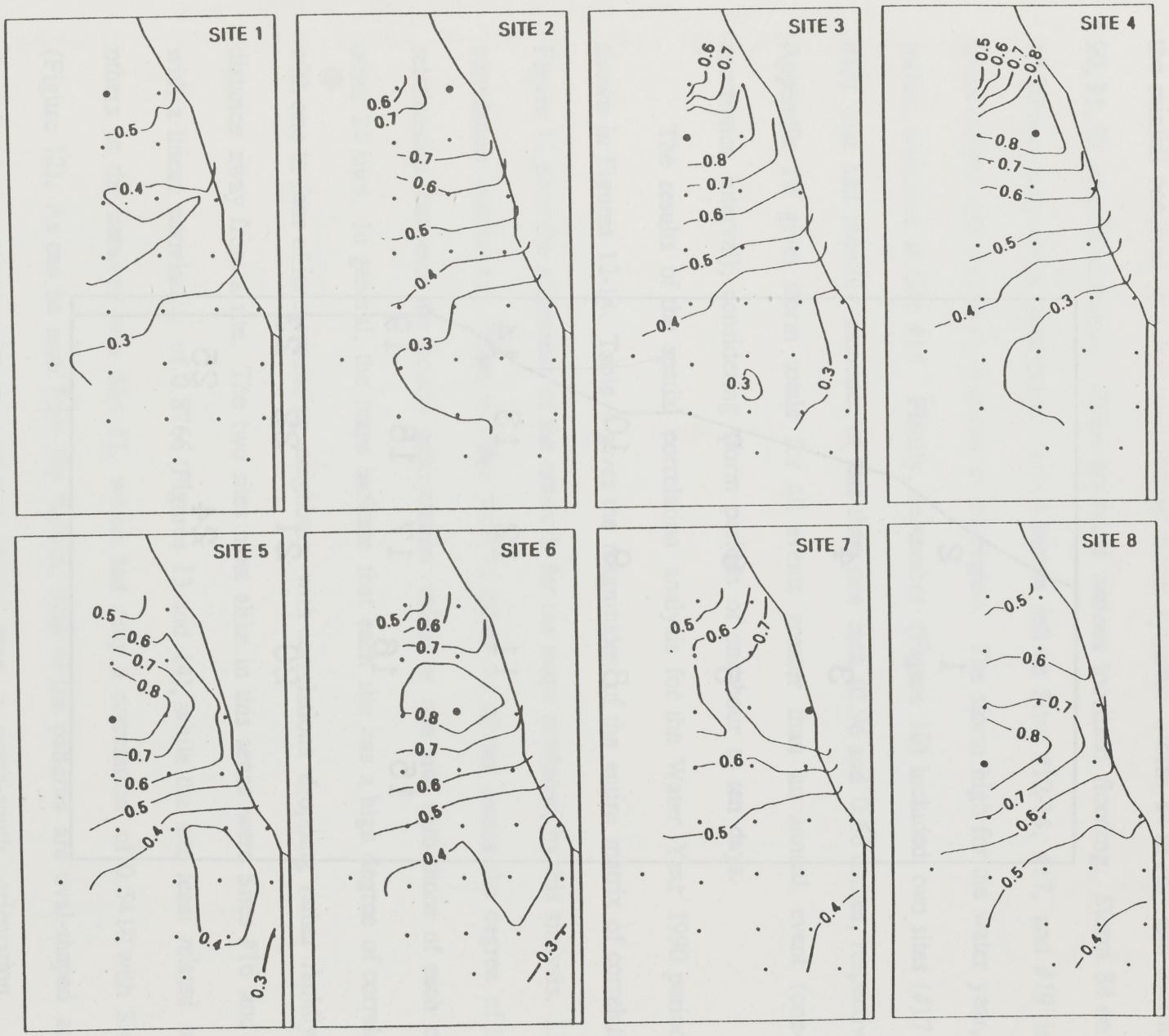


Figure 12. Correlation coefficient patterns for Sites #1-#8. Isopleths (no units) represent linear correlation magnitudes with the labeled site, which is indicated on the map by a large dot. Smaller dots indicate the other sites.



Figure 13. Correlation coefficient patterns for Sites #9-#16. Isopleths (no units) represent linear correlation magnitudes with the labeled site, which is indicated on the map by a large dot. Smaller dots indicate the other sites.



Figure 14. Correlation coefficient patterns for Sites #17-#25. Isopleths (no units) represent linear correlation magnitudes with the labeled site, which is indicated on the map by a large dot. Smaller dots indicate the other sites.

Table 4. Matrix of Correlation Coefficients for the Entire 25-site Network from Actual Data.

	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15
Gage 1	1.0000	0.5418	0.4951	0.4559	0.3931	0.3960	0.4005	0.4563	0.4034	0.3685	0.3712	0.2990	0.2941	0.2370	0.3109
Gage 2	0.5418	1.0000	0.7985	0.7941	0.5623	0.6316	0.5948	0.5001	0.4805	0.4568	0.4171	0.3001	0.2787	0.2515	0.3643
Gage 3	0.4951	0.7985	1.0000	0.8586	0.7120	0.7104	0.6080	0.5569	0.5326	0.5067	0.4507	0.3317	0.3041	0.2908	0.3947
Gage 4	0.4559	0.7941	0.8586	1.0000	0.6151	0.6786	0.6634	0.5462	0.5079	0.4923	0.4461	0.3199	0.3020	0.2872	0.3888
Gage 5	0.3931	0.5623	0.7120	0.6151	1.0000	0.8260	0.5827	0.6489	0.6806	0.6133	0.4981	0.3840	0.3730	0.3652	0.4360
Gage 6	0.3960	0.6316	0.7104	0.6786	0.8260	1.0000	0.7722	0.6794	0.6752	0.6374	0.5295	0.4461	0.4133	0.3970	0.4348
Gage 7	0.4005	0.5948	0.6080	0.6634	0.5827	0.7722	1.0000	0.6123	0.6070	0.6676	0.5548	0.4972	0.4488	0.4375	0.4581
Gage 8	0.4563	0.5001	0.5569	0.5462	0.6489	0.6794	0.6123	1.0000	0.8356	0.6756	0.7402	0.5984	0.5260	0.5056	0.5953
Gage 9	0.4034	0.4805	0.5326	0.5079	0.6806	0.6752	0.6070	0.8356	1.0000	0.7785	0.7183	0.6700	0.6711	0.6122	0.5960
Gage 10	0.3685	0.4568	0.5067	0.4923	0.6133	0.6374	0.6676	0.6756	0.7785	1.0000	0.6448	0.6758	0.6406	0.6288	0.5928
Gage 11	0.3712	0.4171	0.4507	0.4461	0.4981	0.5295	0.5548	0.7402	0.7183	0.6448	1.0000	0.8162	0.6466	0.6730	0.7772
Gage 12	0.2990	0.3001	0.3317	0.3199	0.3840	0.4461	0.4972	0.5984	0.6700	0.6758	0.8162	1.0000	0.7668	0.7760	0.6601
Gage 13	0.2941	0.2787	0.3041	0.3020	0.3730	0.4133	0.4488	0.5260	0.6711	0.6406	0.6466	0.7668	1.0000	0.8260	0.5647
Gage 14	0.2370	0.2515	0.2908	0.2872	0.3652	0.3970	0.4375	0.5056	0.6122	0.6288	0.6730	0.7760	0.8260	1.0000	0.5889
Gage 15	0.3109	0.3643	0.3947	0.3888	0.4360	0.4348	0.4581	0.5953	0.5960	0.5928	0.7772	0.6601	0.5647	0.5889	1.0000
Gage 16	0.2778	0.2893	0.3244	0.3130	0.3792	0.3835	0.4237	0.5339	0.5836	0.5651	0.7172	0.7042	0.6305	0.7377	0.7600
Gage 17	0.2649	0.2821	0.3027	0.2888	0.3289	0.3581	0.4349	0.4877	0.5369	0.5439	0.6529	0.7004	0.6349	0.7385	0.6416
Gage 18	0.2648	0.2712	0.3133	0.2732	0.4768	0.4648	0.4779	0.4588	0.5878	0.5526	0.5925	0.6600	0.6260	0.7199	0.6136
Gage 19	0.2681	0.2716	0.2928	0.2849	0.3619	0.3851	0.4231	0.4871	0.5967	0.5250	0.6042	0.6700	0.6603	0.7733	0.5834
Gage 20	0.2797	0.3132	0.3421	0.3156	0.3605	0.3725	0.4415	0.5032	0.5249	0.5310	0.5837	0.5571	0.5068	0.5318	0.6403
Gage 21	0.2617	0.2753	0.2950	0.2781	0.3301	0.3352	0.4257	0.4356	0.4758	0.5319	0.4976	0.4890	0.4690	0.4781	0.5714
Gage 22	0.2757	0.2787	0.3146	0.2679	0.4926	0.4111	0.4155	0.4208	0.5108	0.4971	0.4552	0.4498	0.4284	0.4155	0.4697
Gage 23	0.2751	0.2858	0.2894	0.2617	0.2898	0.2897	0.3792	0.3765	0.4085	0.4296	0.4161	0.4094	0.4074	0.4174	0.4185
Gage 24	0.2911	0.3308	0.3398	0.3206	0.3269	0.3582	0.4883	0.4557	0.5160	0.5326	0.5151	0.5095	0.4813	0.4815	0.5384
Gage 25	0.2290	0.2354	0.2467	0.2301	0.3009	0.2826	0.3434	0.3322	0.3843	0.4057	0.3810	0.3923	0.3662	0.3678	0.4026

	Gage 16	Gage 17	Gage 18	Gage 19	Gage 20	Gage 21	Gage 22	Gage 23	Gage 24	Gage 25
Gage 1	0.2778	0.2649	0.2648	0.2681	0.2797	0.2617	0.2757	0.2751	0.2911	0.2290
Gage 2	0.2893	0.2821	0.2712	0.2716	0.3132	0.2753	0.2787	0.2858	0.3308	0.2354
Gage 3	0.3244	0.3027	0.3133	0.2928	0.3421	0.2950	0.3146	0.2894	0.3398	0.2467
Gage 4	0.3130	0.2888	0.2732	0.2849	0.3156	0.2781	0.2679	0.2617	0.3206	0.2301
Gage 5	0.3792	0.3289	0.4768	0.3619	0.3605	0.3301	0.4926	0.2898	0.3269	0.3009
Gage 6	0.3835	0.3581	0.4648	0.3851	0.3725	0.3352	0.4111	0.2897	0.3582	0.2826
Gage 7	0.4237	0.4349	0.4779	0.4231	0.4415	0.4257	0.4155	0.3792	0.4883	0.3434
Gage 8	0.5339	0.4877	0.4588	0.4871	0.5032	0.4356	0.4208	0.3765	0.4557	0.3322
Gage 9	0.5836	0.5369	0.5878	0.5967	0.5249	0.4758	0.5108	0.4085	0.5160	0.3843
Gage 10	0.5651	0.5439	0.5526	0.5250	0.5310	0.5319	0.4971	0.4296	0.5326	0.4057
Gage 11	0.7172	0.6529	0.5925	0.6042	0.5837	0.4976	0.4552	0.4161	0.5151	0.3810
Gage 12	0.7042	0.7004	0.6600	0.6700	0.5571	0.4890	0.4498	0.4094	0.5095	0.3923
Gage 13	0.6305	0.6349	0.6260	0.6603	0.5068	0.4690	0.4284	0.4074	0.4813	0.3662
Gage 14	0.7377	0.7385	0.7199	0.7733	0.5318	0.4781	0.4155	0.4174	0.4815	0.3678
Gage 15	0.7600	0.6416	0.6136	0.5834	0.6403	0.5714	0.4697	0.4185	0.5384	0.4026
Gage 16	1.0000	0.8766	0.7554	0.7629	0.7582	0.6528	0.5485	0.4718	0.5981	0.4471
Gage 17	0.8766	1.0000	0.7852	0.7719	0.7256	0.6171	0.5102	0.4586	0.5897	0.4221
Gage 18	0.7554	0.7852	1.0000	0.8513	0.6728	0.6518	0.7079	0.5768	0.6201	0.5372
Gage 19	0.7629	0.7719	0.8513	1.0000	0.7087	0.6613	0.6302	0.6285	0.6401	0.5937
Gage 20	0.7582	0.7256	0.6728	0.7087	1.0000	0.8457	0.7247	0.6213	0.7545	0.6022
Gage 21	0.6528	0.6171	0.6518	0.6613	0.8457	1.0000	0.7864	0.6844	0.8003	0.7046
Gage 22	0.5485	0.5102	0.7079	0.6302	0.7247	0.7864	1.0000	0.7069	0.7423	0.6832
Gage 23	0.4718	0.4586	0.5768	0.6285	0.6213	0.6844	0.7069	1.0000	0.7089	0.7775
Gage 24	0.5981	0.5897	0.6201	0.6401	0.7545	0.8003	0.7423	0.7089	1.0000	0.7025
Gage 25	0.4471	0.4221	0.5372	0.5937	0.6022	0.7046	0.6832	0.7775	0.7025	1.0000

spring and summer), reflecting the seasonal differences in the region's preferred storm tracks. A study of this phenomenon is needed. The west-east orientations for the water year shown here likely represent a combination of patterns. There is a general low relationship (correlations less than 0.40 or even 0.30) between far northern Sites #1-#7 and far southern Sites #15-#25 (see Figures 12-14 and Table 4). This demonstrates the great precipitation variability that can occur over relatively small areas given the physics controlling the scale of precipitation. Though water year amounts can be similar, the timing of the precipitation events (storms) affecting the far northern and southern regions was likely quite different. If seasonal patterns were constructed, time-lag relationships may be revealed that could give further insight into the region's precipitation variability.

Several questions arise.

- Is the 6-mile spacing used between raingages satisfactory, more than adequate, or inadequate?
- Is there a sufficient degree of autocorrelation in the individual sites' precipitation time-series to inflate the significance of the correlation values?
- Would these patterns be replicated by other water years or by "reshuffling" the current water year's data, or are they simply some random statistical artifact?

Each question will be addressed in turn and in context to results from two papers (Huff and Shipp, 1969; Huff, 1979) where spatial correlations were computed on raingage networks similar to the present one.

Concerning the adequacy of grid spacing of the network, it is suggested that if neighboring stations were very highly interrelated, the correlations between them would have been much closer to 1.0 (e.g., at least 0.95 or higher). However, no inter-site correlation

exceeded 0.8766, corresponding to a shared variance of 76.84 percent (Sites #16 and #17). Only 9 of 300 possible site pairings produced correlation values at or above 0.80 corresponding to a shared variance of 64 percent (refer to Table 4 for all correlation values). Site #1, in the northwest corner of the domain, had as its highest inter-site correlation just 0.5418 with Site #2 (shared variance of only 29.35 percent), as previously mentioned. All other sites had at least one inter-site correlation of 0.70 or higher (approximately 50 percent of the shared variance). These numbers are not overwhelmingly high for a rather fine time resolution (hourly). All of this suggests that when considering an entire water year, the 6-mile grid spacing is probably inadequate or just barely adequate to properly capture the nature of the area's precipitation, and that it is inadequate in some of the network's fringe regions. If computed for a seasonal data breakdown, the correlations would most likely be lower during the summer months when convective-type precipitation systems, which are more highly variable in space and time, dominate, and higher during winter when larger-scale systems provide most of the area's precipitation, which tends to be more widespread and continuous. It is presumed that a tighter grid spacing, advocated by Huff (1970, 1979) for convective-type precipitation, would produce higher inter-site correlations, thereby allowing a more detailed examination of the precipitation that falls on Cook County.

Indeed, Huff and Shipp (1969) and Huff (1979) examined the spatial and temporal correlation patterns of precipitation over all of Illinois and for some specific raingage networks deployed. Correlations of annual precipitation between Chicago and 35 sites across Illinois for 1906-1955 displayed a southwest-northeast orientation, reflecting prevailing storm movement, with correlation maxima above 0.80 in the northeastern Illinois region.

Correlations were also computed between 49 raingages in the East Central Illinois Network (ECIN) operating from 1955-1967, which encompassed an area of 400 square miles. Raingages were spaced approximately 3 miles apart. Data were subdivided into the warm season (May-September) and the cool season (October-April), and precipitation events were further subdivided into basic synoptic weather categories such as low-center passages, frontal storms, and air-mass storms. For the center gage in this network, correlations between it and all other stations were well over 0.90 for low centers, with a large area of 0.98 and higher in the center of the domain. For frontal storms, a large area of 0.95 and greater correlations was observed, but values dropped to 0.75 at the boundaries of the network. For convective air-mass storms, a small area of 0.95 and higher correlations existed near the center raingage, but the correlations dropped to less than 0.60 in some parts of the network near its boundaries. When precipitation was subdivided into thunderstorms/rainshowers and steady rain, the patterns looked very much like those for fronts and low centers, respectively. Correlations were also generally less for decreasing storm duration (below 12 hours) and increasing storm duration (above 12 hours). Thus, the correlations from the ECIN, which had a tighter grid spacing than the Cook County network uses, are larger than those for the new network. It is suggested that by using a tighter spacing in the new network, correlations could also be expected to be higher, explaining more of the precipitation variance. Huff (1979) and Huff and Shipp (1969) also found that correlations were lower for convective precipitation than for more continuous and widespread types, and thus were lower during the warm season. This characteristic would also be expected for data from the new network if stratified like this.

Huff and Shipp (1969) and Huff (1979) were further able to determine raingage

sampling requirements for a desired level of explained precipitation variance. It was found that warm-season precipitation requires a substantially greater density than cool-season precipitation. For all storms, 90 percent of the precipitation variance (correlation of 0.95) can be explained by a gage spacing of 2 miles during the warm season and only 6 miles during the cool season. Thus, in the new raingage network, some information is being lost during the warm season by using the 6-mile spacing, which theoretically explains about 78 percent of the precipitation variance (correlation of 0.88). But during the cool season this spacing is adequate to explain 90 percent of the variance (correlation of 0.95). The largest correlations found in the new network for the entire water year were nearer the warm-season level. For particular precipitation types, the recommended grid spacings vary. Air-mass storms require a one-mile grid spacing during both seasons to adequately capture this convective event; frontal storms require grid spacings of 2 and 4 miles during the warm and cool seasons, respectively; and low centers only require spacings of 10 and 8 miles, respectively. Therefore, it appears that it would also be highly instructive to subdivide the current data not only by season but also by storm type to more accurately determine how effective the new network is at capturing the Chicago region's precipitation.

Concerning the second question, autocorrelation within a time-series can artificially inflate the significance of correlation coefficients computed between time-series. By falsely assuming that the individual observations in the time-series are independent, a large correlation may have much less significance than was computed initially. In fact, the "effective" sample size (size of the sample's independent information) of the time-series may be much smaller than the actual number of data observations due to autocorrelation, rendering the result of a significance calculation much less impressive. This study uses 8,760

hourly data values for each of the 25 sites. Using this value in a one-sided t-test produced significance levels of less than 0.000001 for every unique pairing of sites (300 total). In other words, each correlation is at least 99.99999 percent significant. Because significance typically is compared to the 0.05 or 0.01 levels (95 or 99 percent significant), all of the correlations computed here on the actual data are extremely significant. The amount of autocorrelation in each time-series can be assessed and an effective sample size computed by comparing the autocorrelation functions computed from each of the 25 precipitation time-series to those functions computed from a series of pseudorandom standard normal variates. These autocorrelation functions were plotted (the only example shown is Site #4 in Figure 15). The degree of autocorrelation for each time-series was found by first noting the largest deviation on the standard normal autocorrelation function plot, from the level of "white-noise" or randomness associated with a sample size of 8,760 observations (one divided by the square root of 8,760, or 0.0107). This deviation was 0.0360. Each of the 25 autocorrelation function plots was then analyzed to find the first occurrence of the difference of the autocorrelation function with the white-noise level dropping below 0.0360. The lag there, minus one, denotes the length of the autocorrelation in the series, in hours. Lags of up to 48 hours were computed. Other subsequent large differences were also noted and will be discussed below.

All stations had autocorrelations within their hourly precipitation data ranging from 6 to 12 hours (Table 5). Thus, the effective sample size is about an order of magnitude less than the sample size of 8,760 (8,760 divided by the lag). In other words, the amount of independent information contained in each time-series is much less than the total number of observations would indicate. Within this range of autocorrelation, succeeding values are

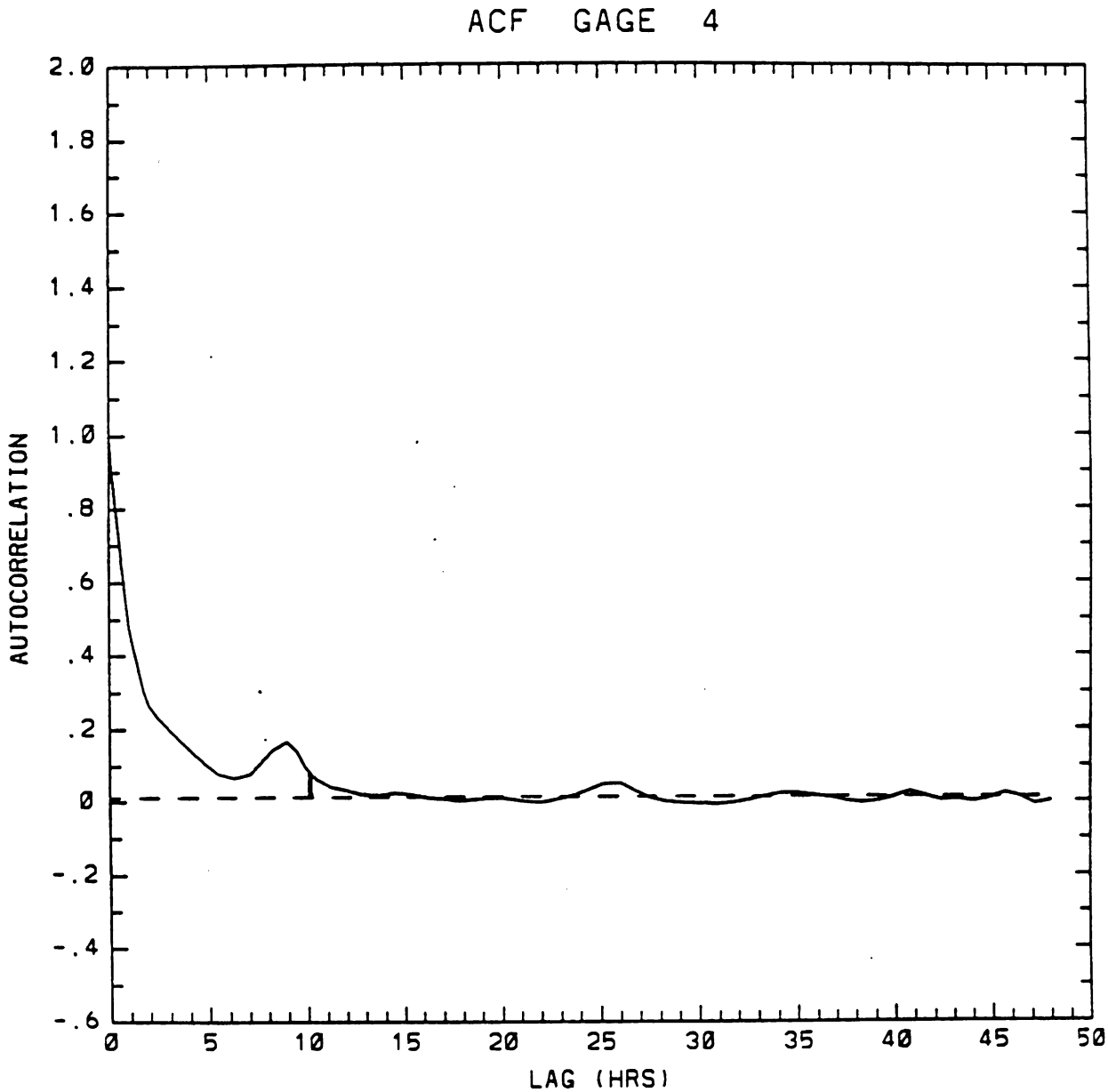


Figure 15. Autocorrelation function for Site #4 (solid curve). Dashed line represents the level of white noise associated with a sample size of 8,760 hourly data values. Vertical line between the autocorrelation function and white-noise line at lag 10 represents the last lag (hour) of autocorrelation greater than the maximum found from a similar function for pseudorandom standard normal variates.

Table 5. Autocorrelations Found within the Hourly Precipitation Time-series, in Hours.

Site Number	Autocorrelation Lag	Other Lags of Note
1	7	21-24/37-38
2	10	8/18/23-26
3	11	9/26/40
4	10	9/25
5	8	25/43
6	9	40
7	12	9
8	10	7-8
9	9	7/12
10	11	6
11	9	5
12	9	6-7
13	8	6-7
14	8	6-7
15	8	6/31
16	9	7/12/32
17	8	7/15
18	8	6-7
19	9	6-7
20	7	---
21	7	---
22	6	15/41
23	8	---
24	9	14
25	8	---

dependent on previous ones, which is meteorologically represented by the passing of a storm through the area. Significance values computed for the site-to-site correlations are thus inflated: the correlations themselves are likely less significant statistically than the traditional t-tests would indicate and may not be terribly physically significant. Paragraphs at the end of this section will discuss a bootstrapping method for determining the robustness of the correlation patterns shown in Figures 12-14, a technique that will attempt to replicate the patterns given by the actual data while trying to minimize the autocorrelation within the data.

The autocorrelations within the precipitation data tended to be longer (9-12 hours) in the northern half of the study region, roughly defined as Sites #11 and #12 northward. Sites to the south had autocorrelations generally in the range of 6 to 9 hours. A possible explanation is that storms passing through the northern half of the region either had longer durations or were slower moving than those traversing the southern part of the network, which may be the result of a lake effect.

Additional interesting features were indicated in many of the autocorrelation plots (Table 5). At nearly all of the sites north of those bounded by Sites #20-#25, a relatively high autocorrelation "peak" occurred within the autocorrelation lag period, usually centered on hours 5 to 9. These peaks seemed to be more pronounced at sites closer to Lake Michigan, Sites #2, #4 (see Figure 15), #7, #13, #14, and #19, and could be indicative of some type of interaction between passing storms and the lake, perhaps the collision of storm flows with lake flows that were present. It should be noted that water year totals at some of the sites nearer the lake were lower than those at more inland locations (Figure 4), particularly in the northern part of the network. The early-peak phenomenon also occurred

to some extent at Sites #9, #12, #16, #17, and #18 in the south central part of the network. Less pronounced peaks in autocorrelation magnitude also occurred around hours 12-15 at a few scattered sites (Table 5), and also at hours 21-26, 31-32, and 37-43. The peaks during hours 21-26, which generally occurred at the northernmost sites (#1-#5), are likely indicative of some diurnal cycle, while those occurring at the other lags require further investigation to determine their causes (e.g., analysis of weather maps for the period).

To answer the final question, a technique called "bootstrapping" (e.g., Wu, 1986) was used to determine the robustness of the inter-site correlation patterns if the data covering the 8,760 hours of the water year were statistically reshuffled. As mentioned before, this technique should help mitigate the presence of autocorrelation in the data while maintaining the integrity of the data observations over the network at any particular hour. If the same general patterns reappear after each reshuffling of the data, then the original pattern can be said to be fairly reliable, or robust.

Using a uniform random number generator over the range of integer values (1 to 8,760) a number was drawn from within that range 8,760 times, placing the drawn number back for consideration each time ("sampling with replacement"), creating a time-series of integer numbers. Using this series of integers, a reshuffling of the precipitation data over the water year could be determined. For example, if the integer 567 was drawn as the first uniform random number, all 25 precipitation values for hour 567 of the original data series were assigned to hour 1. If integer 2,306 was drawn second, the precipitation values for hour 2,306 in the original data series were assigned to hour 2, and so on, thus creating a new time-series of precipitation data over the network, with autocorrelation theoretically randomized. This process was carried out 10 times to produce 10 reordered data samples

on which correlation coefficients were computed to see if the correlation patterns and magnitudes produced by the actual precipitation data set held up, or if they were simply some statistical artifact. If they did hold, this would provide evidence for the robustness of the patterns produced by the actual data, in spite of autocorrelation in the data. This analysis was performed for the entire water year but should also be performed for seasonal periods within the water year.

Ten sets of 25 correlation patterns each, similar to the set shown in Figures 12-14, were computed and drawn from the bootstrapped data samples. General results will be discussed, but only those patterns for Site #17 are shown as examples (Figure 16). Overall, the patterns held up very well for most of the sites, the exceptions being in the extreme northern portion of the network (Sites #1-#3), at two of the Lake Michigan proximity locations (Sites #7 and #10), and some sites in the extreme southern part of the network (Sites #20 and #22). In a correlation magnitude sense, the highest inter-site correlations for each site in the bootstrapped samples fell within some range around the highest values found from the actual data, with more values slightly below the actual ones, indicative of a slight inflating effect in the actual data by autocorrelation. Thus, the effect of the bootstrapping, which attempted to randomize the autocorrelation of the time-series, was to not significantly dilute the correlations in the actual data, so it appears that most of the correlation patterns and magnitudes in the actual data are "real". The sites that had the most resilient correlation structures are generally in the central part of the network, including Site #17 (Figure 16). Site #1 seems to have had the poorest correlation structure of any raingage location. The correlation patterns within close proximity of a site were generally elongated from west-to-east, rather than north-to-south, as in the actual data.

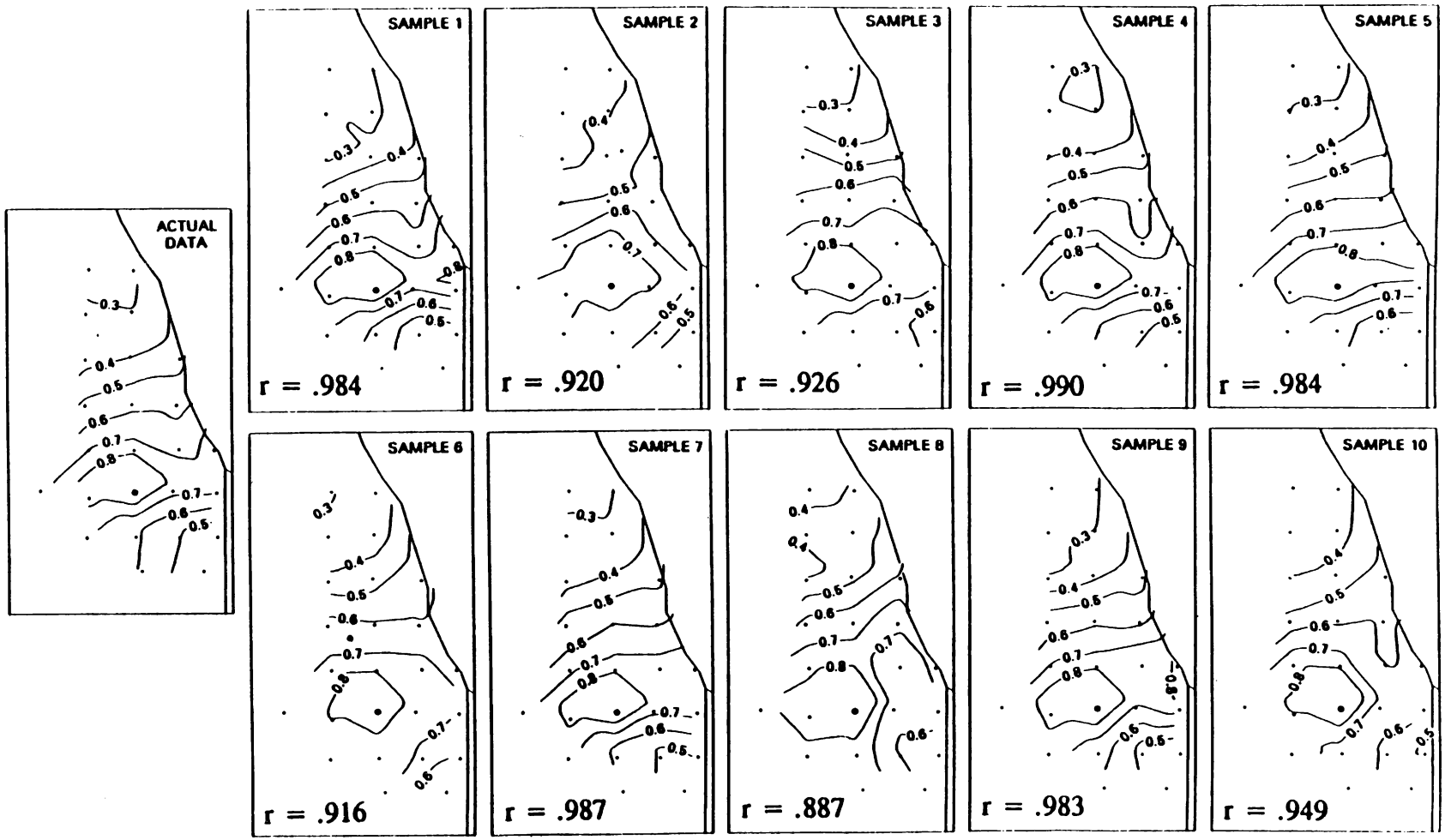


Figure 16. Correlation coefficient patterns for Site #17 from the original data (left panel) and from ten replication samples extracted from the original data. Isopleths (no units) represent linear correlation magnitudes with the labeled site, which is indicated on the map by a large dot. Smaller dots indicate the other sites. Numbers in lower left corners of the sample panels indicate the correlation

Again, it would be highly instructive to repeat this exercise on a seasonal basis.

Looking specifically at Site #17 (Figure 16), which had the highest inter-site correlation with a neighboring raingage in the network (Site #16), the pattern given by the actual data is quite well duplicated by most of the ten bootstrapped data sets, particularly by samples 1, 4, 5, 7, and 9. The values given in the lower left corner of each of the ten replication map panels are the correlations of the respective sample map with the map from the actual data. As can be seen, samples 1, 4, 5, 7, and 9 all replicated the actual pattern with correlations of 0.983 or higher. Sample 8 gave the least duplication, with a correlation of "only" 0.887. The duplication in the patterns decreased slightly with distance away from Site #17, as was expected, since most of the correlations dropped fairly rapidly with distance away from a site throughout this network. Samples 4 and 7 gave remarkably good pattern duplication (correlations of 0.990 and 0.987, respectively).

Thus, the results of the statistical analyses performed revealed that the correlation patterns and magnitudes found were fairly robust despite the presence of autocorrelation in the data. The 6-mile grid spacing between raingages is probably the largest one able to resolve the detail in the precipitation of the region. The correlation patterns produced give some sense, on a water-year basis, of how storm patterns are aligned over the network, so that a finer time resolution analysis, for seasons or even months, would likely yield further important information about the precipitation affecting the Cook County region.

6. EVALUATION OF THE NETWORK TO DATE

Some individual station comparisons were made between values at network sites and available totals from nearby NWS and CC raingages to evaluate the effectiveness of the network's purported better catchment ability. Unfortunately, most of the improvement shown by the network would have been revealed in comparisons with data from MWRDGC sites, which were not available. The NWS and CC sites analyzed include Midway 3 SW, Chicago O'Hare Airport, Chicago Botanical Gardens, Chicago University, Park Forest (NWS), Mayfair Pumping Station, Springfield Pumping Station, Jardine Water Purification Plant, South Water Purification Plant, and Roseland Pumping Station (CC). See Figure 1 for the locations of all of these sites except for Chicago Botanical Gardens in extreme northeastern Cook County and Jardine Water Purification Plant, which is very near the Erie Sanitary District Office (SDO). Plots of monthly precipitation totals were compared at each of these raingages and the nearest corresponding network site(s) for the water year for the NWS gages (Figure 17), and for the period October 1989 - April 1990 for the CC gages (Figure 18).

With the exception of a few noticeable differences, trends and amounts of NWS and network raingages agreed closely (Figure 17). Discrepancies of note occurred in February 1990 between Chicago O'Hare AP and Sites #3 and #5, in July-September 1990 between Midway 3 SW and Site #12, and during much of the period March-September 1990 between Chicago University and Sites #13 and #14 when Chicago O'Hare AP, Midway 3 SW, and Chicago University, respectively, had less precipitation. Other discrepancies occurred in February 1990 between Chicago Botanical Gardens and Site #2, and in May-

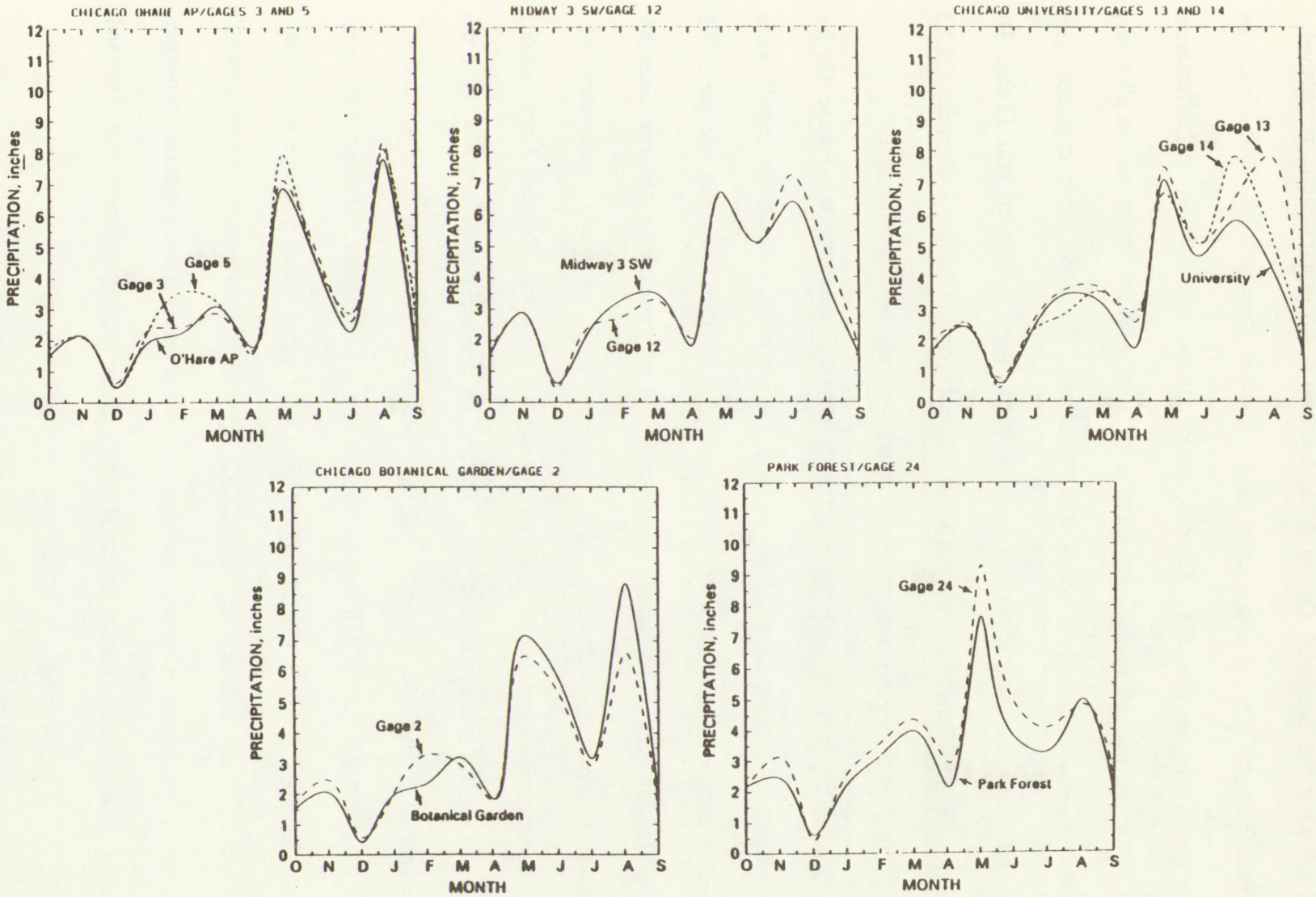


Figure 17. Comparisons of monthly precipitation amounts (inches) between network raingages and nearby NWS raingages, spanning the entire Water Year 1990 period.

September 1990 when the Botanical Gardens had less and more precipitation, respectively. Finally, the Park Forest NWS site consistently underestimated precipitation throughout the water year in comparison to Site #24. Overall, these comparisons reveal that differences were usually within the natural spatial variability associated with precipitation. In all cases, except for the Chicago Botanical Gardens (39.95 inches), the NWS sites captured less precipitation than did their nearby network counterparts over the water year (Table 3): Chicago O'Hare AP (35.59 inches); Midway 3 SW (39.60 inches); Chicago University (38.73 inches); and Park Forest (approximately 38.50 inches [March 1990 is missing]).

Comparisons between network raingages and their CC counterparts (Figure 18) for October 1989-April 1990 were much more variable, as expected. Although trends were generally similar between sites, amounts varied much more than they did for the NWS comparisons. Mayfair PS overestimated precipitation in comparison to network Sites #4 and #6 during October 1989-January 1990; Jardine WPP generally underestimated precipitation in comparison to Sites #7 and #10 throughout the entire October-April period; and Roseland PS greatly underestimated precipitation in comparison to Site #18 over the period. Amounts at Springfield PS tended to vary between amounts at Sites #6 and #7, while Site #14 and South WPP, which are located near each other, showed very good agreement. It had been noted in the adjustment work for other water years (e.g., Peppler, 1991) that some CC sites (and several MWRDGC ones) had consistently underestimated precipitation, due to both poor raingage exposure and to the use of tipping-bucket raingages, which inherently tend to underestimate precipitation. These months indicate that Roseland PS and Jardine WPP particularly underestimated precipitation.

Data records for the entire water year from the CC and previously unavailable

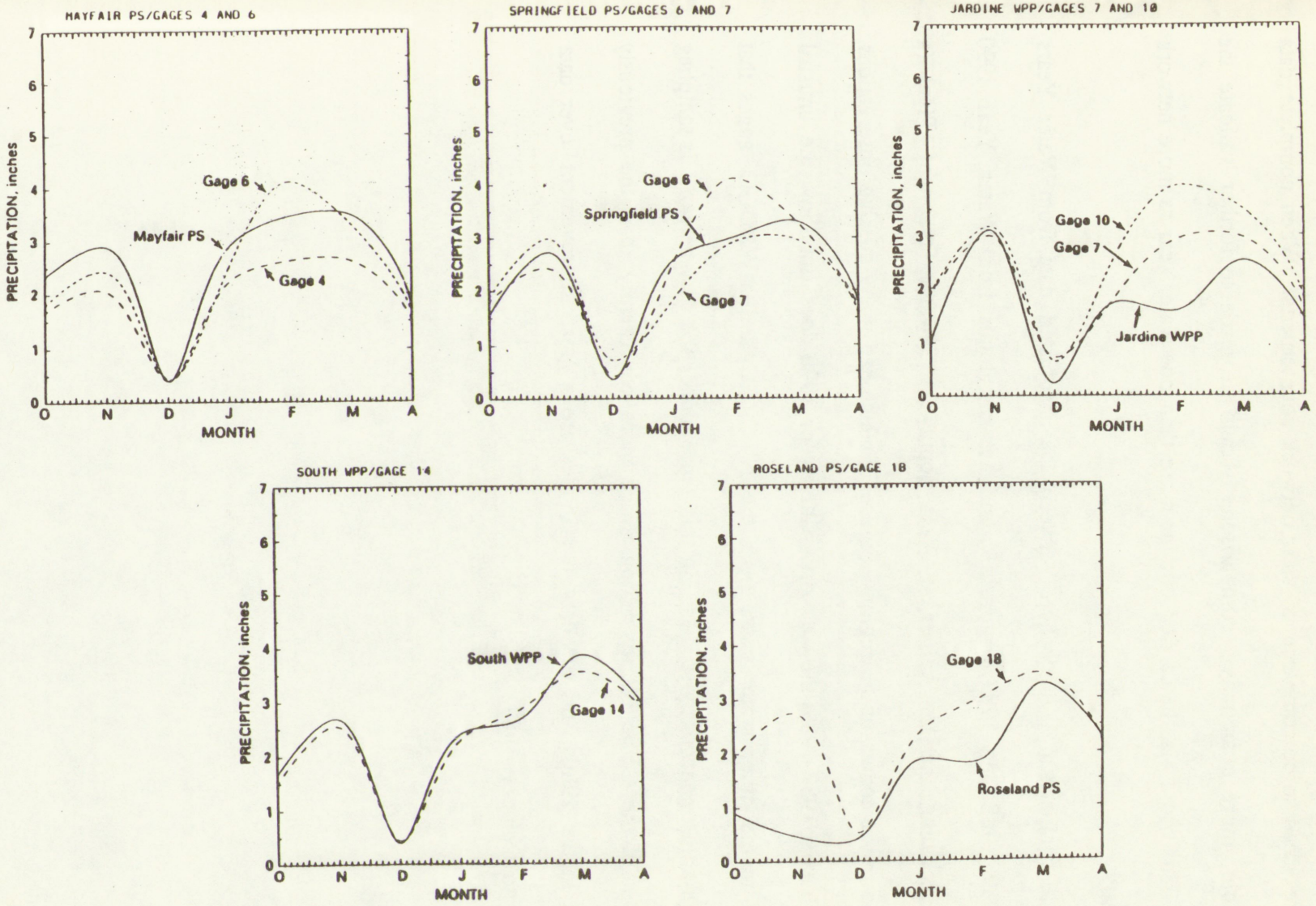


Figure 18. Comparisons of monthly precipitation amounts (inches) between network raingages and nearby CC raingages for the period October 1989-April 1990. Jardine WPP is located just east of Erie SDO (see Figure 1).

MWRDGC sites need to be compared to their network counterparts. When complete data sets are available, more quantitative comparisons should be made to further validate the network sites. This could also be carried out for more than one water year as more network data is collected.

One final comparison is made here between the unadjusted data from Water Years 1984, 1985, 1987, 1988, and 1989 (Figure 19) and network totals from Water Year 1990 (Figure 4). Excluding the low values at West Southwest Sewage Treatment Plant (a MWRDGC site in the center of the region - Figure 1) which had many missing observations during Water Years 1987-1989, the network pattern for 1990 does **not** show the unusual patterns found in the other water years from the NWS, CC, and MWRDGC gages that necessitated the adjustment of those data. In this sense alone, the new network is sampling the precipitation of the Cook County region **much** more consistently than the previously used gages did in the 1980s, and the data it provides are a great improvement over data collected prior to October 1989.

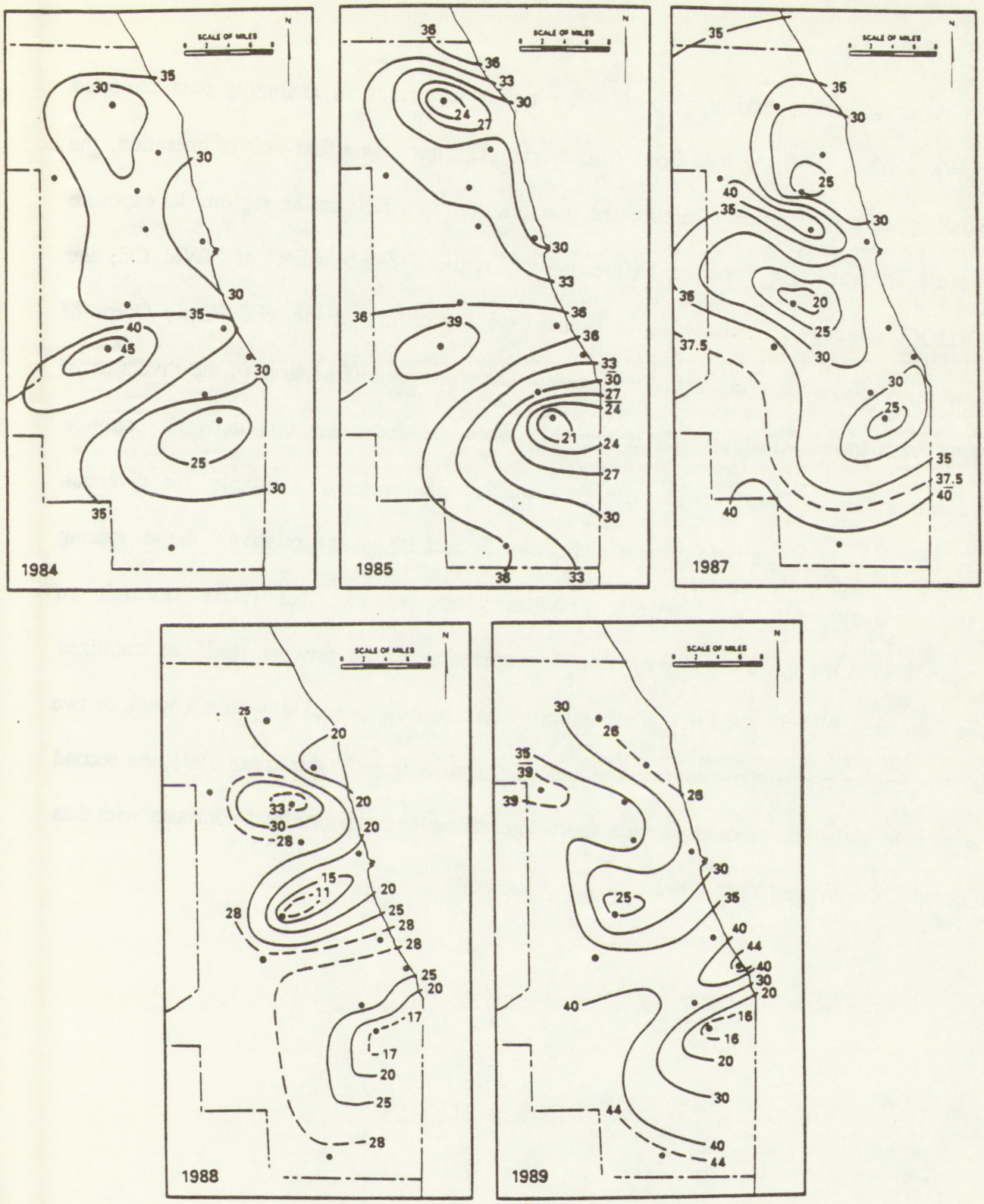


Figure 19. Raw precipitation patterns (inches) for Water Years 1984-1989 from NWS, CC, and MWRDGC raingages. Compare to Figure 4 for patterns from Water Year 1990 network raingages.

7. SUMMARY

After collecting, reducing, performing quality-control, and analyzing data collected from Water Year 1990, the new Cook County raingage network's first year of operation, the network appears to accurately capture the precipitation that falls on the region. Its exposure and areal coverage are superior to the previously used combination of NWS, CC, and MWRDGC raingages. This will greatly improve the ability of the U.S. Army Corps of Engineers, Chicago District, to properly assess the storm runoff portion of the diversion of waters from Lake Michigan. It is strongly recommended that this network continue operation indefinitely to provide the best quality precipitation available for diversion accounting. And, as has been shown in Section 5, because of the relatively dense spacing of the raingages, the new network provides great potential for future research on precipitation in the Cook County region. The operation of the network itself has stabilized since October 1989; final hourly precipitation data are now available within a week or two after the end of a calendar month. The analysis of data from Water Year 1991, the second year of data collection, should provide more interesting comparisons and contrasts with data collected from Water Year 1990.

8. ACKNOWLEDGEMENTS

This work was contracted by the U.S. Army Corps of Engineers, Chicago District, under Grant DACW 23-89-C-0005. Diane Peppler computed and plotted storm totals, disaggregated storm values interpolated at missing sites into hourly ones, and finally performed data entry and correction. Doug Ward established the digitizing system, including software, and made the digitization of charts an extremely simple task. His various computer assistance and advice are greatly appreciated. Linda Hascall skillfully drafted all figures, and Eva Kingston carefully edited the report. Statistical advice from Mike Richman was of great help in the analysis of results. Tim Lormand, U.S. Army Corps of Engineers, provided the support for this project and helpful suggestions, as well as editing the draft version of this report. Floyd Huff and Doug Jones were mainly responsible for bringing this project to fruition, and their continuing advice and editing of this report are greatly appreciated.

9. REFERENCES

Changnon, S. A. 1961: Precipitation contrasts between the Chicago urban area and an offshore station in southern Lake Michigan. Bulletin of the American Meteorological Society, 42, 1-10.

Changnon, S. A., 1968: Precipitation Climatology of Lake Michigan Basin. Illinois State Water Survey Bulletin 52, Champaign, 46 pp.

Changnon, S. A., 1980: Evidence of urban and lake influences on precipitation in the Chicago area. Journal of Applied Meteorology, 19, 1137-1159.

Changnon, S. A., 1984: Urban and lake effects on summer rainfall in the Chicago area. Physical Geography, 4 (2), 1-23.

Huff, F. A., 1967: Time distribution of rainfall in heavy storms. Water Resources Research, 3, 1007-1019.

Huff, F. A., 1970: Sampling errors in measurement of mean precipitation. Journal of Applied Meteorology, 9, 35-44.

Huff, F. A., 1979: Spatial and Temporal Correlation of Precipitation in Illinois. Illinois State Water Survey Circular 141, Champaign, 14 pp.

Huff, F. A., and J. R. Angel, 1989: Frequency Distributions and Hydroclimatic Characteristics of Heavy Rainstorms in Illinois. Illinois State Water Survey Bulletin 70, 177 pp.

Huff, F. A., and S. A. Changnon, 1973: Precipitation modification by major urban areas. Bulletin of the American Meteorological Society, 54, 1220-1232.

Huff, F. A., and W. L. Shipp, 1969: Spatial correlations of storm, monthly and seasonal precipitation. Journal of Applied Meteorology, 8, 542-550.

Huff, F. A., and J. L. Vogel, 1976: Hydrometeorology of Heavy Rainstorms in Chicago and Northeastern Illinois. Illinois State Water Survey Report of Investigation 82, 63 pp.

Jones, D. M. A., 1969: Effect of housing shape on the catch of recording gages. Monthly Weather Review, 97, 604-606.

Koschmieder, H, 1934: Methods and results of definite rain measurements. Monthly Weather Review, 62, 5.

Legates, D. R., and C. J. Willmott, 1990: Mean seasonal and spatial variability in gauge-corrected, global precipitation. International Journal of Climatology, 10, 111-127.

Lindroth, A., 1991: Reduced loss in precipitation measurements using a new wind shield for raingages. Journal of Atmospheric and Oceanic Technology, 8, 444-451.

Peppler, R. A., 1990: Reduction of 1987 Water Year Precipitation Data for Lake Michigan Diversion Accounting. Illinois State Water Survey Contract Report 498, Champaign, 16 pp.

Peppler, R. A., 1991: Reduction and Adjustment of Water Year 1988 and Water Year 1989 Precipitation Data for Lake Michigan Diversion Accounting. Illinois State Water Survey Contract Report 510, 24 pp.

Vogel, J. L., 1986: Significant Storm Distribution in Chicago 1949-1979. Illinois State Water Survey Contract Report 388, Champaign, 30 pp.

Vogel, J. L., 1988: An Examination of Chicago Precipitation Patterns for Water Year 1984. Illinois State Water Survey Contract Report 449, Champaign, 44 pp.

Vogel, J. L., 1989: Reduction of 1985 Water Year Precipitation Data for Chicago. Illinois State Water Survey Contract Report 459, Champaign, 15 pp.

Vogel, J. L., 1991: Reduction of 1986 Water Year Precipitation Data for Chicago. Illinois State Water Survey Contract Report, Champaign, in preparation.

Wu, C. F. J., 1986: Jackknife, bootstrap and other resampling methods in regression analysis. Annals of Statistics, 14, 1261-1350.

APPENDIX I

Contained below are descriptions of the 25 raingage network sites. All represent the current siting as of publication. Sites that have been relocated since the beginning of network operation in October 1989 are noted in the "Placement" section of the description. Descriptions are current as of July 1991.

SITE DESCRIPTION		
Site Number: 1		
County: Cook	Township: 42N	Range: 12E
Section: 20	Lat/Long: 42°06'38" / 87°52'05"	Quadrangle: Park Ridge
Property Owner: Mission Brook Sanitary District, Attn: John Tomaras		
Address: P.O. Box 2362, Northbrook, Illinois 60665		
Telephone: 708/272-2956		
Permission Date: September 14, 1989		
Installation Date: September 27, 1989		
Gage Mfrs. No.: 7378	Gage ID No.: 6561	Clock Mfrs. No.: E 7345
Placement: Placed in southeast corner of pump station lawn at southwest corner of intersection of Post and Cornflower Streets. Tri-State Tollway fence is just to the west. Enter area from west at Landwehr Road (north of Willow Road) at Sunset Ridge.		

SITE DESCRIPTION		
Site Number: 2		
County: Cook	Township: 42N	Range: 13E
Section: 19	Lat/Long: 42°06'28" / 87°45'05"	Quadrangle: Park Ridge
Property Owner: Winnetka Park District, Attn: Richard Blust		
Address: 510 Green Bay Road, Winnetka, Illinois 60093		
Telephone: 708/446-2397		
Permission Date: September 14, 1989		
Installation Date: October 3, 1989		
Gage Mfrs. No.: 7379	Gage ID No.: 6560	Clock Mfrs. No.: E 7420
Placement: Anchored on concrete pavement of the maintenance storage yard. The yard closes at 1600 local time on workdays. Enter facility west off of Hibbard Street, north of Willow Road.		

SITE DESCRIPTION		
Site Number: 3		
County: Cook	Township: 41N	Range: 12E
Section: 28	Lat/Long: 42°01'20" / 87°52'38"	Quadrangle: Arlington Heights
Property Owner: Private Residence		
Address: 1885 Riverview Avenue, Des Plaines, Illinois 60018		
Telephone: 708/824-1093		
Permission Date: September 14, 1989		
Installation Date: September 28, 1989		
Gage Mfrs. No.: 4730	Gage ID No.: 5062	Clock Mfrs. No.: E 7419
Placement: Placed in northwest corner of the yard by the fence. Enter Riverview west off of Des Plaines River Road.		

SITE DESCRIPTION		
Site Number: 4		
County: Cook	Township: 41N	Range: 13 E
Section: 21	Lat/Long: 42°01'35" / 87°45'22"	Quadrangle: Park Ridge
Property Owner: Village of Skokie, Attn: Eddy Nakai		
Address: 5127 Oakton Street, Skokie, Illinois 60077		
Telephone: 708/673-0500		
Permission Date: September 18, 1989		
Installation Date: September 27, 1989		
Gage Mfrs. No.: 4656	Gage ID No.: 5040	Clock Mfrs. No.: E 7392
Placement: Located in grassy strip between municipal parking lot and Floral Street just north of Oakton Street (across from Village Hall). Enter from Oakton Street.		

SITE DESCRIPTION		
Site Number: 5		
County: Cook	Township: 40N	Range: 12E
Section: 28	Lat/Long: 41°55'57" / 87°52'42"	Quadrangle: Elmhurst
Property Owner: Private Residence		
Address: 2925 North Sarah Drive, Franklin Park, Illinois 60131		
Telephone: 708/455-1226		
Permission Date: September 13, 1989		
Installation Date: September 28, 1989		
Gage Mfrs. No.: 4717	Gage ID No.: 5105	Clock Mfrs. No.: E 7413
Placement: Placed in northeast corner of backyard near a fence and a hedge. Enter Schiller Avenue east off of Mannheim Road, then south on Sarah Drive (one-way).		

SITE DESCRIPTION		
Site Number: 6		
County: Cook	Township: 40N	Range: 13E
Section: 28	Lat/Long: 41°56'17" / 87°45'38"	Quadrangle: River Forest
Property Owner: Private Residence		
Address: 5340 West Fletcher Street, Chicago, Illinois 60641		
Telephone: 312/736-0106		
Permission Date: September 28, 1989		
Installation Date: September 28, 1989		
Gage Mfrs. No.: 5300	Gage ID No.: 5304	Clock Mfrs. No.: E 7625
Placement: Placed in middle of backyard along walkway halfway between house and garage. Was closer to alley before garage was built (9-28-89 through 9-19-90). Enter alley east off of Long Street, which is south off of Belmont Avenue.		

SITE DESCRIPTION		
Site Number: 7		
County: Cook	Township: 40N	Range: 14E
Section: 28	Lat/Long: 41°56'03" / 87°38'02"	Quadrangle: Chicago Loop
Property Owner: Chicago Park District, Attn: Tom Creech		
Address: 425 McFetridge Drive, Chicago, Illinois 60605		
Telephone: 312-294-2282		
Permission Date: June 28, 1991		
Installation Date: June 28, 1991		
Gage Mfrs. No.:	Gage ID No.: 148832	Clock Mfrs. No.: E 7375
Placement: Placed in secluded playground adjacent to Diversey Driving Range, just north of Diversey Harbor. Park in lot between harbor and driving range, walk up service drive (north) to fenced-in playground (no gate). Gage in north corner of playground near driving range fence. Was located at Chicago Park District site at Belmont Inner Harbor (10-3-98 through 12-27-89) and then at the Lincoln Park Gun Club (12-27-89 through 6-28-91).		

SITE DESCRIPTION		
Site Number: 8		
County: Cook	Township: 39N	Range: 12E
Section: 29	Lat/Long: 41°50'41" / 87°52'51"	Quadrangle: Hinsdale
Property Owner: Cook County Forest Preserve District, Attn: Frank Grippo		
Address: 10400 Windsor, Westchester, Illinois 60154		
Telephone: 312/562-7628		
Permission Date: September 21, 1989		
Installation Date: September 27, 1989		
Gage Mfrs. No.: 4669	Gage ID No.: 5070	Clock Mfrs. No.: E 7292
Placement: Located in southeast corner of backyard between pool and grape hedge. Enter Windsor east from Belleview Drive, south from Cermak Road and into Forest Preserve residence facility. Just west of Salt Creek and parallel bike trail.		

SITE DESCRIPTION		
Site Number: 9		
County: Cook	Township: 39E	Range: 13E
Section: 28	Lat/Long: 41°50'48" / 87°45'26"	Quadrangle: Berwyn
Property Owner: Mary Queen of Heaven Parish, c/o Father John Price		
Address: 5314 West 24th Place, Cicero, Illinois 60650		
Telephone: 708/863-6608		
Permission Date: May 24, 1990		
Installation Date: May 24, 1990		
Gage Mfrs. No.: 7376	Gage ID No.: 6559	Clock Mfrs. No.: E 7370
Placement: Located in southwest corner of school yard about 12 feet from south fence line and along a west fence, west of the Nunnery. Was located at 5530 West 24th Street (9-28-89 through 5-24-89). Enter 24th Place (one-way east) from Central Avenue, south from Cermak Road.		

SITE DESCRIPTION		
Site Number: 10		
County: Cook	Township: 39N	Range: 14E
Section: 28	Lat/Long: 41°50'42" / 87°38'27"	Quadrangle: Englewood
Property Owner: Rental Residence		
Address: 527 West 26th Street, Chicago, Illinois 60616		
Telephone: 312/225-8066		
Permission Date: September 13, 1989		
Installation Date: September 28, 1989		
Gage Mfrs. No.: 4720	Gage ID No.: 5113	Clock Mfrs. No.: E 7416
Placement: Placed in backyard near edge of walk north of a garage and east of a spruce tree. Enter off of alley south of 26th Street, where locked gate is to be entered (observer has key). In Chinatown area, block between Wallace and Normal.		

SITE DESCRIPTION		
Site Number: 11		
County: Cook	Township: 38N	Range: 12E
Section: 28	Lat/Long: 41°45'30" / 87°52'18"	Quadrangle: Berwyn
Property Owner: Private Residence		
Address: 10180 5th Avenue Cutoff, LaGrange, Illinois 60525		
Telephone: 708/354-3161		
Permission Date: September 13, 1989		
Installation Date: September 29, 1989		
Gage Mfrs. No.: 3348	Gage ID No.: 4452	Clock Mfrs. No.: E 7297
Placement: Placed 6 feet east of clothesline pole in center of backyard near edge of a large garden. Access from Willow Springs Road, south of Joliet Road (parcel of land is between Interstate-55 and Tri-State Tollway).		

SITE DESCRIPTION		
Site Number: 12		
County: Cook	Township: 38N	Range: 13E
Section: 28	Lat/Long: 41°45'29" / 87°45'08"	Quadrangle: Berwyn
Property Owner: Boyle Midway		
Address: 5151 West 73rd Street, Bedford Park, Illinois 60638		
Telephone: 312/594-1100		
Permission Date: September 13, 1989		
Installation Date: September 28, 1989		
Gage Mfrs. No.: 4661	Gage ID No.: 5111	Clock Mfrs. No.: E 7369
Placement: Located 50 feet southwest of truck scale platform in the third fenced-in area. Facility is locked on the weekend. Access from gate on 73rd Street (few blocks west of Cicero Avenue).		

SITE DESCRIPTION		
Site Number: 13		
County: Cook	Township: 38N	Range: 14E
Section: 28	Lat/Long: 41°45'34" / 87°38'07"	Quadrangle: Englewood
Property Owner: Private Residence		
Address: 7409 South Eggleston, Chicago, Illinois 60621		
Telephone: 312/224-3807		
Permission Date: September 13, 1989		
Installation Date: September 29, 1989		
Gage Mfrs. No.: 4687	Gage ID No.: 5058	Clock Mfrs. No.: E 7353
Placement: Located just north of barbecue pit, 3 feet from 8-foot high chain-link fence. About the only secure location in the vicinity. Entry is through a locked garage gate (observer has key). Otherwise, ring upper bell at front door. Was placed east of barbecue pit, 4 feet from fence (9-29-89 through 8-23-90) and 7 feet southeast of present location. Enter Eggleston south from 74th Street.		

SITE DESCRIPTION		
Site Number: 14		
County: Cook	Township: 38N	Range: 15E
Section: 29	Lat/Long: 41°45'27" / 87°32'40"	Quadrangle: Jackson Park
Property Owner: City of Chicago - South Water Purification Plant, Attn: Robert Sambol		
Address: 3300 East Chiltenham Place, Chicago, Illinois 60649		
Telephone: 312/933-7107		
Permission Date: September 12, 1989		
Installation Date: September 28, 1989		
Gage Mfrs. No.: 3370	Gage ID No.: 4453	Clock Mfrs. No.: E 7624
Placement: Located in center of large grassy area (turf-covered roof) over sand filtration beds. Two distant buildings are east and west of the site. Enter facility east off of 79th Street from South Shore Drive.		

SITE DESCRIPTION		
Site Number: 15		
County: Cook	Township: 37N	Range: 11E
Section: 20	Lat/Long: 41°40'38" / 87°59'52"	Quadrangle: Sag Bridge
Property Owner: Metropolitan Water Reclamation District of Greater Chicago, Attn: Jim Ivers		
Address: 13 Stephen Street, Lemont, Illinois 60439		
Telephone: 708/257-7371		
Permission Date: September 11, 1989		
Installation Date: September 27, 1989		
Gage Mfrs. No.: 3373	Gage ID No.: 4421	Clock Mfrs. No.: E 7323
Placement: Placed about 100 feet east of entrance road, and several hundred feet south of MWRDGC building. Just north of Illinois and Michigan Canal. Access from Stephen Street in downtown Lemont. Exit Interstate-55 south on Lemont Road and then downtown, or enter from east on McCarthy Road. Hours are 0700-1530 local time.		

SITE DESCRIPTION		
Site Number: 16		
County: Cook	Township: 37N	Range: 12E
Section: 28	Lat/Long: 41°39'47" / 87°52'14"	Quadrangle: Palos Park
Property Owner: Private Residence		
Address: 240 Timber Edge Lane, Palos Park, Illinois 60464		
Telephone: 708/361-0853		
Permission Date: September 11, 1989		
Installation Date: September 27, 1989		
Gage Mfrs. No.: 4733	Gage ID No.: 5022	Clock Mfrs. No.: E 7423
Placement: Placed along west edge of lawn in backyard, about 20 feet south of property line and utility. Was moved about 2 feet on 4-26-91 to facilitate landscaping. Enter subdivision from 125th Street (off of Route 45), just south of McCarthy Road. West-southwest of Papoose Lake.		

SITE DESCRIPTION		
Site Number: 17		
County: Cook	Township: 37N	Range: 13E
Section: 28	Lat/Long: 41°40'33" / 87°45'03"	Quadrangle: Palos Park
Property Owner: Sardee Industries, Attn: Andy Chakonas		
Address: 11900 South Laramie Street, Alsip, Illinois 60658		
Telephone: 708/597-7330		
Permission Date: September 11, 1989		
Installation Date: September 27, 1989		
Gage Mfrs. No.: 4719	Gage ID No.: 5415	Clock Mfrs. No.: E 7300
Placement: Placed about 50 feet west of last loading dock in grassy field northwest of factory. Enter Laramie from 122nd Street, west of Cicero Avenue. Northeast of Tri-State Tollway, just south of Restvale Cemetery.		

SITE DESCRIPTION		
Site Number: 18		
County: Cook	Township: 37N	Range: 14E
Section: 29	Lat/Long: 41°40'35" / 87°39'06"	Quadrangle: Blue Island
Property Owner: Ingersoll Products Company, Attn: Don Recupido		
Address: 1000 West 120th Street, Chicago, Illinois 60643		
Telephone: 312/264-7800		
Permission Date: September 12, 1989		
Installation Date: September 27, 1989		
Gage Mfrs. No.: 7130	Gage ID No.: None	Clock Mfrs. No.: E 7410
Placement: Located at southwest end of property just southwest of a truck scale and east of property fence. Must enter at guarded gate on 119th Street.		

SITE DESCRIPTION		
Site Number: 19		
County: Cook	Township: 37N	Range: 15E
Section: 20	Lat/Long: 41°40'20" / 87°32'21"	Quadrangle: Lake Calumet
Property Owner: Graycor Industries		
Address: 12233 Avenue O, Chicago, Illinois 60603		
Telephone: 312/221-8400		
Permission Date: September 11, 1989		
Installation Date: September 26, 1989		
Gage Mfrs. No.: 5298	Gage ID No.: 5291	Clock Mfrs. No.: E 7274
Placement: Placed in grassy area just south of entrance drive and just west of the main parking lot. Office building and shops to north, and shops to southwest.		

SITE DESCRIPTION		
Site Number: 20		
County: Cook	Township: 36N	Range: 12E
Section: 29	Lat/Long: 41°35'08" / 87°52'37"	Quadrangle: Mokena
Property Owner: Private Residence		
Address: 10595 West 167th Street, Orland Park, Illinois 60462		
Telephone: 708/349-9388		
Permission Date: March 16, 1990		
Installation Date: March 16, 1990		
Gage Mfrs. No.: 4667	Gage ID No.: 5061	Clock Mfrs. No.: E 7293
Placement: Sited about 30 feet east of welding shop on rural property. Shop is east building of home/shop complex. Three dachshunds outside. Was located about 0.25 mile southeast on South 104th Avenue (9-26-89 through 3-16-90).		

SITE DESCRIPTION		
Site Number: 21		
County: Cook	Township: 36N	Range: 13E
Section: 28	Lat/Long: 41°35'14" / 87°44'56"	Quadrangle: Harvey
Property Owner: Private Residence		
Address: 16710 Lockwood Road, Tinley Park, Illinois 60477		
Telephone: 708/560-0213		
Permission Date: September 16, 1989		
Installation Date: September 28, 1989		
Gage Mfrs. No.: 4686	Gage ID No.: 5037	Clock Mfrs. No.: E 7374
Placement: Placed in north end of backyard west of (behind) garage. Enter Lockwood south off of 167th Street.		

SITE DESCRIPTION		
Site Number: 22		
County: Cook	Township: 36N	Range: 14E
Section: 21	Lat/Long: 41°35'08" / 87°38'08"	Quadrangle: Harvey
Property Owner: U.S. Army Reserve Center, Attn: Commander Al Dixon		
Address: 400 East 167th Street, Harvey, Illinois 60426		
Telephone: 708/339-0001		
Permission Date: September 12, 1989		
Installation Date: September 26, 1989		
Gage Mfrs. No.: 4676	Gage ID No.: 5035	Clock Mfrs. No.: E 7334
Placement: Located between parking lot and reserve building, just north of fenced-in reserve storage lot, about 150 feet south of 167th Street. Was located about 100 feet northwest on Army property, just west of parking lot before a building was constructed on property just to the west (9-26-89 through 11-2-90). Enter 167th Street east off of Halsted Avenue.		

SITE DESCRIPTION		
Site Number: 23		
County: Cook	Township: 36N	Range: 15E
Section: 29	Lat/Long: 41°35'10" / 87°32'16"	Quadrangle: Calumet City
Property Owner: City of Lansing Public Works, Attn: Al Poortenga		
Address: 3300 East 171st Street, Lansing, Illinois 60438		
Telephone: 708/895-7190		
Permission Date: September 12, 1989		
Installation Date: September 26, 1989		
Gage Mfrs. No.: 4660	Gage ID No.: 5043	Clock Mfrs. No.: E 7344
Placement: Placed 6 feet from east fence in northeast corner of storage yard of Public Works complex. Northeast of office and maintenance building. Enter north gate east off of 170th Street. Closes at 1530 local time.		

SITE DESCRIPTION		
Site Number: 24		
County: Cook	Township: 35N	Range: 13E
Section: 16	Lat/Long: 41°31'16" / 87°43'59"	Quadrangle: Harvey
Property Owner: Village of Matteson, Attn: Frank W. Denman		
Address: 3625 West 215th Street, Matteson, Illinois 60443		
Telephone: 708/748-1411		
Permission Date: September 12, 1989		
Installation Date: September 26, 1989		
Gage Mfrs. No.: 7573	Gage ID No.: WMU81122	Clock Mfrs. No.: E 7573
Placement: Located at Police Station on Cicero Avenue, about 0.5 mile north of U.S. 30. Placed 5 feet west of telephone terminal box on grass north of parking lot and northeast of police station.		

SITE DESCRIPTION		
Site Number: 25		
County: Cook	Township: 35N	Range: 14E
Section: 13	Lat/Long: 41°31'14"/ 87°34'26"	Quadrangle: Calumet City
Property Owner: Big John's Farm Stand, Attn: John DeBoer		
Address: 1754 East Joe Orr Road, Chicago Heights, Illinois 60411		
Telephone: 708/758-2711		
Permission Date: September 12, 1989		
Installation Date: September 26, 1989		
Gage Mfrs. No.: 7467	Gage ID No.: WMU80955	Clock Mfrs. No.: E 7628
Placement: Placed just northeast of farm stand parking lot, northwest of house and northeast of farm stand. Small ditch between parking lot and gage, and large trees near house. Just east of Interstate-394 and Stony Island Avenue, and west of Torrence Avenue.		

APPENDIX II

Instructions for Raingage Technicians

1. **Supplies required for proper servicing of the instruments in the Cook County raingage network:**

   a. A supply of 24-hour rotation raingage charts (Belfort number 5-4047-B)
   b. A bottle of raingage ink (Belfort #10 Purple)
   c. A roll of paper towels or similar absorbent material for ink spills
   d. A ball-point pen or pencil
   e. Grass clippers and/or sickle
   f. A clipboard
   g. A 12-quart bucket

2. **Make sure you have the correct time in the Central Standard Time zone:**

   Please coordinate your watch with the broadcast tone from WMAQ or WGN, etc., on the hour, before starting a day's servicing schedule and recheck if possible when out in the field. Try to be within 15 seconds of the correct time.

3. **Order of servicing upon arrival at a site (try to complete within 5-10 minutes of arrival):**

   1) Cut the grass around the raingage if necessary or applicable. Do this to the specifications of the landowner or below the level of the raingage door, whichever is shorter.

   2) Open the sliding door on the side of the instrument case by pushing out on the hinge lock and pulling up on the door handle, depress the bucket platform upright casting to ink the OFF time on the chart (a vertical line). Note the time on your watch, and move the pen point and arm away from the chart by pushing out on the pen bracket. Lift up on the drum cylinder to disengage it from the electric chart drive, and remove it out the door. Write the OFF date and time on the chart. Carefully remove the chart from the drum to avoid smearing the fresh ink at the end of the trace.

   3) Write this OFF time as the ON time on a new chart, and apply it to the drum cylinder, making sure the crease at the right end of the chart is sharp and the chart is tight on the cylinder. This helps prevent skipping when the pen point travels over the drum clip, as well as

preventing spurious indications of a rain event. Make a small mark with your pen or pencil on the chart near the zero inch line to indicate the ON time. Try to match the chart reading with the ON time as closely as possible. Reinstall the chart cylinder onto the electric chart drive, making sure the chart cylinder gear and the chart drive gear mesh.

4) - Quickly, remove the collector from the top of the gage by rotating it clockwise to disengage the tongue-and-grove assembly, set it down, and then carefully lift the bucket off of the weighing platform (if there is water in it) and dump the water on the ground. Reposition the bucket on the platform and reinstall the collector by setting it on top of the raingage case and turning counterclockwise until the tongue-and-groove assembly meshes. During wintertime operation when a charge of antifreeze is in the bucket, leave the antifreeze in until the chart reading passes the 6-inch mark. At that point, dump the bucket contents into a large plastic bucket and dispose of properly. DO NOT POUR SOLUTION ONTO THE GROUND! If wintertime conditions prevail, recharge the empty bucket with a quart of antifreeze. At any time of the year, once the collector is repositioned, check the gage to make sure the collector orifice top edge is level. With a level positioned on the collector orifice, depress the stakes on the side(s) reading high with your shoe or boot, lightly or firmly depending on how much out of level the gage is and how soft the ground is.

5) Re-ink the pen with a drop or two of ink. If the pen point appeared dirty or the previous chart's trace was rough, pull a small sheet of lint-free paper through the pen nibs to clean them. Move the pen arm and point over near the chart cylinder and rotate the cylinder counterclockwise until the pen point coincides with the pencil mark on the chart denoting the ON time. Let the pen point rest on the chart there, and depress the platform casting again to make a vertical pen line at the ON time. This also assures that the pen point is writing correctly. If not, check the tip of the pen point again to see why the ink is not drawing. It helps if the word "ON" is written on the chart near the ON line for later chart editing purposes. Remember not to overfill the pen reservoir: extra ink will only spill and create a blotch on the new chart. Rezero the pen point if necessary by turning the fine adjustment screw. It isn't a bad idea to "zero" the pen near the 0.25 inch mark instead to prevent evaporation from taking the pen point below the zero line.

6) Wipe up any excess ink from the base of the gage to keep it relatively clean. Check the just-removed chart for any irregularities and note them on the upper right corner of the chart. As you are doing this, keep an eye on the new chart to make sure the drum is rotating

and the pen is writing. When you are sure everything is operating correctly, carefully close the gage door and push the hinge lock in to secure it. Make sure you have removed all supplies and tools from the site before moving on to the next one.

4. **Disposition of recorded raingage charts:**

When a complete set of 25 charts has been collected for a week, place them in numerical order, put them in one of the postage-paid envelopes provided, and mail them to the State Water Survey, noting the name of the project director on the envelope. If any serious problems were encountered during servicing, please call the director "collect" to relay the information to him. Situations worthy of immediate attention include chart-drive stoppages, unauthorized movement of the raingage, vandalism, and theft. Repairs will then be scheduled as soon as possible. Go ahead and make minor repairs (e.g., pen point stuck under drum cylinder, debris in the collection bucket, etc.). Major repairs will require the attention of the State Water Survey.

5. **Change in site status:**

If you become aware that there has been or will be a change of status of one of the sites in the network, or one of the landowners requests movement of the raingage, please alert the State Water Survey immediately so that the project director can contact the landowner to work out a new arrangement. It is important to try to keep the sites as permanent as possible during the course of this project.

6. **Public relations:**

As a representative of the State of Illinois, it is imperative that you make your contacts with the landowners and others as cordial as possible and respect their property. They are providing an important service by agreeing to have the instrumentation on their property, so please keep their good will. Any questions from them concerning the project and your job that you are unable to answer should be referred to the project director.

APPENDIX III

This appendix gives complete documentation of all maintenance work carried out at each of the sites in the raingage network since the inception of the network in October 1989, including visits when no action was taken. Organized chronologically by site number, this documentation is current through June 1991.

SITE #1: MISSION BROOK SANITARY DISTRICT

10-89: Straightened dented raingage case near sliding door. Cause of dent unknown.

7-23-90: Taped collector joint. Sprayed ants residing in gage with ant killer. Reset upper limit stop, since the downward traverse was too short.

11-2-90: Replaced the vandalized outer case.

SITE #2: WINNETKA PARK DISTRICT

10-89: Original siting destroyed by a truck backing over the gage. Replaced entire instrument except for chart drive. Bolted case bottom into concrete.

5-18-90: Readjusted the pen for zero.

6-13-90: Corrected binding in the meshing of the chart drive and drum gears to prevent chart-drive stoppages.

7-23-90: Rezeroed pen, and adjusted level of reversal.

9-19-90: Outer case found damaged upon arrival, as 3 of 5 case screw holes were stripped. Retightened everything for time being.

11-2-90: Replaced outer case.

5-29-91: Chart drive fine.

SITE #3: DES PLAINES

4-6-90: Turned pen arm toward drum slightly to remedy skipping problem.

5-18-90: Replaced chart drive and rezeroed pen.

7-3-90: Replaced pen arm and point. Raised clock to match time line and reversal. Releveled gage and tightened outer case screws. Shortened clip to prevent interference during drum rotation.

7-23-90: Releveled gage. Calibration fine.

4-26-91: Replaced chart drive and rezeroed pen. Also releveled gage.

SITE #4: VILLAGE OF SKOKIE

7-23-90: Reset upper and lower limit stops. Releveled gage.

8-23-90: Gage operating properly.

11-2-90: Releveled gage.

6-28-91: Releveled gage, checked chart-drive operation.

SITE #5: FRANKLIN PARK

7-3-90: Releveled gage and cleaned pen point.

7-23-90: Calibration fine.

5-29-91: Chart drive fine. Releveled gage.

SITE #6: WEST FLETCHER STREET

7-3-90: Reset chart drive height. Removed locking screw to prevent interference with lower limit stop. Releveled gage.

7-23-90: Exchanged drum to lower the chart level in relation to the pen traverse.

8-17-90: Pen tip had been getting caught under drum lip. Found to be fine.

9-19-90: Repositioned gage on walkway between house and new garage. Had been moved (unauthorized) by garage builder. New drum installed, pen arm adjusted, and pen rezeroed.

2-5-91: Replaced chart drive. Releveled gage.

3-27-91: Releveled gage.

SITE #7: CHICAGO PARK DISTRICT

10-89: Straightened bent case (boat had rammed it) - Belmont Harbor site.

11-89: Water had entered dashpot during October 1989 accident, causing weighing mechanism to lock up when freezing occurred. Replaced dashpot - Belmont Harbor site.

12-27-89: Removed destroyed instrument from Belmont Harbor site, and relocated a new instrument at the Lincoln Park Gun Club. Gage needs underpinning timbers to provide stability.

1-11-90: Attached "t-base" of 2x4 lumber sections to base of gage. Stability greatly increased.

3-16-90: Evidence of water on chart drive. Replaced chart drive, dried interior of gage.

5-18-90: Adjusted and lubricated pen arm.

5-24-90: Checked site, but gained access by ladder since Gun Club door lock was broken.

7-24-90: Lubricated drum shaft. Relieved locking screw and locking nut.

11-2-90: Replaced chart drive.

1-15-91: Gage operating correctly.

2-5-91: Replaced chart drive again. Gage checked out otherwise.

3-27-91: Once again, replaced chart drive.

6-28-91: Removed gage from Lincoln Park Gun Club site to area in Lincoln Park north of Diversey Harbor, just west of old site. Replaced chart drive. Site now located at ground level.

SITE #8: COOK COUNTY FOREST PRESERVE DISTRICT - WESTCHESTER

5-18-90: Replaced chart drive and rezeroed pen.

7-3-90: Replaced chart drive upon finding the old one dead. Chart drive height fine. Releveled gage after bending collector tab.

7-23-90: Releveled gage. Calibration fine.

4-26-91: Replaced chart drive. Releveled gage.

5-29-91: Replaced chart drive.

SITE #9: MARY QUEEN OF HEAVEN PARISH - CICERO

12-27-89: De-iced and dried dashpot. Replaced it and filled pot with Mobil 90 oil - 24th Street site. The bucket had disappeared and then reappeared, causing rain to reach the gage workings during its absence.

1-31-90: Replaced fine adjustment thumb screw - 24th Street site.

5-18-90: Replaced chart drive. Gage moved to facilitate landscaping, to spot between garages with bucket over collector (out-of-service) - 24th Street site.

5-24-90: Moved site to Mary Queen of Heaven Parish schoolyard, about 2 blocks northeast of previous site.

7-23-90: Calibration fine.

11-2-90: Loosened collector fit onto case at request of observer.

3-27-91: Replaced chart drive batteries. Releveled gage.

6-28-91: Checked chart drive. Reinked pen.

SITE #10: WEST 26th STREET

7-24-90: Calibration fine. Releveled gage.

SITE #11: LAGRANGE

7-24-90: Calibration fine. Releveled gage.

12-17-90: Site visited, but everything fine.

2-5-91: Replaced chart drive. Releveled gage.

4-26-91: Straightened out drum clip to prevent further skipping on the trace.

SITE #12: BOYLE MIDWAY - BEDFORD PARK

7-24-90: Releveled gage. Trace about 0.10" short on reversal, but not changed.

5-29-91: Replaced chart drive. Releveled gage. Large dirt mounds near gage.

SITE #13: SOUTH EGGLESTON AVENUE

7-3-90: Corrected chart drive height and rezeroed gage.

8-23-90: Moved gage at request of owner to new spot in backyard, approximately 7 feet northwest of old location. Reset pen arm up a little. Gage operation explained to landowner.

4-26-91: Bent pen arm down a bit to prevent shakiness in trace. Releveled gage.

SITE #14: SOUTH WATER PURIFICATION PLANT

7-24-90: Pen point off slightly with respect to following time line on chart. Efforts to correct failed. Releveled gage.

11-2-90: Stakes refortified to prevent a shaky trace. Releveled gage.

12-17-90: Replaced chart drive. Fortified stakes again and releveled gage.

2-5-91: Pushed stakes down, and releveled gage.

3-27-91: Releveled gage, pushed stakes down a bit. Loose soil at this site, above the filtration bed.

SITE #15: MWRDGC - LEMONT

11-89: Straightened vandalized sliding door on outer case.

12-27-89: Replaced outer case with one having a better sliding door.

3-16-90: Replaced outer case again for same reason. Could be some tampering. Rerouted chart drive battery wires to prevent drum clip from getting caught on them. Instructed Jim Ivers in reading of device.

4-6-90: Replaced drum to prevent dragging of clip.

5-3-90: Replaced chart drive. Installed evaporation shield for observer.

7-23-90: Reversal about 0.10" high. Left as is. Time trace fine.

12-17-90: Replaced chart drive. Releveled gage.

2-5-91: Replaced chart drive again, and drum cylinder.

SITE #16: PALOS PARK

10-89: Spilled oil indicated a dashpot leak. Dashpot replaced and refilled with oil. Shorted out chart drive when reinstalling, so replaced it and batteries.

4-6-90: Replaced bucket and rezeroed. Old bucket was rubbing against case and causing pen vibrations.

7-24-90: Calibration fine. Releveled gage.

3-27-91: Replaced chart drive. Releveled gage. New home going up next door.

4-26-91: Unauthorized move of gage made by contractor to facilitate landscaping. Gage relocated very near original site during visit. Raised drum for better time line trace. Relubricated moving parts of gage.

5-29-91: Replaced chart drive again. Clip fine. Releveled gage. Tilted pen down a bit and cleaned it.

SITE #17: SARDEE INDUSTRIES - ALSIP

5-24-90: Rezeroed pen.

6-13-90: Cleaned pen nibs to prevent skipping and rezeroed pen.

7-24-90: Reversal and both stops a little high, but not changed. Releveled gage.

3-27-91: Changed batteries.

SITE #18: INGERSOLL PRODUCTS - WEST 120th STREET

4-6-90: Rezeroed pen with pseudo-bucket weight since bucket filled with antifreeze.

7-24-90: Releveled gage with sledgehammer. Reversal and upper limit stop a bit high, but not changed.

8-17-90: Raingage stolen sometime on August 11 or 12 during factory break-in. New gage installed at previous site. Original stakes and nuts and bolts were left on site by thieves. The stolen chart drive was the main casualty. This location has an 8-foot high chain link and barbed wire fence around perimeter, along with a guarded security gate.

12-17-90: Site visited, but gage fine.

1-15-91: Replaced chart drive.

SITE #19: GRAYCOR INDUSTRIES - AVENUE O

4-6-90: Rezeroed pen with pseudo-bucket weight since bucket filled with antifreeze.

7-24-90: Calibration fine.

8-17-90: Skipping problem previously, but fine at this visit.

9-19-90: Tightened pen arm, tilted in towards drum, and cleaned pen tip nibs.

3-27-91: Pen arm again loose (looked like tampering), so tilted it in a bit towards drum. Releveled.

6-28-91: Straightened drum clip.

SITE #20: ORLAND PARK

1-31-90: Replaced batteries and antifreeze - S. 104th Avenue site.

3-16-90: Moved gage from S. 104th Avenue site to present location on 167th Street in Orland Park, roughly 0.25 miles northwest.

7-24-90: Calibration fine. Releveled gage.

1-15-91: Replaced chart drive. Changed pen point. Gage was badly out of level.

3-27-91: Releveled gage again, tilted pen arm in a bit towards drum.

4-26-91: Releveled gage again and cleaned pen nibs.

SITE #21: TINLEY PARK

4-6-90: Cleaned pen point and well.

7-24-90: Lowered drum a bit. Upper limit stop a little high, but left as is. Releveled gage.

1-15-91: Replaced chart drive.

SITE #22: U.S. ARMY RESERVE CENTER - HARVEY

7-24-90: Calibration fine. Releveled gage.

9-19-90: Found drum not rotating, but chart drive operating. Observer had not correctly meshed the chart drive and drum gears. Business development going up next door, very near gage. Cleaned pen point.

11-2-90: Unauthorized move of gage made by building contractor in late October, and destroyed outer case while doing so. Moved gage about 100 feet southeast between the parking lot and the Reserve building. Replaced the outer case and applied a new drum gear (other one stolen).

12-17-90: A bent lower traverse rod was causing the reversal to occur at 2.35 inches instead of at 6 inches. Straightened the traverse and recalibrated the gage.

SITE #23: CITY OF LANSING PUBLIC WORKS

4-6-90: Cleaned pen point.

5-3-90: Straightened pen arm and recleaned pen point. Installed evaporation shield for observer.

5-18-90: Changed pen point. Inking difficult.

7-24-90: Calibration fine. Releveled gage.

9-19-90: Replaced pen point (had been skipping). Rezeroed pen.

11-23-90: Pen wouldn't zero. Removed case and recalibrated.

SITE #24: VILLAGE OF MATTESON

1-31-90: Replaced chart drive. Later found batteries to be reversed by observer in dead drive.

4-6-90: Straightened pen point and cleaned nibs.

5-18-90: Rezeroed pen.

7-24-90: Calibration fine. Releveled gage. Sprayed ants with killer.

3-27-91: Releveled gage. Pen point fine (had been caught under drum lip).

6-28-91: Changed chart drive.

SITE #25: BIG JOHN'S FARM STAND - CHICAGO HEIGHTS

6-13-90: Releveled gage, and rezeroed pen with a dry bucket. Cleaned pen nibs.

7-24-90: Calibration fine. Releveled.

9-19-90: Checked out fine (chart drive had been intermittent).

11-2-90: Readjusted fine adjustment screw, repositioned battery pack lines under pen arm.

11-23-90: Rezeroed pen.

12-17-90: Replaced chart drive.

APPENDIX IV

Included here are storm totals for all events greater than an annual event (one-year recurrence interval) during Water Year 1990. Storm durations of one hour to ten days were considered. The rainfall amounts for a one-year recurrence interval and these storm durations in northeastern Illinois are given below (Huff and Angel, 1989).

Storm Duration	Rainfall Amounts (inches)
1 hour	1.18
2 hours	1.48
3 hours	1.60
6 hours	1.88
12 hours	2.18
18 hours	2.30
24 hours	2.51
48 hours	2.70
72 hours	2.93
5 days	3.25
10 days	4.12

The values listed in the table below exceed the numbers above for the given storm duration. An "E" indicates a partial or full estimate for that particular site and storm. The last column indicates whether a particular storm total exceeded other events greater than an annual event (2-year to 100-year recurrence intervals considered).

Storm Totals

Storm #	Site #	Date	Duration (hours)	Amount (inches)	Other Events Exceeded
8	24	11-14/15-89	38	2.67	
55	1	5-9/10-90	21	2.58	
	2		21	3.10 E	2-year
	3		22	3.30 E	2-year
	4		22	3.56	2-year
	5		21	3.66	5-year
	6		22	3.96	5-year
	7		20	2.94	2-year
	8		22	3.88	5-year
	9		22	3.45 E	2-year
	10		21	2.95	2-year
	11		22	3.27	2-year
	12		21	2.46	
	13		19	3.05	2-year
	14		21	2.45	
	15		21	3.08	2-year
	17		20	2.40 E	
	18		19	2.76	
	21		19	2.77	
	22		19	2.34	
	24		20	3.35	2-year
84	11	7-19/20-90	18	3.26	2-year
	12		19	4.44	10-year
	13		17	4.41	10-year
	14		21	5.72	25-year
	15		23	2.81	
	16		29	3.85	2-year
	17		28	4.13	5-year
	18		18	3.58	5-year
	19		24	4.06	5-year
90	5	8-10-90	4	2.05	
	6		3	1.80 E	
	13		5	2.77	2-year
	18		2	1.89 E	2-year
	22		1	1.25	

Storm Totals (Concluded)

Storm #	Site #	Date	Duration (hours)	Amount (inches)	Other Events Exceeded
92	1	8-17/18-90	24	3.41	2-year
	2		23	3.21	2-year
	3		22	2.79	
	4		16	3.50	5-year
	6		24	2.83	
	25		23	2.84	
94	6	8-19/21-90	44	2.79	

APPENDIX D

DIVERSION ACCOUNTING UPDATE FOR THE NEW 25-GAGE PRECIPITATION NETWORK

Diversion Accounting Update for the New 25-Gage Precipitation Network

October 1993

Prepared for:
U.S. Army Corps of Engineers
Chicago District

Prepared by:
RUST Environment & Infrastructure
111 North Canal Street, Suite 305
Chicago, IL 60606

TABLE OF CONTENTS

Title	Page No.
INTRODUCTION	1
Study Area and Background	2
Topics Covered in this Report	4
PROJECT MAPPING	5
SPECIAL CONTRIBUTING AREAS (SCA)	6
PRECIPITATION GAGE ASSIGNMENT	11
Objective	11
Gage Identification and Numbering	11
Gage Locations and Mapping	13
Delineation of Gage Areas	13
LAND COVER ANALYSIS	14
Objective	14
Approach	14
Land Use Delineation	15
Land Cover Computations	17
Results	18
Land Cover Analysis for the Ungaged Watersheds	19
COMPUTER FILE REVISIONS	20
HSPF Revisions	21
SCALP Revisions	21
REFERENCES	22

LIST OF TABLES

No.	Title	Page No.
1	Review Listing of SCA's, Sewer Systems and Input Files	8-10
2	Identification of the 25-Gage Precipitation Network and Temporary TSS Data Set Numbers	12
3	Land Cover Proportions for Each Land Use	16
4	Land Cover Results by Major Drainage Area	21
5	Land Cover Results for the Ungaged Watersheds	22

LIST OF FIGURES

No.	Title	Page No.
1	Diversion Accounting Major Watersheds and MWRDGC Planning Area and Boundaries	3
2	Special Contributing Areas and Precipitation Gage Thiessen Boundaries	Pocket
3	Index to the 1" = 400' Scale Land Use Delineation Maps	Pocket

LIST OF APPENDICES

Appendix	Title
A	Index to CAD Drawing Levels
B	Coordinate Conversion Computer Output
C	Raw Land Use Data in Acres as Measured Directly from the 1" = 400' Scale Aerial Photo Overlays
D	Land Use Areas in Acres for Each SCA and Ungaged Partition
E	Land Cover Results in Acres for Each SCA and Ungaged Partition
F	Land Cover in Acres for Each Gage Assignment Partition
G	Land Cover in Square Miles for Each Gage Assignment Partition

INTRODUCTION

This report describes revisions and updates to portions of the computer model inputs used in the Lake Michigan diversion accounting program of the U.S. Army Corps of Engineers, Chicago District. The primary purpose of the revisions is to implement a new 25-gage precipitation network in the diversion accounting analysis. Assignment of gage data to land areas is interconnected with land cover assignments used in the analysis. Because it would be necessary to recalculate land cover assignments to determine precipitation gage assignments, the project also includes a complete update of land cover assignments.

Major elements of the project were the following:

- Review and correction of the Computer Aided Drafting (CAD) mapping system used in diversion accounting
- Delineation of precipitation gage area assignments for the 25-gage network
- Manual land use delineation
- Calculation of land cover
- Revision of computer model inputs to reflect the updates

The above tasks formed the major work effort of the project. Deliverable results of this effort included updated CAD mapping computer files, 1" = 400' scale land use maps for the study region, and the revised computer model input files.

There were additional tasks included in the project not related to the land cover/gage delineation work. These tasks included the following items:

- Review of model parameters used in the hydrologic models
- Preliminary delineation of separate sewer areas in the Calumet facility planning area

The results of these additional tasks are presented in technical memoranda which are included as attachments to this report.

Study Area and Background

The diversion accounting program of the U.S. Army Corps of Engineers is an analysis procedure used to determine the annual volume of flow diversion from the Lake Michigan watershed to the Illinois River watershed. The overall program is extremely complex, and this report does not attempt to provide a complete explanation of the procedure. However, a simplified, partial explanation follows to provide sufficient background information for interpretation of this report.

Water is transferred from the Lake Michigan watershed to the Illinois River system by several mechanisms. The largest contribution is due to reversal of the flow direction in 1900 of the Chicago River and construction of a canal outlet from the Chicago River to the Des Plaines River, a tributary of the Illinois River. As a result, 673 square miles of land formerly tributary to Lake Michigan drains to the Illinois River system. This diverted watershed is shown in Figure 1.

The focus of this project is the combined sewer areas serviced by the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC). The MWRDGC facility planning areas (FPA) are also outlined in Figure 1. The study area for this project covers most of the North, Central and Calumet FPAs. In about 80 percent of these planning areas, combined sewers collect both stormwater and sanitary sewage and convey the flow to MWRDGC treatment plants. After treatment, the water is discharged to the reversed Chicago River system. The sanitary sewage portions of the treatment plant discharges also contribute to the diversion because the potable water supply is pumped from Lake Michigan. This pumping is the second major component of the diversion. However, MWRDGC service boundaries extend beyond the boundary of the diverted Lake Michigan watershed. Therefore, not all stormwater discharges from the treatment plants are part of the diversion.

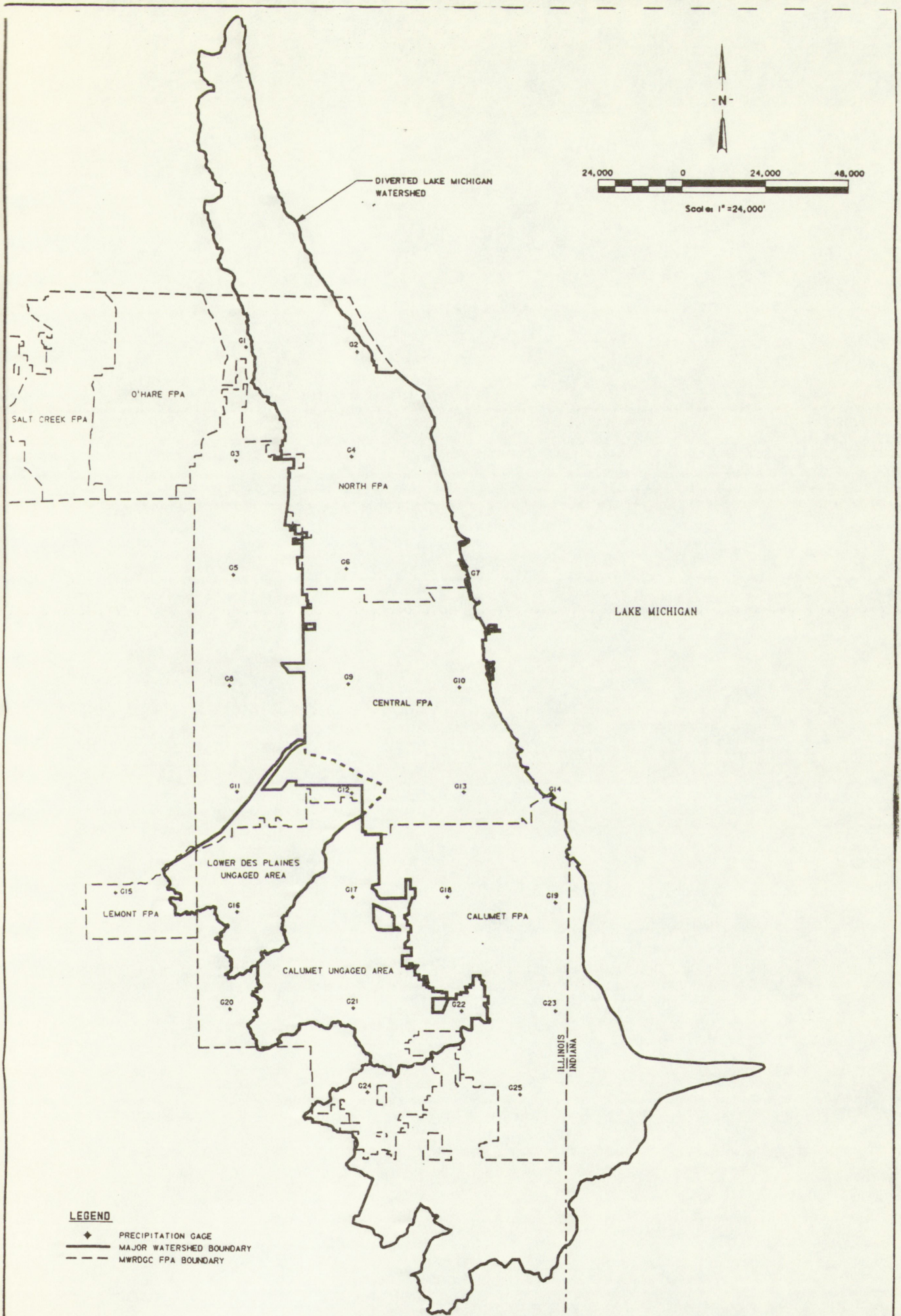


FIGURE 1

DIVERSION ACCOUNTING MAJOR WATERSHEDS AND MWRDGC PLANNING AREA BOUNDARIES
DIVERSION ACCOUNTING UPDATE FOR THE NEW 25-GAGE PRECIPITATION NETWORK
US ARMY CORPS OF ENGINEERS CHICAGO, ILLINOIS

RUST ENVIRONMENT & INFRASTRUCTURE

The Corps of Engineers, Chicago District has developed a hydrologic and hydraulic computer modeling strategy to track stormwater and sanitary sewage discharges through the MWRDGC system. The strategy is used to determine the portion of treatment plant discharges and flows at certain stream gages that is attributable to the Lake Michigan diversion. The project described in this report deals with updates to the hydrologic and hydraulic models of specific portions of the MWRDGC sewer system and local systems connected to it. It must be stressed that this is just one component of the overall diversion accounting procedure.

Topics Covered in this Report

This report presents computations of impervious and pervious areas for over three hundred watershed subdivisions. These subdivisions are the result of portioning subbasins according to a new gage delineation also completed for this project. A key tool in the land cover/gage delineation analysis is the computer mapping system used in diversion accounting which is introduced in the following section. Next, the basic watershed units, the special contributing areas (SCA), are described. This is followed by report sections describing the delineation of precipitation gage areas and the land cover analysis. Finally, the report presents procedures for revising the computer model input files.

The report includes six appendices. Appendix A is an index to the computer mapping system. Appendices B through G contain interim and final data associated with the land cover computations. Additional project deliverables that accompany the report are the following.

- Revised CAD mapping on disk
- Land cover database on disk
- SCA database on disk
- 161 1" = 400' scale aerial photo maps with overlays showing land use delineations
- Updated HSPF and SCALP computer model data files on disk

PROJECT MAPPING

The contributing watersheds have been digitized into a computer format map. The Corps provided the mapping in AutoCAD which was converted to Intergraph Microstation format by the contractor during the project. New and revised map layers were delivered to the Corps in both formats.

The base layer of the mapping appears to be of USGS origin. The imbedded coordinate system is the Universal Transverse Mercator (UTM) grid coordinates, zone 16. The UTM System is metric and grid distances on the drawings are expressed in meters. Layers showing water courses, railroads and airports appear to have been part of the original base map. The base mapping is useful for identifying physical locations and for lining up the mapping with other maps such as USGS quadrangles. However, the base mapping has no function in diversion accounting.

Important subarea delineations have been incorporated into the digital maps. These are as follows.

- The diverted Lake Michigan Watershed
- 214 Special Contributing Areas (SCA)
- 2 "Ungaged" Watersheds:
  - Lower Des Plaines River
  - Calumet River
- 7 MWRDGC Facility Planning Areas (FPA)
- The MWRDGC Poplar Creek Service Area

The focus of the present study is on the SCA's and ungaged watersheds. The MWRDGC FPA boundaries are really legal delineations rather than watershed boundaries because MWRDGC collection facilities occasionally carry flow across their FPA boundaries.

The digital maps also contain numerous features of the MWRDGC collection system, including interceptor sewers, TARP tunnels, connecting structures, drop shafts, pumping

stations and sewage treatment plants. These features were digitized previously and not used or modified in the present study. An index to Intergraph format drawing levels and AutoCAD format drawing layers is provided in Appendix A.

SPECIAL CONTRIBUTING AREAS (SCA)

The SCA's are sewered watersheds that drain to the MWRDGC TARP system either at present or in the future. The SCA's are virtually all combined sewer areas, but a few SCA's in the Calumet area are served by separate sanitary sewers and storm sewers. There are 214 SCA's that have been delineated. They are grouped according to the portion of the TARP system to which they drain or will drain when the system is complete. Each group is identified by a one or two letter prefix as follows.

- CA - Calumet
- DP - Des Plaines
- M - Mainstream
- MN - Mainstream, North Leg
- O - O'Hare

The SCA's in the O'Hare group are not addressed in this analysis.

The function of TARP is to capture the part of storm runoff events which overload the combined sewer system. If an event is large enough to fill the available storage in the TARP system, then excess flow discharges to area rivers and streams through combined sewer overflows. The diversion accounting program employs hydrologic and hydraulic models to track runoff and determine the runoff portions that flow directly to the treatment plants, indirectly to the treatment plants via TARP, or are discharged to the rivers as overflows. Both the source location and ultimate fate of the runoff are important in the computations.

The diversion accounting program uses the HSPF hydrologic model to estimate watershed runoff and the SCALP model to perform initial hydraulic computations. The HSPF model

calculates storm runoff per unit area on an hourly basis. Contributing areas and baseflows are defined in the SCALP models. Total flows are routed through the lateral, submain and main sewer systems using SCALP. SCALP also determines the hourly flow to the treatment facility and the quantity that becomes overflow. Subsequent modeling of the TARP system identifies the portion of the overflow that is diverted to TARP for later treatment and the portion that is discharged to area waterways.

Thus, a major input in the treatment plant/TARP/overflow modeling is the definition and delineation of the SCAs. The study described in this report included a detailed review of SCA delineations and development of a database that provides pertinent information about each SCA. A map showing the SCA delineations is provided in Figure 2 which can be found after the appendices at the end of this report. This map indicates the identifying designation of each SCA and the area of the SCA in acres. The SCA designation consists of two parts: a prefix identifying the TARP branch to which the SCA drains and an identification number. Letters A and B have been used to indicate SCA's that were formerly a single SCA and have since been divided.

A review database of SCA characteristics is provided in Table 1. This table lists the SCA identifier and the surface area of each SCA. The map area was determined digitally using the CAD software. The model area indicates the area that was entered in SCALP model inputs for WY 1989 submitted to the contractor by the Corps. Differences between the map and WY 1989 model are noted, with differences exceeding 5 percent highlighted. Correction of SCA areas to agree with "map areas" is among the revisions to SCALP inputs completed in this project. Table 1 also identifies whether the SCA is served by separate or combined sanitary sewers. Four separate sewer areas in Calumet are not mapped in the CAD system, and there is no map area shown. Table 1 also gives the file name where the SCALP input for each SCA resides.

TABLE 1

REVIEW LISTING OF SCA'S, SEWER SYSTEMS AND INPUT FILES

SUBBASIN	ID #	MAP AREA acres	MAP AREA sq. miles	MODEL AREA sq. miles	DIFFERENCE sq. miles	PERCENT DIFFERENCE	SEWER SYSTEM	SCALP FILE	COMMENTS
CA	2	718	1.122	1.122	–	–	COMB.	CAL–LM	
CA	4			3.080			SEP.	CAL–LMDP	
CA	5	201	0.314	0.314	–	–	COMB.	CAL–LM	
CA	6	681	1.064	1.064	–	–	COMB.	CAL–LM	
CA	7	741	1.158	1.158	–	–	COMB.	CAL–LM	
CA	7 M	659	1.030	1.030	–	–	COMB.	CAL–LM	Former CA(C–7M)
CA	7 Q	4063	6.348	6.348	–	–	COMB.	CAL–LM	Former CA(C–7Q)
CA	7 V	847	1.323	1.323	–	–	COMB.	CAL–LM	Former CA(C–7V)
CA	7 W	2497	3.902	3.902	–	–	COMB.	CAL–LM	Former CA(C–7W)
CA	8	747	1.167	1.167	–	–	COMB.	CAL–LM	
CA	9	203	0.317	0.317	–	–	COMB.	CAL–LM	
CA	10	5439	8.498	8.498	–	–	COMB.	CAL–LM	
CA	11 A	567	0.886	0.886	–	–	COMB.	CAL–LM	
CA	11 B	844	1.319	1.319	–	–	COMB.	CAL–LM	
CA	12	2622	4.097	4.097	–	–	COMB.	CAL–LM	
CA	13	3726	5.822	5.822	–	–	COMB.	CAL–LM	
CA	14	187	0.292	0.292	–	–	COMB.	CAL–LM	
CA	15 A	960	1.500	1.500	–	–	COMB.	CAL–LM	Former 15–1
CA	15 B	79	0.123	0.123	–	–	COMB.	CAL–LM	Former 15–2
CA	16	227	0.355	0.355	–	–	COMB.	CAL–LM	
CA	17	527	0.823	0.823	–	–	COMB.	CAL–LM	
CA	18	1062	1.659	1.659	–	–	COMB.	CAL–LM	
CA	18 E–A	3948	6.169	6.169	–	–	COMB.	CAL–LM	
CA	18 E–B	786	1.228	1.228	–	–	COMB.	CAL–LM	
CA	19	1267	1.980	1.980	–	–	COMB.	CAL–LM	
CA	20	249	0.389	0.389	–	–	COMB.	CAL–LM	
CA	21	636	0.994	0.994	–	–	COMB.	CAL–LM	
CA	22	671	1.048	1.048	–	–	COMB.	CAL–LM	
CA	23	603	0.942	0.942	–	–	COMB.	CAL–LM	
CA	24	1453	2.270	2.270	–	–	COMB.	CAL–LM	
CA	25	312	0.488	0.488	–	–	COMB.	CAL–LM	
CA	26	586	0.916	0.916	–	–	COMB.	CAL–LM	
CA	27	390	0.609	0.609	–	–	COMB.	CAL–LM	
CA	28	1329	2.077	2.077	–	–	COMB.	CAL–LM	
CA	29	802	1.253	1.253	–	–	COMB.	CAL–LM	
CA	30	766	1.197	1.197	–	–	COMB.	CAL–LM	
CA	31	947	1.480	1.480	–	–	COMB.	CAL–LM	
CA	32	860	1.344	1.344	–	–	COMB.	CAL–LM	
CA	33	112	0.175	0.175	–	–	COMB.	CAL–LM	33A in Model
CA	34 A	1590	2.484	2.484	–	–	COMB.	CAL–LM	
CA	34 B	100	0.156	0.156	–	–	COMB.	CAL–LM	
CA	36	141	0.220	0.220	–	–	COMB.	CAL–LM	
CA	38	950	1.484	1.484	–	–	COMB.	CAL–LM	
CA	39	896	1.400	1.400	–	–	COMB.	CAL–LM	
CA	40	1207	1.886	1.886	–	–	COMB.	CAL–LM	
CA	41	465	0.727	0.727	–	–	COMB.	CAL–LM	
CA	42 A	680	1.063	1.063	–	–	COMB.	CAL–LM	
CA	42 B	576	0.900	0.900	–	–	COMB.	CAL–LM	
CA	43 A	26	0.041	0.041	–	–	COMB.	CAL–LM	Former 43–1
CA	43 B	44	0.069	0.069	–	–	COMB.	CAL–LM	Former 43–2
CA	44	70	0.109	0.109	–	–	COMB.	CAL–LM	
CA	45	157	0.245	0.245	–	–	COMB.	CAL–LM	
CA	46	343	0.536	0.536	–	–	COMB.	CAL–LM	
CA	47	54	0.084	0.084	–	–	COMB.	CAL–LM	
CA	48	349	0.545	0.545	–	–	COMB.	CAL–LM	
CA	49	267	0.417	0.417	–	–	COMB.	CAL–LM	
CA	50	131	0.205	0.205	–	–	COMB.	CAL–LM	
CA	51	1008	1.575	1.575	–	–	COMB.	CAL–LM	
CA	52			5.560			SEP.	CAL–LMDP	
CA	53	73	0.114	0.114	–	–	COMB.	CAL–LM	
CA	54	105	0.164	0.164	–	–	COMB.	CAL–LM	
CA	55	182	0.284	0.284	–	–	COMB.	CAL–LM	
CA	56	2094	3.272	3.272	–	–	COMB.	CAL–LM	
CA	57	324	0.506	0.506	–	–	COMB.	CAL–LM	
CA	58	997	1.558						No SCALP, Not Sewered
CA	59			1.430			SEP.	CAL–LMDP	
CA	60			1.670			SEP.	CAL–LMDP	
CA	61	9	0.014	0.014	–	–	COMB.	CAL–LM	
CA	62	235	0.367	0.367	–	–	COMB.	CAL–LM	
DP	1	281	0.439	0.553	0.114	20.6%	COMB.	W–UDPPS	
DP	2 A	72	0.113	0.114	0.001	0.9%	COMB.	W–UDPPS	

TABLE 1

REVIEW LISTING OF SCA'S, SEWER SYSTEMS AND INPUT FILES
(continued)

SUBBASIN ID #		MAP AREA acres	MAP AREA sq. miles	MODEL AREA sq. miles	DIFFERENCE sq. miles	PERCENT DIFFERENCE	SEWER SYSTEM	SCALP FILE	COMMENTS
DP	2 B	397	0.620	0.620	–	–	COMB.	W–UDPPS	
DP	3	196	0.306	0.306	–	–	COMB.	W–UDPPS	
DP	4	365	0.570	0.570	–	–	COMB.	W–UDPPS	
DP	5	1216	1.900	1.900	–	–	COMB.	W–UDPPS	
DP	6	705	1.102	1.102	–	–	COMB.	W–UDPPS	
DP	7	955	1.492	1.511	0.019	1.2%	COMB.	W–UDPPS	
DP	8	422	0.659	0.659	–	–	COMB.	W–UDPPS	
DP	9	750	1.172	1.172	–	–	COMB.	W–UDPPS	
DP	10	154	0.241	0.241	–	–	COMB.	W–UDPPS	
DP	11	950	1.484	1.547	0.063	4.0%	COMB.	W–UDPPS	
DP	12	210	0.328	0.328	–	–	COMB.	W–UDPPS	
DP	13	155	0.242	0.242	–	–	COMB.	W–UDPPS	
DP	14	504	0.788	0.788	–	–	COMB.	W–UDPPS	
DP	15	387	0.605	0.613	0.008	1.3%	COMB.	W–UDPPS	
DP	16	145	0.227	0.227	–	–	COMB.	W–UDPPS	
DP	17 A	175	0.273	0.227	−0.046	−20.3%	COMB.	W–UDPPS	
DP	17 B	441	0.689	0.675	−0.014	−2.1%	COMB.	W–UDPPS	
DP	18	253	0.395	0.395	–	–	COMB.	W–UDPPS	
DP	19	1279	1.998	1.998	–	–	COMB.	W–UDPPS	
DP	20	165	0.258	0.258	–	–	COMB.	W–UDPPS	
DP	21	669	1.045	1.045	–	–	COMB.	W–SUBDP	
DP	22	44	0.069	0.069	–	–	COMB.	W–SUBDP	
DP	23	311	0.486	0.486	–	–	COMB.	W–SUBDP	
DP	24	77	0.120	0.120	–	–	COMB.	W–SUBDP	
DP	25 AB	110	0.172	0.17188	–	–	COMB.	W–SUBDP	25A &25B Combined in Model
DP	26	369	0.577	0.577	–	–	COMB.	W–SUBDP	
DP	27	324	0.506	0.473	−0.033	−7.0%	COMB.	W–SUBDP	
DP	28	312	0.488	0.488	–	–	COMB.	W–SUBDP	
DP	29	144	0.225	0.225	–	–	COMB.	W–SUBDP	
DP	30	335	0.523	0.533	0.010	1.9%	COMB.	W–SUBDP	
DP	31	334	0.522	0.522	–	–	COMB.	W–SUBDP	
DP	32	160	0.250	0.250	–	–	COMB.	W–SUBDP	
DP	33	452	0.706	0.706	–	–	COMB.	W–SUBDP	
DP	34	492	0.769	0.806	0.037	4.6%	COMB.	W–SUBDP	
DP	35	281	0.439	0.469	0.030	6.4%	COMB.	W–SUBDP	
DP	36	259	0.405	0.405	–	–	COMB.	W–SUBDP	
DP	37	112	0.175	0.175	–	–	COMB.	W–SUBDP	
DP	38	700	1.094	1.094	–	–	COMB.	W–SUBDP	
DP	39	265	0.414	0.414	–	–	COMB.	W–SUBDP	
DP	40	131	0.205	0.205	–	–	COMB.	W–SUBDP	
DP	41	417	0.652	0.652	–	–	COMB.	W–SUBDP	
DP	42	104	0.163	0.163	–	–	COMB.	W–SUBDP	
DP	43	222	0.347	0.347	–	–	COMB.	W–SUBDP	
DP	44	207	0.323	0.345	0.022	6.4%	COMB.	W–SUBDP	
DP	45	95	0.148	0.148	–	–	COMB.	W–SUBDP	
DP	46	218	0.341	0.341	–	–	COMB.	W–SUBDP	
DP	47	36	0.056	0.041	−0.015	−36.6%	COMB.	W–SUBDP	
DP	48	42	0.066	0.066	–	–	COMB.	W–SUBDP	
DP	49	389	0.608	0.606	−0.002	−0.3%	COMB.	W–SUBDP	
DP	50	172	0.269	0.163	−0.106	−65.0%	COMB.	W–SUBDP	
DP	51	217	0.339	0.358	0.019	5.3%	COMB.	W–SUBDP	
DP	52	751	1.173	1.173	–	–	COMB.	W–SUBDP	
DP	53	150	0.234	0.234	–	–	COMB.	W–SUBDP	
DP	54	153	0.239	0.239	–	–	COMB.	W–SUBDP	
DP	55	442	0.691	0.691	–	–	COMB.	W–SUBDP	
DP	56	151	0.236	0.236	–	–	COMB.	W–SUBDP	
DP	62	51	0.080	0.080	–	–	COMB.	W–SUBDP	
DP	63	473	0.739	0.739	–	–	COMB.	W–SUBDP	
DP	64	50	0.078	0.078	–	–	COMB.	W–SUBDP	
DP	65	252	0.394	0.394	–	–	COMB.	W–SUBDP	
DP	66	37	0.058	0.058	–	–	COMB.	W–SUBDP	
M	1	2919	4.561	4.561	–	–	COMB.	N–LMICH	
M	2	2783	4.348	4.348	–	–	COMB.	N–LMICH	
M	3	1088	1.700	1.700	–	–	COMB.	N–LMICH	
M	4	4393	6.864	6.847	−0.017	−0.2%	COMB.	N–LMICH	
M	5	2184	3.413	3.389	−0.023	−0.7%	COMB.	N–LMICH	
M	6	3354	5.241	5.281	0.040	0.8%	COMB.	N–LMICH	
M	7	1002	1.566	1.566	–	–	COMB.	N–LMICH	
M	8	1045	1.633	1.531	−0.102	−6.7%	COMB.	N–LMICH	
M	9	5362	8.378	8.378	–	–	COMB.	N–LMICH	
M	10 A	3109	4.858	4.884	0.026	0.5%	COMB.	N–LMICH	Former 10–1

TABLE 1

REVIEW LISTING OF SCA'S, SEWER SYSTEMS AND INPUT FILES
(continued)

SUBBASIN ID #		MAP AREA acres	MAP AREA sq. miles	MODEL AREA sq. miles	DIFFERENCE sq. miles	PERCENT DIFFERENCE	SEWER SYSTEM	SCALP FILE	COMMENTS
M	10 B	1465	2.289	2.308	0.019	0.8%	COMB.	N-LMICH	Former 10-2
M	11	1064	1.663	1.655	-0.008	-0.5%	COMB.	N-LMICH	
M	12	219	0.342	0.342	-	-	COMB.	N-LMICH	
M	13	720	1.125	1.111	-0.014	-1.3%	COMB.	N-LMICH	
M	14	691	1.080	1.080	-	-	COMB.	N-LMICH	
M	15	1365	2.133	2.133	-	-	COMB.	N-LMICH	
M	16	5416	8.463	8.511	0.049	0.6%	COMB.	N-LMICH	
M	17	626	0.978	0.978	-	-	COMB.	N-LMICH	
M	18	2107	3.292	3.292	-	-	COMB.	N-LMICH	
M	19	922	1.441	1.441	-	-	COMB.	N-LMICH	
M	20	1572	2.456	2.481	0.025	1.0%	COMB.	N-LMICH	
M	21	1110	1.734	1.748	0.014	0.8%	COMB.	W-LMICH	
M	22	126	0.197	0.194	-0.003	-1.5%	COMB.	W-LMICH	
M	23	1746	2.728	2.767	0.039	1.4%	COMB.	W-LMICH	
M	24	109	0.170	0.181	0.011	6.1%	COMB.	W-LMICH	
M	25	1435	2.242	0.469	-1.773	-378.0%	COMB.	N-LMICH	
M	26			0.150	0.150	100.0%	COMB.	W-LMICH	
M	27	777	1.214	1.222	0.008	0.7%	COMB.	W-LMICH	
M	28	362	0.566	0.416	-0.150	-36.0%	COMB.	W-LMICH	
M	29	846	1.322	1.353	0.031	2.3%	COMB.	W-LMICH	
M	30	811	1.267	1.302	0.035	2.7%	COMB.	W-LMICH	
M	31	338	0.528	0.528	-	-	COMB.	W-LMICH	
M	32	1022	1.597	1.644	0.047	2.9%	COMB.	W-LMICH	
M	33	392	0.613	0.634	0.021	3.3%	COMB.	W-LMICH	
M	34	3263	5.098	5.108	0.010	0.2%	COMB.	W-LMICH	
M	35	1012	1.581	1.553	-0.028	-1.8%	COMB.	W-LMICH	
M	36	2395	3.742	3.662	-0.080	-2.2%	COMB.	W-LMICH	
M	37	1195	1.867	1.828	-0.039	-2.1%	COMB.	W-LMICH	
M	38	268	0.419	0.398	-0.021	-5.3%	COMB.	W-LMICH	
M	39	583	0.911	0.928	0.017	1.8%	COMB.	W-LMICH	
M	40	14252	22.269	22.269	-	-	COMB.	W-LMICH	
M	41	5089	7.952	7.952	-	-	COMB.	W-LMICH	
M	42	1364	2.131	2.131	-	-	COMB.	W-LMICH	
M	43	1561	2.439	2.439	-	-	COMB.	W-LMICH	
M	44	338	0.528	0.488	-0.040	-8.2%	COMB.	W-LMICH	
M	45	977	1.527	1.525	-0.002	-0.1%	COMB.	W-LMICH	
M	46	1657	2.589	2.595	0.006	0.2%	COMB.	W-LMICH	
M	47	3267	5.105	5.145	0.040	0.8%	COMB.	W-LMICH	
M	48 DP	1033	1.614	1.500	-0.114	-7.6%	COMB.	W-LDP	
M	48 LM	8708	13.606	13.667	0.061	0.4%	COMB.	W-LMICH	
M	49 A	1403	2.192	2.175	-0.017	-0.8%	COMB.	W-LMICH	Former 49-1
M	49 BDP	1418	2.216	2.093	-0.123	-5.9%	COMB.	W-LDP	
M	49 BLM	578	0.903	0.999	0.096	9.6%	COMB.	W-LMICH	Former 49-2
M	50	6478	10.122	10.122	-	-	COMB.	W-LMICH	
M	51	11300	17.656	17.656	-	-	COMB.	W-LMICH	
M	52	4450	6.953	6.948	-0.005	-0.1%	COMB.	W-LMICH	
M	53	2613	4.083	4.083	-	-	COMB.	W-13A	
M	55 A	988	1.544	1.539	-0.005	-0.3%	COMB.	W-LDP	Former 55-1
M	55 B	2071	3.236	3.216	-0.020	-0.6%	COMB.	W-LDP	Former 55-2
MN	1 A	302	0.472	0.472	-	-	COMB.	N-LMICH	Former N1-1
MN	1 B	1416	2.213	2.002	-0.211	-10.5%	COMB.	N-LMICH	Former N1-2
MN	1 C	1768	2.763	2.763	-	-	COMB.	N-LMICH	Former N1-3
MN	1 D	544	0.850	0.850	-	-	COMB.	N-LMICH	Former N1-4
MN	1 E	640	1.000	1.000	-	-	COMB.	N-LMICH	Former N1-5
MN	2 A	265	0.414	0.414	-	-	COMB.	N-LMICH	Former N2-1
MN	2 B	669	1.045	1.045	-	-	COMB.	N-LMICH	Former N2-2
MN	2 C	464	0.725	0.725	-	-	COMB.	N-LMICH	Former N2-3
MN	3	1316	2.056	1.475	-0.581	-39.4%	COMB.	N-LM-NBC	
MN	4	1383	2.161	2.228	0.067	3.0%	COMB.	N-LM-NBC	
MN	5	948	1.481	1.481	-	-	COMB.	N-LM-NBC	
O	2	5010	7.828						
O	2 A	1378	2.153						
O	2 A	64	0.100						
O	3 C	223	0.348						
O	301	115	0.180						
O	302	135	0.211						
O	381	767	1.198						
O	382	467	0.730						
O	383	243	0.380						
O	384	211	0.330						

PRECIPITATION GAGE ASSIGNMENT

Objective

Through WY 1989, a network of 13 precipitation gages was the basis of rainfall inputs to the HSPF and SCALP models used for diversion accounting. The current study updates these model inputs to reflect the implementation of a new 25-gage network. This requires assignment of all land areas in the study region to specific gages in the network. The work items involved in the gage assignment task include the following.

- Identify gages and devise a numbering system
- Map gage locations in the CAD system
- Delineate areas to be assigned to each gage and map the delineation

The Thiessen Network or Thiessen Polygon method (Linsley & Franzini, 1979) is the selected approach to delineating and assigning the gages.

Gage Identification and Numbering

A list identifying the 25 precipitation gages in the new network is provided in Table 2. Detailed information describing the location, layout, equipment and maintenance of each gage is provided in an Illinois State Water Survey (ISWS) report titled, " Installation and Operation of a Dense Raingage Network to Improve Precipitation Measurements for Lake Michigan Diversion Accounting: Water Year 1990," (Peppler, 1991). The gage numbering provided in the first column of Table 2 is taken from the ISWS report. Names of eight letters or fewer are needed for computer directory names. TSS (Time Series Store) data set numbers are required to complete the updates of the HSPF and SCALP model inputs. The rainfall TSS number identifies the gage for precipitation input to HSPF. The remaining columns of TSS numbers identify the various HSPF output time series which also serve as SCALP inputs. Forest area HSPF outputs have been assigned only to gages that have forest land cover within their delineation.

TABLE 2

IDENTIFICATION OF THE 25–GAGE PRECIPITATION NETWORK AND TEMPORARY TSS DATA SET NUMBERS

GAGE NUMBER	DIRECTORY NAME	NAME	USGS QUAD MAP	TEMPORARY TSS DATA SET NUMBERS					
						GRASS		FOREST	
				RAINFALL	IMPRO	OLFRO	SUBRO	OLFRO	SUBRO
1	NORTHBRK	NORTHBROOK	PARK RIDGE	10	11	12	13		
2	WINNETKA	WINNETKA	PARK RIDGE	20	21	22	23		
3	DESPLNS	DES PLAINES	ARLINGTON HEIGHTS	30	31	32	33		
4	SKOKIE	SKOKIE	PARK RIDGE	40	41	42	43		
5	FRANKPK	FRANKLIN PARK	ELMHURST	50	51	52	53		
6	BRICKTWN	BRICKTOWN	RIVER FOREST	60	61	62	63		
7	DIVRSYHR	DIVERSEY HARBOR	CHICAGO LOOP	70	71	72	73		
8	WESTCHES	WESTCHESTER	HINSDALE	80	81	82	83		
9	CICERO	CICERO	BERWYN	90	91	92	93	94	95
10	CHINATWN	CHINATOWN	ENGLEWOOD	100	101	102	103		
11	LAGRANGE	LAGRANGE	BERWYN	110	111	112	113	114	115
12	BEDPARK	BEDFORD PARK	BERWYN	120	121	122	123	124	125
13	ENGLEWD	ENGLEWOOD	ENGLEWOOD	130	131	132	133		
14	SOUTHWTR	SOUTH WATER PLANT	JACKSON PARK	140	141	142	143		
15	LEMONT	LEMONT	SAG BRIDGE	150	151	152	153	154	155
16	PALOSPK	PALOS PARK	PALOS PARK	160	161	162	163	164	165
17	ALSIP	ALSIP	PALOS PARK	170	171	172	173	174	175
18	WESTPULL	WEST PULLMAN	BLUE ISLAND	180	181	182	183	184	185
19	POWCONAR	POWERS CONSRV. AREA	LAKE CALUMET	190	191	192	193		
20	ORLANDPK	ORLAND PARK	MOKENA	200	201	202	203	204	205
21	TINLEYPK	TINLEY PARK	HARVEY	210	211	212	213	214	215
22	HARVEY	HARVEY	HARVEY	220	221	222	223	224	225
23	LANSING	LANSING	CALUMET CITY	230	231	232	233	234	235
24	MATTESON	MATTESON	HARVEY	240	241	242	243	244	245
25	CHICHTS	CHICAGO HEIGHTS	CALUMET CITY	250	251	252	253	254	255

Gage Locations and Mapping

To obtain an accurate gage delineation, it is necessary to map the gage locations precisely in the CAD mapping system. Mapping of gage locations was accomplished using longitude and latitude coordinates provided for each gage in the ISWS report. The longitude and latitude coordinates were converted to Universal Transverse Mercator (UTM) grid coordinates for mapping purposes because the UTM grid system is the basis of the available CAD mapping.

The coordinate conversions were performed using Geodimeter Survey Software, Version 4.5 and equations and constants published in NOAA manual NOS NGS 5 - "State Plane Coordinate System of 1983". Computer program output summarizing the computations is provided in Appendix B. The gage locations were then mapped according to UTM grid coordinates.

Gage locations were checked by printing drawings at a scale of 1" = 2,000' and overlaying the drawings on USGS 7-1/2 minute maps. The plotted gage locations were compared with the physical locations described in the ISWS report. This process verified the conversion procedure as well as the latitude and longitude coordinates obtained from the ISWS report.

Delineation of Gage Areas

The Thiessen Polygon method uses triangulation to define the region that is closer to a particular point (e.g. a rain gage) than all other points defined in the network. The triangulation lines are constructed by connecting each gage location with each of its immediate neighbors. This process frequently will result in pairs of lines that intersect. Such intersections are eliminated by deleting the longer of the two intersecting lines. What remains is a system of compact triangles connecting the gage locations. This system is the set of smallest triangles defined by the gage points.

Next, a perpendicular bisector is extended from the midpoint of each triangulation line. Each perpendicular bisector is extended until it intersects another bisector. The bisectors will always meet in groups of three (except in the unusual case where four adjacent gages define a perfect square, in which case four bisectors will intersect in the center). When all of the bisectors have been extended and trimmed, the Thiessen Polygons are complete and the triangulation lines may be discarded.

The completed Thiessen Polygons for the 25-gage network are shown in Figure 2. The gage delineation lines define the area assigned to the gage located in each polygon. Each point in the study area has been assigned to the gage nearest to it. The CAD drawing layers containing the gage locations, triangulation lines and Thiessen Polygons have been added to the diversion accounting CAD mapping system.

LAND COVER ANALYSIS

Objective

The SCALP modeling requires estimates of impervious and pervious land cover areas for each SCA. Modeling of the "ungaged" watersheds requires estimates of forested area as well. With the study area in excess of 480 square miles, it is not cost-effective to conduct a direct delineation of land cover areas. Therefore, an indirect approach based on land use is employed. The following describes the procedure to estimate impervious, pervious and forest cover areas throughout the study area.

Approach

The land cover analysis uses a matrix multiplication procedure to convert land use areas to land cover areas, where land cover is one of impervious, pervious or forest. There are eleven defined land uses. These land uses have been assigned specific proportions of

pervious and impervious area, as shown in Table 3. The information in Table 3 is derived from standard sources such as the Soil Conservation Service Technical Release 55 (TR-55), (SCS, 1981).

Table 3 represents a 11 by 2 matrix that is the key to the land cover computations. Land use areas were determined in each of the SCA's and ungaged areas. These land use assignments yield a matrix that is 209 subbasins by 11 land uses. The 209 by 11 matrix of land uses is multiplied by the 11 by 2 land use to land cover conversion matrix to obtain a 209 by 2 matrix of the desired impervious and pervious areas in each subbasin.

There are slight variations in the above approach when forest land use is considered. Forest land use was assigned only in the ungaged areas where forest land use was translated directly into forest land cover. The ungaged areas were partitioned into 15 subareas for the analysis and are included in the above total of 209 subbasins.

Land Use Delineation

Land use, as defined in Table 3, was delineated for the study area on 1" = 400' scale aerial photographs. Overlay plots showing the SCA and Thiessen Polygon boundaries were prepared at 1" = 400'. Geographic features were also plotted on the overlays to aid in aligning the overlays on the aerial photos. These 161 aerial photo sheets cover the entire study area. Figure 3, located after the appendices at the end of this report, shows the location and numbering of each sheet. The sheets overlap one to two inches on each side. Match lines were drawn on the overlays to coincide with the outlines shown on Figure 3, to ensure there would be no overlap in the land use delineations from sheet to sheet.

Land use areas were delineated on the overlays using the color scheme shown in Table 3. The outline of each land use area was drawn and labelled. For residential areas, the technicians performed spot checks of dwelling unit density until they became proficient at identifying residential densities by eye. The residential densities indicated in Table 3 include streets and small parks in the aggregate lot sizes.

TABLE 3

LAND COVER PROPORTIONS
FOR EACH LAND USE

LAND USE	PERCENT IMPERVIOUS	PERCENT GRASS	AERIAL MAP COLOR
Forest	0	100 *	
Open Space / Park	5	95	
Low Density Residential: (1.1 acre median lot)	19	81	
Medium Density Residential: (1/2 acre median lot)	40	60	
High Density Residential: (1/5 acre median lot)	56	44	
Multifamily and High Rise	70	30	
Commercial	85	15	
Industrial	72	28	
Highway Corridor:			
with Grassed Median	50	50	
No Median	80	20	
Open Water	100	0	OPEN WATER

* In the ungaged watersheds, forest land use is translated to 100 percent forest land cover.

Land use areas within each SCA were measured using an electronic planimeter. Many SCA's extended across several sheets but areas were recorded on a sheet by sheet basis. The measured areas were entered into a spreadsheet titled LUBASE. A listing of this spreadsheet is provided in Appendix C. Note that the forest land use was assigned only in the ungaged watersheds which are indicated by the prefixes CAL (Calumet) and LDP (Lower Des Plaines).

Land Cover Computations

The measured land use areas recorded in LUBASE were sorted by subbasin identifier so that records for SCA's located on several sheets appeared in sequence. The sorted data appear in a file named LUSORT. The sorted data were exported to an ASCII data file for input to a small accumulating program written for the project. The accumulator program, called ACC, computes the total area in each land use by adding together the measured areas obtained from individual map sheets. The result is a database consisting of the area of each land use in each SCA. The land use database is the matrix that is multiplied by the conversion matrix in Table 3 to obtain the two-column matrix of land cover in each SCA. This database is listed in Appendix D.

The next step is to partition each SCA according to gage assignment. The gage assignment breakdown for the SCA's was computed electronically by the CAD software. Of the 197 SCA's analyzed, 78 are split into two or more parts by gage assignment boundaries. At most the SCA's were split into four parts. Land cover in the individual partitions was determined using the ratio of the area of each partition to the total area of the SCA. Each partitioned area has the same percent impervious as the entire SCA it comes from. Exceptions were made for two very large SCA's, M40 and M51. Each gage assignment partition in M40 and M51 was subjected to an independent land use analysis which was converted to land cover by the previously described method. Results for these SCA's differed slightly from results using the area ratio method. Therefore, all other SCA's were divided into gage areas using the area ratio rather than individual gage area analysis.

Results

The matrix multiplication yields land cover areas (impervious area and grass area) for each SCA. The results are provided in Appendix E. An overall summary of the results by major area is in Table 4. As shown, the Calumet, Des Plaines and Mainstream, North Leg combined sewer areas were found to have overall imperviousness of approximately 55 percent while the Mainstream area is approximately 60 percent impervious area. The ungaged areas had significantly less impervious area.

The SCA land cover areas were split in proportion to the area assigned to each gage within the SCA. The results of the partitioning is shown in Appendix F expressed in acres and in Appendix G in square miles. The data in Appendix G was the source of information for updating the SCALP model representation of each SCA.

Land Cover Analysis for the Ungaged Watersheds

The analysis to determine land cover in the ungaged watershed is roughly equivalent to the approach used for the SCA land cover. Key differences are the introduction of forest land cover and that each gage assignment partition was analyzed individually. The relevant analysis data are present in Appendices C and D. The prefixes CAL and LDP are used to identify the Calumet and Lower Des Plaines ungaged watersheds respectively. Areas are numbered according to the precipitation gage assigned to the area. Land cover results for the ungaged areas are provided in Table 5. As shown, the gage assignments partition the Calumet ungaged area into eight areas and the Lower Des Plaines area into seven areas.

COMPUTER FILE REVISIONS

The HSPF and SCALP model input files used for diversion accounting were revised to reflect implementation of the 25-gage precipitation network and the updated land cover computations. The new 25-gage network affects both the HSPF and SCALP models while

TABLE 4

LAND COVER RESULTS BY MAJOR DRAINAGE AREA

BASIN	BASIN AREA (SQ. MI.)	IMPERV. AREA (SQ. MI.)	GRASS AREA (SQ. MI.)	FOREST AREA (SQ. MI.)	PERCENT IMPERV.	PERCENT GRASS	PERCENT FOREST
GAGED BASINS							
CALUMET	88.042	47.711	40.332	0.000	54.2%	45.8%	0.0%
DES PLAINES	32.323	17.960	14.321	0.000	55.6%	44.3%	0.0%
MAINSTREAM	205.741	127.184	82.386	0.000	61.8%	40.0%	0.0%
MAINSTREAM–NORTH LEG	15.180	8.298	6.882	0.000	54.7%	45.3%	0.0%
UNGAGED BASINS							
UNGAGED CALUMET	84.166	33.862	45.696	4.607	40.2%	54.3%	5.5%
UNGAGED LOWER DES PLAINES	57.914	19.308	21.424	17.182	33.3%	37.0%	29.7%

TABLE 5

LAND COVER RESULTS FOR THE UNGAGED WATERSHEDS

BASIN	GAGE NUMBER	BASIN AREA (ACRES)	AREA SQ. MI.	% OF TOTAL AREA	SQ. MI. IMPERV	SQ. MI. GRASS	SQ. MI. FOREST
CAL 12	12	1601	2.502	3.0%	1.421	1.080	0.000
CAL 16	16	1913	2.989	3.6%	0.791	2.137	0.061
CAL 17	17	15870	24.797	29.5%	10.421	12.353	2.023
CAL 18	18	1203	1.880	2.2%	0.632	1.248	0.000
CAL 20	20	5355	8.367	9.9%	3.292	5.068	0.007
CAL 21	21	17775	27.773	33.0%	9.712	15.545	2.516
CAL 22	22	8130	12.703	15.1%	6.395	6.308	0.000
CAL 24	24	2019	3.155	3.7%	1.198	1.957	0.000
CAL TOTAL		53866	84.166	100.0%	33.862	45.696	4.6071
LDP 9	9	9	0.014	0.02%	0.010	0.004	0.000
LDP 11	11	8453	13.208	22.8%	4.716	4.751	3.741
LDP 12	12	6556	10.244	17.7%	6.041	4.202	0.000
LDP 15	15	873	1.364	2.4%	0.372	0.992	0.000
LDP 16	16	18008	28.137	48.6%	5.539	9.175	13.423
LDP 17	17	2547	3.980	6.9%	2.206	1.774	0.000
LDP 20	20	619	0.967	1.7%	0.424	0.525	0.018
LDP TOTAL		37065	57.914	100.0%	19.308	21.424	17.182

the updated land cover areas are reflected only in the SCALP models. The following provides specific information regarding the model input revisions.

HSPF Revisions

HSPF model input files to model impervious and grass (pervious) areas were created for each of the 25 precipitation gages. There are also models for forest area for the 13 gages that have forest area in their gage assignment area. The filenames IMPERV.INP, GRASS.INP and FOREST.INP are used in each case. The gage assignments are differentiated by storing the files in directories bearing the name listed in Table 2. In each file, the TSS number indicating precipitation input was changed to the number given in Table 2. The impervious area HSPF runs produce a single TSS output labeled IMPRO. The grass and forest area HSPF runs produce two output TSS called OLFRO and SUBRO. The EXTERNAL TARGETS references to these outputs in the HSPF inputs were changed to the numbering shown in Table 2. In all, 63 HSPF input files were created and submitted to the Corps.

SCALP Revisions

For diversion accounting, SCALP model inputs are arranged into 15 computer files. Eight of these contain inputs for combined sewer areas. Only the combined sewer SCALP inputs were revised in this study. The main model revisions are for the TSS file references that specify the runoff inputs and applicable land area. The runoff input TSS file numbers are given in the DATA block of the input, and the land areas used to compute total hourly runoff volume are specified in the LANDS subblock of each SCA block. The required land areas are the impervious and pervious land area within each gage assignment area in each SCA. TSS numbering was according to the number given for IMPRO, OLFRO and SUBRO time series in Table 2, and land areas were those given in Appendix G. The land area for SUBRO unit runoff was taken to be the pervious land area. The revised SCALP input files were delivered to the Corps on diskette.

REFERENCES

Linsley, R.F., and L.B. Franzini, Water Resources Engineering, McGraw-Hill, New York, 1979.

Peppler, R.A., "Installation and Operation of a Dense Raingage Network to Improve Precipitation Measurements for Lake Michigan Diversion Accounting: Water Year 1990," Illinois State Water Survey Contract Report 517, Champaign, IL, September 1991.

U.S. Department of Agriculture, Soil Conservation Service, "Urban Hydrology for Small Watersheds," SCS Technical Report No. 55, 1986.

U.S. Department of Commerce - National Oceanic and Atmospheric Administration, "State Plane Coordinate System of 1983," manual NOS NGS 5, 1984.

Microstation is a registered trademark of Intergraph Corp.

AutoCAD is a registered trademark of Autodesk Inc.

R/USARMY/REFERENCE.EDL

APPENDIX A

INDEX TO CAD DRAWING LEVELS

Filename: BORDERS2.DSN

Microstation Level	AutoCAD Layer Name	Description
01	LIBERTYVILLE	USGS Quad Border
02	WAUKEGAN	USGS Quad Border
03	LAKE ZURICH	USGS Quad Border
04	WHEELING	USGS Quad Border
05	HIGHLAND PARK	USGS Quad Border
06	STREAMWOOD	USGS Quad Border
07	PALATINE	USGS Quad Border
08	ARLINGTON HEIGHTS	USGS Quad Border
09	PARK RIDGE	USGS Quad Border
10	EVANSTON	USGS Quad Border
11	ELMHURST	USGS Quad Border
12	RIVER FOREST	USGS Quad Border
13	CHICAGO LOOP	USGS Quad Border
14	HINSDALE	USGS Quad Border
15	BERWYN	USGS Quad Border
16	ENGLEWOOD	USGS Quad Border
17	JACKSON PARK	USGS Quad Border
18	ROMEOVILLE	USGS Quad Border
19	SAG BRIDGE	USGS Quad Border
20	PALOS PARK	USGS Quad Border
21	BLUE ISLAND	USGS Quad Border
22	LAKE CALUMET	USGS Quad Border
23	MOKENA	USGS Quad Border
24	TINLEY PARK	USGS Quad Border
25	HARVEY	USGS Quad Border
26	CALUMET CITY	USGS Quad Border
27	HIGHLAND	USGS Quad Border
28	GARY	USGS Quad Border
29	FRANKFORT	USGS Quad Border
30	STEGER	USGS Quad Border
31	DYER	USGS Quad Border
32	ST. JOHN	USGS Quad Border
33	BEECHER WEST	USGS Quad Border
34	U.S ARMY BORDER	USCOE Standard Border and Title Block

Filename: OVERALL.DGN

Microstation Level	AutoCAD Layer Name	Description
01		Arrowheads
02		
03	AREA–SCA	SCA Boundaries
04	TARP–DS	Drop Shafts
05	TARP–CO	Construction Shafts
06	AREA–DLM	Diverted Lake Michigan Watershed Boundary
07	TARP–TUNN	Tunnels
08	TARP–PS	Pump Stations
09	TARP–STP	Treatment Plants
10	MISC–FCRR	Reservoirs (Flood Control)
11	TARP–RES	Reservoirs (Overflow)
12	MSDG–ISEW	Interceptors
13	AREA–LDP	Ungaged Watershed Boundary
14	TARP–CS	Connecting Structures
15	TARP–AS	Access Shafts
16	MSDG–TPB	FPA Boundaries
17	AREA–NDA	Obsolete (1975) SCA Boundaries
18	TARP–IS	Intercepting Structures
19		
63		Overall Border

Filename: GAGE.DGN

Microstation Level	AutoCAD Layer Name	Description
01	GAGES	Gage Locations
02	GAGEDELIN	Delineations
03	TRIANGLE	Triangulation Lines

Filename: RIVERS.DGN

Microstation Level	AutoCAD Color	Description
01	WHITE, GREEN	Railroads
02	WHITE	Airports
03	WHITE, MAGENTA	Miscellaneous
04	BLUE	Rivers/Streams
05	WHITE	Intermittent Streams

APPENDIX B

COORDINATE CONVERSION COMPUTER OUTPUT

GEODIMETER SUPVEYING SOFTWARE - V4.5
STATE PLANE/UTM - GEODETIC COORDINATE COMPUTATIONS
NAD 1983, UTM, ZONE 16

UTM Northing meters	UTM Easting meters	Geodetic Latitude dd mm ss.sssss	Geodetic Longitude ddd mm ss.sssss	Grid Angle dd mm ss.s	Grid Factor
4662415.762	428233.994	42 06 38.00000	87 52 05.00000	-0 34 55.6	0.9996
4662015.928	437876.694	42 06 28.00000	87 45 03.00000	-0 30 13.8	0.9996
4652615.720	427375.524	42 01 20.00000	87 52 38.00000	-0 35 14.1	0.9996
4652982.669	437406.363	42 01 35.00000	87 45 22.00000	-0 30 22.4	0.9996
4642654.822	427181.387	41 55 57.00000	87 52 42.00000	-0 35 13.1	0.9996
4643178.318	436951.346	41 56 17.00000	87 45 38.00000	-0 30 29.9	0.9996
4642661.139	447448.638	41 56 03.00000	87 38 02.00000	-0 25 25.0	0.9996
4632911.145	426874.077	41 50 41.00000	87 52 51.00000	-0 35 15.5	0.9996
4633029.161	437138.127	41 50 48.00000	87 45 26.00000	-0 30 18.7	0.9996
4632765.469	446798.983	41 50 42.00000	87 38 27.00000	-0 25 39.1	0.9996
4623311.927	427537.821	41 45 30.00000	87 52 18.00000	-0 34 50.0	0.9996
4623187.367	437467.112	41 45 29.00000	87 45 08.00000	-0 30 03.5	0.9996
4623263.169	447189.994	41 45 34.00000	87 38 07.00000	-0 25 23.2	0.9996
4622995.513	454739.540	41 45 27.00000	87 32 40.00000	-0 21 45.3	0.9996
4614420.455	416949.643	41 40 38.00000	87 59 52.00000	-0 39 48.6	0.9996
4612732.802	427523.239	41 39 47.00000	87 52 14.00000	-0 34 43.4	0.9996
4614057.679	437502.961	41 40 33.00000	87 45 03.00000	-0 29 57.3	0.9996
4614052.182	445757.799	41 40 35.00000	87 39 06.00000	-0 25 59.9	0.9996
4613524.879	455119.003	41 40 20.00000	87 32 21.00000	-0 21 30.5	0.9996
4604133.902	426903.854	41 35 08.00000	87 52 37.00000	-0 34 55.5	0.9996
4604218.414	437579.382	41 35 14.00000	87 44 56.00000	-0 29 49.6	0.9996
4603957.620	447024.515	41 35 08.00000	87 38 08.00000	-0 25 18.7	0.9996
4603963.907	455175.005	41 35 010.0000	87 32 16.00000	-0 21 25.0	0.9996
[redacted]					
4596867.342	438836.821	41 31 16.00000	87 43 59.00000	-0 29 09.4	0.9996

1596705.254 452116.694 41 31 14.00000 87 34 26.00000 -0 22 47.6 0.9976

APPENDIX C

RAW LAND USE DATA IN ACRES AS MEASURED DIRECTLY FROM THE 1"=400' SCALE AERIAL PHOTO OVERLAYS

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
1	M 1			938.13				40.30					978.43
3	M 1				75.13								75.13
3	M 4		279.52	374.60	5.19		45.35	300.60			46.07		1051.33
4	M 1			1500.28				185.79				16.65	1702.72
4	M 2			159.03				22.45				52.10	233.58
4	M 4			121.72				1.70					123.42
5	M 1			56.79	55.88							50.05	162.72
5	M 2			23.49	231.39		162.04	173.64				21.56	612.12
6	DP 1			201.64			17.80	43.25					262.69
6	DP 2 A			36.19									36.19
6	DP 2 A			19.43									19.43
6	DP 3			10.90				11.96					22.86
6	DP 4			4.69				7.21	7.33				19.23
7	DP 1			11.72				6.59					18.31
7	DP 2 A			16.38									16.38
7	DP 2 B			258.41	70.16			43.75		6.63			378.95
7	DP 3			12.19				8.39					20.58
8	DP 2 B			18.05									18.05
8	MN 4		131.57	549.64	62.33			46.92					790.46
8	DP 5			36.76									36.76
9	MN 3				102.00				43.11	11.31			156.42
9	MN 4			198.50				7.12					205.62
9	MN 5			622.89					234.77				857.66
9	M 6			1.65					2.06	0.39			4.10
10	MN 3			83.48									83.48
10	M 4		192.13	1564.45			51.41	62.87	183.55		44.46		2098.87
10	MN 5			62.65			14.22				13.47		90.34
11	M 2			396.75				74.97	61.85			55.38	588.95
11	M 3			379.75				23.88	179.56			60.78	643.97
11	M 4			687.99	116.13			63.78	131.07				998.97
11	M 6			3.95					8.82				12.77
12	M 2			332.31			684.53	98.02					1114.86
12	M 3			50.04			6.45						56.49
13	DP 3			111.83									111.83
13	DP 4			18.98	120.76				206.03				345.77
14	DP 3			34.08				6.65					40.73
14	DP 5			375.52	66.79			53.10					495.41
14	DP 6		18.89	122.06			35.20	12.69					188.84
14	DP 7		46.70		166.38			34.57					247.65
15	MN 1 A			302.00									302.00
15	MN 1 B			385.76				68.49					454.25
15	MN 1 C			5.30									5.30
15	MN 4			180.90									180.90
15	DP 5		125.18	23.99	419.71	51.31		63.64					683.83
15	DP 6			293.57	117.97			104.62					516.16
15	DP 7			452.34				43.10					495.44

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
16	MN 1 B		376.70	160.83				102.10					639.63
16	MN 1 C			31.80				4.61					36.41
16	MN 2 A				265.00								265.00
16	MN 2 B				152.19								152.19
16	MN 3		142.85						738.71				881.56
16	MN 4			148.48				57.54					206.02
16	M 6								175.92	11.64			187.56
16	M 25		201.03										201.03
17	MN 2 B		6.95	47.43	117.21								171.59
17	MN 3			45.21	3.83			10.48	134.39		0.63		194.54
17	M 4			48.77			11.16	60.48					120.41
17	M 6		128.41	995.98	504.00		8.41	282.03	26.01		81.12		2025.96
18	M 3			174.39					132.92			22.46	329.77
18	M 5		13.56	802.19				131.02	0.00			131.79	1078.56
18	M 6			428.81				97.95	596.85				1123.61
19	M 2		88.55	63.18			64.68		17.08				233.49
19	M 3			28.57			29.20						57.77
19	M 5		87.22	85.78			482.33	56.61	4.07				716.01
19	M 9		1.91				766.47	6.34	39.92				814.64
20	MN 1 B			150.12				55.26					205.38
20	MN 1 C			323.46				108.10		73.04			504.60
20	DP 7			156.86	55.05								211.91
20	DP 8			201.58	126.33			94.09					422.00
20	DP 9			253.23	233.19		194.12	64.02		5.44			750.00
21	MN 1 B			62.79	28.04			25.91					116.74
21	MN 1 C			806.02	199.78			166.92		48.97			1221.69
21	MN 1 D			254.97				6.62					261.59
21	MN 1 E			140.80				16.71		16.49			174.00
21	M 10 A		14.43	517.85						4.09			536.37
21	M 25		168.82	3.85									172.67
22	MN 1 D		21.01	163.02				8.00	90.38				282.41
22	MN 1 E		22.54	338.73				47.76	30.10		26.87		466.00
22	MN 2 B			194.92							4.91		199.83
22	MN 2 C				428.97				7.44		27.59		464.00
22	M 8			50.25					156.45				206.70
22	M 10 A			218.19				108.66	10.32		16.49		353.66
22	M 10 B							1.06					1.06
22	M 25		622.70	62.45	15.32						21.86		722.33
23	MN 2 B			78.54				54.30	12.55				145.39
23	M 5			165.18								10.46	175.64
23	M 7		19.14	548.06				127.59				21.22	716.01
23	M 8		180.41	473.27				164.05	2.51			18.06	838.30
23	M 9			173.29					63.33			6.42	243.04
23	M 10 A			70.64				45.58				3.64	119.86
23	M 25		150.69	142.92				45.36					338.97
24	M 5		16.48				124.00	73.31					213.79
24	M 7		271.50	14.49									285.99
24	M 9		204.71	1390.17			427.93	183.68	51.31	37.16			2294.96
26	DP 12			120.71				89.29					210.00
26	DP 13			103.90				19.72	31.38				155.00

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
26	DP 14			236.95				11.81	123.50				372.26
26	DP 15							0.97					0.97
27	DP 10			154.00									154.00
27	M 10 A								64.66				64.66
27	DP 11		14.64	781.60									796.24
27	M 16			89.95				10.24					100.19
28	M 10 A		172.57	737.02				117.06					1026.65
28	M 10 B		59.46	184.48				33.16					277.10
28	M 16		56.86	1077.55				156.35					1290.76
29	M 10 A			563.89				141.52	77.57		70.34		853.32
29	M 10 B		40.08	473.13				133.72	16.54		29.84		693.31
29	M 16			824.07				138.23	121.06				1083.36
30	M 9			1.54				1.83				1.44	4.81
30	M 10 A			104.72				25.35	17.74		6.67		154.48
30	M 10 B		18.03	364.98				110.30	0.22				493.53
30	M 11		3.65	177.30				34.81	22.68			29.38	267.82
30	M 12			171.60				42.03	5.37				219.00
30	M 13			18.62				41.90	86.05			9.09	155.66
30	M 14		35.99	417.43				158.08	78.34		1.16		691.00
30	M 15							121.57	3.55			12.77	137.89
30	M 16			205.32				42.91	44.65		32.22		325.10
30	M 17			42.11				24.42	42.86		9.71		119.10
31	M 9		131.37	9.99			246.56	108.13					496.05
31	M 11		9.47	506.48			85.20	195.03					796.18
31	M 13			379.02				185.32					564.34
31	M 15			285.81				112.84					398.65
32	M 9		227.61				539.78				55.01		822.40
32	M 15							6.22					6.22
33	DP 14			80.03					51.71				131.74
33	DP 15		3.60	309.83					72.60				386.03
33	DP 16		4.58	129.20					11.22				145.00
33	DP 17 A			67.77									67.77
33	DP 17 B			41.93	205.68			48.24	145.15				441.00
33	DP 20								86.63				86.63
34	DP 11			73.54			27.81	52.41					153.76
34	DP 17 A		36.34	67.50				3.39					107.23
34	DP 18			162.90	53.76							36.34	253.00
34	DP 19			1062.90				72.03					1134.93
34	DP 20							78.37					78.37
35	M 16		59.77	1097.68				186.47	160.73				1504.65
35	DP 19			107.71				36.36					144.07
35	M 51		33.14	757.04				106.71	155.90				1052.79
36	M 16		32.12	806.45				151.85	71.49				1061.91
36	M 18		16.81	521.30			11.42	36.01	201.78				787.32
36	M 47			101.21				9.34	111.24				221.79
36	M 51		1.62	300.12				61.56	144.96				508.26
37	M 15							35.36	15.60			3.69	54.65
37	M 16			50.03									50.03
37	M 17			328.10				15.09	62.46		63.29	4.16	471.10

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
37	M 18			892.11				111.02	63.14	36.68	4.75		1107.70
37	M 20			780.75				74.48	14.94	31.44			901.61
37	M 23							6.04					6.04
37	M 47			10.73				1.65					12.38
38	M 9						4.95						4.95
38	M 15			459.59			103.14	37.20	133.79			18.05	751.77
38	M 17						35.80						35.80
38	M 18			36.69			4.90		151.79		18.60		211.98
38	M 19				443.74		84.76	36.25	118.89			22.49	706.13
38	M 20			289.28				37.80	91.46		9.01		427.55
38	M 21			84.49					78.31			6.99	169.79
38	M 22			17.84					84.84		3.74		106.42
38	M 23			85.06				41.79	2.06		0.89		129.80
38	M 24								0.10				0.10
39	M 9		368.11	1.23			311.81						681.15
39	M 15						15.82						15.82
39	M 19		19.08	91.06			63.57	42.16					215.87
39	M 21			67.42			217.74	23.03					308.19
40	DP 21						69.17		94.98				164.15
41	DP 21		16.03	309.70				30.10	149.02				504.85
41	DP 27			191.69					2.78				194.47
41	DP 31			179.09									179.09
41	DP 52			405.53	79.08				130.06				614.67
42	DP 22			44.00									44.00
42	DP 23		56.06	183.01			18.63	53.30					311.00
42	DP 24			77.00									77.00
42	DP 25 A			80.00									80.00
42	DP 25 B							30.00					30.00
42	DP 26		9.86	145.45	156.60			16.97	40.12				369.00
42	DP 27		14.35	82.88			10.47		21.83				129.53
42	DP 28			22.90			123.42		19.55				165.87
42	DP 29			139.86					4.14				144.00
42	DP 31			61.68			22.46						84.14
42	DP 52				136.33								136.33
42	DP 63		11.37		302.48		12.28	40.05					366.18
42	DP 64			39.24				6.21	4.55				50.00
43	M 51		43.15	1740.00	179.62		188.43	370.54	26.94				2548.68
43	DP 63				106.82								106.82
44	M 47			470.02			43.03	12.72	457.41				983.18
44	M 51		13.84	1091.76			151.74	31.12	442.31				1730.77
45	M 20		7.25	209.00					6.20				222.45
45	M 23		202.47	755.87					196.28				1154.62
45	M 27			207.99					13.09				221.08
45	M 32			6.83					17.91				24.74
45	M 36		58.87	229.46			0.79		349.96				639.08
45	M 47		33.29	216.37			51.79		50.78				352.23
46	M 20			20.39									20.39
46	M 21			1.40					88.80			17.42	107.62
46	M 22			5.86					13.72				19.58
46	M 23			380.25				33.45	31.59		10.25		455.54
46	M 24								99.70			6.96	106.66

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
46	M 27			468.29				16.27	58.33		11.03	2.00	555.92
46	M 28								149.34		8.91	12.82	171.07
46	M 29		11.80	294.42			14.87	77.96	283.59		55.80		738.44
46	M 32		5.06				152.21	73.03	168.13				398.43
47	M 21		35.63					428.69	30.40		29.68		524.40
47	M 24								2.24				2.24
47	M 28						20.77	25.10	131.78			13.28	190.93
47	M 29							69.15	28.39		10.02		107.56
47	M 30		10.77	611.74			67.62		81.49		25.99	13.39	811.00
47	M 31							116.16				30.50	146.66
47	M 32							43.12	0.71		7.25		51.08
47	M 44		92.75					143.62			15.53	20.32	272.22
48	DP 31			25.30									25.30
48	DP 32		0.68	16.66									17.34
48	DP 33		33.52	246.40							25.66		305.58
48	DP 34			438.37				40.34	13.29				492.00
49	DP 28			128.45				17.68					146.13
49	DP 30		17.20	261.78				38.14	3.69		14.19		335.00
49	DP 31			45.47									45.47
49	DP 32			122.86							19.80		142.66
49	DP 33			146.42									146.42
50	M 51		92.58	1695.54				195.98	9.94		64.80		2058.84
50	M 52		60.77	414.40				40.75	47.27		24.84		588.03
51	M 45			77.02			15.42						92.44
51	M 47			552.91			18.96	40.57	342.47		38.20		993.11
51	M 51			322.86				34.95					357.81
51	M 52			358.33				160.90	584.12				1103.35
52	M 36		9.51	497.92			53.46	28.29	66.41		58.50		714.09
52	M 37								43.22				43.22
52	M 42		162.99	197.62			78.67	144.61	247.54		5.15		836.58
52	M 43			294.27			141.73	44.21	72.70		22.54		575.45
52	M 45			18.43				2.29					20.72
52	M 47			227.45			27.70		14.77		24.88		294.80
53	M 32			55.22			135.27	266.98					457.47
53	M 35			98.08			135.59	169.33	126.32		21.23		550.55
53	M 36			127.14	114.58		204.91	333.29	125.90		56.53		962.35
53	M 37		19.28	307.82	13.39		7.99		167.32				515.80
53	M 42			18.23	7.79				2.12				28.14
54	M 31							162.07	8.18			21.09	191.34
54	M 32							84.04			6.24		90.28
54	M 33							140.08	222.59		18.71	10.62	392.00
54	M 34		149.38					701.79	298.97			21.06	1171.20
54	M 35							13.41	144.36		52.78	4.36	214.91
54	M 44		27.89					37.89					65.78
54	M 50							71.95	18.60				90.55
55	DP 53			23.88									23.88
55	DP 54			81.40									81.40
56	DP 35			98.00									98.00
56	DP 37			112.00									112.00
56	DP 38			555.52	43.55		59.69	39.62	1.62				700.00

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
56	DP 39			265.00									265.00
56	DP 40			131.00									131.00
56	DP 46			182.97					16.35				199.32
56	DP 50			11.75									11.75
56	DP 53			126.12									126.12
56	DP 54			71.60									71.60
57	DP 35			183.00									183.00
57	DP 36		48.42	161.89			24.00		24.69				259.00
57	DP 41			54.62	344.54			17.84					417.00
57	DP 42		2.85		100.44			0.71					104.00
57	DP 43			1.35	188.17			32.48					222.00
57	DP 44			7.81	199.19								207.00
57	DP 45			41.93	53.07								95.00
57	DP 47			17.75	17.85			0.40					36.00
57	DP 48							25.33					25.33
57	DP 49		10.60		15.41			23.27					49.28
57	DP 50		49.75	13.82									63.57
57	DP 51			4.15									4.15
57	DP 62				39.83			11.17					51.00
57	DP 66				25.67			11.33					37.00
58	M 51			1350.15				196.19	130.86				1677.20
58	M 52			666.18				37.93	275.33				979.44
59	M 46								16.70			25.54	42.24
59	M 45			279.17					306.06			21.66	606.89
59	M 47			65.55					343.96				409.51
59	M 52			679.99				199.29	731.09				1610.37
60	M 37								48.26				48.26
60	M 41			227.35					431.15		44.65		703.15
60	M 42			240.59				15.59	243.10				499.28
60	M 43		55.13	527.98					402.44				985.55
60	M 46			6.55					91.32			3.46	101.33
60	M 45			155.62					74.15		13.70	13.48	256.95
61	M 34			355.02					193.55		26.58	4.64	579.79
61	M 35								201.54				201.54
61	M 36								60.24			19.24	79.48
61	M 37			155.16					397.31		5.60	29.65	587.72
61	M 38			37.48				1.43	199.08		16.77	13.24	268.00
61	M 39		8.28	324.43				51.90	189.82		8.57		583.00
61	M 40								37.18				37.18
61	M 41		65.27	16.13					19.52				100.92
61	M 50			44.08					191.53			5.41	241.02
62	M 34			397.21			463.34	112.86	299.02		195.58		1468.01
62	M 35								37.29		7.71		45.00
62	M 50		41.73	5.98			735.62	49.60	92.42		64.02		989.37
63	M 50		69.63					41.54	20.00		18.84		150.01
64	M 53		122.90	369.79	136.37								629.06
64	DP 55			400.98				41.02					442.00
64	DP 56			150.03							0.97		151.00
64	DP 65		42.64	205.92							3.44		252.00
65	DP 46			1.89					16.79				18.68
65	DP 50			68.06				17.21	2.73				88.00

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
65	DP 51			191.79				21.06					212.85
65	M 53		96.17	1093.32	100.19			135.92	127.64				1553.24
66	LDP 9								9.00				9.00
66	LDP 11								121.96				121.96
66	LDP 12								81.00				81.00
66	DP 48								16.67				16.67
66	DP 49			300.60				39.12					339.72
66	DP 50			8.68									8.68
66	M 53			396.23					34.47				430.70
66	M 55 A			84.76					95.89			24.64	205.29
67	M 48 LM			5.22				3.88	24.77				33.87
67	M 49 A			254.08				34.18	584.13	55.81		29.29	957.49
67	M 49 BLM			293.96				35.28					329.24
67	M 51		270.19	439.23					511.99			42.41	1263.82
68	M 46				282.18			37.36	330.56	10.41			660.51
68	M 48 LM			611.21			46.31	123.63	503.49	32.54		17.44	1334.62
68	M 49 A			183.89				2.38	165.39	17.79		4.93	374.38
68	M 51								93.20			8.63	101.83
68	M 52								162.89			5.92	168.81
69	M 41			1158.93				126.05	660.99				1945.97
69	M 46			222.71				33.19	387.31				643.21
70	M 40		64.34	784.72					393.69				1242.75
70	M 41			327.13					686.47				1013.60
70	M 50								366.08				366.08
71	M 40		136.42	90.75			447.71		162.28				837.16
71	M 50		26.27	364.03			821.69		412.46		116.35		1740.80
72	M 40						98.04						98.04
72	M 50		101.75		87.11		1075.59				63.28		1327.73
73	LDP 11								184.58			9.49	194.07
73	M 55 B								71.72			7.43	79.15
74	LDP 11								358.02			38.08	396.10
74	LDP 12								146.39				146.39
74	M 55 A			327.71					455.00				782.71
74	M 55 B			55.64					868.71				924.35
75	M 48 DP			6.98									6.98
75	LDP 12								282.80				282.80
75	M 48 LM			134.63									134.63
75	M 49 A			71.13									71.13
75	M 49 BLM			157.28				21.04					178.32
75	M 55 B								690.98				690.98
75	M 49 BDP			1004.88				49.75	224.57				1279.20
76	LDP 12								195.62				195.62
76	M 46			86.41				0.97					87.38
76	M 48 LM			764.78				100.94	610.85				1476.57
76	M 49 BDP			62.64				21.84	54.32				138.80
76	M 55 B								376.52				376.52
76	M 49 BLM								70.44				70.44
76	M 48 DP			136.10				9.64	273.53				419.27

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
77	M 41			475.83				73.71	21.81				571.35
77	M 46			121.33					1.00				122.33
77	M 48 LM		314.42	1361.38				210.90					1886.70
77	M 48 DP			13.15				4.60					17.75
78	M 40		59.27	1358.43				128.99	176.98				1723.67
78	M 41			432.69				46.86	274.46				754.01
78	M 48 LM			67.74				18.44					86.18
79	M 40		203.06	321.82			1563.03	52.42	324.19		95.52		2560.04
79	M 50			18.24							4.10		22.34
80	M 40		283.86	40.42			482.14						806.42
80	M 50		598.07	18.97			724.11	35.42					1376.57
83	LDP 11	230.27	155.93	82.29	728.88				500.03	109.83		97.73	1904.96
84	LDP 11		515.19	696.03	318.36		80.23		107.29		38.84		1755.94
84	LDP 12			22.08	416.94			20.59	345.71				805.32
85	LDP 12		149.93	1586.21	120.52			72.37	647.93				2576.96
86	LDP 12			618.55				53.92	362.14				1034.61
86	CAL 12			164.23				14.41					178.64
86	M 48 LM		38.32	602.10				16.79	26.60				683.81
86	M 48 DP		1.78	122.01				30.40	381.84				536.03
87	CA 10		14.55	13.32									27.87
87	M 48 LM		227.73	1643.01					634.81				2505.55
87	M 48 DP			25.05					27.92				52.97
88	CA 10		5.43					5.34					10.77
88	M 40		45.28	1727.37				282.93	180.63				2236.21
88	M 48 LM		92.02	79.93					57.47				229.42
89	CA 7 W								4.62				4.62
89	CA 13			24.37									24.37
89	M 40		39.30	1776.81			36.05	95.21	555.75		87.97		2591.09
90	CA 7 Q			718.34			73.70	66.90	191.39				1050.33
90	CA 7 W			1.91				4.10					6.01
90	CA 34 A								10.90				10.90
90	M 40		20.05	1058.37			171.18	38.25	102.78				1390.63
90	M 50			171.15			2.38						173.53
91	CA 7 V			8.67					404.08				412.75
91	CA 34 A			363.83				12.68	47.57				424.08
91	CA 36				5.42			1.17	3.45				10.04
91	M 40			332.58			309.42	35.92	0.36				678.28
92	LDP 11								133.98				133.98
93	LDP 11	893.27	21.72						55.58			141.53	1112.10
93	LDP 16	882.18										59.11	941.29
94	LDP 11	1270.68			365.71	28.53			59.35			18.79	1743.06
94	LDP 16	659.41				122.89						23.46	805.76
95	LDP 11		126.81		865.29			73.48	12.75		12.50		1090.83
95	LDP 12			88.72	63.60				254.76		47.06		454.14

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
95	LDP 16		48.21		265.75		77.16						391.12
95	LDP 17				135.49			13.85	444.50	43.54			637.38
96	CAL 12			243.18				103.34		9.13	18.19		373.84
96	LDP 12		50.81	525.52	303.47		29.47	50.59		19.30			979.16
96	LDP 17			170.90				61.19	249.06				481.15
96	CAL 17		125.01	402.96				69.94	38.32				636.23
97	CAL 12		34.97		660.03		25.69	327.83					1048.52
97	CAL 17			1051.56			26.65	70.68					1148.89
97	CA 18 E-A			39.81			33.14	12.40					85.35
97	M 48 LM			311.96			24.69						336.65
98	CA 2		12.01	647.14				58.85					718.00
98	CA 10			7.82									7.82
98	CA 18 E-A		581.50	1001.95				138.44					1721.89
99	CA 10		123.75	1899.66				151.31	9.29	26.70			2210.71
99	CA 12			51.07				12.10		6.43			69.60
99	CA 13		40.27	189.82				37.08					267.17
99	CA 18 E-A			23.69									23.69
100	CA 7 Q			12.40					5.12				17.52
100	CA 7 W		9.86	140.86			7.42	52.78	30.41	6.78			248.11
100	CA 12			380.01				34.41		30.05			444.47
100	CA 13		107.02	1291.16			89.75	75.85	176.72	116.42			1856.92
101	CA 7 Q		163.13	1015.75				47.98	663.48	15.90			1906.24
101	CA 7 W		64.81	9.79			41.22	37.49	140.55	67.41			361.27
101	CA 34 A		10.03	165.11					1.62		5.53		182.29
101	M 40			1.18					15.85				17.03
102	CA 7 Q			270.72					250.25				520.97
102	CA 7 V		108.12	6.60					290.07			29.46	434.25
102	CA 32		155.97	49.14					90.64				295.75
102	CA 33			35.46					76.54				112.00
102	CA 34 A		12.69	429.62					415.75			31.08	889.14
102	CA 34 B			74.41					25.59				100.00
102	CA 36			105.25					25.71				130.96
102	M 40			14.95					18.55				33.50
106	LDP 15		528.31			35.05			241.98			28.65	833.99
106	LDP 16	629.95	140.76			357.82						20.73	1149.26
107	LDP 16	2041.34	96.33									447.16	2584.83
108	LDP 16	2083.63	322.28			36.82	56.74	41.50				64.70	2605.67
109	LDP 16	178.96	260.07	455.72		148.52	192.61	46.01	150.14			35.87	1467.90
109	CAL 17		63.75	21.28				6.27	86.94			14.85	193.09
109	LDP 17		295.67	155.84	255.05			43.97	198.76			8.95	958.24
110	LDP 17			250.66	62.42		21.92	35.27	75.54		22.50		468.31
110	CAL 17		553.54	542.93	455.83		49.46	161.19	244.36		51.63	18.45	2077.39
111	CAL 17		567.20	1019.72			71.69	30.60	418.88				2108.09
111	CA 18 E-A		101.83	200.59			13.89						316.31
112	CA 10		31.81	424.76									456.57
112	CAL 18		246.18										246.18

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
112	CA 18 E-A		319.48	1171.41	185.12			42.70	42.05				1760.76
113	CA 10			1339.79	222.40		23.88	53.98	180.64	65.49			1886.18
113	CA 12			587.68					61.49	4.88			654.05
114	CA 7 W			36.53					72.47				109.00
114	CA 11 B			407.03				48.40					455.43
114	CA 12			828.25				86.24					914.49
114	CA 13		80.45	602.30				117.14	286.53				1086.42
115	CA 7 Q		45.97	41.07					169.38	8.75		111.97	377.14
115	CA 7 W		602.30	10.32					213.01	159.25			984.88
115	CA 13								100.56				100.56
115	CA 29								205.44				205.44
115	CA 30		154.39						245.13			125.50	525.02
115	CA 31		275.94	12.60					410.66			122.09	821.29
116	CA 7 M			569.53								89.47	659.00
116	CA 7 Q			32.06					158.74				190.80
116	CA 28		436.38	243.00					204.25				883.63
116	CA 29		32.37						25.60				57.97
116	CA 30		76.93						143.42				220.35
116	CA 31			40.61					85.10				125.71
116	CA 32		13.22	537.32					13.71				564.25
116	CA 34 A			83.59									83.59
118	LDP 15		39.01										39.01
118	LDP 16	102.95	260.48			99.13						20.12	482.68
119	LDP 16	416.40	362.09		301.91	189.74						60.36	1330.50
120	LDP 16	972.28	553.77		361.73	589.61	49.47	49.08	142.24			57.08	2775.26
121	CAL 16				254.11	543.69		24.95					822.75
121	LDP 16				184.54	811.78		20.32	42.91				1059.55
121	CAL 17	154.99				659.00		23.74					837.73
121	LDP 17					1.92							1.92
122	CAL 17		896.08		1195.84	81.06	49.54	159.64	172.68		26.45	43.29	2624.58
123	CAL 17		111.52	708.90			44.79		1228.28		99.52	66.95	2259.96
123	CA 18 E-B			24.32					348.32			7.25	379.89
124	CA 5		30.02	89.42					72.88			8.68	201.00
124	CA 7		42.31	400.62				25.05	6.19			6.45	480.62
124	CA 8			73.15					15.94				89.09
124	CAL 17		226.90	199.35	116.38		42.37		208.70			5.78	799.48
124	CAL 18		293.01	36.39					245.74			22.01	597.15
124	CA 18 E-B		24.84	11.57					347.17			22.53	406.11
125	CA 7			180.54				47.73	27.90			4.21	260.38
125	CA 8		90.58	139.52					0.70		4.71	21.66	257.17
125	CA 9		10.84	128.10				0.29	54.87			8.90	203.00
125	CA 10		25.28	274.43					473.26		45.05	21.06	839.08
125	CA 11 A		98.02	341.69					55.04			0.45	495.20
125	CA 12			162.64					113.63				276.27
125	CA 15 A		207.47						67.04		2.66		277.17
125	CA 58		12.77						107.94		3.16		123.87
126	CA 7 W			0.91			5.44		381.71		5.88		393.94

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
126	CA 11 A			71.80									71.80
126	CA 11 B		40.73	347.84									388.57
126	CA 12		47.59	215.53									263.12
126	CA 13			150.26					240.30				390.56
126	CA 14			10.54			9.33		167.13				187.00
126	CA 15 A		118.26	83.31					273.92				475.49
126	CA 16		10.91	92.11			60.95		63.03				227.00
126	CA 17			46.33					105.79				152.12
126	CA 18								59.72				59.72
126	CA 25		46.88	16.83			126.17						189.88
126	CA 58			4.27					11.85				16.12
127	CA 7 W		1.11				0.74	8.74	297.24	81.34			389.17
127	CA 18		55.46										55.46
127	CA 20		15.94						32.41				48.35
127	CA 25		59.87					62.25					122.12
127	CA 26		156.48						183.81				340.29
127	CA 27								29.43				29.43
127	CA 29		153.50						299.44			21.48	474.42
127	CA 38		426.89					24.71	244.19	35.80			731.59
128	CA 20			6.51					151.94				158.45
128	CA 22		207.87	337.20					12.75				557.82
128	CA 26		20.45	127.46					97.80				245.71
128	CA 27		158.04	121.08					81.45				360.57
128	CA 28		421.62	0.85					22.90				445.37
128	CA 29								64.17				64.17
128	CA 30								20.63				20.63
129	LDP 16	275.49	56.31			135.86						18.91	486.57
129	LDP 20		5.63		10.07	56.04			29.45				101.19
130	LDP 16	348.05	189.58		535.52		61.11	73.95	117.99			253.48	1579.68
130	CAL 16				178.67								178.67
130	CAL 20				119.92		71.78	94.03					285.73
130	LDP 20		70.70		276.63			86.94					434.27
131	LDP 16		70.71	239.19	38.03								347.93
131	CAL 16	39.25	299.23	0.89	402.77	169.44							911.58
131	CAL 17		276.06										276.06
131	CAL 20	4.31	258.76	10.33	45.93	37.85							357.18
131	CAL 21	391.25	131.04		38.19	18.51							578.99
132	CAL 17	1139.54			6.43								1145.97
132	CAL 21	835.91	323.77	163.36	24.56	47.23							1394.83
133	CAL 17		163.21	114.06	390.65	92.57	52.65	25.14	352.22		18.48		1208.98
133	CAL 21		193.84	396.54	351.01	338.93		140.42	4.90				1425.64
134	CA 6		67.76	301.92	104.95			9.23	108.48		20.74		613.08
134	CA 8		42.87						161.32				204.19
134	CAL 17		55.37	68.00	367.05				47.07		16.06		553.55
134	CAL 18		31.88		250.08				71.28		6.43		359.67
134	CAL 21		216.46	393.48	6.42			101.09			20.38		737.83
134	CAL 22		39.44								20.63		60.07
134	CA 39		3.08						6.74	15.41			25.23
134	CA 40		24.78	6.53									31.31
135	CA 6			38.05					29.87				67.92
135	CA 8		36.01	25.29					135.25				196.55

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
135	CA 15 A								25.90				25.90
135	CA 39		155.75	205.29	111.62				346.85		51.26		870.77
135	CA 40		49.58	310.93				2.83	124.45				487.79
135	CA 41		2.56	226.22				54.00	38.24				321.02
135	CA 42 A			77.41				7.54					84.95
135	CA 42 B		30.64	60.81									91.45
135	CA 58		272.02	10.41					494.88		16.20		793.51
135	CA 62			2.30									2.30
136	CA 15 A			135.45					45.99				181.44
136	CA 15 B			64.04					14.96				79.00
136	CA 17			263.72					111.16				374.88
136	CA 18			20.40					129.43			29.77	179.60
136	CA 42 A			24.54					14.78				39.32
136	CA 42 B			409.75					74.80				484.55
136	CA 43 A			26.00									26.00
136	CA 43 B			14.75					29.25				44.00
136	CA 44			3.68					66.32				70.00
136	CA 45			77.17					57.83				135.00
136	CA 46								36.06				36.06
136	CA 47			8.56					5.06				13.62
136	CA 48			40.96					278.62				319.58
136	CA 51		36.44	249.56				24.20	72.50			17.86	400.56
136	CA 58								63.50				63.50
136	CA 62			211.12					21.58				232.70
137	CA 18			388.25			2.62	21.82	300.54	53.99			767.22
137	CA 19			536.64			37.03	47.58	143.20	16.19			780.64
137	CA 21			98.15					41.65				139.80
137	CA 38								210.65	7.76			218.41
137	CA 51			239.20		33.12	1.15	113.68		14.53			401.68
138	CA 19		54.27	277.33									331.60
138	CA 20			7.58					34.62				42.20
138	CA 21		259.57	136.92					99.71				496.20
138	CA 22		58.88	16.21					38.09				113.18
138	CA 23		391.41	50.50					110.60			50.49	603.00
138	CA 24		223.54	385.34				34.35	182.46				825.69
139	LDP 20	11.71							43.92				55.63
140	CAL 20		262.94	486.80	43.63				225.25				1018.62
140	LDP 20			27.59					0.32				27.91
141	CAL 20		138.35	808.17	105.90	21.55	121.12		130.09				1325.18
141	CAL 21		281.16	710.84			211.56	51.32					1254.88
142	CAL 21	80.21	702.43	962.43	307.53		39.66	119.41	308.97				2520.64
143	CAL 21	303.07	810.23	443.99	525.01			357.76	72.76	33.49		41.30	2587.61
144	CAL 21		336.89	238.97	692.25	74.76		87.05	303.56	102.72			1836.20
144	CAL 22		481.38	173.46	57.07	161.56		48.75	33.05	100.41			1055.68
144	CA 40		27.55	84.66									112.21
145	CAL 22			319.68		82.95		24.58	608.02				1035.23
145	CA 40			413.75		7.63		46.97	107.34				575.69
145	CA 41			109.55				34.43					143.98
145	CA 42 A			223.70				63.49	268.54				555.73
145	CA 57			306.32					17.68				324.00

SHEET #	BASIN #	FOREST	OPEN SPACE	RESIDENTIAL				COMMERCIAL	INDUSTRIAL	HIGHWAY		OPEN WATER	TOTAL
				HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI-FAMILY HIGH RISE			GRASS MEDIAN	NO MEDIAN		
146	CAL 22			206.57	44.53			94.19	969.06				1314.35
146	CA 45			6.11					15.89				22.00
146	CA 46			269.34				2.58	35.02				306.94
146	CA 47			12.75					27.63				40.38
146	CA 48			2.81					26.61				29.42
146	CA 49			79.70				45.50	47.67				172.87
146	CA 50			47.19					83.81				131.00
146	CA 51			23.24									23.24
147	CA 19		30.93	22.48					13.50				66.91
147	CA 51		15.14	152.18					12.46	2.74			182.52
147	CA 61			9.00									9.00
148	CA 19		59.40	14.24					14.21				87.85
148	CA 24		188.97	438.34									627.31
148	CA 53			10.64	37.53			24.83					73.00
148	CA 54			70.71				34.29					105.00
148	CA 55		25.99	135.65				20.36					182.00
148	CA 56		10.34	266.17				14.77	35.80				327.08
149	CAL 20		769.36	597.58					10.51				1377.45
150	CAL 20		368.44	542.14	80.26								990.84
150	CAL 21		312.47	178.33	113.08								603.88
151	CAL 21		265.89	401.04	159.32	148.59		58.53	28.85				1062.22
152	CAL 21		948.86	382.01	443.84	504.83			60.34	239.27			2579.15
152	CAL 24		23.90	10.55	10.60								45.05
153	CAL 21		157.72	606.95	68.51	154.43	2.00	134.06	12.25	47.21			1183.13
153	CAL 22		265.18	197.40	423.71	53.01		23.29	71.80	71.61			1106.00
153	CAL 24				108.21	73.50	18.03	29.09					228.83
154	CAL 22		527.84	670.57	317.10			128.45	700.48	69.01			2413.45
155	CAL 22		133.51	59.78					344.14	52.04			589.47
155	CA 49			44.98				49.15					94.13
156	CA 56			36.09				49.76					85.85
157	CA 56		136.57	1154.77				170.82	84.85	50.59			1597.60
158	CAL 21		3.01						6.99				10.00
158	CAL 24		115.09	177.72	205.23		40.55	18.86	6.94				564.39
159	CAL 22			112.03				23.13					135.16
159	CAL 24		406.30	319.78	248.33	75.47		128.85					1178.73
160	CAL 22			420.59									420.59
160	CAL 24			2.00									2.00
161	CA 56			83.47									83.47
													0.00
TOTALS		13945.10	34216.81	131988.49	22267.10	6044.70	15693.61	19926.40	56253.28	2083.80	2544.72	3412.99	308377.00

APPENDIX D

LAND USE AREAS IN ACRES
FOR EACH SCA AND UNGAGED PARTITION

BASIN	FOREST	OPEN AREA	HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI FAMILY	COMMERCIAL	INDUSTRIAL	HIGHWAY GRASS MED.	HIGHWAY NO MED.	OPEN WATER	TOTAL
CA 2	0.00	12.01	647.14	0.00	0.00	0.00	58.85	0.00	0.00	0.00	0.00	718.00
CA 5	0.00	30.02	89.42	0.00	0.00	0.00	0.00	72.88	0.00	0.00	8.68	201.00
CA 6	0.00	67.76	339.97	104.95	0.00	0.00	9.23	138.35	0.00	20.74	0.00	681.00
CA 7	0.00	42.31	581.16	0.00	0.00	0.00	72.78	34.09	0.00	0.00	10.66	741.00
CA 7 M	0.00	0.00	569.53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	89.47	659.00
CA 7 Q	0.00	209.10	2090.34	0.00	0.00	73.70	114.88	1438.36	24.65	0.00	111.97	4063.00
CA 7 V	0.00	108.12	15.27	0.00	0.00	0.00	0.00	694.15	0.00	0.00	29.46	847.00
CA 7 W	0.00	678.08	200.32	0.00	0.00	54.82	103.11	1140.01	314.78	5.88	0.00	2497.00
CA 8	0.00	169.46	237.96	0.00	0.00	0.00	0.00	313.21	0.00	4.71	21.66	747.00
CA 9	0.00	10.84	128.10	0.00	0.00	0.00	0.29	54.87	0.00	0.00	8.90	203.00
CA 10	0.00	200.82	3959.78	222.40	0.00	23.88	210.63	663.19	92.19	45.05	21.06	5439.00
CA 11 A	0.00	98.02	413.49	0.00	0.00	0.00	0.00	55.04	0.00	0.00	0.45	567.00
CA 11 B	0.00	40.73	754.87	0.00	0.00	0.00	48.40	0.00	0.00	0.00	0.00	844.00
CA 12	0.00	47.59	2225.18	0.00	0.00	0.00	132.75	175.12	41.36	0.00	0.00	2622.00
CA 13	0.00	227.74	2257.91	0.00	0.00	89.75	230.07	804.11	116.42	0.00	0.00	3726.00
CA 14	0.00	0.00	10.54	0.00	0.00	9.33	0.00	167.13	0.00	0.00	0.00	187.00
CA 15 A	0.00	325.73	218.76	0.00	0.00	0.00	0.00	412.85	0.00	2.66	0.00	960.00
CA 15 B	0.00	0.00	64.04	0.00	0.00	0.00	0.00	14.96	0.00	0.00	0.00	79.00
CA 16	0.00	10.91	92.11	0.00	0.00	60.95	0.00	63.03	0.00	0.00	0.00	227.00
CA 17	0.00	0.00	310.05	0.00	0.00	0.00	0.00	216.95	0.00	0.00	0.00	527.00
CA 18	0.00	55.46	408.65	0.00	0.00	2.62	21.82	489.69	53.99	0.00	29.77	1062.00
CA 18 E-A	0.00	1002.81	2437.45	185.12	0.00	47.03	193.54	42.05	0.00	0.00	0.00	3908.00
CA 18 E-B	0.00	24.84	35.89	0.00	0.00	0.00	0.00	695.49	0.00	0.00	29.78	786.00
CA 19	0.00	144.60	850.69	0.00	0.00	37.03	47.58	170.91	16.19	0.00	0.00	1267.00
CA 20	0.00	15.94	14.09	0.00	0.00	0.00	0.00	218.97	0.00	0.00	0.00	249.00
CA 21	0.00	259.57	235.07	0.00	0.00	0.00	0.00	141.36	0.00	0.00	0.00	636.00
CA 22	0.00	266.75	353.41	0.00	0.00	0.00	0.00	50.84	0.00	0.00	0.00	671.00
CA 23	0.00	391.41	50.50	0.00	0.00	0.00	0.00	110.60	0.00	0.00	50.49	603.00
CA 24	0.00	412.51	823.68	0.00	0.00	0.00	34.35	182.46	0.00	0.00	0.00	1453.00
CA 25	0.00	106.75	16.83	0.00	0.00	126.17	62.25	0.00	0.00	0.00	0.00	312.00
CA 26	0.00	176.93	127.46	0.00	0.00	0.00	0.00	281.61	0.00	0.00	0.00	586.00
CA 27	0.00	158.04	121.08	0.00	0.00	0.00	0.00	110.88	0.00	0.00	0.00	390.00
CA 28	0.00	858.00	243.85	0.00	0.00	0.00	0.00	227.15	0.00	0.00	0.00	1329.00
CA 29	0.00	185.87	0.00	0.00	0.00	0.00	0.00	594.65	0.00	0.00	21.48	802.00
CA 30	0.00	231.32	0.00	0.00	0.00	0.00	0.00	409.18	0.00	0.00	125.50	766.00
CA 31	0.00	275.94	53.21	0.00	0.00	0.00	0.00	495.76	0.00	0.00	122.09	947.00
CA 32	0.00	169.19	586.46	0.00	0.00	0.00	0.00	104.35	0.00	0.00	0.00	860.00

BASIN	FOREST	OPEN AREA	HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI FAMILY	COMMERCIAL	INDUSTRIAL	HIGHWAY GRASS MED.	HIGHWAY NO MED.	OPEN WATER	TOTAL
CA 33	0.00	0.00	35.46	0.00	0.00	0.00	0.00	76.54	0.00	0.00	0.00	112.00
CA 34 A	0.00	22.72	1042.15	0.00	0.00	0.00	12.68	475.84	0.00	5.53	31.08	1590.00
CA 34 B	0.00	0.00	74.41	0.00	0.00	0.00	0.00	25.59	0.00	0.00	0.00	100.00
CA 36	0.00	0.00	105.25	5.42	0.00	0.00	1.17	29.16	0.00	0.00	0.00	141.00
CA 38	0.00	426.89	0.00	0.00	0.00	0.00	24.71	454.84	43.56	0.00	0.00	950.00
CA 39	0.00	158.83	205.29	111.62	0.00	0.00	0.00	353.59	15.41	51.26	0.00	896.00
CA 40	0.00	101.91	815.87	0.00	7.63	0.00	49.80	231.79	0.00	0.00	0.00	1207.00
CA 41	0.00	2.56	335.77	0.00	0.00	0.00	88.43	38.24	0.00	0.00	0.00	465.00
CA 42 A	0.00	0.00	325.65	0.00	0.00	0.00	71.03	283.32	0.00	0.00	0.00	680.00
CA 42 B	0.00	30.64	470.56	0.00	0.00	0.00	0.00	74.80	0.00	0.00	0.00	576.00
CA 43 A	0.00	0.00	26.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	26.00
CA 43 B	0.00	0.00	14.75	0.00	0.00	0.00	0.00	29.25	0.00	0.00	0.00	44.00
CA 44	0.00	0.00	3.68	0.00	0.00	0.00	0.00	66.32	0.00	0.00	0.00	70.00
CA 45	0.00	0.00	83.28	0.00	0.00	0.00	0.00	73.72	0.00	0.00	0.00	157.00
CA 46	0.00	0.00	269.34	0.00	0.00	0.00	2.58	71.08	0.00	0.00	0.00	343.00
CA 47	0.00	0.00	21.31	0.00	0.00	0.00	0.00	32.69	0.00	0.00	0.00	54.00
CA 48	0.00	0.00	43.77	0.00	0.00	0.00	0.00	305.23	0.00	0.00	0.00	349.00
CA 49	0.00	0.00	124.68	0.00	0.00	0.00	94.65	47.67	0.00	0.00	0.00	267.00
CA 50	0.00	0.00	47.19	0.00	0.00	0.00	0.00	83.81	0.00	0.00	0.00	131.00
CA 51	0.00	51.58	664.18	0.00	33.12	1.15	137.88	84.96	17.27	0.00	17.86	1008.00
CA 53	0.00	0.00	10.64	37.53	0.00	0.00	24.83	0.00	0.00	0.00	0.00	73.00
CA 54	0.00	0.00	70.71	0.00	0.00	0.00	34.29	0.00	0.00	0.00	0.00	105.00
CA 55	0.00	25.99	135.65	0.00	0.00	0.00	20.36	0.00	0.00	0.00	0.00	182.00
CA 56	0.00	146.91	1540.50	0.00	0.00	0.00	235.35	120.65	50.59	0.00	0.00	2094.00
CA 57	0.00	0.00	306.32	0.00	0.00	0.00	0.00	17.68	0.00	0.00	0.00	324.00
CA 58	0.00	284.79	14.68	0.00	0.00	0.00	0.00	678.17	0.00	19.36	0.00	997.00
CA 61	0.00	0.00	9.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	9.00
CA 62	0.00	0.00	213.42	0.00	0.00	0.00	0.00	21.58	0.00	0.00	0.00	235.00
DP 1	0.00	0.00	213.36	0.00	0.00	17.80	49.84	0.00	0.00	0.00	0.00	281.00
DP 2 A	0.00	0.00	72.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	72.00
DP 2 B	0.00	0.00	276.46	70.16	0.00	0.00	43.75	0.00	6.63	0.00	0.00	397.00
DP 3	0.00	0.00	169.00	0.00	0.00	0.00	27.00	0.00	0.00	0.00	0.00	196.00
DP 4	0.00	0.00	23.67	120.76	0.00	0.00	7.21	213.36	0.00	0.00	0.00	365.00
DP 5	0.00	125.18	436.27	486.50	51.31	0.00	116.74	0.00	0.00	0.00	0.00	1216.00
DP 6	0.00	18.89	415.63	117.97	0.00	35.20	117.31	0.00	0.00	0.00	0.00	705.00
DP 7	0.00	46.70	609.20	221.43	0.00	0.00	77.67	0.00	0.00	0.00	0.00	955.00
DP 8	0.00	0.00	201.58	126.33	0.00	0.00	94.09	0.00	0.00	0.00	0.00	422.00

BASIN	FOREST	OPEN AREA	HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI FAMILY	COMMERCIAL	INDUSTRIAL	HIGHWAY GRASS MED.	HIGHWAY NO MED.	OPEN WATER	TOTAL
DP 9	0.00	0.00	253.23	233.19	0.00	194.12	64.02	0.00	5.44	0.00	0.00	750.00
DP 10	0.00	0.00	154.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	154.00
DP 11	0.00	14.64	855.14	0.00	0.00	27.81	52.41	0.00	0.00	0.00	0.00	950.00
DP 12	0.00	0.00	120.71	0.00	0.00	0.00	89.29	0.00	0.00	0.00	0.00	210.00
DP 13	0.00	0.00	103.90	0.00	0.00	0.00	19.72	31.38	0.00	0.00	0.00	155.00
DP 14	0.00	0.00	316.98	0.00	0.00	0.00	11.81	175.21	0.00	0.00	0.00	504.00
DP 15	0.00	3.60	309.83	0.00	0.00	0.00	0.97	72.60	0.00	0.00	0.00	387.00
DP 16	0.00	4.58	129.20	0.00	0.00	0.00	0.00	11.22	0.00	0.00	0.00	145.00
DP 17 A	0.00	36.34	135.27	0.00	0.00	0.00	3.39	0.00	0.00	0.00	0.00	175.00
DP 17 B	0.00	0.00	41.93	205.68	0.00	0.00	48.24	145.15	0.00	0.00	0.00	441.00
DP 18	0.00	0.00	162.90	53.76	0.00	0.00	0.00	0.00	0.00	0.00	36.34	253.00
DP 19	0.00	0.00	1170.61	0.00	0.00	0.00	108.39	0.00	0.00	0.00	0.00	1279.00
DP 20	0.00	0.00	0.00	0.00	0.00	0.00	78.37	86.63	0.00	0.00	0.00	165.00
DP 21	0.00	16.03	309.70	0.00	0.00	69.17	30.10	244.00	0.00	0.00	0.00	669.00
DP 22	0.00	0.00	44.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	44.00
DP 23	0.00	56.06	183.01	0.00	0.00	18.63	53.30	0.00	0.00	0.00	0.00	311.00
DP 24	0.00	0.00	77.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	77.00
DP 25 AB	0.00	0.00	80.00	0.00	0.00	0.00	30.00	0.00	0.00	0.00	0.00	110.00
DP 26	0.00	9.86	145.45	156.60	0.00	0.00	16.97	40.12	0.00	0.00	0.00	369.00
DP 27	0.00	14.35	274.57	0.00	0.00	10.47	0.00	24.61	0.00	0.00	0.00	324.00
DP 28	0.00	0.00	151.35	0.00	0.00	123.42	17.68	19.55	0.00	0.00	0.00	312.00
DP 29	0.00	0.00	139.86	0.00	0.00	0.00	0.00	4.14	0.00	0.00	0.00	144.00
DP 30	0.00	17.20	261.78	0.00	0.00	0.00	38.14	3.69	0.00	14.19	0.00	335.00
DP 31	0.00	0.00	311.54	0.00	0.00	22.46	0.00	0.00	0.00	0.00	0.00	334.00
DP 32	0.00	0.68	139.52	0.00	0.00	0.00	0.00	0.00	0.00	19.80	0.00	160.00
DP 33	0.00	33.52	392.82	0.00	0.00	0.00	0.00	0.00	0.00	25.66	0.00	452.00
DP 34	0.00	0.00	438.37	0.00	0.00	0.00	40.34	13.29	0.00	0.00	0.00	492.00
DP 35	0.00	0.00	281.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	281.00
DP 36	0.00	48.42	161.89	0.00	0.00	24.00	0.00	24.69	0.00	0.00	0.00	259.00
DP 37	0.00	0.00	112.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	112.00
DP 38	0.00	0.00	555.52	43.55	0.00	59.69	39.62	1.62	0.00	0.00	0.00	700.00
DP 39	0.00	0.00	265.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	265.00
DP 40	0.00	0.00	131.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	131.00
DP 41	0.00	0.00	54.62	344.54	0.00	0.00	17.84	0.00	0.00	0.00	0.00	417.00
DP 42	0.00	2.85	0.00	100.44	0.00	0.00	0.71	0.00	0.00	0.00	0.00	104.00
DP 43	0.00	0.00	1.35	188.17	0.00	0.00	32.48	0.00	0.00	0.00	0.00	222.00
DP 44	0.00	0.00	7.81	199.19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	207.00

BASIN	FOREST	OPEN AREA	HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI FAMILY	COMMERCIAL	INDUSTRIAL	HIGHWAY GRASS MED.	HIGHWAY NO MED.	OPEN WATER	TOTAL
DP 45	0.00	0.00	41.93	53.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	95.00
DP 46	0.00	0.00	184.86	0.00	0.00	0.00	0.00	33.14	0.00	0.00	0.00	218.00
DP 47	0.00	0.00	17.75	17.85	0.00	0.00	0.40	0.00	0.00	0.00	0.00	36.00
DP 48	0.00	0.00	0.00	0.00	0.00	0.00	25.33	16.67	0.00	0.00	0.00	42.00
DP 49	0.00	10.60	300.60	15.41	0.00	0.00	62.39	0.00	0.00	0.00	0.00	389.00
DP 50	0.00	49.75	102.31	0.00	0.00	0.00	17.21	2.73	0.00	0.00	0.00	172.00
DP 51	0.00	0.00	195.94	0.00	0.00	0.00	21.06	0.00	0.00	0.00	0.00	217.00
DP 52	0.00	0.00	405.53	215.41	0.00	0.00	0.00	130.06	0.00	0.00	0.00	751.00
DP 53	0.00	0.00	150.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	150.00
DP 54	0.00	0.00	153.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	153.00
DP 55	0.00	0.00	400.98	0.00	0.00	0.00	41.02	0.00	0.00	0.00	0.00	442.00
DP 56	0.00	0.00	150.03	0.00	0.00	0.00	0.00	0.00	0.00	0.97	0.00	151.00
DP 62	0.00	0.00	0.00	39.83	0.00	0.00	11.17	0.00	0.00	0.00	0.00	51.00
DP 63	0.00	11.37	0.00	409.30	0.00	12.28	40.05	0.00	0.00	0.00	0.00	473.00
DP 64	0.00	0.00	39.24	0.00	0.00	0.00	6.21	4.55	0.00	0.00	0.00	50.00
DP 65	0.00	42.64	205.92	0.00	0.00	0.00	0.00	0.00	0.00	3.44	0.00	252.00
DP 66	0.00	0.00	0.00	25.67	0.00	0.00	11.33	0.00	0.00	0.00	0.00	37.00
M 1	0.00	0.00	2495.20	131.01	0.00	0.00	226.09	0.00	0.00	0.00	66.70	2919.00
M 2	0.00	88.55	974.76	231.39	0.00	911.25	369.08	78.93	0.00	0.00	129.04	2783.00
M 3	0.00	0.00	632.75	0.00	0.00	35.65	23.88	312.48	0.00	0.00	83.24	1088.00
M 4	0.00	471.65	2797.53	121.32	0.00	107.92	489.43	314.62	0.00	90.53	0.00	4393.00
M 5	0.00	117.26	1053.15	0.00	0.00	606.33	260.94	4.07	0.00	0.00	142.25	2184.00
M 6	0.00	128.41	1430.39	504.00	0.00	8.41	379.98	809.66	12.03	81.12	0.00	3354.00
M 7	0.00	290.64	562.55	0.00	0.00	0.00	127.59	0.00	0.00	0.00	21.22	1002.00
M 8	0.00	180.41	523.52	0.00	0.00	0.00	164.05	158.96	0.00	0.00	18.06	1045.00
M 9	0.00	933.71	1576.22	0.00	0.00	2297.50	299.98	154.56	37.16	55.01	7.86	5362.00
M 10 A	0.00	187.00	2212.31	0.00	0.00	0.00	438.17	170.29	4.09	93.50	3.64	3109.00
M 10 B	0.00	117.57	1022.59	0.00	0.00	0.00	278.24	16.76	0.00	29.84	0.00	1465.00
M 11	0.00	13.12	683.78	0.00	0.00	85.20	229.84	22.68	0.00	0.00	29.38	1064.00
M 12	0.00	0.00	171.60	0.00	0.00	0.00	42.03	5.37	0.00	0.00	0.00	219.00
M 13	0.00	0.00	397.64	0.00	0.00	0.00	227.22	86.05	0.00	0.00	9.09	720.00
M 14	0.00	35.99	417.43	0.00	0.00	0.00	158.08	78.34	0.00	1.16	0.00	691.00
M 15	0.00	0.00	745.40	0.00	0.00	118.96	313.19	152.94	0.00	0.00	34.51	1365.00
M 16	0.00	148.75	4151.05	0.00	0.00	0.00	686.05	397.93	0.00	32.22	0.00	5416.00
M 17	0.00	0.00	368.21	0.00	0.00	35.80	39.51	105.32	0.00	73.00	4.16	626.00
M 18	0.00	16.81	1450.10	0.00	0.00	16.32	147.03	416.71	36.68	23.35	0.00	2107.00
M 19	0.00	19.08	91.06	443.74	0.00	148.33	78.41	118.89	0.00	0.00	22.49	922.00

BASIN	FOREST	OPEN AREA	HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI FAMILY	COMMERCIAL	INDUSTRIAL	HIGHWAY GRASS MED.	HIGHWAY NO MED.	OPEN WATER	TOTAL
M 20	0.00	7.25	1299.42	0.00	0.00	0.00	112.28	112.60	31.44	9.01	0.00	1572.00
M 21	0.00	35.63	153.31	0.00	0.00	217.74	451.72	197.51	0.00	29.68	24.41	1110.00
M 22	0.00	0.00	23.70	0.00	0.00	0.00	0.00	98.56	0.00	3.74	0.00	126.00
M 23	0.00	202.47	1221.18	0.00	0.00	0.00	81.28	229.93	0.00	11.14	0.00	1746.00
M 24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	102.04	0.00	0.00	6.96	109.00
M 25	0.00	1143.24	209.22	15.32	0.00	0.00	45.36	0.00	0.00	21.86	0.00	1435.00
M 27	0.00	0.00	676.28	0.00	0.00	0.00	16.27	71.42	0.00	11.03	2.00	777.00
M 28	0.00	0.00	0.00	0.00	0.00	20.77	25.10	281.12	0.00	8.91	26.10	362.00
M 29	0.00	11.80	294.42	0.00	0.00	14.87	147.11	311.98	0.00	65.82	0.00	846.00
M 30	0.00	10.77	611.74	0.00	0.00	67.62	0.00	81.49	0.00	25.99	13.39	811.00
M 31	0.00	0.00	0.00	0.00	0.00	0.00	278.23	8.18	0.00	0.00	51.59	338.00
M 32	0.00	5.06	62.05	0.00	0.00	287.48	467.17	186.75	0.00	13.49	0.00	1022.00
M 33	0.00	0.00	0.00	0.00	0.00	0.00	140.08	222.59	0.00	18.71	10.62	392.00
M 34	0.00	149.38	752.23	0.00	0.00	463.34	814.65	791.54	0.00	222.16	25.70	3219.00
M 35	0.00	0.00	98.08	0.00	0.00	135.59	182.74	509.51	0.00	81.72	4.36	1012.00
M 36	0.00	68.38	854.52	114.58	0.00	259.16	361.58	602.51	0.00	115.03	19.24	2395.00
M 37	0.00	19.28	462.98	13.39	0.00	7.99	0.00	656.11	0.00	5.60	29.65	1195.00
M 38	0.00	0.00	37.48	0.00	0.00	0.00	1.43	199.08	0.00	16.77	13.24	268.00
M 39	0.00	8.28	324.43	0.00	0.00	0.00	51.90	189.82	0.00	8.57	0.00	583.00
M 40	0.00	851.58	7507.40	0.00	0.00	3107.57	633.72	1968.24	0.00	183.49	0.00	14252.00
M 41	0.00	65.27	2638.06	0.00	0.00	0.00	246.62	2094.40	0.00	44.65	0.00	5089.00
M 42	0.00	162.99	456.44	7.79	0.00	78.67	160.20	492.76	0.00	5.15	0.00	1364.00
M 43	0.00	55.13	822.25	0.00	0.00	141.73	44.21	475.14	0.00	22.54	0.00	1561.00
M 44	0.00	120.64	0.00	0.00	0.00	0.00	181.51	0.00	0.00	15.53	20.32	338.00
M 45	0.00	0.00	530.24	0.00	0.00	15.42	2.29	380.21	0.00	13.70	35.14	977.00
M 46	0.00	0.00	967.24	282.18	0.00	15.42	73.81	1232.64	10.41	13.70	38.60	2634.00
M 47	0.00	33.29	1644.24	0.00	0.00	141.48	64.28	1320.63	0.00	63.08	0.00	3267.00
M 48 DP	0.00	1.78	303.29	0.00	0.00	0.00	44.64	683.29	0.00	0.00	0.00	1033.00
M 48 LM	0.00	672.49	5581.96	0.00	0.00	71.00	474.58	1857.99	32.54	0.00	17.44	8708.00
M 49 A	0.00	0.00	509.10	0.00	0.00	0.00	36.56	749.52	73.60	0.00	34.22	1403.00
M 49 BDP	0.00	0.00	1067.52	0.00	0.00	0.00	71.59	278.89	0.00	0.00	0.00	1418.00
M 49 BLM	0.00	0.00	451.24	0.00	0.00	0.00	56.32	70.44	0.00	0.00	0.00	578.00
M 50	0.00	837.45	622.45	87.11	0.00	3359.39	198.51	1101.09	0.00	266.59	5.41	6478.00
M 51	0.00	454.52	7696.70	179.62	0.00	340.17	997.05	1516.10	0.00	64.80	51.04	11300.00
M 52	0.00	60.77	2118.90	0.00	0.00	0.00	438.87	1800.70	0.00	24.84	5.92	4450.00
M 53	0.00	219.07	1859.34	236.56	0.00	0.00	135.92	162.11	0.00	0.00	0.00	2613.00
M 55 A	0.00	0.00	412.47	0.00	0.00	0.00	0.00	550.89	0.00	0.00	24.64	988.00

BASIN	FOREST	OPEN AREA	HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI FAMILY	COMMERCIAL	INDUSTRIAL	HIGHWAY GRASS MED.	HIGHWAY NO MED.	OPEN WATER	TOTAL
M 55 B	0.00	0.00	55.64	0.00	0.00	0.00	0.00	2007.93	0.00	0.00	7.43	2071.00
MN 1 A	0.00	0.00	302.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	302.00
MN 1 B	0.00	376.70	759.50	28.04	0.00	0.00	251.76	0.00	0.00	0.00	0.00	1416.00
MN 1 C	0.00	0.00	1166.58	199.78	0.00	0.00	279.63	0.00	122.01	0.00	0.00	1768.00
MN 1 D	0.00	21.01	417.99	0.00	0.00	0.00	14.62	90.38	0.00	0.00	0.00	544.00
MN 1 E	0.00	22.54	479.53	0.00	0.00	0.00	64.47	30.10	16.49	26.87	0.00	640.00
MN 2 A	0.00	0.00	0.00	265.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	265.00
MN 2 B	0.00	6.95	320.89	269.40	0.00	0.00	54.30	12.55	0.00	4.91	0.00	669.00
MN 2 C	0.00	0.00	0.00	428.97	0.00	0.00	0.00	7.44	0.00	27.59	0.00	464.00
MN 3	0.00	142.85	128.69	105.83	0.00	0.00	10.48	916.21	11.31	0.63	0.00	1316.00
MN 4	0.00	131.57	1077.52	62.33	0.00	0.00	111.58	0.00	0.00	0.00	0.00	1383.00
MN 5	0.00	0.00	685.54	0.00	0.00	14.22	0.00	234.77	0.00	13.47	0.00	948.00

APPENDIX D UNGAGED AREAS

BASIN	FOREST	OPEN AREA	HIGH DENSITY	MEDIUM DENSITY	LOW DENSITY	MULTI FAMILY	COMMERCIAL	INDUSTRIAL	HIGHWAY GRASS MED.	HIGHWAY NO MED.	OPEN WATER	TOTAL
CAL 12	0.00	34.97	407.41	660.03	0.00	25.69	445.58	0.00	9.13	18.19	0.00	1601.00
CAL 16	39.25	299.23	0.89	835.55	713.13	0.00	24.95	0.00	0.00	0.00	0.00	1913.00
CAL 17	1294.53	3038.64	4128.76	2532.18	832.63	337.15	547.20	2797.45	0.00	212.14	149.32	15870.00
CAL 18	0.00	571.07	36.39	250.08	0.00	0.00	0.00	317.02	0.00	6.43	22.01	1203.00
CAL 20	4.31	1797.85	2445.02	395.64	59.40	192.90	94.03	365.85	0.00	0.00	0.00	5355.00
CAL 21	1610.44	4683.77	4877.94	2729.72	1287.28	253.22	1049.64	798.62	422.69	20.38	41.30	17775.00
CAL 22	0.00	1447.35	2160.08	842.41	297.52	0.00	342.39	2726.55	293.07	20.63	0.00	8130.00
CAL 24	0.00	545.29	510.05	572.37	148.97	58.58	176.80	6.94	0.00	0.00	0.00	2019.00
LDP 9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	9.00	0.00	0.00	0.00	9.00
LDP 11	2394.22	819.65	778.32	2278.24	28.53	80.23	73.48	1533.54	109.83	51.34	305.62	8453.00
LDP 12	0.00	200.74	2841.08	904.53	0.00	29.47	197.47	2316.35	19.30	47.06	0.00	6556.00
LDP 15	0.00	567.32	0.00	0.00	35.05	0.00	0.00	241.98	0.00	0.00	28.65	873.00
LDP 16	8590.64	2360.59	694.91	1687.48	2492.17	437.09	230.86	453.28	0.00	0.00	1060.98	18008.00
LDP 17	0.00	295.67	577.40	452.96	1.92	21.92	154.28	967.86	43.54	22.50	8.95	2547.00
LDP 20	11.71	76.33	27.59	286.70	56.04	0.00	86.94	73.69	0.00	0.00	0.00	619.00

APPENDIX E

LAND COVER RESULTS
FOR EACH SCA AND UNGAGED PARTITION

BASIN	ACRES IMPERV.	ACRES GRASS	ACRES FOREST	PERCENT IMPERV.	PERCENT GRASS	PERCENT FOREST
CA 2	413.02	304.98	0.00	57.52%	42.48%	0.00%
CA 5	112.73	88.27	0.00	56.08%	43.92%	0.00%
CA 6	359.80	321.20	0.00	52.83%	47.17%	0.00%
CA 7	424.63	316.37	0.00	57.31%	42.69%	0.00%
CA 7 M	408.41	250.59	0.00	61.97%	38.03%	0.00%
CA 7 Q	2490.20	1572.80	0.00	61.29%	38.71%	0.00%
CA 7 V	543.21	303.79	0.00	64.13%	35.87%	0.00%
CA 7 W	1255.00	1242.00	0.00	50.26%	49.74%	0.00%
CA 8	392.67	354.33	0.00	52.57%	47.43%	0.00%
CA 9	120.93	82.07	0.00	59.57%	40.43%	0.00%
CA 10	3092.92	2346.08	0.00	56.87%	43.13%	0.00%
CA 11 A	276.53	290.47	0.00	48.77%	51.23%	0.00%
CA 11 B	465.90	378.10	0.00	55.20%	44.80%	0.00%
CA 12	1508.08	1113.92	0.00	57.52%	42.48%	0.00%
CA 13	2171.37	1554.63	0.00	58.28%	41.72%	0.00%
CA 14	132.77	54.23	0.00	71.00%	29.00%	0.00%
CA 15 A	438.17	521.83	0.00	45.64%	54.36%	0.00%
CA 15 B	46.63	32.37	0.00	59.03%	40.97%	0.00%
CA 16	140.17	86.83	0.00	61.75%	38.25%	0.00%
CA 17	329.83	197.17	0.00	62.59%	37.41%	0.00%
CA 18	661.34	400.66	0.00	62.27%	37.73%	0.00%
CA 18 E-A	1716.87	2191.13	0.00	43.93%	56.07%	0.00%
CA 18 E-B	551.87	234.13	0.00	70.21%	29.79%	0.00%
CA 19	681.13	585.87	0.00	53.76%	46.24%	0.00%
CA 20	166.35	82.65	0.00	66.81%	33.19%	0.00%
CA 21	246.40	389.60	0.00	38.74%	61.26%	0.00%
CA 22	247.85	423.15	0.00	36.94%	63.06%	0.00%
CA 23	177.97	425.03	0.00	29.51%	70.49%	0.00%
CA 24	642.46	810.55	0.00	44.22%	55.78%	0.00%
CA 25	155.99	156.01	0.00	50.00%	50.00%	0.00%
CA 26	282.98	303.02	0.00	48.29%	51.71%	0.00%
CA 27	155.54	234.46	0.00	39.88%	60.12%	0.00%
CA 28	343.00	986.00	0.00	25.81%	74.19%	0.00%
CA 29	458.92	343.08	0.00	57.22%	42.78%	0.00%
CA 30	431.68	334.32	0.00	56.35%	43.65%	0.00%
CA 31	522.63	424.37	0.00	55.19%	44.81%	0.00%
CA 32	412.01	447.99	0.00	47.91%	52.09%	0.00%

BASIN	ACRES IMPERV.	ACRES GRASS	ACRES FOREST	PERCENT IMPERV.	PERCENT GRASS	PERCENT FOREST
CA 33	74.97	37.03	0.00	66.93%	33.07%	0.00%
CA 34 A	973.63	616.37	0.00	61.23%	38.77%	0.00%
CA 34 B	60.09	39.91	0.00	60.09%	39.91%	0.00%
CA 36	83.10	57.90	0.00	58.93%	41.07%	0.00%
CA 38	391.61	558.39	0.00	41.22%	58.78%	0.00%
CA 39	470.85	425.15	0.00	52.55%	47.45%	0.00%
CA 40	672.65	534.35	0.00	55.73%	44.27%	0.00%
CA 41	290.86	174.14	0.00	62.55%	37.45%	0.00%
CA 42 A	446.73	233.27	0.00	65.70%	34.30%	0.00%
CA 42 B	318.90	257.10	0.00	55.36%	44.64%	0.00%
CA 43 A	14.56	11.44	0.00	56.00%	44.00%	0.00%
CA 43 B	29.32	14.68	0.00	66.64%	33.36%	0.00%
CA 44	49.81	20.19	0.00	71.16%	28.84%	0.00%
CA 45	99.72	57.28	0.00	63.51%	36.49%	0.00%
CA 46	204.20	138.80	0.00	59.53%	40.47%	0.00%
CA 47	35.47	18.53	0.00	65.69%	34.31%	0.00%
CA 48	244.28	104.72	0.00	69.99%	30.01%	0.00%
CA 49	184.60	82.40	0.00	69.14%	30.86%	0.00%
CA 50	86.77	44.23	0.00	66.24%	33.76%	0.00%
CA 51	586.48	421.52	0.00	58.18%	41.82%	0.00%
CA 53	42.08	30.92	0.00	57.64%	42.36%	0.00%
CA 54	68.74	36.26	0.00	65.47%	34.53%	0.00%
CA 55	94.57	87.43	0.00	51.96%	48.04%	0.00%
CA 56	1182.24	911.76	0.00	56.46%	43.54%	0.00%
CA 57	184.27	139.73	0.00	56.87%	43.13%	0.00%
CA 58	526.23	470.77	0.00	52.78%	47.22%	0.00%
CA 61	5.04	3.96	0.00	56.00%	44.00%	0.00%
CA 62	135.05	99.95	0.00	57.47%	42.53%	0.00%
DP 1	174.31	106.69	0.00	62.03%	37.97%	0.00%
DP 2 A	40.32	31.68	0.00	56.00%	44.00%	0.00%
DP 2 B	223.38	173.62	0.00	56.27%	43.73%	0.00%
DP 3	117.59	78.41	0.00	59.99%	40.01%	0.00%
DP 4	221.31	143.69	0.00	60.63%	39.37%	0.00%
DP 5	554.15	661.85	0.00	45.57%	54.43%	0.00%
DP 6	405.24	299.76	0.00	57.48%	42.52%	0.00%
DP 7	498.08	456.92	0.00	52.15%	47.85%	0.00%
DP 8	243.39	178.61	0.00	57.68%	42.32%	0.00%

BASIN	ACRES IMPERV.	ACRES GRASS	ACRES FOREST	PERCENT IMPERV.	PERCENT GRASS	PERCENT FOREST
DP 9	428.11	321.89	0.00	57.08%	42.92%	0.00%
DP 10	86.24	67.76	0.00	56.00%	44.00%	0.00%
DP 11	543.63	406.37	0.00	57.22%	42.78%	0.00%
DP 12	143.49	66.51	0.00	68.33%	31.67%	0.00%
DP 13	97.54	57.46	0.00	62.93%	37.07%	0.00%
DP 14	313.70	190.30	0.00	62.24%	37.76%	0.00%
DP 15	226.78	160.22	0.00	58.60%	41.40%	0.00%
DP 16	80.66	64.34	0.00	55.63%	44.37%	0.00%
DP 17 A	80.45	94.55	0.00	45.97%	54.03%	0.00%
DP 17 B	251.26	189.74	0.00	56.98%	43.02%	0.00%
DP 18	149.07	103.93	0.00	58.92%	41.08%	0.00%
DP 19	747.67	531.33	0.00	58.46%	41.54%	0.00%
DP 20	128.99	36.01	0.00	78.17%	21.83%	0.00%
DP 21	423.92	245.08	0.00	63.37%	36.63%	0.00%
DP 22	24.64	19.36	0.00	56.00%	44.00%	0.00%
DP 23	163.63	147.37	0.00	52.62%	47.38%	0.00%
DP 24	43.12	33.88	0.00	56.00%	44.00%	0.00%
DP 25 AB	70.30	39.70	0.00	63.91%	36.09%	0.00%
DP 26	187.90	181.10	0.00	50.92%	49.08%	0.00%
DP 27	179.52	144.48	0.00	55.41%	44.59%	0.00%
DP 28	200.25	111.75	0.00	64.18%	35.82%	0.00%
DP 29	81.30	62.70	0.00	56.46%	43.54%	0.00%
DP 30	193.88	141.12	0.00	57.88%	42.12%	0.00%
DP 31	190.18	143.82	0.00	56.94%	43.06%	0.00%
DP 32	94.01	65.99	0.00	58.75%	41.25%	0.00%
DP 33	242.18	209.82	0.00	53.58%	46.42%	0.00%
DP 34	289.35	202.66	0.00	58.81%	41.19%	0.00%
DP 35	157.36	123.64	0.00	56.00%	44.00%	0.00%
DP 36	127.66	131.34	0.00	49.29%	50.71%	0.00%
DP 37	62.72	49.28	0.00	56.00%	44.00%	0.00%
DP 38	405.14	294.86	0.00	57.88%	42.12%	0.00%
DP 39	148.40	116.60	0.00	56.00%	44.00%	0.00%
DP 40	73.36	57.64	0.00	56.00%	44.00%	0.00%
DP 41	183.57	233.43	0.00	44.02%	55.98%	0.00%
DP 42	40.92	63.08	0.00	39.35%	60.65%	0.00%
DP 43	103.63	118.37	0.00	46.68%	53.32%	0.00%
DP 44	73.25	106.75	0.00	35.39%	51.57%	0.00%

BASIN	ACRES IMPERV.	ACRES GRASS	ACRES FOREST	PERCENT IMPERV.	PERCENT GRASS	PERCENT FOREST
DP 45	44.71	50.29	0.00	47.06%	52.94%	0.00%
DP 46	127.38	90.62	0.00	58.43%	41.57%	0.00%
DP 47	17.42	18.58	0.00	48.39%	51.61%	0.00%
DP 48	33.53	8.47	0.00	79.84%	20.16%	0.00%
DP 49	228.06	160.94	0.00	58.63%	41.37%	0.00%
DP 50	76.38	95.62	0.00	44.40%	55.60%	0.00%
DP 51	127.63	89.37	0.00	58.81%	41.19%	0.00%
DP 52	406.90	344.10	0.00	54.18%	45.82%	0.00%
DP 53	84.00	66.00	0.00	56.00%	44.00%	0.00%
DP 54	85.68	67.32	0.00	56.00%	44.00%	0.00%
DP 55	259.42	182.58	0.00	58.69%	41.31%	0.00%
DP 56	84.79	66.21	0.00	56.15%	43.85%	0.00%
DP 62	25.43	25.57	0.00	49.86%	50.14%	0.00%
DP 63	206.93	266.07	0.00	43.75%	56.25%	0.00%
DP 64	30.53	19.47	0.00	61.06%	38.94%	0.00%
DP 65	120.20	131.80	0.00	47.70%	52.30%	0.00%
DP 66	19.90	17.10	0.00	53.78%	46.22%	0.00%
M 1	1708.59	1210.41	0.00	58.53%	41.47%	0.00%
M 2	1780.31	1002.69	0.00	63.97%	36.03%	0.00%
M 3	707.82	380.18	0.00	65.06%	34.94%	0.00%
M 4	2429.24	1963.76	0.00	55.30%	44.70%	0.00%
M 5	1387.04	796.96	0.00	63.51%	36.49%	0.00%
M 6 B	1991.78	1362.22	0.00	59.39%	40.61%	0.00%
M 7	459.23	542.77	0.00	45.83%	54.17%	0.00%
M 8	574.15	470.85	0.00	54.94%	45.06%	0.00%
M 9	2974.33	2387.67	0.00	55.47%	44.53%	0.00%
M 10 A	1823.78	1285.22	0.00	58.66%	41.34%	0.00%
M 10 B	850.97	614.03	0.00	58.09%	41.91%	0.00%
M 11	684.29	379.71	0.00	64.31%	35.69%	0.00%
M 12	135.69	83.31	0.00	61.96%	38.04%	0.00%
M 13	486.86	233.14	0.00	67.62%	32.38%	0.00%
M 14	427.26	263.74	0.00	61.83%	38.17%	0.00%
M 15	911.53	453.47	0.00	66.78%	33.22%	0.00%
M 16	3227.45	2188.55	0.00	59.59%	40.41%	0.00%
M 17	403.23	222.77	0.00	64.41%	35.59%	0.00%
M 18	1286.35	820.65	0.00	61.05%	38.95%	0.00%
M 19	508.01	413.99	0.00	55.10%	44.90%	0.00%

BASIN	ACRES IMPERV.	ACRES GRASS	ACRES FOREST	PERCENT IMPERV.	PERCENT GRASS	PERCENT FOREST
M 20	927.48	644.52	0.00	59.00%	41.00%	0.00%
M 21	814.38	295.62	0.00	73.37%	26.63%	0.00%
M 22	87.23	38.77	0.00	69.23%	30.77%	0.00%
M 23	937.53	808.47	0.00	53.70%	46.30%	0.00%
M 24	80.43	28.57	0.00	73.79%	26.21%	0.00%
M 25	236.50	1198.50	0.00	16.48%	83.52%	0.00%
M 27	454.79	322.21	0.00	58.53%	41.47%	0.00%
M 28	271.51	90.49	0.00	75.00%	25.00%	0.00%
M 29	578.20	267.80	0.00	68.35%	31.65%	0.00%
M 30	483.30	327.70	0.00	59.59%	40.41%	0.00%
M 31	293.98	44.02	0.00	86.97%	13.03%	0.00%
M 32	778.58	243.42	0.00	76.18%	23.82%	0.00%
M 33	304.92	87.08	0.00	77.79%	22.21%	0.00%
M 34	2218.85	1000.15	0.00	68.93%	31.07%	0.00%
M 35	741.75	270.25	0.00	73.30%	26.70%	0.00%
M 36	1561.61	833.39	0.00	65.20%	34.80%	0.00%
M 37	777.71	417.29	0.00	65.08%	34.92%	0.00%
M 38	192.20	75.80	0.00	71.72%	28.28%	0.00%
M 39	369.74	213.26	0.00	63.42%	36.58%	0.00%
M 40	8524.61	5727.39	0.00	59.81%	40.19%	0.00%
M 41	3233.89	1855.11	0.00	63.55%	36.45%	0.00%
M 42	817.02	546.98	0.00	59.90%	40.10%	0.00%
M 43	960.14	600.86	0.00	61.51%	38.49%	0.00%
M 44	193.06	144.94	0.00	57.12%	42.88%	0.00%
M 45	629.52	347.48	0.00	64.43%	35.57%	0.00%
M 46	1670.32	963.68	0.00	63.41%	36.59%	0.00%
M 47	2077.43	1189.57	0.00	63.59%	36.41%	0.00%
M 48 DP	699.84	333.16	0.00	67.75%	32.25%	0.00%
M 48 LM	5683.92	4057.08	0.00	65.27%	46.59%	0.00%
M 49 A	926.85	476.15	0.00	66.06%	33.94%	0.00%
M 49 BDP	859.46	558.54	0.00	60.61%	39.39%	0.00%
M 49 BLM	1210.75	785.25	0.00	209.47%	135.86%	0.00%
M 50	3957.06	2520.94	0.00	61.08%	38.92%	0.00%
M 51	6684.81	4615.19	0.00	59.16%	40.84%	0.00%
M 52	2884.96	1565.04	0.00	64.83%	35.17%	0.00%
M 53	1379.06	1233.94	0.00	52.78%	47.22%	0.00%
M 55	652.26	335.74	0.00	66.02%	33.98%	0.00%

BASIN	ACRES IMPERV.	ACRES GRASS	ACRES FOREST	PERCENT IMPERV.	PERCENT GRASS	PERCENT FOREST
M 55 A	1484.30	586.70	0.00	71.67%	28.33%	0.00%
MN 1 A	169.12	132.88	0.00	56.00%	44.00%	0.00%
MN 1 B	669.37	746.63	0.00	47.27%	52.73%	0.00%
MN 1 C	1031.89	736.11	0.00	58.36%	41.64%	0.00%
MN 1 D	312.63	231.37	0.00	57.47%	42.53%	0.00%
MN 1 E	375.88	264.12	0.00	58.73%	41.27%	0.00%
MN 2 A	106.00	159.00	0.00	40.00%	60.00%	0.00%
MN 2 B	346.92	322.08	0.00	51.86%	48.14%	0.00%
MN 2 C	199.02	264.98	0.00	42.89%	57.11%	0.00%
MN 3	796.28	519.72	0.00	60.51%	39.49%	0.00%
MN 4	729.76	653.24	0.00	52.77%	47.23%	0.00%
MN 5	573.67	374.33	0.00	60.51%	39.49%	0.00%

APPENDIX E UNGAGED AREAS

BASIN	ACRES IMPERV.	ACRES GRASS	ACRES FOREST	PERCENT IMPERV.	PERCENT GRASS	PERCENT FOREST
CAL 12	909.75	691.25	0.00	56.82%	43.18%	0.00%
CAL 16	506.38	1367.37	39.25	26.47%	71.48%	2.05%
CAL 17	6669.43	7906.04	1294.53	42.03%	49.82%	8.16%
CAL 18	404.37	798.63	0.00	33.61%	66.39%	0.00%
CAL 20	2107.01	3243.68	4.31	39.35%	60.57%	0.08%
CAL 21	6215.71	9948.85	1610.44	34.97%	55.97%	9.06%
CAL 22	4092.69	4037.31	0.00	50.34%	49.66%	0.00%
CAL 24	766.43	1252.57	0.00	37.96%	62.04%	0.00%
LDP 9	6.48	2.52	0.00	72.00%	28.00%	0.00%
LDP 11	3017.93	3040.85	2394.22	35.70%	35.97%	28.32%
LDP 12	3866.40	2689.60	0.00	58.98%	41.02%	0.00%
LDP 15	237.90	635.10	0.00	27.25%	72.75%	0.00%
LDP 16	3545.22	5872.14	8590.64	19.69%	32.61%	47.70%
LDP 17	1411.74	1135.26	0.00	55.43%	44.57%	0.00%
LDP 20	271.55	335.74	11.71	43.87%	54.24%	1.89%

APPENDIX F

LAND COVER IN ACRES
FOR EACH GAGE ASSIGNMENT PARTITION

BASIN ID #	TOTAL AREA	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS
CA 2	718.00	12	474.91	66.1%	273.19	201.72	17	161.73	22.5%	93.03	68.70	18	81.36	11.3%	46.80	34.56					
CA 5	201.00	18	201.00	100.0%	112.73	88.27															
CA 6	681.00	18	380.45	55.9%	201.01	179.44	21	128.81	18.9%	68.06	60.75	22	171.74	25.2%	90.74	81.00					
CA 7	741.00	18	741.00	100.0%	424.63	316.37															
CA 7 M	659.00	19	659.00	100.0%	408.41	250.59															
CA 7 Q	4063.00	13	907.53	22.3%	556.22	351.31	14	1954.30	48.1%	1197.78	756.52	18	173.16	4.3%	106.13	67.03	19	1028.01	25.30%	630.06	397.95
CA 7 V	847.00	19	847.00	100.0%	543.21	303.79															
CA 7 W	2497.00	13	248.24	9.9%	124.77	123.47	14	27.23	1.1%	13.69	13.54	18	1459.25	58.4%	733.42	725.83	19	762.28	30.53%	383.12	379.16
CA 8	747.00	18	747.00	100.0%	392.67	354.33															
CA 9	203.00	18	203.00	100.0%	120.93	82.07															
CA 10	5439.00	13	998.98	18.4%	568.08	430.90	18	4440.02	81.6%	2524.84	1915.18										
CA 11 A	567.00	18	567.00	100.0%	276.53	290.47															
CA 11 B	844.00	18	844.00	100.0%	465.90	378.10															
CA 12	2622.00	18	2622.00	100.0%	1508.08	1113.92															
CA 13	3726.00	13	1737.48	46.6%	1012.54	724.94	18	1988.52	53.4%	1158.83	829.69										
CA 14	187.00	18	187.00	100.0%	132.77	54.23															
CA 15 A	960.00	18	960.00	100.0%	438.17	521.83															
CA 15 B	79.00	18	69.34	87.8%	40.93	28.41	22	9.66	12.2%	5.70	3.96										
CA 16	227.00	18	227.00	100.0%	140.17	86.83															
CA 17	527.00	18	310.59	58.9%	194.39	116.20	22	216.41	41.1%	135.44	80.97										
CA 18	1062.00	18	214.18	20.2%	133.38	80.80	19	563.30	53.0%	350.78	212.52	22	164.12	15.5%	102.20	61.92	23	120.40	11.34%	74.98	45.42
CA 18 E-A	3908.00	12	178.97	4.6%	78.63	100.34	13	270.45	6.9%	118.81	151.64	17	2105.60	53.9%	925.03	1180.57	18	1392.98	35.64%	611.97	781.01
CA 18 E-B	786.00	17	786.00	100.0%	551.87	234.13															
CA 19	1267.00	19	243.54	19.2%	130.93	112.61	23	1023.46	80.8%	550.21	473.25										
CA 20	249.00	19	249.00	100.0%	166.35	82.65															
CA 21	636.00	19	243.54	38.3%	94.35	149.19	23	392.46	61.7%	152.05	240.41										
CA 22	671.00	19	671.00	100.0%	247.85	423.15															
CA 23	603.00	19	533.41	88.5%	157.43	375.98	23	69.59	11.5%	20.54	49.05										
CA 24	1453.00	19	20.37	1.4%	9.01	11.36	23	1432.63	98.6%	633.45	799.18										
CA 25	312.00	18	238.14	76.3%	119.07	119.07	19	73.86	23.7%	36.93	36.93										
CA 26	586.00	19	586.00	100.0%	282.98	303.02															
CA 27	390.00	19	390.00	100.0%	155.54	234.46															
CA 28	1329.00	19	1329.00	100.0%	343.00	986.00															
CA 29	802.00	19	802.00	100.0%	458.92	343.08															
CA 30	766.00	19	766.00	100.0%	431.68	334.32															
CA 31	947.00	19	947.00	100.0%	522.63	424.37															
CA 32	860.00	14	116.38	13.5%	55.76	60.62	19	743.62	86.5%	356.25	387.37										
CA 33	112.00	14	112.00	100.0%	74.97	37.03															
CA 34 A	1590.00	14	1281.49	80.6%	784.71	496.78	19	308.51	19.4%	188.91	119.60										
CA 34 B	100.00	14	100.00	100.0%	60.09	39.91															
CA 36	141.00	14	141.00	100.0%	83.10	57.90															
CA 38	950.00	19	950.00	100.0%	391.61	558.39															
CA 39	896.00	18	522.65	58.3%	274.65	248.00	22	373.35	41.7%	196.20	177.15										
CA 40	1207.00	22	1207.00	100.0%	672.65	534.35															
CA 41	465.00	22	465.00	100.0%	290.86	174.14															
CA 42 A	680.00	22	680.00	100.0%	446.73	233.27															
CA 42 B	576.00	18	307.33	53.4%	170.15	137.18	22	268.67	46.6%	148.75	119.92										
CA 43 A	26.00	22	26.00	100.0%	14.56	11.44															
CA 43 B	44.00	22	44.00	100.0%	28.32	14.68															
CA 44	70.00	22	70.00	100.0%	49.81	20.19															

BASIN ID #	TOTAL AREA	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS
CA 45	157.00	22	157.00	100.0%	99.72	57.28															
CA 46	343.00	22	343.00	100.0%	204.20	138.80															
CA 47	54.00	22	54.00	100.0%	35.47	18.53															
CA 48	349.00	22	349.00	100.0%	244.28	104.72															
CA 49	267.00	22	267.00	100.0%	184.60	82.40															
CA 50	131.00	22	131.00	100.0%	88.77	44.23															
CA 51	1008.00	22	767.42	76.1%	446.51	320.91	23	240.58	23.9%	139.98	100.60										
CA 53	73.00	23	73.00	100.0%	42.08	30.92															
CA 54	105.00	23	105.00	100.0%	68.74	36.26															
CA 55	182.00	23	182.00	100.0%	94.57	87.43															
CA 56	2094.00	23	2094.00	100.0%	1182.24	911.76															
CA 57	324.00	22	324.00	100.0%	184.27	139.73															
CA 58	997.00	18	923.76	92.7%	487.57	436.19	22	73.24	7.3%	38.66	34.58										
CA 61	9.00	22	9.00	100.0%	5.04	3.96															
CA 62	235.00	22	235.00	100.0%	135.05	99.95															
DP 1	281.00	3	281.00	100.0%	174.31	106.69															
DP 2 A	72.00	3	72.00	100.0%	40.32	31.68															
DP 2 B	397.00	3	397.00	100.0%	223.38	173.62															
DP 3	196.00	3	196.00	100.0%	117.59	78.41															
DP 4	365.00	3	365.00	100.0%	221.31	143.69															
DP 5	1216.00	3	1212.19	99.7%	552.41	659.78	4	3.71	0.3%	1.69	2.02										
DP 6	705.00	3	705.00	100.0%	405.24	299.76															
DP 7	955.00	3	955.00	100.0%	498.08	456.92															
DP 8	422.00	3	422.00	100.0%	243.39	178.61															
DP 9	750.00	3	317.44	42.3%	181.20	136.24	5	144.97	19.3%	82.75	62.22	6	287.59	38.3%	164.16	123.43					
DP 10	154.00	5	154.00	100.0%	86.24	67.76															
DP 11	950.00	5	549.46	57.8%	314.42	235.04	6	400.54	42.2%	229.20	171.34										
DP 12	210.00	5	210.00	100.0%	143.49	66.51															
DP 13	155.00	5	155.00	100.0%	97.54	57.46															
DP 14	504.00	5	504.00	100.0%	313.70	190.30															
DP 15	387.00	5	387.00	100.0%	226.78	160.22															
DP 16	145.00	5	145.00	100.0%	80.66	64.34															
DP 17 A	175.00	5	175.00	100.0%	80.45	94.55															
DP 17 B	441.00	5	441.00	100.0%	251.26	189.74															
DP 18	253.00	5	253.00	100.0%	149.07	103.93															
DP 19	1278.00	5	415.14	32.5%	242.68	172.46	6	863.86	67.5%	504.99	358.87										
DP 20	165.00	5	165.00	100.0%	128.99	36.01															
DP 21	669.00	5	656.66	98.2%	416.10	240.56	8	12.34	1.8%	7.82	4.52										
DP 22	44.00	5	44.00	100.0%	24.64	19.36															
DP 23	311.00	5	136.48	43.9%	71.80	64.66	6	174.54	56.1%	91.84	82.70										
DP 24	77.00	5	77.00	100.0%	43.12	33.88															
DP 25 AB	110.00	5	74.88	68.1%	47.86	27.02	8	35.12	31.9%	22.44	12.68										
DP 26	369.00	5	89.00	24.1%	45.32	43.68	6	15.44	4.2%	7.86	7.58	8	67.90	18.4%	34.57	33.33	9	196.68	53.30%	100.14	96.52
DP 27	324.00	5	1.49	0.5%	0.83	0.66	6	322.51	99.5%	178.70	143.81										
DP 28	312.00	9	312.00	100.0%	200.25	111.75															
DP 29	144.00	8	75.90	52.7%	42.85	33.05	9	68.10	47.3%	38.45	29.65										
DP 30	335.00	9	335.00	100.0%	193.88	141.12															
DP 31	334.00	8	334.00	100.0%	190.18	143.82															
DP 32	160.00	8	160.00	100.0%	94.01	65.99															
DP 33	452.00	8	452.00	100.0%	242.18	209.82															

BASIN ID #	TOTAL AREA	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS
DP 34	492.00	8	492.00	100.0%	289.35	202.66															
DP 35	281.00	8	281.00	100.0%	157.38	123.64															
DP 36	259.00	8	118.78	45.9%	58.54	60.24	9	140.22	54.1%	69.11	71.11										
DP 37	112.00	8	112.00	100.0%	62.72	49.28															
DP 38	700.00	8	700.00	100.0%	405.14	294.86															
DP 39	265.00	8	265.00	100.0%	148.40	116.60															
DP 40	131.00	8	131.00	100.0%	73.36	57.64															
DP 41	417.00	8	1.01	0.2%	0.44	0.57	9	415.99	99.8%	183.12	232.87										
DP 42	104.00	8	104.00	100.0%	40.92	63.08															
DP 43	222.00	8	183.08	82.5%	85.45	97.61	9	38.94	17.5%	18.18	20.76										
DP 44	207.00	8	122.08	59.0%	49.57	72.51	9	84.92	41.0%	34.48	50.44										
DP 45	95.00	9	95.00	100.0%	44.71	50.29															
DP 46	218.00	8	218.00	100.0%	127.38	90.62															
DP 47	36.00	8	26.00	72.2%	12.58	13.42															
DP 48	42.00	8	42.00	100.0%	33.53	8.47															
DP 49	389.00	8	146.53	37.7%	85.91	60.62	9	242.47	62.3%	142.15	100.32										
DP 50	172.00	8	172.00	100.0%	76.38	95.62															
DP 51	217.00	8	217.00	100.0%	127.63	89.37															
DP 52	751.00	5	751.00	100.0%	406.90	344.10															
DP 53	150.00	8	150.00	100.0%	84.00	66.00															
DP 54	153.00	8	153.00	100.0%	85.68	67.32															
DP 55	442.00	8	424.12	96.0%	248.92	175.20	11	17.88	4.0%	10.49	7.39										
DP 56	151.00	8	151.00	100.0%	84.79	66.21															
DP 62	51.00	8	51.00	100.0%	25.43	25.57															
DP 63	473.00	5	173.37	36.7%	75.85	97.52	6	260.89	55.2%	114.18	148.81	9	38.64	8.2%	16.90	21.74					
DP 64	50.00	8	26.37	52.7%	16.10	10.27	9	23.63	47.3%	14.43	9.20										
DP 65	252.00	8	252.00	100.0%	120.20	131.80															
DP 66	37.00	8	37.00	100.0%	19.90	17.10															
M 1	2919.00	2	2919.00	100.0%	1708.59	1210.41															
M 2	2783.00	2	78.08	2.8%	49.95	28.13	4	2704.92	97.2%	1730.36	974.56										
M 3	1088.00	4	1088.00	100.0%	707.82	380.18															
M 4	4393.00	4	2140.30	48.7%	1183.54	956.76	7	2252.70	51.3%	1245.70	1007.00										
M 5	2184.00	4	2184.00	100.0%	1387.04	796.96															
M 6	3354.00	4	3354.00	100.0%	1991.78	1362.22															
M 7	1002.00	4	1002.00	100.0%	459.23	542.77															
M 8	1045.00	4	511.69	49.0%	281.13	230.56	6	162.82	15.6%	89.46	73.36	7	370.49	35.5%	203.56	166.93					
M 9	5362.00	4	677.75	12.6%	375.95	301.80	6	4684.25	87.4%	2598.38	2085.87										
M 10 A	3109.00	6	3109.00	100.0%	1823.78	1285.22															
M 10 B	1465.00	6	1460.48	99.7%	848.35	612.13	7	4.52	0.3%	2.63	1.89										
M 11	1064.00	6	40.45	3.8%	26.01	14.44	7	1023.55	96.2%	658.27	365.28										
M 12	219.00	6	40.65	18.6%	25.19	15.46	7	178.35	81.4%	110.50	67.85										
M 13	720.00	6	720.00	100.0%	486.86	233.14															
M 14	691.00	6	671.43	97.2%	415.16	256.27	7	19.57	2.8%	12.10	7.47										
M 15	1365.00	6	58.30	4.3%	38.93	19.37	7	1306.70	95.7%	872.60	434.10										
M 16	5416.00	6	5416.00	100.0%	3227.45	2188.55															
M 17	626.00	6	357.50	57.1%	230.28	127.22	7	268.50	42.9%	172.95	95.55										
M 18	2107.00	6	1766.24	83.8%	1078.31	687.93	7	340.76	16.2%	208.04	132.72										
M 19	922.00	7	922.00	100.0%	508.01	413.99															
M 20	1572.00	6	688.86	43.8%	406.43	282.43	7	883.14	56.2%	521.05	362.09										
M 21	1110.00	7	1110.00	100.0%	814.38	295.62															

BASIN ID #	TOTAL AREA	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS	GAGE NUMBER	AREA ACRES	% OF TOTAL	ACRES IMPERV.	ACRES GRASS
M 22	126.00	7	126.00	100.0%	87.23	38.77															
M 23	1746.00	6	610.44	35.0%	327.78	282.66	7	773.56	44.3%	415.37	358.19	9	353.20	20.2%	189.65	163.55	10	8.80	0.50%	4.73	4.07
M 24	109.00	7	109.00	100.0%	80.43	28.57															
M 25	1435.00	4	684.52	47.7%	112.81	571.71	6	750.48	52.3%	123.68	626.80										
M 27	777.00	6	3.13	0.4%	1.83	1.30	7	643.76	82.9%	376.80	266.96	10	130.11	16.7%	76.16	53.95					
M 28	362.00	7	313.62	86.6%	235.22	78.40	10	48.38	13.4%	36.29	12.09										
M 29	846.00	7	225.50	26.7%	154.12	71.38	10	620.50	73.3%	424.08	196.42										
M 30	811.00	7	657.04	81.0%	391.55	265.49	10	153.96	19.0%	91.75	62.21										
M 31	338.00	10	338.00	100.0%	293.98	44.02															
M 32	1022.00	10	1022.00	100.0%	778.58	243.42															
M 33	392.00	10	392.00	100.0%	304.92	87.08															
M 34	3219.00	10	3263.00	101.4%	2249.17	1013.83															
M 35	1012.00	10	1012.00	100.0%	741.75	270.25															
M 36	2395.00	6	1.38	0.1%	0.90	0.48	7	4.22	0.2%	2.75	1.47	9	825.42	34.5%	538.20	287.22	10	1563.98	65.30%	1019.76	544.22
M 37	1195.00	10	1195.00	100.0%	777.71	417.29															
M 38	268.00	10	268.00	100.0%	192.20	75.80															
M 39	583.00	10	583.00	100.0%	369.74	213.26															
M 40	14252.00	10	894.92	6.3%	535.28	359.64	13	11501.39	80.7%	6879.37	4622.02	14	1855.69	13.0%	1109.95	745.74					
M 41	5089.00	9	806.72	15.9%	512.64	294.08	10	1925.20	37.8%	1223.40	701.80	12	699.69	13.7%	444.63	255.06	13	1657.39	32.57%	1053.22	604.17
M 42	1364.00	9	474.48	34.8%	284.21	190.27	10	889.52	65.2%	532.81	356.71										
M 43	1581.00	9	1458.90	93.5%	897.34	561.56	10	102.10	6.5%	62.80	39.30										
M 44	338.00	10	338.00	100.0%	193.06	144.94															
M 45	977.00	9	977.00	100.0%	629.53	347.47															
M 46	1657.00	9	928.80	56.1%	587.41	341.39	12	728.20	43.9%	460.54	267.66										
M 47	3267.00	6	900.81	27.6%	572.81	328.00	9	2368.19	72.4%	1504.62	861.57										
M 48 DP	1033.00	12	1033.00	100.0%	699.84	333.16															
M 48 LM	8708.00	9	631.54	7.3%	361.47	270.07	12	6719.04	77.2%	3845.68	2873.36	13	1357.42	15.6%	778.93	580.49					
M 49 A	1403.00	9	621.07	44.3%	410.29	210.78	12	781.93	55.7%	518.56	265.37										
M 49 BDP	1418.00	12	1418.00	100.0%	859.46	558.54															
M 49 BLM	578.00	12	578.00	100.0%	351.28	226.72															
M 50	6478.00	10	4026.29	62.2%	2459.44	1566.85	13	1117.35	17.2%	682.53	434.82	14	1334.36	20.6%	815.09	519.27					
M 51	11300.00	6	3735.00	33.1%	2209.54	1525.46	9	7544.72	66.8%	4463.28	3081.44	12	20.28	0.2%	12.00	8.28					
M 52	4450.00	9	4450.00	100.0%	2884.96	1565.04															
M 53	2613.00	8	1695.40	64.9%	894.78	800.62	9	29.03	1.1%	15.32	13.71	11	888.57	34.0%	468.96	419.61					
M 55 A	988.00	11	390.08	39.5%	257.53	132.55	12	597.92	60.5%	394.74	203.18										
M 55 B	2071.00	11	826.52	39.9%	592.37	234.15	12	1244.48	60.1%	891.93	352.55										
MN 1 A	302.00	3	109.89	36.4%	61.54	48.35	4	192.11	63.6%	107.58	84.53										
MN 1 B	1416.00	3	199.09	14.1%	94.11	104.98	4	1177.98	83.2%	558.85	621.13	6	38.95	2.8%	18.41	20.54					
MN 1 C	1768.00	3	213.11	12.1%	124.38	88.73	4	745.07	42.1%	434.88	310.21	6	809.82	45.8%	472.65	337.17					
MN 1 D	544.00	4	213.90	39.3%	122.92	90.98	6	330.10	60.7%	189.70	140.40										
MN 1 E	640.00	6	640.00	100.0%	375.88	264.12															
MN 2 A	265.00	4	265.00	100.0%	106.00	159.00															
MN 2 B	669.00	4	669.00	100.0%	346.92	322.08															
MN 2 C	464.00	4	444.35	95.8%	190.59	253.76	6	19.65	4.2%	8.43	11.22										
MN 3	1316.00	4	1316.00	100.0%	796.28	519.72															
MN 4	1383.00	3	440.32	31.8%	232.34	207.98	4	942.68	68.2%	497.42	445.26										
MN 5	948.00	4	948.00	100.0%	573.67	374.33															

APPENDIX G

LAND COVER IN SQUARE MILES FOR EACH GAGE ASSIGNMENT PARTITION

BASIN ID #	TOTAL AREA	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS
CA 2	1.122	12	0.742	66.1%	0.427	0.315	17	0.253	22.5%	0.145	0.107	18	0.127	11.3%	0.073	0.054					
CA 5	0.314	18	0.314	100.0%	0.176	0.138															
CA 6	1.064	18	0.594	55.9%	0.314	0.280	21	0.201	18.9%	0.106	0.095	22	0.268	25.2%	0.142	0.127					
CA 7	1.158	18	1.158	100.0%	0.663	0.494															
CA 7 M	1.030	19	1.030	100.0%	0.638	0.392															
CA 7 Q	6.348	13	1.418	22.3%	0.869	0.549	14	3.054	48.1%	1.872	1.182	18	0.271	4.3%	0.166	0.105	19	1.606	25.30%	0.984	0.622
CA 7 V	1.323	19	1.323	100.0%	0.849	0.475															
CA 7 W	3.902	13	0.388	9.9%	0.195	0.193	14	0.043	1.1%	0.021	0.021	18	2.280	58.4%	1.148	1.134	19	1.191	30.53%	0.599	0.592
CA 8	1.167	18	1.167	100.0%	0.614	0.554															
CA 9	0.317	18	0.317	100.0%	0.189	0.128															
CA 10	8.498	13	1.561	18.4%	0.688	0.673	18	6.938	81.6%	3.945	2.992										
CA 11 A	0.886	18	0.886	100.0%	0.432	0.454															
CA 11 B	1.319	18	1.319	100.0%	0.728	0.591															
CA 12	4.097	18	4.097	100.0%	2.358	1.740															
CA 13	5.822	13	2.715	46.6%	1.582	1.133	18	3.107	53.4%	1.811	1.296										
CA 14	0.292	18	0.292	100.0%	0.207	0.085															
CA 15 A	1.500	18	1.500	100.0%	0.685	0.815															
CA 15 B	0.123	18	0.108	87.8%	0.064	0.044	22	0.015	12.2%	0.009	0.006										
CA 16	0.355	18	0.355	100.0%	0.219	0.136															
CA 17	0.823	18	0.485	58.9%	0.304	0.182	22	0.338	41.1%	0.212	0.127										
CA 18	1.659	18	0.335	20.2%	0.208	0.126	19	0.880	53.0%	0.548	0.332	22	0.256	15.5%	0.160	0.097	23	0.188	11.34%	0.117	0.071
CA 18 E-A	6.106	12	0.280	4.6%	0.123	0.157	13	0.423	6.9%	0.186	0.237	17	3.290	53.9%	1.445	1.845	18	2.177	35.64%	0.958	1.220
CA 18 E-B	1.228	17	1.228	100.0%	0.862	0.366															
CA 19	1.980	19	0.381	19.2%	0.205	0.176	23	1.599	80.8%	0.860	0.739										
CA 20	0.389	19	0.389	100.0%	0.260	0.129															
CA 21	0.994	19	0.381	38.3%	0.147	0.233	23	0.613	61.7%	0.238	0.376										
CA 22	1.048	19	1.048	100.0%	0.387	0.661															
CA 23	0.942	19	0.833	88.5%	0.246	0.587	23	0.109	11.5%	0.032	0.077										
CA 24	2.270	19	0.032	1.4%	0.014	0.018	23	2.238	98.6%	0.990	1.249										
CA 25	0.488	18	0.372	76.3%	0.186	0.186	19	0.115	23.7%	0.058	0.058										
CA 26	0.916	19	0.916	100.0%	0.442	0.473															
CA 27	0.609	19	0.609	100.0%	0.243	0.366															
CA 28	2.077	19	2.077	100.0%	0.536	1.541															
CA 29	1.253	19	1.253	100.0%	0.717	0.536															
CA 30	1.197	19	1.197	100.0%	0.674	0.522															
CA 31	1.480	19	1.480	100.0%	0.817	0.663															
CA 32	1.344	14	0.182	13.5%	0.087	0.095	19	1.162	86.5%	0.557	0.605										
CA 33	0.175	14	0.175	100.0%	0.117	0.058															
CA 34 A	2.484	14	2.002	80.6%	1.226	0.776	19	0.482	19.4%	0.295	0.187										
CA 34 B	0.156	14	0.156	100.0%	0.094	0.062															
CA 36	0.220	14	0.220	100.0%	0.130	0.090															
CA 38	1.484	19	1.484	100.0%	0.612	0.872															
CA 39	1.400	18	0.817	58.3%	0.429	0.387	22	0.583	41.7%	0.307	0.277										
CA 40	1.886	22	1.888	100.0%	1.051	0.835															
CA 41	0.727	22	0.727	100.0%	0.454	0.272															
CA 42 A	1.063	22	1.063	100.0%	0.698	0.364															
CA 42 B	0.900	18	0.480	53.4%	0.266	0.214	22	0.420	46.6%	0.232	0.187										
CA 43 A	0.041	22	0.041	100.0%	0.023	0.018															
CA 43 B	0.069	22	0.069	100.0%	0.048	0.023															
CA 44	0.109	22	0.109	100.0%	0.078	0.032															

BASIN ID #	TOTAL AREA	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS
CA 45	0.245	22	0.245	100.0%	0.156	0.090															
CA 46	0.536	22	0.536	100.0%	0.319	0.217															
CA 47	0.084	22	0.084	100.0%	0.055	0.029															
CA 48	0.545	22	0.545	100.0%	0.382	0.164															
CA 49	0.417	22	0.417	100.0%	0.288	0.129															
CA 50	0.205	22	0.205	100.0%	0.136	0.069															
CA 51	1.575	22	1.199	76.1%	0.698	0.501	23	0.376	23.9%	0.219	0.157										
CA 53	0.114	23	0.114	100.0%	0.066	0.048															
CA 54	0.164	23	0.164	100.0%	0.107	0.057															
CA 55	0.284	23	0.284	100.0%	0.148	0.137															
CA 56	3.272	23	3.272	100.0%	1.847	1.425															
CA 57	0.506	22	0.506	100.0%	0.288	0.218															
CA 58	1.558	18	1.443	92.7%	0.762	0.682	22	0.114	7.3%	0.060	0.054										
CA 61	0.014	22	0.014	100.0%	0.008	0.006															
CA 62	0.367	22	0.367	100.0%	0.211	0.156															
DP 1	0.439	3	0.439	100.0%	0.272	0.167															
DP 2 A	0.113	3	0.113	100.0%	0.063	0.050															
DP 2 B	0.620	3	0.620	100.0%	0.349	0.271															
DP 3	0.306	3	0.306	100.0%	0.184	0.123															
DP 4	0.570	3	0.570	100.0%	0.346	0.225															
DP 5	1.900	3	1.894	99.7%	0.863	1.031	4	0.006	0.3%	0.003	0.003										
DP 6	1.102	3	1.102	100.0%	0.633	0.468															
DP 7	1.492	3	1.492	100.0%	0.778	0.714															
DP 8	0.659	3	0.659	100.0%	0.380	0.279															
DP 9	1.172	3	0.496	42.3%	0.283	0.213	5	0.227	19.3%	0.129	0.097	6	0.449	38.3%	0.256	0.193					
DP 10	0.241	5	0.241	100.0%	0.135	0.106															
DP 11	1.484	5	0.859	57.8%	0.491	0.367	6	0.626	42.2%	0.358	0.268										
DP 12	0.328	5	0.328	100.0%	0.224	0.104															
DP 13	0.242	5	0.242	100.0%	0.152	0.090															
DP 14	0.788	5	0.788	100.0%	0.490	0.297															
DP 15	0.605	5	0.605	100.0%	0.354	0.250															
DP 16	0.227	5	0.227	100.0%	0.126	0.101															
DP 17 A	0.273	5	0.273	100.0%	0.126	0.148															
DP 17 B	0.689	5	0.689	100.0%	0.393	0.296															
DP 18	0.395	5	0.395	100.0%	0.233	0.162															
DP 19	1.998	5	0.649	32.5%	0.379	0.269	6	1.350	67.5%	0.789	0.561										
DP 20	0.258	5	0.258	100.0%	0.202	0.056															
DP 21	1.045	5	1.026	98.2%	0.650	0.376	8	0.019	1.8%	0.012	0.007										
DP 22	0.069	5	0.069	100.0%	0.039	0.030															
DP 23	0.486	5	0.213	43.9%	0.112	0.101	6	0.273	56.1%	0.143	0.129										
DP 24	0.120	5	0.120	100.0%	0.067	0.053															
DP 25 AB	0.172	5	0.117	68.1%	0.075	0.042	8	0.055	31.9%	0.035	0.020										
DP 26	0.577	5	0.139	24.1%	0.071	0.068	6	0.024	4.2%	0.012	0.012	8	0.106	18.4%	0.054	0.052	9	0.307	53.30%	0.156	0.151
DP 27	0.506	5	0.002	0.5%	0.001	0.001	6	0.504	99.5%	0.279	0.225										
DP 28	0.488	9	0.488	100.0%	0.313	0.175															
DP 29	0.225	8	0.119	52.7%	0.067	0.052	9	0.106	47.3%	0.060	0.046										
DP 30	0.523	9	0.523	100.0%	0.303	0.220															
DP 31	0.522	8	0.522	100.0%	0.297	0.225															
DP 32	0.250	8	0.250	100.0%	0.147	0.103															
DP 33	0.706	8	0.706	100.0%	0.378	0.328															

BASIN ID #	TOTAL AREA	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV	SQ. MI GRASS
DP 34	0.769	8	0.769	100.0%	0.452	0.317															
DP 35	0.439	8	0.439	100.0%	0.246	0.193															
DP 36	0.405	8	0.186	45.9%	0.091	0.094	9	0.219	54.1%	0.108	0.111										
DP 37	0.175	8	0.175	100.0%	0.098	0.077															
DP 38	1.094	8	1.094	100.0%	0.633	0.461															
DP 39	0.414	8	0.414	100.0%	0.232	0.182															
DP 40	0.205	8	0.205	100.0%	0.115	0.090															
DP 41	0.652	8	0.002	0.2%	0.001	0.001	9	0.650	99.8%	0.288	0.364										
DP 42	0.163	8	0.163	100.0%	0.064	0.099															
DP 43	0.347	8	0.286	82.5%	0.134	0.153	9	0.061	17.5%	0.028	0.032										
DP 44	0.323	8	0.191	59.0%	0.077	0.113	9	0.133	41.0%	0.054	0.079										
DP 45	0.148	9	0.148	100.0%	0.070	0.079															
DP 46	0.341	8	0.341	100.0%	0.199	0.142															
DP 47	0.058	8	0.041	72.2%	0.020	0.021															
DP 48	0.066	8	0.066	100.0%	0.052	0.013															
DP 49	0.608	8	0.229	37.7%	0.134	0.095	9	0.379	62.3%	0.222	0.157										
DP 50	0.269	8	0.269	100.0%	0.119	0.149															
DP 51	0.339	8	0.339	100.0%	0.199	0.140															
DP 52	1.173	5	1.173	100.0%	0.636	0.538															
DP 53	0.234	8	0.234	100.0%	0.131	0.103															
DP 54	0.239	8	0.239	100.0%	0.134	0.105															
DP 55	0.691	8	0.663	96.0%	0.389	0.274	11	0.028	4.0%	0.016	0.012										
DP 56	0.236	8	0.236	100.0%	0.132	0.103															
DP 62	0.080	8	0.080	100.0%	0.040	0.040															
DP 63	0.739	5	0.271	36.7%	0.119	0.152	6	0.408	55.2%	0.178	0.229	9	0.060	8.2%	0.026	0.034					
DP 64	0.078	8	0.041	52.7%	0.025	0.016	9	0.037	47.3%	0.023	0.014										
DP 65	0.394	8	0.394	100.0%	0.188	0.206															
DP 66	0.058	8	0.058	100.0%	0.031	0.027															
M 1	4.581	2	4.581	100.0%	2.670	1.891															
M 2	4.348	2	0.122	2.8%	0.078	0.044	4	4.226	97.2%	2.704	1.523										
M 3	1.700	4	1.700	100.0%	1.106	0.594															
M 4	6.864	4	3.344	48.7%	1.849	1.495	7	3.520	51.3%	1.948	1.573										
M 5	3.413	4	3.413	100.0%	2.167	1.245															
M 6	5.241	4	5.241	100.0%	3.112	2.128															
M 7	1.566	4	1.566	100.0%	0.718	0.848															
M 8	1.633	4	0.800	49.0%	0.439	0.360	6	0.254	15.6%	0.140	0.115	7	0.579	35.5%	0.318	0.261					
M 9	8.378	4	1.059	12.6%	0.587	0.472	6	7.319	87.4%	4.060	3.259										
M 10 A	4.858	6	4.858	100.0%	2.850	2.008															
M 10 B	2.289	6	2.282	99.7%	1.326	0.956	7	0.007	0.3%	0.004	0.003										
M 11	1.663	6	0.063	3.8%	0.041	0.023	7	1.599	96.2%	1.029	0.571										
M 12	0.342	6	0.064	18.6%	0.039	0.024	7	0.279	81.4%	0.173	0.106										
M 13	1.125	6	1.125	100.0%	0.761	0.364															
M 14	1.080	6	1.049	97.2%	0.649	0.400	7	0.031	2.8%	0.019	0.012										
M 15	2.133	6	0.091	4.3%	0.061	0.030	7	2.042	95.7%	1.363	0.678										
M 16	8.463	6	8.463	100.0%	5.043	3.420															
M 17	0.978	6	0.559	57.1%	0.360	0.199	7	0.420	42.9%	0.270	0.149										
M 18	3.292	6	2.760	83.8%	1.685	1.075	7	0.532	16.2%	0.325	0.207										
M 19	1.441	7	1.441	100.0%	0.794	0.647															
M 20	2.456	6	1.078	43.8%	0.635	0.441	7	1.380	56.2%	0.814	0.566										
M 21	1.734	7	1.734	100.0%	1.272	0.462															

BASIN ID #	TOTAL AREA	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS	GAGE NUMBER	AREA SQ. MILES	% OF TOTAL	SQ. MI IMPERV.	SQ. MI GRASS
M 22	0.197	7	0.197	100.0%	0.136	0.061															
M 23	2.728	8	0.954	35.0%	0.512	0.442	7	1.209	44.3%	0.649	0.560	9	0.552	20.2%	0.296	0.256	10	0.014	0.50%	0.007	0.006
M 24	0.170	7	0.170	100.0%	0.126	0.045															
M 25	2.242	4	1.070	47.7%	0.176	0.893	6	1.173	52.3%	0.193	0.979										
M 27	1.214	6	0.005	0.4%	0.003	0.002	7	1.006	82.9%	0.589	0.417	10	0.203	16.7%	0.119	0.084					
M 28	0.568	7	0.490	86.6%	0.368	0.122	10	0.078	13.4%	0.057	0.019										
M 29	1.322	7	0.352	26.7%	0.241	0.112	10	0.970	73.3%	0.663	0.307										
M 30	1.267	7	1.027	81.0%	0.612	0.415	10	0.241	19.0%	0.143	0.097										
M 31	0.528	10	0.528	100.0%	0.459	0.069															
M 32	1.597	10	1.597	100.0%	1.217	0.380															
M 33	0.613	10	0.613	100.0%	0.476	0.136															
M 34	5.030	10	5.098	101.4%	3.514	1.584															
M 35	1.581	10	1.581	100.0%	1.159	0.422															
M 36	3.742	6	0.002	0.1%	0.001	0.001	7	0.007	0.2%	0.004	0.002	9	1.290	34.5%	0.841	0.449	10	2.444	65.30%	1.593	0.850
M 37	1.867	10	1.867	100.0%	1.215	0.652															
M 38	0.419	10	0.419	100.0%	0.300	0.118															
M 39	0.911	10	0.911	100.0%	0.578	0.333															
M 40	22.269	10	1.398	6.3%	0.836	0.562	13	17.971	80.7%	10.748	7.222	14	2.900	13.0%	1.734	1.165					
M 41	7.952	9	1.261	15.9%	0.801	0.459	10	3.008	37.8%	1.912	1.097	12	1.093	13.7%	0.695	0.399	13	2.590	32.57%	1.646	0.944
M 42	2.131	9	0.741	34.8%	0.444	0.297	10	1.390	65.2%	0.833	0.557										
M 43	2.439	9	2.280	93.5%	1.402	0.877	10	0.160	6.5%	0.098	0.061										
M 44	0.528	10	0.528	100.0%	0.302	0.226															
M 45	1.527	9	1.527	100.0%	0.984	0.543															
M 46	2.589	9	1.451	56.1%	0.918	0.533	12	1.138	43.9%	0.720	0.418										
M 47	5.105	6	1.408	27.6%	0.895	0.513	9	3.697	72.4%	2.351	1.346										
M 48 DP	1.614	12	1.614	100.0%	1.094	0.521															
M 48 LM	13.606	9	0.967	7.3%	0.565	0.422	12	10.499	77.2%	6.009	4.490	13	2.121	15.6%	1.214	0.907					
M 49 A	2.192	9	0.970	44.3%	0.641	0.329	12	1.222	55.7%	0.807	0.415										
M 49 BDP	2.216	12	2.216	100.0%	1.343	0.873															
M 49 BLM	0.903	12	0.903	100.0%	0.549	0.354															
M 50	10.122	10	6.291	62.2%	3.843	2.448	13	1.748	17.2%	1.068	0.679	14	2.085	20.6%	1.274	0.811					
M 51	17.656	6	5.838	33.1%	3.452	2.384	9	11.789	66.8%	6.974	4.815	12	0.032	0.2%	0.019	0.013					
M 52	6.953	9	6.953	100.0%	4.508	2.445															
M 53	4.083	8	2.649	64.9%	1.398	1.251	9	0.045	1.1%	0.024	0.021	11	1.388	34.0%	0.733	0.656					
M 55 A	1.544	11	0.610	39.5%	0.402	0.207	12	0.934	60.5%	0.617	0.317										
M 55 B	3.236	11	1.291	39.9%	0.928	0.368	12	1.945	60.1%	1.384	0.551										
MN 1 A	0.472	3	0.172	36.4%	0.096	0.076	4	0.300	63.6%	0.168	0.132										
MN 1 B	2.213	3	0.311	14.1%	0.147	0.164	4	1.841	83.2%	0.870	0.971	6	0.061	2.8%	0.029	0.032					
MN 1 C	2.763	3	0.333	12.1%	0.194	0.139	4	1.164	42.1%	0.679	0.485	6	1.265	45.8%	0.739	0.527					
MN 1 D	0.850	4	0.334	39.3%	0.192	0.142	6	0.516	60.7%	0.296	0.219										
MN 1 E	1.000	6	1.000	100.0%	0.587	0.413															
MN 2 A	0.414	4	0.414	100.0%	0.166	0.248															
MN 2 B	1.045	4	1.045	100.0%	0.542	0.503															
MN 2 C	0.725	4	0.694	95.8%	0.298	0.397	6	0.031	4.2%	0.013	0.018										
MN 3	2.056	4	2.056	100.0%	1.244	0.812															
MN 4	2.161	3	0.688	31.8%	0.363	0.325	4	1.473	68.2%	0.777	0.696										
MN 5	1.481	4	1.481	100.0%	0.896	0.585															

APPENDIX E

COMPARISON, ANALYSIS, AND ESTIMATION OF DISCHARGE DATA FROM TWO ACOUSTIC VELOCITY METERS ON THE CHICAGO SANITARY AND SHIP CANAL AT ROMEOVILLE, ILLINOIS

COMPARISON, ANALYSIS, AND ESTIMATION OF DISCHARGE DATA FROM TWO ACOUSTIC VELOCITY METERS ON THE CHICAGO SANITARY AND SHIP CANAL AT ROMEOVILLE, ILLINOIS

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 93-4048

Prepared in cooperation with the
U.S. ARMY CORPS OF ENGINEERS,
CHICAGO DISTRICT



COMPARISON, ANALYSIS, AND ESTIMATION OF DISCHARGE DATA FROM TWO ACOUSTIC VELOCITY METERS ON THE CHICAGO SANITARY AND SHIP CANAL AT ROMEOVILLE, ILLINOIS

by Charles S. Melching and Kevin A. Oberg

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 93-4048

Prepared in cooperation with the
U.S. ARMY CORPS OF ENGINEERS,
CHICAGO DISTRICT



Urbana, Illinois

1993

U.S. DEPARTMENT OF THE INTERIOR
BRUCE BABBITT, Secretary

U.S. GEOLOGICAL SURVEY
Dallas L. Peck, Director

For additional information
write to:

District Chief
U.S. Geological Survey
102 E. Main St., 4th Floor
Urbana, IL 61801

Copies of the report can be
purchased from:

U.S. Geological Survey
Earth Science Information Center
Open-File Reports Section
Box 25286, MS 517
Denver Federal Center
Denver, CO 80225

CONTENTS

	Page
Abstract	1
Introduction	2
Purpose and scope	5
Approach	5
Description of site	6
Acknowledgments	6
Comparison and analysis of discharge data	7
Discharge estimates made by the Sarasota and ORE acoustic velocity meters	8
Regression relations between discharge estimates at Romeoville and Lockport	14
Vertical velocity distributions	18
Correction of discharge for channel-width and depth errors	20
Regression equations for estimation of missing data	26
Data used to develop equations	26
Form of the estimation equations	31
Verification of estimation equations	35
Estimation of missing record	39
Future considerations	39
Summary and conclusions	39
References cited	41
Appendix 1: History of inoperative periods for the acoustic velocity meter at Romeoville, water years 1984-91	44
Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative	46

ILLUSTRATIONS

Figure 1.	Map showing the Chicago Sanitary and Ship Canal and its tributary area and the locations of the acoustic velocity meter at Romeoville, Ill., and the Lockport lock, dam, and controlling works	3
2.	Diagram showing the locations of the transducers and gage house for the acoustic-velocity-meter installation at Romeoville, Ill.	4
3-7.	Graphs showing:	
3.	Discharge-duration curves for unadjusted daily mean discharges measured by Sarasota and ORE acoustic velocity meters for all discharge regimes, Chicago Sanitary and Ship Canal at Romeoville, Ill.	10

ILLUSTRATIONS

Page

Figures

3-7. Graphs showing:--Continued

4. Discharge-duration curves for unadjusted daily mean discharges measured by Sarasota and ORE acoustic velocity meters for the turbine, lockage, and leakage discharge regime, Chicago Sanitary and Ship Canal at Romeoville, Ill. 11

5. Discharge-duration curves for unadjusted daily mean discharges measured by Sarasota and ORE acoustic velocity meters for the turbine, lockage, leakage, and sluice-gate discharge regime, Chicago Sanitary and Ship Canal at Romeoville, Ill. 12

6. Nondimensional vertical velocity distribution and 1 standard-deviation confidence limits (bounds) based on measurements by the Sarasota and ORE acoustic velocity meters 19

7. Nondimensional vertical velocity distribution of the main paths measured by the Sarasota and ORE acoustic velocity meters and the best-fit power-equation representation of the data 21

8. Cross sections measured on June 4 and 5, 1991, at the upstream transducers, midway between the transducers, and the downstream transducers for the acoustic velocity meter on the Chicago Sanitary and Ship Canal at Romeoville, Ill. 23

9-13. Graphs showing:

9. Sluice-gate discharge estimates by Metropolitan Water Reclamation District of Greater Chicago and corrected sluice-gate discharge estimated from the acoustic-velocity-meter measurements . . . 29

10. Sluice-gate plus controlling-works discharge estimates by Metropolitan Water Reclamation District of Greater Chicago and corrected sluice-gate plus controlling-works discharge estimated from the acoustic-velocity-meter measurements . . . 30

11. Sluice-gate plus controlling-works discharge estimates by Metropolitan Water Reclamation District of Greater Chicago and corrected sluice-gate plus controlling-works discharge estimated from the acoustic-velocity-meter measurements (includes the 6 days of estimated sluice-gate discharge greater than 5,000 cubic feet per second with no controlling-works discharge) 33

12. Daily mean discharge estimated by the best-fit multiple linear-regression equations and the measured daily mean discharge for October 1, 1991, through May 31, 1992 36

13. Daily mean discharge estimated by the regression equations derived from corrected discharges and the measured daily mean discharge for October 1, 1991, through May 31, 1992 37

TABLES

Page

Table 1. Statistical comparison of 10 data sets for the Sarasota acoustic velocity meter, June 12, 1984, through November 2, 1988, and the ORE acoustic velocity meter, November 18, 1988, through September 30, 1990 9

2. Regression coefficients and standard errors of the multiple linear regression of the acoustic-velocity-meter discharges at Romeoville, Ill., approximated on the basis of discharge estimates made by the Metropolitan Water Reclamation District of Greater Chicago for Lockport, Ill., for the various discharge regimes (combinations of outlet works); separate equations are provided for the periods of operation for the Sarasota and ORE acoustic velocity meters 15

3. Closeness of fit of the regression equation for the various discharge regimes for the Sarasota acoustic velocity meter, June 12, 1984, through November 2, 1988, and the ORE acoustic velocity meter, November 18, 1988, through September 30, 1990 16

4. Calculated bottom-elevation data for the Chicago Sanitary and Ship Canal elevations measured at Romeoville, Ill., June 4 and 5, 1991 22

5. Regression coefficients and standard errors of the multiple linear regression of the adjusted acoustic-velocity-meter discharges at Romeoville, Ill., approximated on the basis of discharge estimates made by the Metropolitan Water Reclamation District of Greater Chicago for Lockport, Ill., for the various discharge regimes (combinations of outlet works); separate equations are provided for the periods of operation for the Sarasota and ORE acoustic velocity meters 27

6. Regression coefficients and standard errors of the multiple linear regression of the adjusted acoustic-velocity-meter discharges at Romeoville, Ill., approximated on the basis of discharge estimates made by the Metropolitan Water Reclamation District of Greater Chicago for Lockport, Ill., for the various discharge regimes (combinations of outlet works) 32

7. Discharge statistics for October 1, 1991, through May 31, 1992, for the measured discharges at Romeoville, Ill., and the estimated discharges using three sets of regression equations and equations derived from corrected discharges 38

CONVERSION FACTORS

Multiply	By	To obtain
inch (in.)	25.4	millimeter
foot (ft)	0.3048	meter
mile (mi)	1.609	kilometer
square foot (ft^2)	0.09294	square meter
square mile (mi^2)	2.590	square kilometer
cubic foot per second (ft^3/s)	0.02832	cubic meter per second

COMPARISON, ANALYSIS, AND ESTIMATION OF DISCHARGE DATA FROM TWO ACOUSTIC VELOCITY METERS ON THE CHICAGO SANITARY AND SHIP CANAL AT ROMEOVILLE, ILLINOIS

by Charles S. Melching and Kevin A. Oberg

ABSTRACT

Acoustic velocity meters (AVM's) were installed on the Chicago Sanitary and Ship Canal at Romeoville, Ill., to aid in the accounting of water diverted from Lake Michigan to the Illinois River. This report describes the analyses performed to establish the most accurate estimates of discharge possible at this time on the Canal at Romeoville for water years 1986 through 1991 (October 1985-September 1991). The first AVM at Romeoville was operational on June 12, 1984. On November 3, 1988, the AVM was shut down because of numerous maintenance problems; on November 17, 1988, it was replaced by an AVM made by another manufacturer. The AVM's were and are occasionally rendered inoperative because of power surges, damage by barges, and other causes. During these periods of AVM inoperation, discharge estimates made by the Metropolitan Water Reclamation District of Greater Chicago (MWRD) at the Lockport powerhouse, lock, and controlling works are used to approximate the flow at Romeoville through a series of estimation equations that are updated and improved as described in this report.

Naturally caused differences in discharges between the period of operation of the first and second AVM's are far greater than any differences that may be attributed to the performance of the two AVM's. The regression equations relating AVM discharge estimates at Romeoville with MWRD discharge estimates at Lockport for the period of operation were comparable. The accuracy of the simulated data set is nearly the same as the best-fit regression equations. The vertical velocity-distribution measurements made by the two AVM's are consistent and are within the range of values expected from open-channel flow theory. Therefore, no difference in the discharge computed using the two AVM's can be supported, and no correction factor is needed to adjust one AVM to be consistent with the other.

Two different forms of equations for estimating discharge at Romeoville on the basis of MWRD discharge estimates at Lockport were compared. The first form was from standard multiple-linear regression between AVM discharge estimates at Romeoville and MWRD estimates of discharge at Lockport through the various outlet components--turbines, lockage, and leakage; powerhouse sluice gates; and controlling works. In the second form, the regression relation between MWRD estimates of turbine, lockage, and leakage (TLL) discharge at Lockport and AVM discharge estimates at Romeoville derived for days when the sluice gate and (or) controlling works were not in operation was used to define an error relation for the TLL discharge regime. This error relation was used to determine corrected

discharges through the sluice gates and (or) controlling works. Regression equations were derived relating the corrected discharges and the MWRD estimates of discharge through these outlet components. The equations derived from corrected discharges performed as well as the first form of regression equations in terms of closeness of fit for the calibration period and estimation quality for the verification period. The equations derived from corrected discharges estimated the mean, standard deviation, and skewness coefficient of the daily mean discharges at Romeoville for the verification period within 0.22, 5.15, and 0.66 percent, respectively. The equations derived from the corrected discharges were used to estimate discharge at Romeoville for days when the AVM was not operational because of their strong physical basis and excellent verification results.

The AVM discharge estimates were adjusted for the width and depth errors that were detected during a 1991 canal survey, and the discharge was estimated using the equations derived from corrected discharges for the 545 days on which the AVM was not operational. The final best estimates of discharge have been entered into the discharge record for the station.

INTRODUCTION

The modified U.S. Supreme Court Decree of December 1, 1980, limits the diversion of water from Lake Michigan by the State of Illinois and the city of Chicago to a 40-year average of 3,200 ft^3/s with the cumulative algebraic sum of the average annual diversions minus 3,200 ft^3/s during the first 39 years to be no more than 2,000 ft^3/s-years. The U.S. Army Corps of Engineers, Chicago District (Corps), has been charged with the diversion accounting. As part of the accounting procedure, the Supreme Court ordered the Corps to convene a three-member Technical Committee at least every 5 years to review the accounting procedure and to ensure that the accounting procedure is "state of the art."

The acoustic velocity meter (AVM) on the Chicago Sanitary and Ship Canal at Romeoville, Ill. (fig. 1), is a key part of the Lake Michigan diversion-accounting procedure. The operation and maintenance of the AVM is the responsibility of the U.S. Geological Survey (USGS) by agreement with the Corps. The AVM operates on the principle that point-to-point traveltime of an acoustic signal is greater when the signal is traveling upstream than when traveling downstream. The difference in traveltime is due to the motion of the water relative to the transducers that receive the acoustic signals. Thus, the AVM can be used to determine the average velocity across the width of the canal for a given elevation (path). The instantaneous average velocity and discharge can be computed by placing transducers at several elevations to represent the variation of velocity with depth. The acoustic signals from the AVM installation at Romeoville are directed in three paths downstream and in one cross path upstream (fig. 2).

The first AVM at Romeoville, manufactured by Sarasota Automation,[1] was installed during the week of March 18-23, 1984, and became operational on June 12, 1984. On November 3, 1988, the Sarasota AVM was shut down because of

[1]Use of firm and trade names in this report is for identification purposes only and does not constitute endorsement by the U.S. Geological Survey.

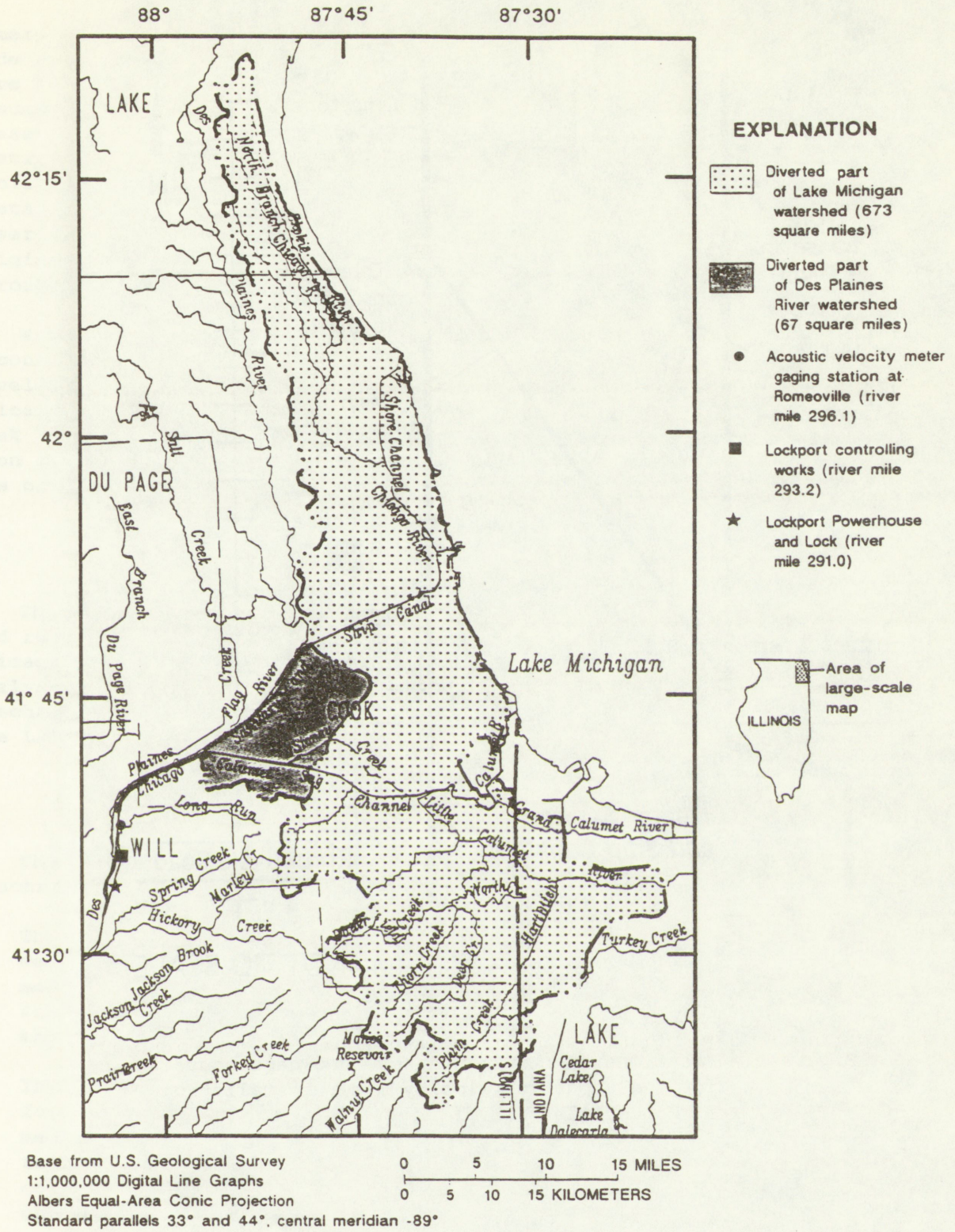


Figure 1.--Chicago Sanitary and Ship Canal and its tributary area and the locations of the acoustic velocity meter at Romeoville, Ill., and the Lockport lock, dam, and controlling works.

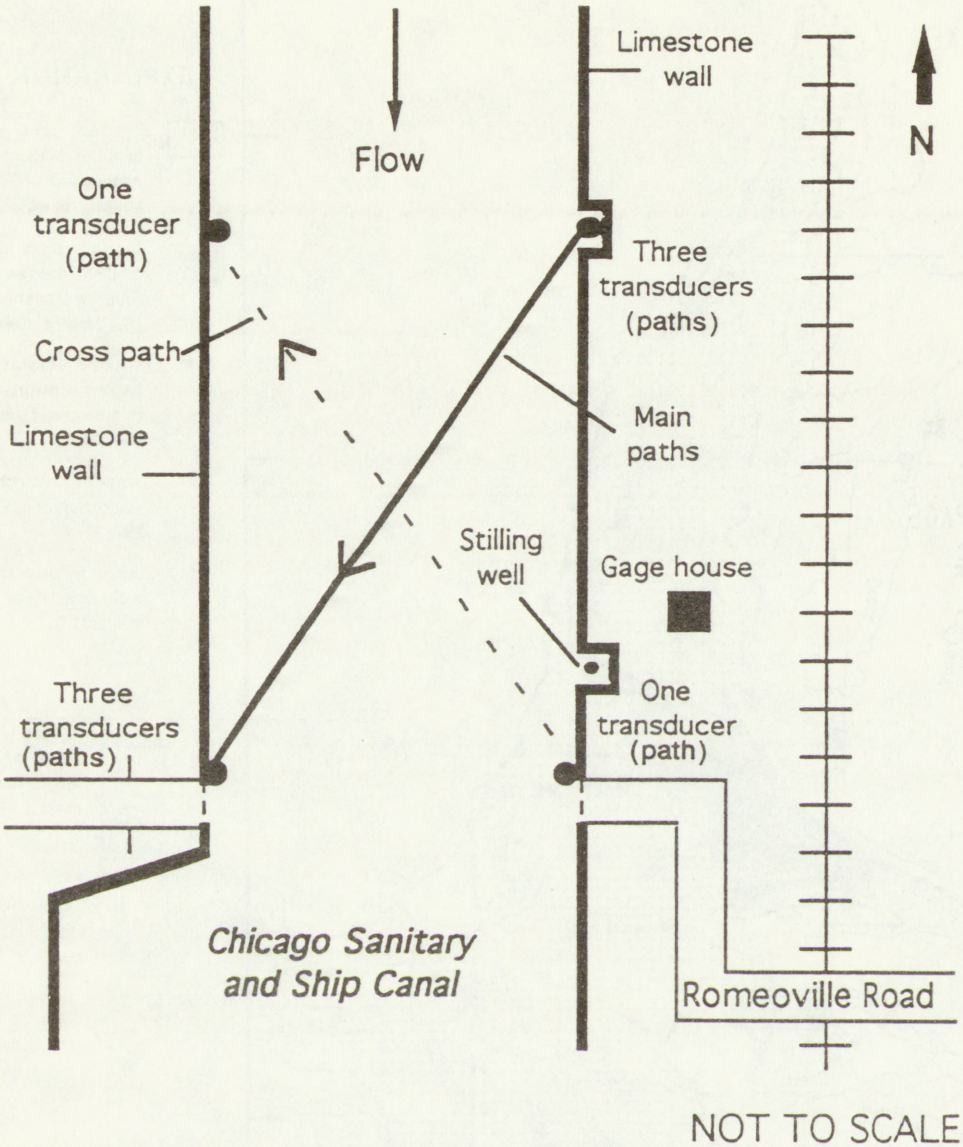


Figure 2.--Locations of the transducers and gage house for the acoustic-velocity-meter installation at Romeoville, Ill.

numerous maintenance problems; on November 17, 1988, it was replaced by an AVM made by ORE, Inc. The installed AVM was and is occasionally rendered inoperative by power surges, damage by barges, and other causes. (See appendix 1 for a summary of periods when the AVM's were inoperative and the causes.) During these periods, discharge estimates made by the Metropolitan Water Reclamation District of Greater Chicago (MWRD) at the Lockport powerhouse, lock, and controlling works, 5.1, 5.1, and 2.9 mi, respectively, downstream from the AVM installation (fig. 1), are used to approximate the flow at Romeoville through a series of regression equations (U.S. Army Corps of Engineers, 1989). The original regression equations were derived from flow data from June 12, 1984, through September 30, 1987, and required updating.

With the convening of the Technical Committee for review of the diversion accounting for water years 1986-90[2], the Corps and USGS have cooperatively developed the most accurate daily mean discharges possible at this time for the Chicago Sanitary and Ship Canal at Romeoville for water years 1986-91. This task included development of a new set of regression equations for the estimation of discharge for periods when the AVM was inoperative. The replacement of the original Sarasota AVM with the ORE AVM in 1988 complicated the task.

Purpose and Scope

This report describes the methods and results of a study to compare, adjust, and revise the daily mean discharge data estimated by the two AVM's on the Chicago Sanitary and Ship Canal at Romeoville during water years 1986-91. The analyses described in this report presently provide the most accurate daily mean discharges possible for the Chicago Sanitary and Ship Canal at Romeoville for the Lake Michigan diversion accounting for water years 1986-91.

Approach

The procedures listed below were followed in the analysis of the AVM discharge data:

1. The discharges computed by the Sarasota AVM and with the use of the ORE AVM were compared by statistical methods and vertical velocity distributions measured by each AVM were compared. Such a comparative analysis ensured that the performance of each AVM was consistent and that the replacement of the Sarasota AVM with the ORE AVM did not introduce a significant bias.

2. The USGS computation programs were used to adjust the discharge estimates for the ORE AVM period of operation using the ORE AVM velocity and stage measurements and the corrected width and depth. The discharge estimates for the Sarasota AVM period of operation were adjusted on the basis of a correction equation applied to the cross-sectional average velocity. A channel cross-section survey was made on June 4 and 5, 1991. The survey

[2]The water year is the 12-month period from October 1 through September 30 and is designated by the calendar year in which it ends and which includes 9 of the 12 months.

revealed that the cross section between the AVM transducers that measure velocity is 1.55 ft deeper than had been determined from engineering plans and 2 ft narrower than measured from the Romeoville Road bridge (fig. 2).

3. Equations were developed and verified for estimation of discharges computed by the AVM at Romeoville on the basis of discharges estimated by MWRD at Lockport. These equations followed a format similar to the regression equations by discharge regime developed by the Corps (U.S. Army Corps of Engineers, 1989).

4. The estimation equations were applied to estimate values of daily mean discharge for all periods when the AVM was not operational.

Description of Site

The Chicago Sanitary and Ship Canal, hereafter referred to as "the Canal", was constructed from 1892 to 1900 to reverse the flow of the Chicago River and to carry wastewater from Chicago away from water-supply inlets in Lake Michigan and into the Illinois River Basin. The Canal is 28 mi long, and the last 15 mi are cut into bedrock. In 1910, the 8-mi-long North Shore Channel was constructed and connected to the Canal to carry wastewater from north shore communities and additional water from Lake Michigan for dilution of wastewater. The 18-mi-long Calumet Sag Channel was constructed during 1910-20 to connect the Calumet River to the Canal and to reverse the flow of the Calumet River and carry wastewater from areas south of Chicago to the Illinois River Basin. These channels and the area drained by the Canal are shown in figure 1. The area drained by the Canal at the Romeoville AVM site is 739 mi^2.

The AVM at Romeoville is at river mile 296.1, 5.1 mi upstream from the end of the Canal at the Lockport Lock and Dam (fig. 1). The AVM is in the part of the Canal carved into bedrock. Initially, the transducers were mounted directly into small slots chiseled into the Canal wall. However, the transducers were not adequately protected from damage by barges (see appendix 1). Divers were then required to service the damaged transducers. Therefore, on November 2, 1990, the three main transducers were relocated into large recesses formed in the Canal wall during Canal construction. The transducers were installed into a 6-in. inside-diameter PVC pipe that allowed the transducers to be serviced without the use of divers. Current locations of the transducers and gage house at Romeoville are shown in figure 2. Analysis of the velocity measurements and discharge calculations in water year 1991 showed that the transducer relocation did not introduce a bias in the discharge record.

Acknowledgments

The authors wish to acknowledge the Metropolitan Water Reclamation District of Greater Chicago for assisting in this project by providing discharge estimates for the outlet works at Lockport, for providing a tour of the facilities at Lockport, and for explaining the current methods of estimating discharge through the various outlet works.

COMPARISON AND ANALYSIS OF DISCHARGE DATA

Because of the frequent failures of various components of the Sarasota AVM (see appendix 1) and difficulties in getting it serviced, it was replaced by the ORE AVM. The two systems differ, however, in the way discharge is calculated, the depths of the transducers, and in the internal operation of the meters. These differences raise questions regarding the consistency of the values of daily mean discharge obtained from the two AVM's. The two AVM's were never operated simultaneously. Such operation would be problematic because of interference between acoustic signals from transducers of two AVM's at nearly the same depths. Thus, the consistency of discharge estimates made by the two AVM's can be examined in a generalized statistical manner only.

The discharge calculations that are most directly compatible were used to perform the comparison. The originally calculated ORE AVM discharges for water years 1989 and 1990 did not include adjustments for the width and depth errors detected in 1991; thus, these discharges are considered directly comparable to the Sarasota AVM discharges. In addition, the method used to compute discharge during these years with the ORE AVM was similar to that used by the Sarasota AVM. All discharges were adjusted to correct for the errors in width and depth; however, the adjustment procedure was different for each AVM (as discussed later). Therefore, comparing Sarasota AVM discharges to the ORE AVM discharges for water years 1989 and 1990 is the least biased comparison. The comparison was done in three ways:

1. The actual discharges for the various discharge regimes were compared between the periods of operation for the Sarasota AVM (June 12, 1984, through November 2, 1988) and the ORE AVM (November 18, 1988, through September 30, 1990).

2. The regression equations with MWRD discharge estimates for the various discharge regimes (combinations of outlet works) were compared. In addition, (a) the Sarasota AVM record was estimated with the ORE AVM regressions and (b) the ORE AVM record was estimated with the Sarasota AVM regressions. The residuals, defined as the difference between the AVM discharge and the discharges estimated by the regression relations, for cases (a) and (b) also were compared.

3. The vertical velocity distributions measured by the two AVM's were compared.

If no significant difference between the discharge estimates derived from each AVM can be demonstrated, then the adjustment of the Sarasota AVM and the ORE AVM discharge estimates to account for the width and depth errors can be discussed. The adjustment for the ORE AVM is straightforward, but the adjustment for the Sarasota AVM is complicated because this AVM internally calculated the average 15-minute and daily discharges.

Discharge Estimates Made by the Sarasota and ORE Acoustic Velocity Meters

Table 1 lists the number of days, mean value, variance, standard deviation, and skewness coefficient for each data series; and Student's t for the t-test and the F-statistic for each pair (Sarasota AVM and ORE AVM) of data series. These statistics are given for the following data sets:

1. AVM discharge for all discharge regimes;
2. MWRD discharge for all discharge regimes;
3. AVM discharge minus MWRD discharge for all discharge regimes;
4. AVM discharge for days of turbine, lockage, and leakage (TLL) discharge only;
5. MWRD discharge for days of TLL discharge only;
6. AVM discharge minus MWRD discharge for days of TLL discharge only;
7. AVM discharge for days of turbine, lockage, leakage, and sluice gate (TLL+SG) discharge;
8. MWRD discharge for days of TLL+SG discharge;
9. AVM discharge minus MWRD discharge for days of TLL+SG discharge; and
10. Discharge for Des Plaines River at Riverside.

The discharge-duration curves for discharges measured by the Sarasota AVM and ORE AVM for all discharge regimes, the TLL discharge, and the TLL+SG discharge, are shown in figures 3-5, respectively.

In the skewness test of normality (Salas and others, 1980, p. 93) for data sets of various sizes, if the absolute value of the skewness coefficient is greater than the values given below, the hypothesis that the data series is normally distributed can be rejected at the 2-percent level of significance.

Number of observations	Skewness-coefficient bound
100	0.567
150	.464
500	.255
600	.233
800	.202
1,000	.180

Table 1.--Statistical comparison of 10 data sets for the Sarasota acoustic velocity meter, June 12, 1984, through November 2, 1988, and the ORE acoustic velocity meter, November 18, 1988, through September 30, 1990

[AVM, acoustic velocity meter; MWRD, Metropolitan Water Reclamation District of Greater Chicago; stat., statistic; ft^3/s, cubic feet per second; ft^6/s^2, cubic feet per second squared]

AVM	Number of days operational	Mean (ft^3/s)	Variance (ft^6/s^2)	Standard deviation (ft^3/s)	Skewness coefficient	Z-stat., t-test	F-stat., F-test
Data set 1, AVM discharge (all discharge regimes)							
Sarasota	952	3,696	2,334,168	1,528	3.332	3.59	1.064
ORE	616	3,415	2,193,893	1,481	2.445		
Data set 2, MWRD discharge (all discharge regimes)							
Sarasota	952	3,492	5,682,810	2,384	7.071	1.20	1.184
ORE	616	3,338	6,726,640	2,594	5.010		
Data set 3, AVM discharge minus MWRD discharge (all discharge regimes)							
Sarasota	952	204	1,262,207	1,123	-10.547	2.02	1.440
ORE	616	77	1,818,197	1,348	-6.640		
Data set 4, AVM discharge (turbine, lockage, and leakage)							
Sarasota	792	3,317	729,753	854	.548	5.81	1.226
ORE	497	3,021	894,626	946	1.005		
Data set 5, MWRD discharge (turbine, lockage, and leakage)							
Sarasota	792	2,981	590,238	768	.404	6.95	1.217
ORE	497	2,663	718,166	847	.945		
Data set 6, AVM discharge minus MWRD discharge (turbine, lockage, and leakage)							
Sarasota	792	336	28,554	169	4.537	2.35	1.272
ORE	497	358	22,453	150	.463		
Data set 7, AVM discharge (turbine, lockage, leakage, and sluice gates)							
Sarasota	138	4,930	1,544,442	1,243	.752	3.20	1.231
ORE	100	4,428	1,255,039	1,120	1.289		
Data set 8, MWRD discharge (turbine, lockage, leakage, and sluice gates)							
Sarasota	138	4,715	1,861,043	1,364	1.117	.40	1.947
ORE	100	4,630	3,623,578	1,904	2.786		
Data set 9, AVM discharge minus MWRD discharge (turbine, lockage, leakage, and sluice gates)							
Sarasota	138	215	166,820	408	-2.798	4.40	6.037
ORE	100	-203	1,007,069	1,004	-3.769		
Data set 10, Discharge for Des Plaines River at Riverside							
Sarasota	1,044	713	859,813	927	4.312	2.34	1.950
ORE	682	617	440,991	664	3.408		

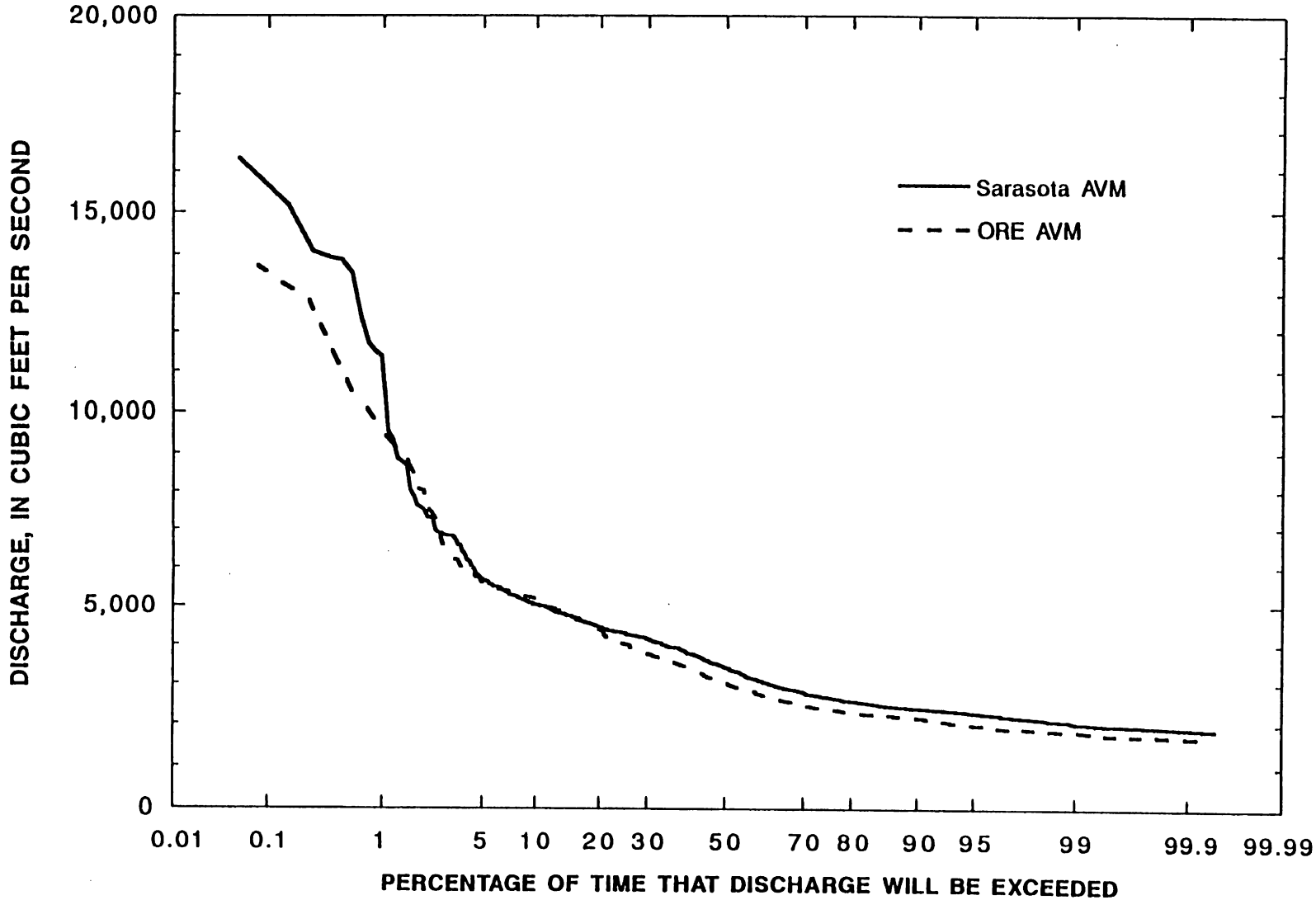


Figure 3.--Discharge-duration curves for unadjusted daily mean discharges measured by Sarasota and ORE acoustic velocity meters for all discharge regimes, Chicago Sanitary and Ship Canal at Romeoville, Ill.

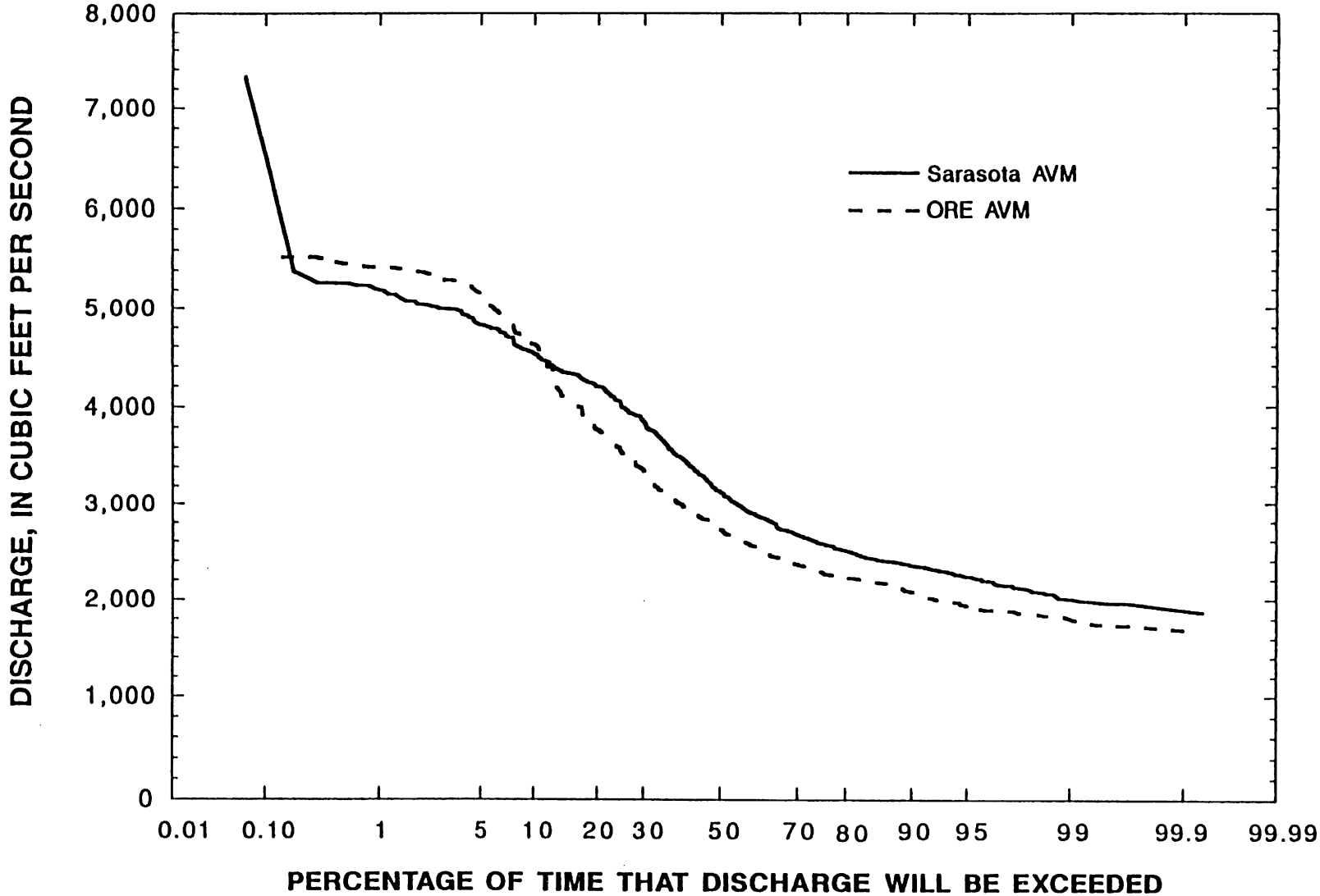


Figure 4.--Discharge-duration curves for unadjusted daily mean discharges measured by Sarasota and ORE acoustic velocity meters for the turbine, lockage, and leakage discharge regime, Chicago Sanitary and Ship Canal at Romeoville, Ill.

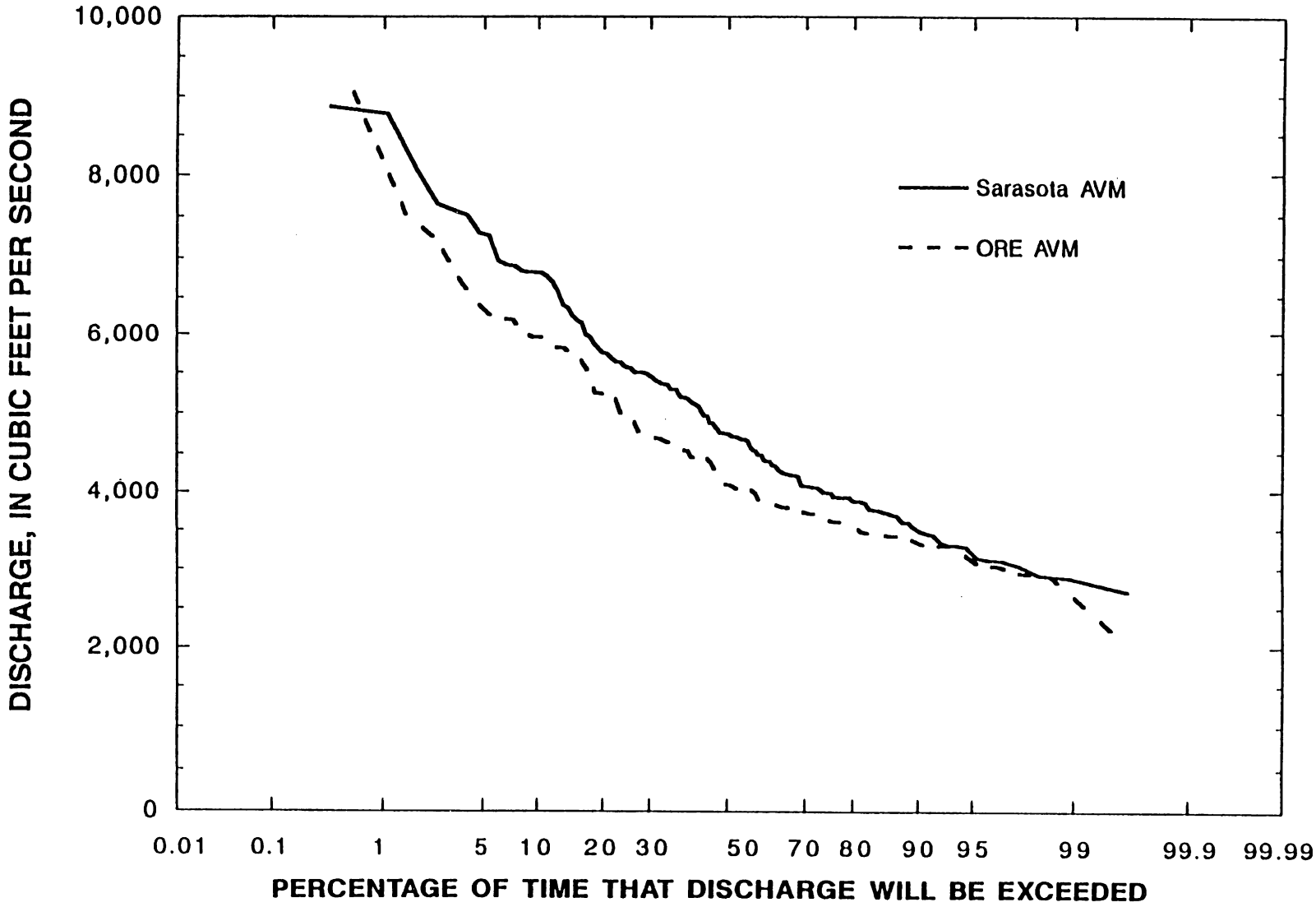


Figure 5.--Discharge-duration curves for unadjusted daily mean discharges measured by Sarasota and ORE acoustic velocity meters for the turbine, lockage, leakage, and sluice-gate discharge regime, Chicago Sanitary and Ship Canal at Romeoville, Ill.

Comparison with the skewness coefficient values in table 1 indicates these data are not normally distributed. Computation of skewness coefficients for the logarithms of the flows failed to reduce the skewness to nonrejection levels for most of the data series. Further, serial correlation between the daily discharges measured by each AVM is high. Thus, the t-test and F-test are not strictly valid for these data series. Nevertheless, the test statistics can still be useful in assessing the magnitude of the differences between the Sarasota AVM and the ORE AVM data-collection periods.

In comparing data series, if both series contained 121 days of discharge (that is, had 120 degrees of freedom), the hypothesis that the variances of the two series are the same could be rejected at the 1-percent significance level if the F-statistic value is greater than 1.53. This threshold becomes smaller as the data sets become larger. Thus, data sets 8, 9, and 10 have significantly different variances. Further, if a data set fails the F-test, the results of the standard t-test used here are invalid because the standard t-test is based on the assumption that the two series have the same variance. It should be noted that an adjusted t-test is available for the case of nonconstant variance; however, because the main question is whether the data are drawn from the same population, such a test is not warranted in this analysis.

If Student's t of the standard t-test is greater than 2.358 or 2.326 for cases of 120 and infinite degrees of freedom, respectively, the hypothesis that the means of the two series are the same can be rejected at the 1-percent significance level. The number of degrees of freedom is equal to the sum of the number of days of discharge in the series being compared minus two. Thus, data sets 1, 4, 5, 6, 7, 9, and 10 have significantly different means.

The only data sets that do not seem to have significantly different statistics for the ORE AVM period relative to the Sarasota AVM period are those for the MWRD discharge estimates and the difference between AVM and MWRD discharge estimates for all discharge regimes. This finding could be considered evidence of an inconsistency in the discharges between the two periods on the part of the MWRD discharge estimates because the two periods were not significantly different, whereas all other discharge data indicate changes between periods; however, because the individual subsets of the discharge data reflecting the different components of the discharge released at Lockport indicate significant differences between the two periods, the overall lack of a significant difference appears to be an artifact of the data. That is, the difference between the AVM and MWRD discharge estimates increases slightly for the ORE AVM period of operation compared to the Sarasota AVM period of operation for days of TLL discharge only, whereas this difference greatly decreases for days when the sluice gates were in operation. These significant increases and decreases cancel one another if the overall discharge data series is considered.

Preliminary analyses by the Corps indicated that discharges computed by the ORE AVM generally were less than those computed by the Sarasota AVM. This conclusion raised the question of whether the Sarasota AVM discharge computations were biased to be higher than those for the ORE AVM. As shown in table 1, the mean value in all the data series in the ORE AVM period of operation are lower than those for the Sarasota AVM period of operation. Thus, the period of ORE AVM operation seems to have been somewhat drier on average than the period of Sarasota AVM operation. Discharges calculated at a nearby streamflow-gaging

station, the Des Plaines River at Riverside, show the same trend. The average difference between AVM and MWRD estimates for TLL flow is 127 ft^3/s less in the ORE AVM period than in the Sarasota AVM period, but whether this difference is because of differences between AVM's or entirely because of natural variation cannot be determined.

From table 1, it is clear that distinct differences in flow between the two periods are due to natural variation despite the high degree of regulation of the Canal. This difference is confirmed by the comparisons made by use of discharge data for the Des Plaines River at Riverside, a relatively unregulated river. The natural variations between periods are much greater in magnitude than any difference between discharges computed by the two meters. It seems highly unlikely that statistical methods could be used to prove a difference in data series because of differences in the meters alone, given the problems with serial correlation of daily flows and non-normality of the data.

Regression Relations Between Discharge Estimates at Romeoville and Lockport

As per the recommendations of the Second Technical Committee (Espey and others, 1987) and the procedures followed by the U.S. Army Corps of Engineers (1989), regression relations were derived between AVM values and MWRD values of daily mean discharge for three different discharge regimes. These regression equations follow.

1. TLL discharge only:

$$Q_{AVM} = a_0 + a_1 \times Q_{TLL}, \quad (1)$$

where Q_{AVM} is the AVM discharge,
Q_{TLL} is the MWRD estimate of TLL discharge,
a_0 is the intercept of linear regression line, and
a_1 is the regression coefficient relating Q_{AVM} and Q_{TLL}.

2. TLL+SG discharge:

$$Q_{AVM} = a_0 + a_1 \times Q_{TLL} + a_2 \times Q_{SG}, \quad (2)$$

where Q_{SG} is the MWRD estimate of sluice-gate (SG) discharge, and
a_2 is the regression coefficient relating Q_{AVM} and Q_{SG}.

3. Turbine, lockage, leakage, sluice-gate, and controlling-works (TLL+SG+CW) discharge:

$$Q_{AVM} = a_0 + a_1 \times Q_{TLL} + a_2 \times Q_{SG} + a_3 \times Q_{CW}, \quad (3)$$

where Q_{CW} is the MWRD estimate of the controlling-works (CW) discharge, and
a_3 is the regression coefficient relating Q_{AVM} and Q_{CW}.

The values of a_0, a_1, and a_2 are different for each of equations 1-3.

To test the consistency of the Sarasota AVM and the ORE AVM discharges, equations 1-3 were fit to data for the Sarasota AVM and the ORE AVM from water years 1989 and 1990. The STATIT statistical package (Statware, Inc., 1990) was used for the regression analysis. Regression coefficients a_0-a_3 and their standard errors for each of the discharge regimes and time periods (Sarasota AVM and ORE AVM) are listed in table 2. The closeness of fit between the regression equation and the data is measured by the coefficient of determination (R^2), standard error of regression (S_e), and standard error as a fraction of mean discharge (Q_M) for the given discharge regime (S_e/Q_M), which are listed in table 3.

Table 2.--Regression coefficients and standard errors of the multiple linear regression of acoustic-velocity-meter discharges at Romeoville, Ill., approximated on the basis of discharge estimates made by the Metropolitan Water Reclamation District of Greater Chicago for Lockport, Ill., for the various discharge regimes (combinations of outlet works); separate equations are provided for the periods of operation for the Sarasota and ORE acoustic velocity meters

[AVM, acoustic velocity meter; a_0, intercept of the multiple linear regression; a_1, regression coefficient between the AVM discharge and the Metropolitan Water Reclamation District of Greater Chicago (MWRD) estimate of turbine, lockage, and leakage discharge for the given data set; a_2, regression coefficient between the AVM discharge and the MWRD estimate of sluice-gate discharge for the given data set; a_3, regression coefficient between the AVM discharge and the MWRD estimate of controlling-works discharge; all coefficients are used to estimate daily mean discharge in cubic feet per second; --, no value calculated]

Discharge regime	Regression coefficient	Sarasota AVM[1] Coefficient value	Sarasota AVM[1] Standard error	ORE AVM[2] Coefficient value	ORE AVM[2] Standard error
Turbine, lockage, and leakage	a_0	55.88	21.78	72.33	17.66
	a_1	1.094	.0071	1.107	.0063
Sluice gates	a_0	100.5	96.16	804.1	101.3
	a_1	1.136	.0262	.976	.0319
	a_2	.736	.0190	.411	.0158
Controlling works	a_0	1,332	848.6	4,127	354.2
	a_1	.9943	.2318	--	--
	a_2	.6647	.0340	.3384	.0373
	a_3	--	--	.5121	.0703

[1]Period of operation considered, June 12, 1984, through November 2, 1988.
[2]Period of operation considered, November 18, 1988, through September 30, 1990.

For the equations involving CW discharges, blank values for a coefficient indicate that the value of the regression coefficient was not significantly different from zero. In this analysis the multiple linear regression is repeated, but the weakly correlated independent variable is omitted to produce a closer fit. For the Sarasota AVM period, an a_3 value of -0.0425 with a standard error of 0.159 was calculated for the multiple regression including a_3. For the Sarasota AVM period, the CW discharges generally were much less than the SG discharges for 20 of 22 days when the CW was in operation. For these 20 days, the CW discharges were, on average, equal to 36 percent of the SG discharges, with 5 days less than 10 percent. On days when discharges were

Table 3.--Closeness of fit of the regression equation for the various discharge regimes for the Sarasota acoustic velocity meter, June 12, 1984, through November 2, 1988, and the ORE acoustic velocity meter, November 18, 1988, through September 30, 1990

[AVM, acoustic velocity meter; R^2, coefficient of determination for the regression equation; S_e, standard error of the regression equation; Q_M, mean discharge for the given discharge regime; ft^3/s, cubic feet per second]

AVM	Discharge regime	R^2	S_e (ft^3/s)	S_e/Q_M (percent)
Sarasota	Turbine, lockage, and leakage	0.968	153.0	4.61
ORE	Turbine, lockage, and leakage	.984	119.4	3.95
Sarasota	Turbine, lockage, leakage, and sluice gates	.959	251.7	5.11
ORE	Turbine, lockage, leakage, and sluice gates	.947	258.3	5.83
Sarasota	Turbine, lockage, leakage, sluice gates, and controlling works	.947	928.7	9.69
ORE	Turbine, lockage, leakage, sluice gates, and controlling works	.916	768.8	9.14

lower, the CW discharges were negligible; on the days when discharges were high, the CW discharges were strongly correlated to the SG discharges. The combination of the two were much greater than the AVM discharges. Thus, the CW contributed little to the estimation of AVM discharges. Similar results were found by the Corps (U.S. Army Corps of Engineers, 1989). For the ORE AVM period, an a_1 value of 0.222 with a standard error of 0.268 was calculated for the multiple linear regression including a_1. The TLL influence on the total discharge is implicitly included in the regression equation in the intercept, a_0, value of 4,127. The average TLL discharge during days with CW discharge for the ORE AVM period is 3,159 ft^3/s. Subtracting this value from a_0 yields 968, which is similar to the a_0 value for the Sarasota AVM period and to the a_0 value for days of SG discharge. The average value of TLL discharge is important because of an operational difference between the water years. For the first 10 days of water year 1989 on which the CW was operational, the TLL discharge ranged from 2,000 to 2,600 ft^3/s, whereas on the last day of CW operation in water year 1989 and during the 8 days of CW operation in water year 1990, the TLL discharge ranged from 3,500 to 4,500 ft^3/s. Therefore, a regression equation calculated from the individual values of TLL discharge produced a poor fit, whereas a regression equation implicitly incorporating the average value of TLL discharge resulted in a closer fit.

The regression relations for the TLL discharge regime are nearly identical for the Sarasota AVM and the ORE AVM periods. Practically, the difference in intercepts is only 16.45 ft^3/s, which is only 0.5 percent of the average TLL discharge for the two periods, and the difference in slope is only 1.2 percent. Statistically, the difference in coefficient values for each regression equation is small relative to the standard errors of the coefficient values.

The difference in the regression equations for the other discharge regimes is statistically significant; however, this is primarily because of the large errors in the MWRD estimates of SG and CW discharge, which have been discussed in detail by the First and Second Technical Committees (Espey and others, 1981; 1987). An example of the high uncertainty in defining a relation between AVM discharges at Romeoville and the MWRD discharge estimates at Lockport, when the sluice gates are operating, is the Corps regression (U.S. Army Corps of Engineers, 1989) of data through September 30, 1987, which produced an a_2 value of 0.796. Thus, by adding another year of data (an additional 28 data points), the regression relation changed by more than three standard-error values. As discussed previously, the difference in the regression equations for days of CW operation is partly because of the implicit inclusion of the average TLL discharge in the intercept of the ORE AVM equation. Subtracting this average value indicates that the intercepts of the equations for the two periods are similar.

In a further comparison of the regression equations between the two periods, the ORE AVM regression equations were used to estimate discharges for the Sarasota AVM period. The Sarasota AVM regression equations, in turn, were used to estimate discharges for the ORE AVM period. Two sets of computations were made for each comparison--one for the TLL discharge only and one for all discharge regimes.

The mean, standard deviation, and skewness coefficient of the residuals (measured value minus estimated value) for the ORE AVM period, TLL discharge only, were 55.69 ft^3/s, 119.7 ft^3/s, and -0.0835, respectively. The mean, standard deviation, and skewness coefficient of the residuals for the Sarasota AVM period, TLL discharge only, were -55.90 ft^3/s, 153.1 ft^3/s, and 4.769, respectively. The hypothesis that the regression equations are equivalent for practical purposes is further supported by the near equality of the standard deviations of the residuals and the standard errors of the regression equations for the appropriate periods. The mean values of the residuals seem to indicate that the ORE AVM estimates tend to be about 50 ft^3/s higher than the Sarasota AVM estimates. The product of the difference in slopes (a_1) and the average TLL discharge for the entire period (3,202 ft^3/s) is 42.3 ft^3/s. Thus, the 50-ft^3/s difference is mainly because of the difference in slopes of the linear regression line. Further, because the slopes are not significantly different, it can be concluded that differences between the discharge computations made by the two AVM's cannot be documented.

The mean, standard deviation, and skewness coefficient of the residuals for the ORE AVM period for all discharge regimes were 165.8 ft^3/s, 421.4 ft^3/s, and -5.480, respectively. The mean, standard deviation, and skewness coefficient of the residuals for the Sarasota AVM period for all discharge regimes were 18.29 ft^3/s, 396.6 ft^3/s, and 5.358, respectively.

The regression equations derived for the periods of operation of the Sarasota AVM and the ORE AVM were further compared by considering the absolute difference between the discharge values estimated by each set of regression equations divided by the AVM discharge estimate on that day. The absolute relative difference, ARD, is defined as

$$ARD = |Q_{SAR} - Q_{ORE}| / Q_{AVM}, \quad (4)$$

where Q_{SAR} is the discharge estimated by the regression equations developed for the period of Sarasota AVM operation and Q_{ORE} is the discharge estimated by the regression equations developed for the period of ORE AVM operation. For the period of Sarasota AVM operation, the mean and standard deviation of the ARD are 1.39 and 0.66 percent, respectively, for days of TLL discharge only and 2.71 and 4.23 percent, respectively, for all days. For the period of ORE AVM operation, the mean and standard deviation of the ARD are 1.35 and 0.68 percent, respectively, for days of TLL discharge only and 2.89 and 4.66 percent, respectively, for all days. From this comparison, it is clear that the difference between use of either set of equations to predict either period is, on average, less than 3 percent. This small difference supports the practical equivalence of the two sets of regression equations.

On the basis of the practical and statistical consideration of the regression equations and the testing of the regression equations for each AVM against the measurements made by the other AVM, no significant differences between the two AVM's can be detected. Further, from a practical point of view, the differences in the estimates from the two sets of regression equations are less than 5 percent of the total flow, which is approximately the accuracy of the AVM equipment.

Vertical Velocity Distributions

Path-ratio velocity analyses were done for ORE AVM data (39,528 sets of four-path velocity measurements made from July 1991 through September 1992) and for Sarasota AVM data (12,353 sets of four-path velocity measurements made from March through October 1987). In these analyses, ratios were determined between the velocity of a given path and the cross-sectional average velocity. Because of the relocation of the transducers of the AVM equipment at the site between 1988 and 1991, these ratios are not directly comparable. The mean and 1 standard deviation confidence limits of the ratio of path velocity to cross-sectional average velocity are shown as a function of transducer elevation in figure 6. This linear plot shows consistency between the vertical velocity-distribution measurements made by the Sarasota AVM and the ORE AVM.

The middle path estimates the average velocity of the flow at a given elevation by measuring the traveltime of acoustic signals moving with the flow, whereas the cross path estimates the average velocity of the flow at a given elevation by measuring the traveltime of acoustic signals moving in opposition to the flow. The difference between the velocities measured by these two paths at nearly the same elevation indicates the angularity of the flow. In a preliminary analysis of 576 sets of four-path velocity data from July 1991, which yielded nearly identical ratios to those for the entire ORE data set, D.A. Stedfast (U.S. Geological Survey, written commun., 1991) concluded that the difference between the middle-path and cross-path velocities was sufficiently small that it could easily be because of equipment precision errors and (or) errors in the measurement of the path angles and length. Thus, he concluded that there is no significant difference between the path velocities. Similar conclusions can be drawn for the 1988 analysis of Sarasota data.

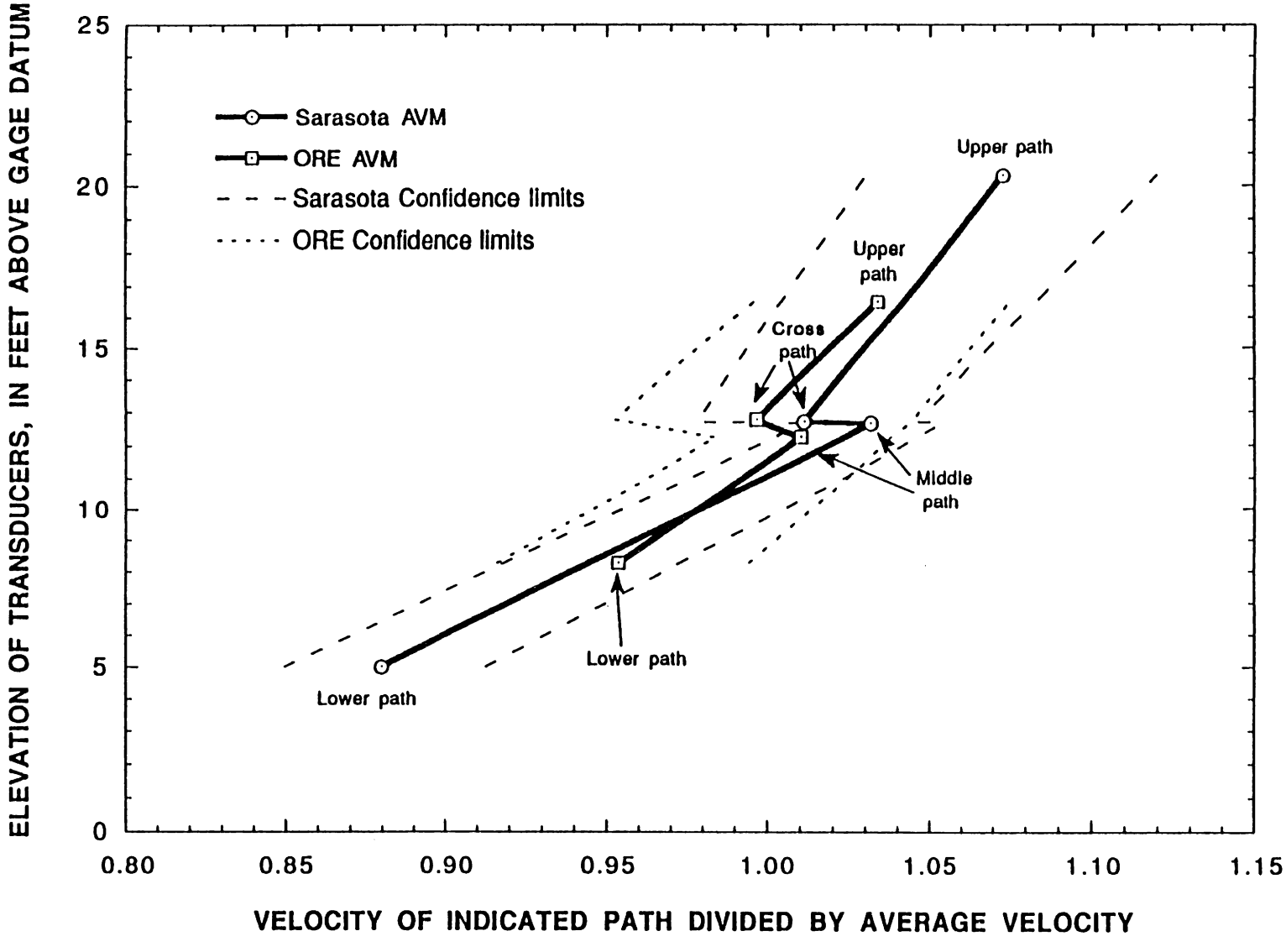


Figure 6.--Nondimensional vertical velocity distribution and 1 standard-deviation confidence limits (bounds) based on measurements by the Sarasota and ORE acoustic velocity meters.

Figure 7 shows the results of deleting the cross-path velocity and fitting a simple equation of the form

$$u(y)/\bar{u} = ay^{1/n}, \quad (5)$$

where $u(y)$ is velocity at depth y,
$\bar{u}$ is cross-sectional average flow velocity,
a is a constant coefficient, and
n is a constant exponent.

The cross-path velocity was deleted because it includes some effects of flow angularity, and these effects, although small, could bias the fitting of equation 5. Equation 5 has a power-law form for the velocity distribution similar to that first proposed by Prandtl in 1925 for flow in pipes (see Prandtl and Tietjens, 1934, p. 70-72). Several power-law formulations of various levels of complexity have been proposed for open-channel flow (Keulegan, 1938; Yen, 1992). Equation 5 is a formulation of the type that Chiu (1991) refers to as one of the most widely used in open-channel flow. The two curves in figure 7 are in close agreement. The values of n are 6.84 and 8.42 for the Sarasota AVM and the ORE AVM curves, respectively. These n values are in the general range reported for open-channel flows.

The agreement in vertical-velocity distributions between the two AVM's is only relative to the cross-sectional average velocity. No conclusions can be made regarding the velocity-magnitude measurements made by the two AVM's.

Correction of Discharge for Channel-Width and Depth Errors

On the basis of preliminary measurements made by the Rock Island District of the Corps, the Second Technical Committee (Espey and others, 1987) suggested that the Canal could be deeper than had been determined from the original design of the Canal. On June 4 and 5, 1991, the channel cross sections were measured at three points: (1) the upstream transducers, (2) the downstream transducers, and (3) a point midway between the transducers. The channel was found to be somewhat deeper than indicated by the original engineering plans and 2 ft narrower than had been estimated from measurements at the Romeoville Road bridge (now closed) just downstream from the gage. The measured bottom elevations relative to the original gage datum are given in table 4 and figure 8. The average bottom elevations at each location (determined by trapezoidal integration) and the average bottom elevation for the reach also are given. The average bottom elevation for the reach was computed by trapezoidal integration of the upstream, middle, and downstream elevations. From the corrected average bottom elevation and width, the flow area, A, in square feet, can be computed as

$$A = 162 \times (GH + 1.55), \quad (6)$$

where GH is gage height, in feet, above the original gage datum. The flow area was previously computed as

$$A = 164 \times GH. \quad (7)$$

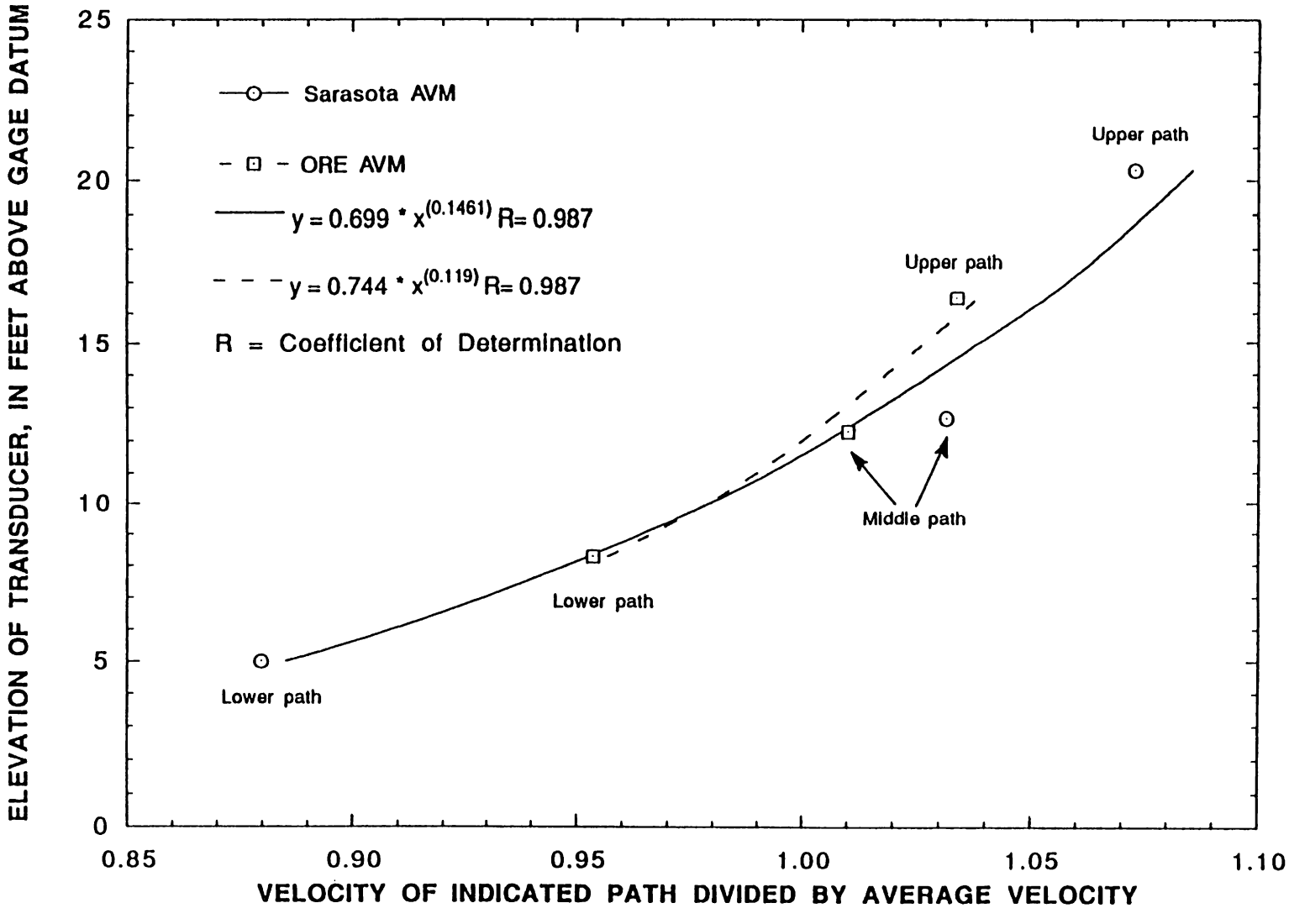


Figure 7.--Nondimensional vertical velocity distribution of the main paths measured by the Sarasota and ORE acoustic velocity meters and the best-fit power-equation representation of the data.

Table 4.--Calculated bottom-elevation data for the Chicago Sanitary and Ship Canal elevations measured at Romeoville, Ill., June 4 and 5, 1991

[Station, position of measurement point relative to a fixed point on the bank of the canal; elevation, elevation of the canal bottom relative to the original gage datum at the given station; average elevation, average elevation relative to the original gage datum between the current and previous stations; area, product of the average elevation and the distance between the current and previous stations; width, canal width at the given location; ft, feet; ft^2, square feet; --, not applicable]

Upstream transducer				Point midway between transducers				Downstream transducer			
Station (ft)	Elevation (ft)	Average elevation (ft)	Area (ft^2)	Station (ft)	Elevation (ft)	Average elevation (ft)	Area (ft^2)	Station (ft)	Elevation (ft)	Average elevation (ft)	Area (ft^2)
6	30	--	--	6	30	--	--	6	30	--	--
6	1.5	--	--	6	-2.4	--	--	6	3.7	--	--
8	.9	1.2	2.4	10	-1.9	-2.15	-8.6	9	6.6	5.15	15.45
10	.3	.6	1.2	14	-2.2	-2.05	-8.2	11	6.6	6.6	13.2
14	.3	.3	1.2	18	-1.5	-1.85	-7.4	15	3.1	4.85	19.4
18	.5	.4	1.6	22	-1.3	-1.4	-5.6	19	1.4	2.25	9.0
22	.7	.6	2.4	26	-1.6	-1.45	-5.8	23	1.5	1.45	5.8
26	.3	.5	2.0	30	-1.8	-1.7	-6.8	27	.2	.85	3.4
30	-.3	0	0	34	-2.0	-1.9	-7.6	31	.5	.35	1.4
34	-1.3	-.8	-3.2	38	-2.4	-2.2	-8.8	35	-.7	-.1	-.4
38	-1.7	-1.5	-6.0	42	-3.0	-2.7	-10.8	39	-.8	-.75	-3.0
42	-1.8	-1.75	-7.0	46	-3.1	-3.05	-12.2	43	-1.2	-1.0	-4.0
46	-2.0	-1.9	-7.6	50	-1.9	-2.5	-10.0	47	-1.1	-1.15	-4.6
50	-2.0	-2.0	-8.0	54	-3.0	-2.45	-9.8	51	-1.5	-1.3	-5.2
54	-2.0	-2.0	-8.0	58	-3.0	-3.0	-12.0	55	-1.4	-1.45	-5.8
58	-1.6	-1.8	-7.2	62	-2.7	-2.85	-11.4	59	-1.5	-1.45	-5.8
62	-1.8	-1.7	-6.8	66	-2.9	-2.8	-11.2	63	-1.3	-1.4	-5.6
66	-2.1	-1.95	-7.8	70	-2.7	-2.8	-11.2	67	-1.4	-1.35	-5.4
70	-2.1	-2.1	-8.4	74	-2.8	-2.75	-11.0	71	-1.2	-1.3	-5.2
74	-1.9	-2.0	-8.0	78	-3.0	-2.9	-11.6	75	-1.6	-1.4	-5.6
78	-2.0	-1.95	-7.8	82	-2.8	-2.9	-11.6	79	-1.6	-1.6	-6.4
82	-2.0	-2.0	-8.0	86	-3.0	-2.9	-11.6	83	-1.4	-1.5	-6.0
86	-2.0	-2.0	-8.0	90	-2.6	-2.8	-11.2	87	-1.7	-1.55	-6.2
90	-1.8	-1.9	-7.6	94	-2.4	-2.5	-10.0	91	-1.8	-1.75	-7.0
94	-1.8	-1.8	-7.2	98	-2.5	-2.45	-9.8	95	-.9	-1.35	-5.4
98	-1.9	-1.85	-7.4	102	-2.4	-2.45	-9.8	99	-1.3	-1.1	-4.4
102	-2.2	-2.05	-8.2	106	-2.1	-2.25	-9.0	103	-1.5	-1.4	-5.6
106	-2.3	-2.25	-9.0	110	-2.2	-2.15	-8.6	107	-1.2	-1.35	-5.4
110	-2.1	-2.2	-8.8	114	-2.1	-2.15	-8.6	111	-1.3	-1.25	-5.0
114	-2.3	-2.2	-8.8	118	-1.8	-1.95	-7.8	115	-1.7	-1.5	-6.0
118	-2.4	-2.35	-9.4	122	-2.0	-1.9	-7.6	119	-1.7	-1.7	-6.8
122	-1.9	-2.15	-8.6	126	-1.9	-1.95	-7.8	123	-1.8	-1.75	-7.0
126	-2.5	-2.2	-8.8	130	-1.8	-1.85	-7.4	127	-1.8	-1.8	-7.2
130	-2.6	-2.55	-10.2	134	-1.7	-1.75	-7.0	131	-1.5	-1.65	-6.6
134	-2.6	-2.6	-10.4	138	-1.6	-1.65	-6.6	135	-1.8	-1.65	-6.6
138	-2.2	-2.4	-9.6	142	-1.7	-1.65	-6.6	139	-1.2	-1.5	-6.0
142	-2.1	-2.15	-8.6	146	-1.4	-1.55	-6.2	143	-1.2	-1.2	-4.8
146	-1.9	-2.0	-8.0	150	-1.2	-1.3	-5.2	147	-.9	-1.05	-4.2
150	-1.7	-1.8	-7.2	154	-.5	-.85	-3.4	151	-.3	-.6	-2.4
154	-1.7	-1.7	-6.8	158	-.9	-.7	-2.8	155	.4	.05	.2
158	-1.9	-1.8	-7.2	162	-1.0	-.95	-3.8	159	1.1	.75	3.0
162	-1.8	-1.85	-7.4	164	-1.1	-1.05	-2.1	163	2.2	1.65	6.6
166	-1.8	-1.8	-7.2	168	-.7	-.9	-3.6	168	4.0	3.1	15.5
168.5	-1.4	-1.6	-4.0	168	30			168	30		
168.5	30										
Total area = -261.4 ft^2				Total area = -338.1 ft^2				Total area = -66.65 ft^2			
Width = 162.5 ft				Width = 162 ft				Width = 162 ft			
Mean elevation = -1.609 ft				Mean elevation = -2.087 ft				Mean elevation = -0.411 ft			

Average elevation for the three cross sections = (-1.609 + 2(-2.087) -0.411)/4 = -1.549

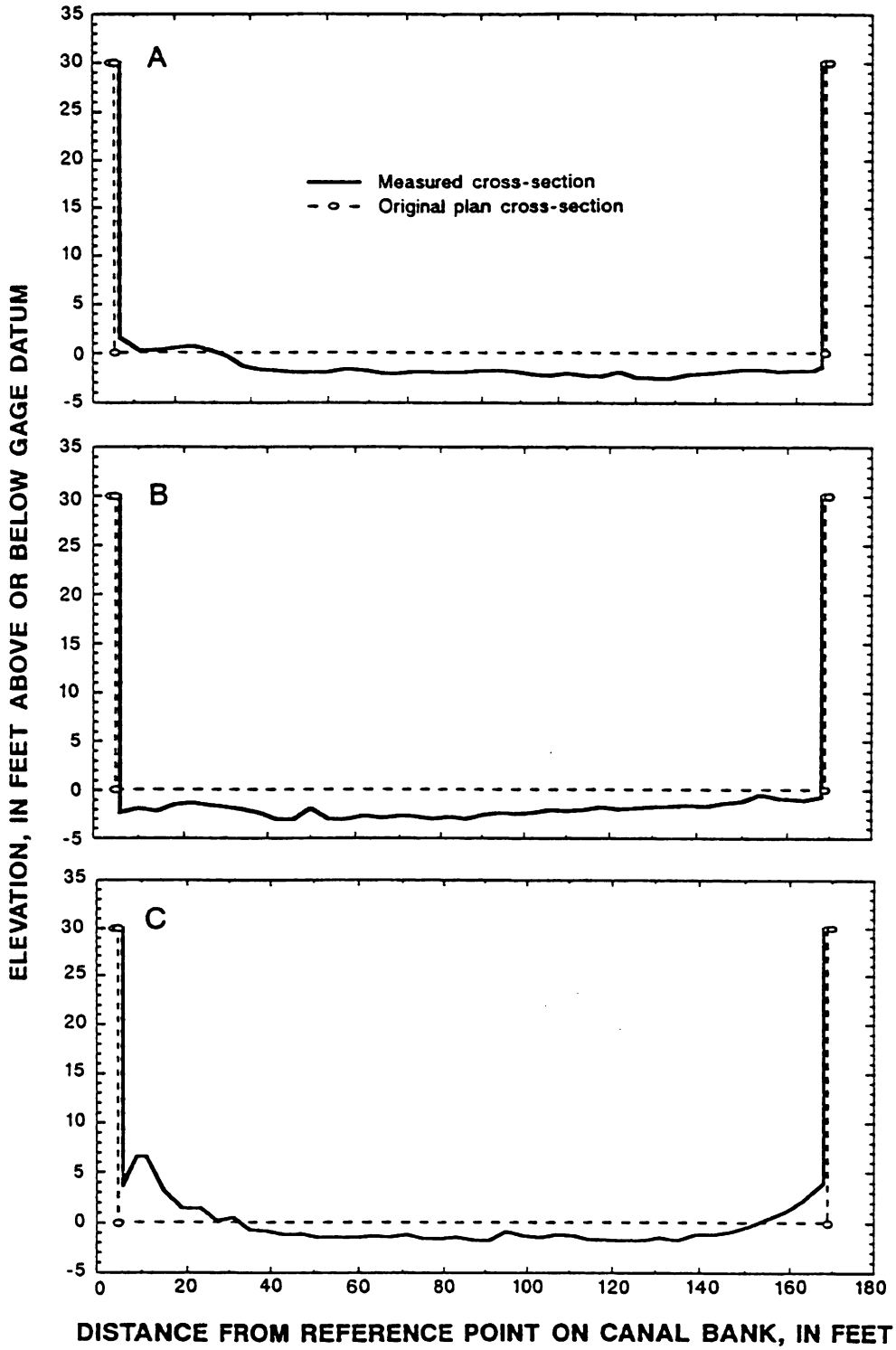


Figure 8.--Cross sections measured on June 4 and 5, 1991, at (A) the upstream transducers, (B) midway between the transducers, and (C) the downstream transducers for the acoustic velocity meter on the Chicago Sanitary and Ship Canal at Romeoville, Ill.

Two methods were used to compute discharges for the ORE AVM from the stage and velocities measured by the AVM. For water years 1989 and 1990, discharge was computed in horizontal slices of the cross section corresponding to each of the four velocity paths of the AVM (if a path is not functioning, the extent of the slices is adjusted). Discharge was computed by summing each of the horizontal slices. For water year 1991 to present (March 1993), discharge was computed by use of the index-velocity method (Rantz and others, 1982, p. 429-470). In this method, the average velocity of all functional paths (the index velocity) is first computed. The index velocity is then adjusted to the cross-sectional average velocity by a curve derived from USGS flow measurements. These two methods were used to recompute the discharges for the ORE AVM by reexecuting the computer programs developed by the USGS incorporating the gage height and area corrections indicated in equation 6.

Sarasota AVM discharges were not as easily corrected. The Sarasota AVM internally calculated the discharge from measured velocities, gage height, and several correction coefficients. The values of these correction coefficients represented a meter calibration. The coefficients were set by the manufacturer and were never provided to the USGS. These calculations were done at short time intervals, and 15-minute average values of the discharge, velocities for functioning paths, gage height, and daily mean discharge were printed and stored in a cassette-tape recorder. Data were stored on cassette tapes beginning in November 1986. Because the details of the internal calculations and the short time intervals are not known, the following approach was used to correct the daily mean discharge.

During water years 1987 and 1988, the daily mean discharges had been adjusted to account for a gage-height correction of 0.24 ft. This correction was determined by computing the difference between simultaneous gage-height readings from the wire-weight gage on the Romeoville Road bridge (fig. 1) and the AVM stage transducer. Originally, the correction was made by increasing the daily mean discharges by 1.2 percent; that is, by multiplying the daily mean discharge by 1.012. Instead of this method of adjustment, the gage-height correction of 0.24 ft was included directly in the area correction. Thus, the corrected gage height used to recalculate the daily mean discharges for water years 1987 and 1988 was 1.79 ft.

The Sarasota AVM computed discharge by means of the approach of horizontal slices of the cross section, as was done with the ORE AVM in water years 1989 and 1990. In the horizontal-slice method of discharge computation used with the ORE AVM, the discharge below the lowest transducer is calculated as

$$Q = V_b\ B\ (h/2) + 0.8V_b\ B\ (h/2), \qquad (8)$$

where V_b is the velocity measured at the lowest functioning transducer,
B is the canal width, and
h is the depth from the lowest functioning transducer to the canal bottom.

The first term in equation 8 represents the area between the lowest transducer and halfway to the bottom, and the second term represents the area between the bottom of the Canal and the lowest functioning transducer. The increase in

depth detected by the June 1991 survey is thus subject to an average velocity of 0.9 V_b. Application of this same procedure to the correction of Sarasota AVM discharge measurements results in

$$Q_{adj} = Q_{old} + 162 \times 1.79 \times 0.9 \times V_b = Q_{old} + 261 \times V_b, \quad (9)$$

where Q_{adj} is the adjusted discharge in cubic feet per second and
Q_{old} is the original discharge in cubic feet per second.

Further, comparisons of the velocity measured at the lowest transducer with the corresponding cross-sectional average velocity, V_m, of 12,353 measurements when all four velocity paths were operational show that the velocity measurement made by the lowest transducer was equal to $0.88V_m$ on average. Thus, the adjusted discharge for the Sarasota AVM was obtained from

$$Q_{adj} = Q_{old} + 229.7 \times V_m. \quad (10)$$

The use of $0.8V_b$ for the lowest depths is proper for the ORE AVM; however, for the Sarasota AVM, the lowest transducer is 5.26 ft lower than that for the ORE AVM (in water years 1989 and 1990 when the horizontal-slice method was used; in 1991, the lowest path was lowered to the position shown in figs. 6 and 7). Thus, a value less than $0.8V_b$ is more appropriate for the Sarasota AVM, but this value was used internally by the Sarasota AVM and is not known. Further, it would be more proper to multiply the gage-height correction of 0.24 ft by the velocity measured at the top transducer. The underestimation of the increase in discharge because of the gage-height correction probably is compensated for by the overestimation of discharge in the bottom parts of the channel. Although this calculation is somewhat crude, it is likely that the overall error is no more than 1 percent of the long-term average flow.

The cross-sectional average velocity to be used with equation 10 is then determined by one of the following methods:

1. For days for which complete 15-minute data were stored on cassette tapes and could be restored to the computer, the 15-minute average discharge was divided by the old flow area from equation 7 to determine a 15-minute average velocity. This 15-minute average velocity was then used in equation 10 to determine the adjusted discharge. These adjusted 15-minute average discharges were then averaged for the day to determine the adjusted daily mean discharge. The average of the unadjusted 15-minute average discharge did not equal the unadjusted daily mean discharge because of the differences in the averaging process; however, the differences usually are less than 1 percent and are unbiased. Therefore, it is unlikely that this error in the computation of the daily mean discharge will significantly affect the estimation of long-term average discharges at Romeoville.

2. Unfortunately, for 128 days in water years 1987 and 1988, the information stored on cassette tapes could not be recovered, and the printer output had to be used. Attempts to electronically scan the printer output directly into a machine-readable form were unsuccessful, primarily because optical character-recognition software could not reliably interpret the printout. Therefore, it was decided to calculate a daily mean gage height and then divide the original daily mean discharge by the unadjusted daily mean area

from equation 7 to determine the daily mean velocity. This daily mean velocity was then used in equation 8 to determine the adjusted daily mean discharge. Results from this approach were compared with the daily mean of the adjusted 15-minute discharges for a 300-day period of restored data, and it was found to have a median error (50 percent of errors smaller and 50 percent larger than this value) of 0.26 ft^3/s and an average error of 1.92 ft^3/s. Therefore, the difference in procedures is unlikely to seriously affect the estimation of long-term average discharges at Romeoville.

REGRESSION EQUATIONS FOR ESTIMATION OF MISSING DATA

A technique was needed for estimating daily mean discharge for days on which the AVM was either inoperative or not functioning properly (appendix 1). The technique devised for estimating missing AVM record is based on a set of equations that relate AVM-computed discharges at Romeoville to MWRD discharge estimates at the powerhouse, lock, and controlling works at Lockport. Development of this set of equations required answers to two questions: What data should be used to develop these equations, and what is the proper form of these equations?

Before discussing these questions, the goal of developing statistical-estimation equations must be defined. As described by Larimore and Mehra (1985), this goal is

> "...to obtain a model of the predictable behavior of the process but to avoid incorporating the random characteristics of the particular data set...Beyond a certain complexity, the model ends up fitting the noise in the data trying to explain every wiggle in the data."

Therefore, equations are sought that are effective estimators but not necessarily the best fit to a given set of data.

Data Used to Develop Equations

As discussed previously, there are no demonstrable, significant differences in the performance of the Sarasota AVM compared to the ORE AVM; however, the procedure for adjusting the discharge measurements made by the two AVM's is different and, in the case of the Sarasota AVM, involves certain assumptions. Therefore, the use of separate sets of equations for estimation of missing AVM record for the period of Sarasota AVM operation (October 1, 1985, through November 2, 1988) and the period of ORE AVM operation (November 18, 1988, through September 30, 1991) was examined. The advantage of combining the two data sets is that a broader range of flows at the AVM site at Romeoville and at Lockport is considered in the complete data set, especially for days when the sluice gates and (or) controlling works are in operation. Thus, if all the data are used, the resulting set of equations provides more robust estimations than if the data sets are kept separate.

In table 5, regression coefficients and standard errors are listed for a multiple linear regression of adjusted AVM discharges at Romeoville on the basis of MWRD discharge estimates at Lockport for the various discharge regimes. For days when the sluice gates and (or) controlling works are in operation, the regression-coefficient values and their differences between the Sarasota AVM period of operation and the ORE AVM period of operation are similar to those reported for the unadjusted data in table 2 despite the differences in the periods used to derive the coefficients in tables 2 and 5.

Table 5.--Regression coefficients and standard errors of the multiple linear regression of adjusted acoustic-velocity-meter discharges at Romeoville, Ill., approximated on the basis of discharge estimates made by the Metropolitan Water Reclamation District of Greater Chicago for Lockport, Ill., for the various discharge regimes (combinations of outlet works); separate equations are provided for the periods of operation for the Sarasota and ORE acoustic velocity meters

[AVM, acoustic velocity meter; a_0, intercept of the multiple linear regression; a_1, regression coefficient between the AVM discharge and the Metropolitan Water Reclamation District of Greater Chicago (MWRD) estimate of turbine, lockage, and leakage discharge for the given data set; a_2, regression coefficient between the AVM discharge and the MWRD estimate of sluice-gate discharge for the given data set; a_3, regression coefficient between the AVM discharge and the MWRD estimate of controlling-works discharge; all coefficients are used to estimate daily mean discharge, in cubic feet per second; coef., coefficient; --, no value calculated]

Discharge regime	Regression coefficient	Sarasota AVM[1] Number of days	Sarasota AVM[1] Coef. value	Sarasota AVM[1] Standard error	ORE AVM[2] Number of days	ORE AVM[2] Coef. value	ORE AVM[2] Standard error
Turbine, lockage, and leakage	a_0	569	-58.52	26.75	777	147.9	18.79
	a_1		1.168	.00863		1.104	.00651
Sluice gates	a_0	92	99.32	110.1	156	745.9	98.33
	a_1		1.145	.0313		1.023	.0294
	a_2		.8316	.0252		.4486	.0139
Controlling works	a_0	14	2,055	1,530	32	3,426	725.7
	a_1		.9037	.4460		.4645	.1920
	a_2		.6692	.0521		.3802	.0371
	a_3			--	--	.3264	.0560

[1]Period of operation considered October 2, 1986, through November 2, 1988.
[2]Period of operation considered November 18, 1988, through September 30, 1991.

For days of TLL discharge only, the difference in the slope (a_1) and intercept (a_0) of the regression equations between the Sarasota AVM period of operation and the ORE AVM period of operation is more than 7 standard errors. This is a significant change relative to the previous comparison of the two AVM's for which the difference was less than 2 standard errors. Therefore, statistically, the hypothesis that the adjusted Sarasota AVM and ORE AVM discharges are from the same population for days of TLL discharges only is

questionable. From a practical point of view, however, the regression equations for one AVM provide a reasonably good estimator of the data for the other AVM. Estimation of discharges for the ORE AVM period by use of the Sarasota AVM period regression equations yields a mean and standard deviation of the residuals equal to 29.2 and 157.8 ft^3/s, respectively, for days of TLL discharge only. Estimation of discharges for the Sarasota AVM period by use of the ORE AVM period regression equations yields a mean and standard deviation of the residuals of -14.1 and 165.1 ft^3/s, respectively, for days of TLL discharge only. The mean values of the residuals are less than 1 percent of the average AVM-measured discharge for each period. The standard deviations of the residuals are similar to the standard error of the linear regression equations for the same period: 165.1 to 157.9 ft^3/s for the Sarasota AVM period and 157.8 to 149.0 ft^3/s for the ORE AVM period. The mean and standard deviation of the ARD for days of TLL discharge only are 1.26 and 1.02 percent, respectively, for the period of Sarasota AVM operation and 1.75 and 1.50 percent, respectively, for the period of ORE AVM operation.

The consistency of the adjusted discharges for the Sarasota AVM and ORE AVM periods for days with SG and (or) CW discharges is best shown graphically. This comparison is facilitated by the assumption that the linear regression between AVM discharge measurements and MWRD discharge estimates at Lockport for days of TLL discharge only define an error relation for the TLL discharge regime. This error relation is assumed to hold for the TLL outlets even on days when the sluice gates and (or) controlling works are active. (The validity of this assumption is discussed in the section on Form of the Estimation Equations.) Subtraction of the corrected TLL discharges from the AVM discharge estimate yields corrected discharges through the SG and (or) CW. The corrected SG and (or) SG+CW discharge can then be compared to the MWRD estimates of these discharges. The corrected SG discharge and the MWRD-estimated SG discharge for days when the CW was not operating are shown in figure 9. The corrected SG+CW discharge and the MWRD-estimated SG+CW discharge are shown in figure 10. These figures show similar values and variabilities of corrected SG discharge compared to MWRD SG discharge and corrected SG+CW discharge and the MWRD SG+CW discharge for the Sarasota AVM and ORE AVM periods of operation. The SG data for the ORE AVM period that appear to be significantly different from the Sarasota period are the six highest MWRD SG daily mean discharge estimates. For these 6 days, the SG's were discharging at a high rate that in all other cases were associated with days when the CW also was operating. Therefore, this inconsistency is because of high discharge rates and not because of changes in AVM measurements relative to MWRD discharge estimates.

Estimation of discharges for the period of Sarasota AVM operation by the set of regression equations developed for the ORE AVM period of operation and for the period of ORE AVM operation by the set of regression equations developed for the Sarasota AVM period yields the following summary statistics:

	Mean (ft^3/s)		Standard deviation (ft^3/s)	
AVM	Measured	Estimated	Measured	Estimated
Sarasota	3,817	3,768	1,590	1,343
ORE	3,645	3,718	1,631	1,957

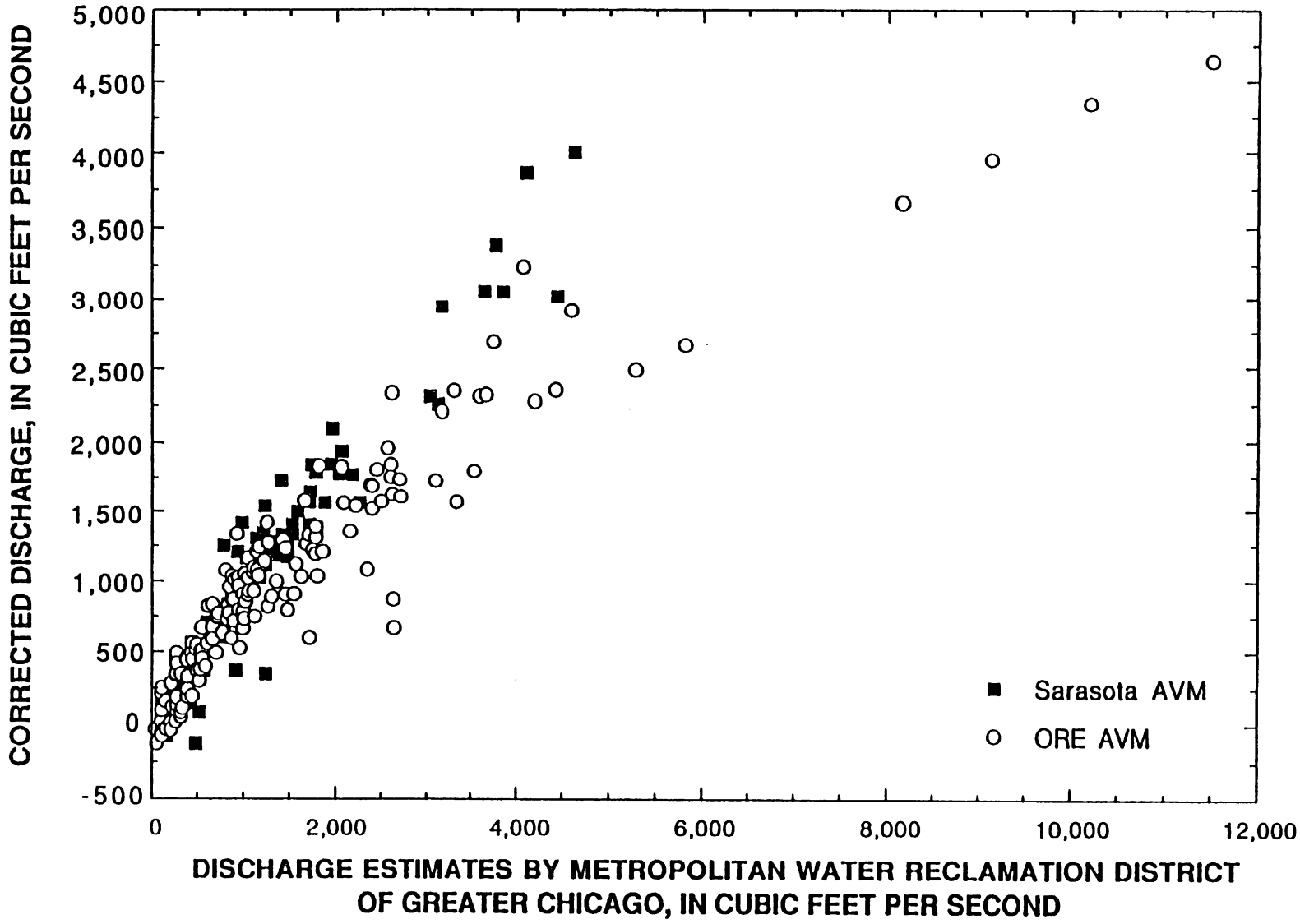


Figure 9.--Sluice-date discharge estimates by Metropolitan Water Reclamation District of Greater Chicago and corrected sluice-gate discharge estimated from the acoustic-velocity-meter (AVM) measurements.

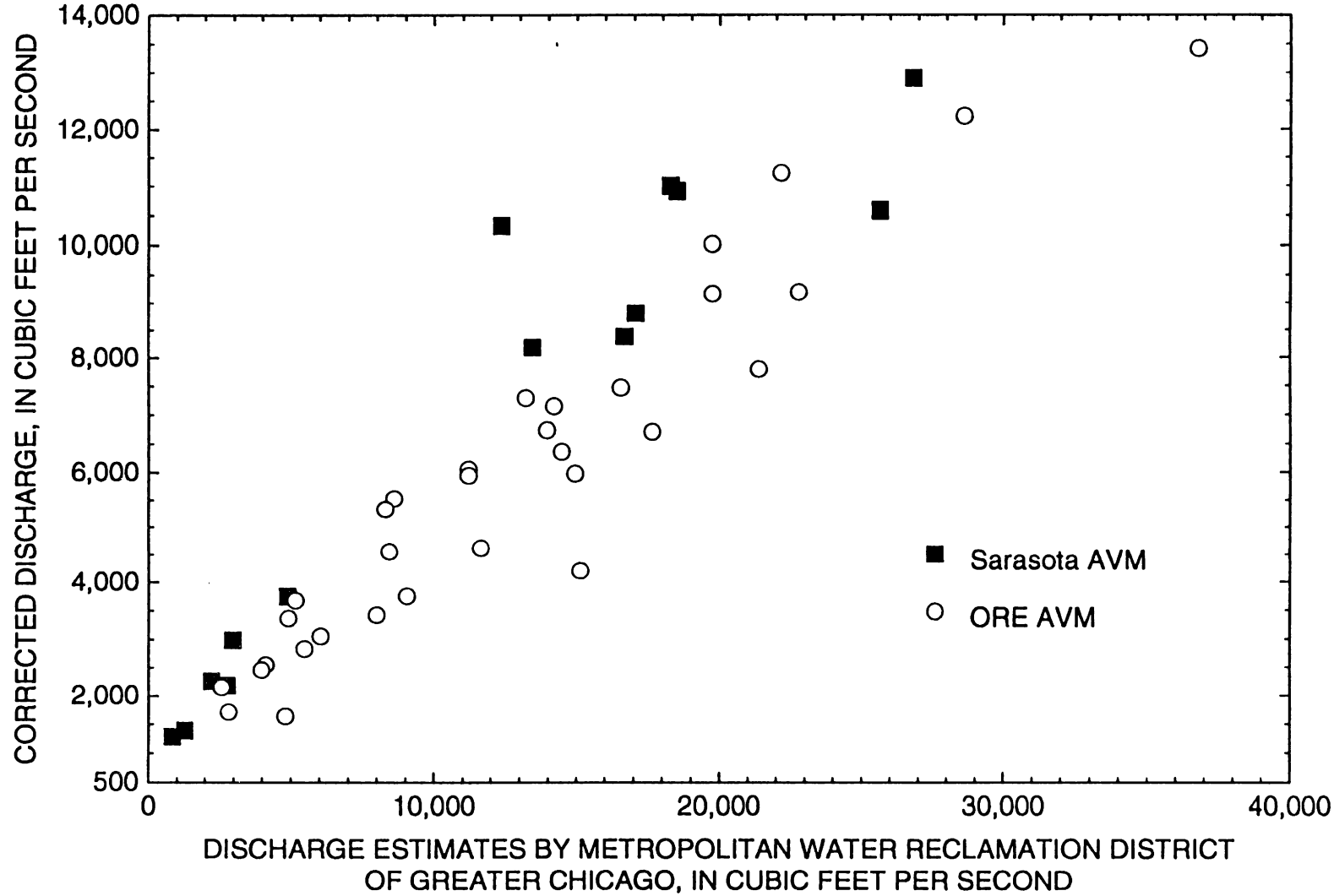


Figure 10.--Sluice-gate plus controlling-works discharge estimates by Metropolitan Water Reclamation District of Greater Chicago and corrected sluice-gate plus controlling-works discharge estimated from the acoustic-velocity-meter (AVM) measurements.

In each case, the error in the estimated mean is 2 percent or less. The mean and standard deviation of the ARD for all days are 2.22 and 2.96 percent, respectively, for the period of Sarasota AVM operation and 3.14 and 4.72 percent, respectively, for the period of ORE AVM operation. Considering the overall accuracy of estimating discharges for one period by use of the regression equations for the other period and the comparisons of figures 9 and 10, it is reasonable to combine the Sarasota AVM and ORE AVM discharge measurements when developing the equations for estimating the days of missing record. Further demonstration of the advantages of combining the two data sets is discussed in the section on Verification of Estimation Equations.

Form of the Estimation Equations

The Second Technical Committee (Espey and others, 1987) recommended that the data be subdivided into groups according to the discharge regime (that is, combination of outlet works in operation), and then the development of simple linear-regression equations between AVM and MWRD discharges for each discharge regime. The Corps (U.S. Army Corps of Engineers, 1989) went one step further by developing multiple linear-regression equations between AVM discharges and the discharge through each of the outlet-works components as given by equations 1-3. As discussed previously, this study followed the Corps' approach. However, the form of the regression equation for days when the CW was operating often appeared (tables 2 and 5) to be more of a "best fit of a particular data set" than a "model of the predictable behavior of the process," the latter being the goal of statistical-estimation-model development (Larimore and Mehra, 1985). This conclusion is evidenced by the large changes in regression-coefficient values as the discharge regime changes from sluice gates to controlling works.

A more physically based model (William Kirby, U.S. Geological Survey, Office of Surface Water, written commun., 1992), which might be a better estimator of "the predictable behavior of the process," also was developed and tested. In this physically based model, the linear-regression equation developed between AVM-measured discharges and MWRD discharge estimates for days when only TLL discharge occurs is assumed to define an error relation for the TLL discharge regime. This error relation is assumed to hold for the TLL outlets even on days when the sluice gates and (or) controlling works are active. Subtraction of the corrected TLL discharges from the AVM discharge estimates yields corrected discharges through the SG and (or) CW. Linear regression or multiple linear regression is then performed between the corrected discharge values and the MWRD estimates of the discharges through the outlet components.

The discharge through the turbines is not the same for days when the other outlet works are in operation as for days of only the turbines and lock operation. The headwater elevation is lower and the tailwater elevation is probably higher for the turbines on days when the other outlet works are in operation. On days with high flows, the headwater elevation can decrease as much as 5 ft, greatly changing the efficiency of the turbines and the error in the estimate of turbine discharge. The results of the multiple linear regression on days when the sluice gates, but not the controlling works, are in operation indicate that the error of holding the coefficient between TLL discharge and AVM discharge constant is not statistically significant. Thus,

the hydraulic error of assuming the relation between MWRD and AVM discharge estimates of TLL discharge constant for all discharge regimes is probably small as well. Conversely, the high hydraulic uncertainties in the SG discharge estimates, shown by the large variation of corrected SG discharge relative to the MWRD-estimated discharge in figure 9, made separation of corrected SG and CW discharges unwise.

The regression coefficients and standard errors of the multiple linear-regression equations between the AVM discharges and the MWRD flow estimates and the number of days used to develop each equation are listed in table 6. These equations were derived using all AVM-measured discharges in water years 1987 through 1991. Further, the discharge regimes were altered slightly, such that the days with MWRD estimates of SG discharge greater than 5,000 ft^3/s were shifted to the CW discharge regime. This shift was made because, as shown in figures 9 and 10, these discharges through the SG were of a magnitude that in all other cases was associated with days of CW operation. The corrected SG+CW discharge and the MWRD-estimated SG+CW discharge for the CW discharge regime and the 6 days of MWRD-estimated SG discharge greater than 5,000 ft^3/s are shown in figure 11. These 6 days of high MWRD-estimated SG discharge show good agreement with the CW discharge regime.

Table 6.--Regression coefficients and standard errors of the multiple linear regression of adjusted acoustic-velocity-meter discharges at Romeoville, Ill., approximated on the basis of discharge estimates made by the Metropolitan Water Reclamation District of Greater Chicago for Lockport, Ill., for the various discharge regimes (combinations of outlet works)

[a_0, intercept of the multiple linear regression; a_1, regression coefficient between the acoustic-velocity-meter (AVM) discharge and the Metropolitan Water Reclamation District of Greater Chicago (MWRD) estimate of turbine, lockage, and leakage discharge for the given data set; a_2, regression coefficient between the AVM discharge and the MWRD estimate of sluice-gate discharge for the given data set; a_3, regression coefficient between the AVM discharge and the MWRD estimate of controlling-works discharge; all coefficients are used to estimate daily mean discharge in cubic feet per second; <, less than; >, greater than; ft^3/s, cubic feet per second]

Discharge regime	Coefficient	Number of days	Coefficient value	Standard error
Turbine, lockage, and leakage	a_0	1,346	75.48	15.59
	a_1		1.127	.00523
Sluice gates (< 5,000 ft^3/s)	a_0	242	245.0	80.27
	a_1		1.120	.0229
	a_2		.6831	.0187
Controlling works or sluice gates (> 5,000 ft^3/s)	a_0	52	2,584	807.0
	a_1		.6883	.2163
	a_2		.4167	.0435
	a_3		.3455	.0666

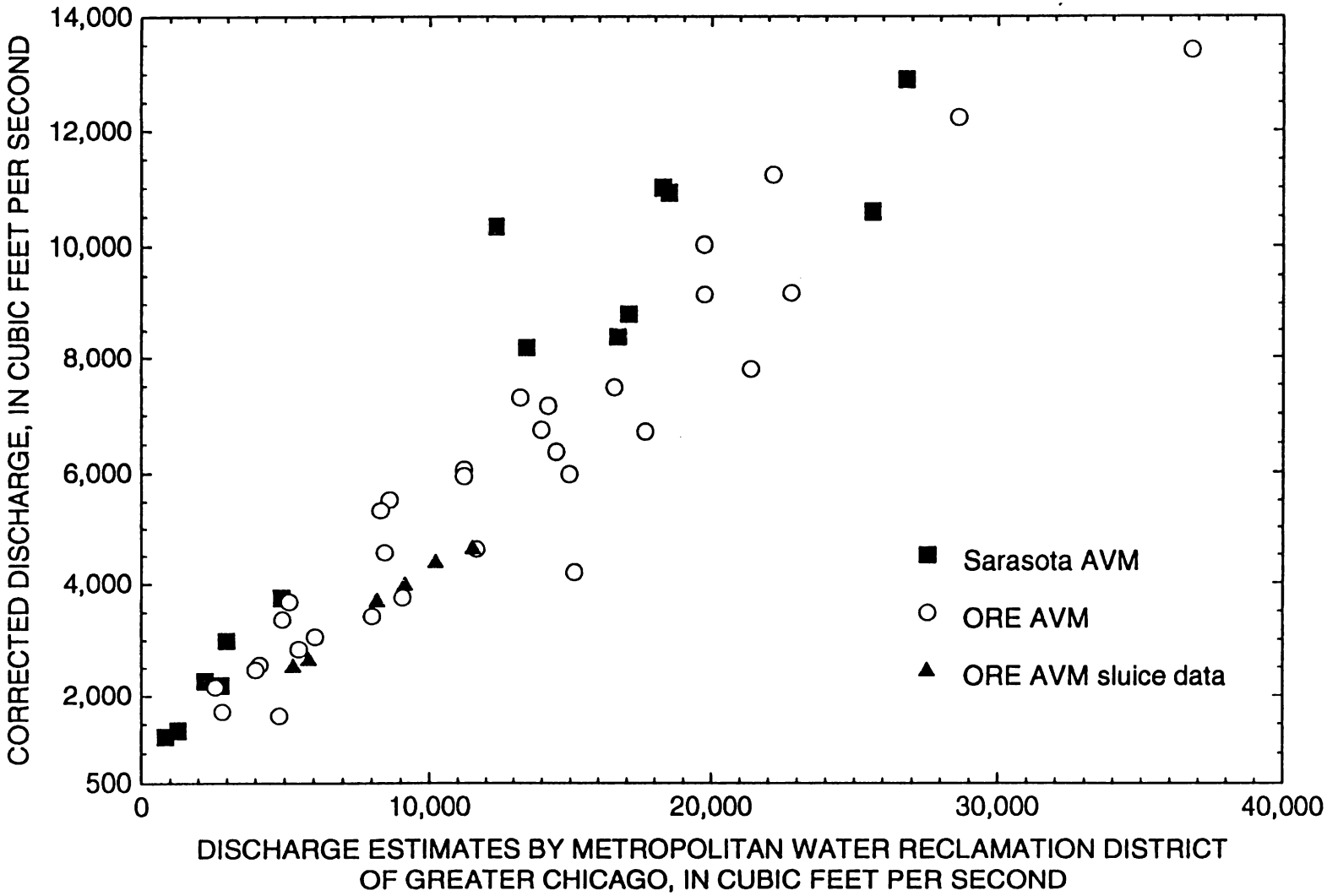


Figure 11.--Sluice-gate plus controlling-works discharge estimates by Metropolitan Water Reclamation District of Greater Chicago and corrected sluice-gate plus controlling-works discharge estimated from the acoustic-velocity-meter (AVM) measurements (includes the 6 days of estimated sluice-gate discharge greater than 5,000 cubic feet per second with no controlling-works discharge).

The approach of regression of the corrected discharges with the MWRD-estimated discharges resulted in the following equations for days with MWRD-estimated SG discharge less than 5,000 ft^3/s:

$$Q = 1.1270 \times Q_{TLL} + 0.6842 \times Q_{SG} + 219.7, \quad (11)$$

and for days with MWRD-estimated SG discharge greater than 5,000 ft^3/s or CW in operation:

$$Q = 1.1270 \times Q_{TLL} + 0.4361 \times Q_{SG} + 0.3228 \times Q_{CW} + 1,086. \quad (12)$$

It is interesting to note that all the coefficients in equation 11 are within 1 standard error of the values from multiple linear regression reported in table 6, whereas the coefficients in equation 12 are considerably different from the values in table 6. The standard errors of the equation (S_e) and the standard error of the equation as a fraction of mean flow for the given flow regime (S_e/Q_M) of the various equations are as follows:

[<, less than; >, greater than; ft^3/s, cubic feet per second]

	S_e (ft^3/s)	S_e/Q_M (percent)
Turbine, lockage, and leakage (table 6)	155.0	4.69
Turbine, lockage, leakage, and sluice gates (< 5,000 ft^3/s)		
Regression equation (table 6)	295.5	6.24
Corrected discharge equation (equation 11)	295.9	6.25
Turbine, lockage, leakage, sluice gates, and controlling works or sluice gates (> 5,000 ft^3/s)		
Regression equation (table 6)	1,195	12.5
Corrected discharge equation (equation 12)	1,245	13.1

The standard errors of equations 11 and 12 are nearly equal to those for the multiple linear-regression equations. Thus, it is clear that the equations derived from the corrected discharges provide nearly as close a fit to the AVM discharges as do the multiple linear-regression equations and a much better physical basis than the multiple linear-regression equations.

The residuals of either the multiple linear-regression equations or the equations derived from the corrected discharges are correlated in time (especially for the TLL discharge regime) and are not normally distributed. Similar results were found for the regression equations previously developed by the Corps (U.S. Army Corps of Engineers, 1989). Theoretically, this would invalidate the regression analyses performed in the sense that there is a better way to extract the maximum estimation capability from the given set of data (Troutman, 1985). That is, a model which incorporates the serial (temporal) correlation and transforms the residuals to be normally distributed would provide the maximum estimation capability from the given set of data. However, as pointed out by the Hydrologic Engineering Center (1990) in their review of the Corps (U.S. Army Corps of Engineers, 1989) regression equations, because the residuals are not independent or normally distributed does not mean that these regression equations should not be applied or accepted. The Hydrologic Engineering Center (1990) concluded that the regression equations

"...probably do not provide maximum likelihood estimates of the regression coefficients because of the failure of the residuals to meet certain stringent distributional requirements. However, the regression equations fit the data very well, and better than the previous attempts to develop regression relationships. Attempts to transform the data did not obtain a better distribution for the residuals or a better fit, nor do the transformed equations provide better forecasts. Consequently, the recommendation is to accept Chicago's proposed regressions as being the best available."

From this it is clear that equations derived for transformed data are unlikely to have improved properties or to provide improved estimates. Further, the regression equations and equations derived from the corrected discharges fit the data very closely, and, in the case of the equations derived from the corrected discharges, from a more correct physical basis than the regression equations that have been developed previously for the discharge through the Canal (Espey and others, 1987; U.S. Army Corps of Engineers, 1989).

Verification of Estimation Equations

The ultimate test of the regression equations and equations derived from corrected discharges is to estimate data not used in equation development; that is, verification of these equations. Data for water year 1992 were not used in the derivation of the estimation equations because these data have not been finalized and approved for final distribution. However, preliminary discharge estimates for the Lockport powerhouse and controlling works and the AVM at Romeoville are available for the period October 1, 1991, through May 31, 1992. The relatively small errors in these data (on random days) should not affect the use of these data in verifying the general estimation performance of the regression equations and the equations derived from corrected discharges.

The statistics of the AVM-measured discharges and the two sets of estimated discharges for the 238-day period, which included 66 days of SG operation (no discharges greater than 5,000 ft^3/s) and 6 days of CW operation, in the 1992 water year, are given below.

	Mean (cubic feet per second)	Standard deviation (cubic feet per second)	Skewness coefficient
AVM measurements	3,592	1,708	1.52
Regression equations estimates	3,594	1,598	1.45
Corrected discharge equations estimates	3,600	1,620	1.51

The standard deviation of the residuals of the estimated series are 216 ft^3/s for the regression equations and 206 ft^3/s for the equations derived from corrected discharges. The hydrographs of the measured and estimated discharges for the regression equations and the equations derived from corrected discharges are shown in figures 12 and 13, respectively.

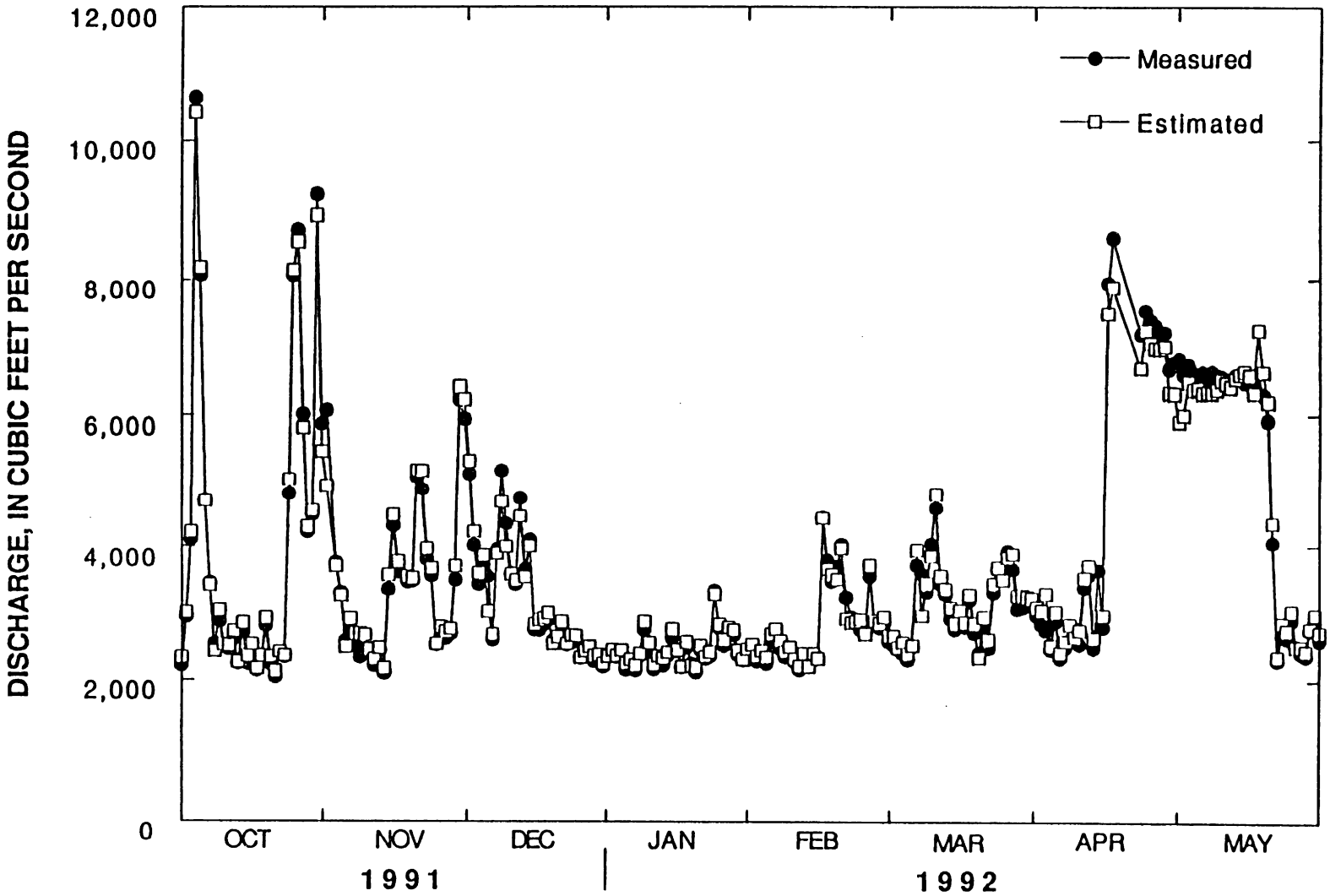


Figure 12.--Daily mean discharge estimated by the best-fit multiple linear-regression equations and the measured daily mean discharge for October 1, 1991, through May 31, 1992.

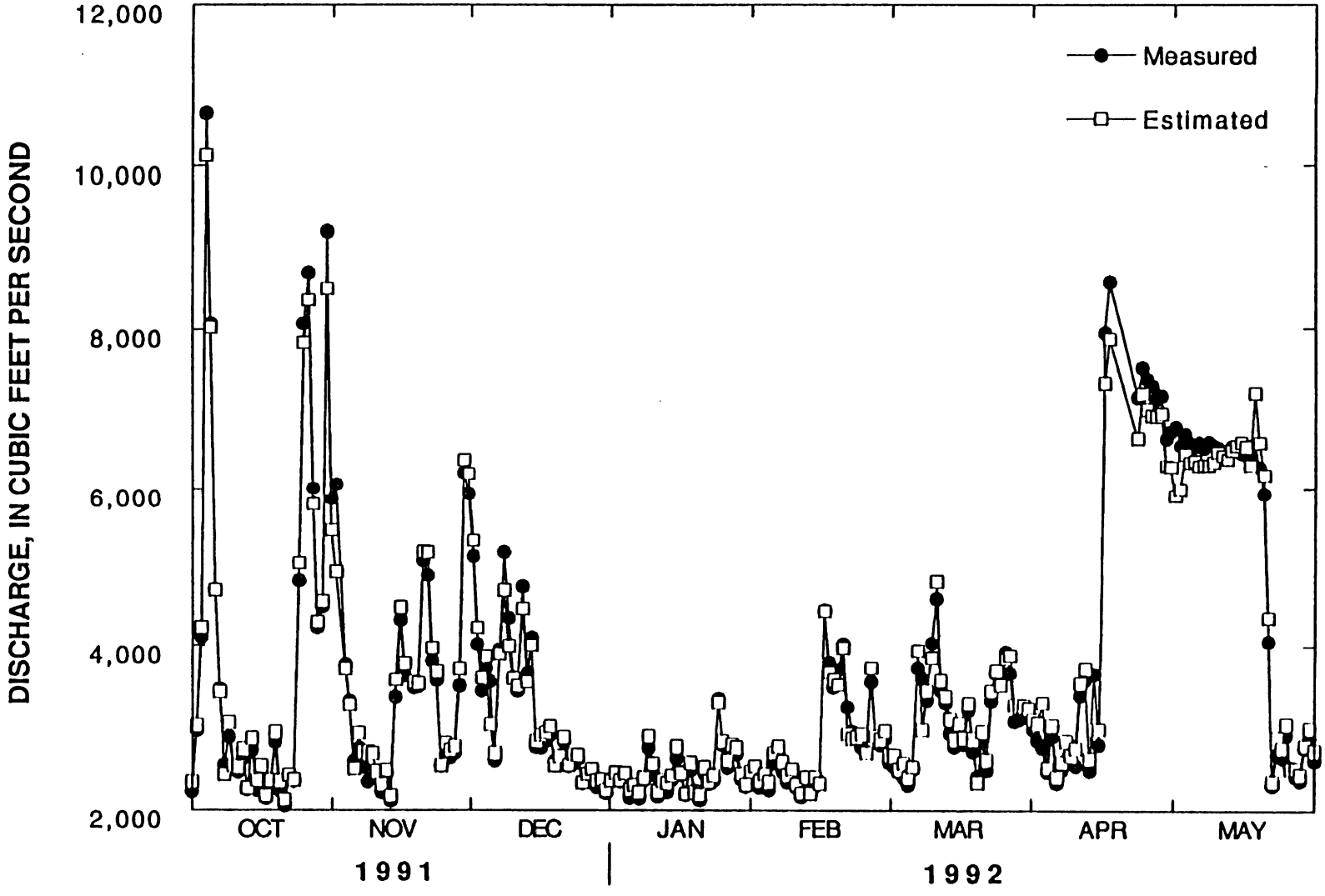


Figure 13.--Daily mean discharge estimated by the regression equations derived from corrected discharges and the measured daily mean discharge for October 1, 1991, through May 31, 1992.

Figures 12 and 13 and the statistics presented above clearly indicate that either equation offers an excellent estimation of discharges for October 1, 1991, through May 31, 1992. This is an especially stringent comparison because from April 15 through May 20, 1992, the Corps was performing repairs on the lock and the SG's were used to keep water levels low. Thus, the comparison period includes a disproportionate number of days of SG operation, and the hydraulic conditions for SG operation are different from the typical operation of the SG for release of high flows caused by stormwater runoff. Therefore, the comparison period represents a significantly perturbed condition relative to the calibration data, which forms the ideal type of verification data (Thomann, 1982).

The verification period also was estimated by three other sets of equations. Those sets were derived with the 6 days of MWRD-estimated discharge through the SG greater than 5,000 ft^3/s still included in the SG discharge regime. The sets are (1) regression equations and (2) equations derived from corrected discharges. The three cases considered for development of the equations are (1) Sarasota AVM discharge, (2) ORE AVM discharge, and (3) all AVM discharge. The mean, standard deviation, and skewness coefficient of the measured and estimated data and the standard deviation of residuals of the estimated data are shown in table 7. The equations derived from all the AVM data provide a better estimation than those derived from the data from only one AVM as shown in table 7. Further, comparison of values in table 7 with the results reported previously indicates that the shifting of the 6 days of MWRD-estimated high SG discharge also improves the estimates obtained.

Table 7.--Discharge statistics for October 1, 1991, through May 31, 1992, for the measured discharges at Romeoville, Ill., and the estimated discharges using three sets of regression equations and equations derived from corrected discharges

[AVM, acoustic velocity meter; ft^3/s, cubic feet per second]

Discharge series	Mean (ft^3/s)	Standard deviation (ft^3/s)	Skewness coefficient	Residual standard deviation (ft^3/s)
Measured	3,592	1,708	1.52	
Estimated regression, Sarasota AVM data	3,664	1,839	1.71	270
Estimated corrected, Sarasota AVM data	3,646	1,791	1.60	212
Estimated regression, ORE AVM data	3,537	1,442	1.43	348
Estimated corrected, ORE AVM data	3,548	1,484	1.51	317
Estimated regression, all data	3,572	1,555	1.52	267
Estimated corrected, all data	3,579	1,581	1.60	266

Estimation of Missing Record

The AVM's were not operational for a total of 545 days in water years 1986 through 1991. The entire 1986 water year makes up the majority of the non-operational period; all periods are listed in appendix 1. Because of the closeness of fit to the data for water years 1987 through 1991, estimation accuracy for the verification period of October 1, 1991, through May 31, 1992, and physical basis, the equations derived from corrected discharges were used to estimate the missing record. The complete estimated record is given in appendix 2.

Future Considerations

The equations derived from corrected discharges are valid for estimating missing record only up to August 20, 1992. In February 1992, the MWRD installed AVM's in the intakes to the turbines at Lockport. These AVM's started collecting data on March 19, 1992, and, beginning on August 21, 1992, the turbine AVM discharge measurements replaced the flow estimates from the turbine rating tables in the official MWRD report of flows at Lockport. Thus, estimation of missing record at the Romeoville AVM in the future will require the derivation of new equations of the form developed in this or similar studies. These equations cannot be developed until sufficient data have been collected at the Lockport turbine AVM's. Because the controlling works and (or) sluice gates are used sparingly in any given year, it could take several years to collect sufficient data for derivation of accurate new estimation equations. In the interim, a new relation between the MWRD rating estimate of turbine discharges and the true turbine discharges measured at the turbine AVM's could be developed for data from March through August 1992, and the equations derived from corrected discharges could then be reworked.

SUMMARY AND CONCLUSIONS

Discharge and regression analyses of acoustic-velocity-meter (AVM) data were done to obtain the most accurate estimates of discharge possible for the Chicago Sanitary and Ship Canal at Romeoville, Ill., for water years 1986-91. The analyses included (1) a check of the consistency of discharge estimates made by two AVM's used during this period, (2) adjustment of the discharge estimates to account for errors in the Canal width and depth used prior to Canal geometry measurement in June 1991, (3) development and verification of equations for estimating discharge on days when the AVM was not functioning properly, and (4) estimation of discharge for all days when the AVM's were not functioning properly.

The examination of the consistency of discharge estimates made by the two AVM's involved three analyses.

1. The discharge series estimated by the Sarasota AVM and the ORE AVM were compared for changes in the mean and variance by the t-test and F-test, respectively, and in the discharge-duration curves for several different combinations of operating outlet works. Trends in the discharge series

between the periods of Sarasota AVM and ORE AVM operation were compared to those trends at a nearby streamflow-gaging station (Des Plaines River at Riverside). Natural variations in discharge between the periods of operation of the two AVM's are far larger than any likely difference between the two meters.

2. Multiple linear-regression equations were derived relating AVM discharge estimates at Romeoville and Metropolitan Water Reclamation District of Greater Chicago (MWRD) discharge estimates at Lockport for the periods of operation of the two AVM's. The regression equations for the Sarasota AVM were used to estimate the ORE AVM period of operation, and the regression equations for the ORE AVM were used to estimate the Sarasota AVM period of operation. The regression equations developed and the estimates made applying these equations indicated that the regression equations for the two AVM's were practically indistinguishable.

3. Vertical velocity-distribution measurements made by the two AVM's were compared. The vertical velocity distributions measured by the two AVM's were consistent and within the range of values expected from open-channel flow theory.

In summary, no difference in the discharge computed using the two AVM's can be supported, and no correction factor is needed to adjust one AVM to be consistent with the other AVM.

The measurements of the Canal width and depth performed on June 4 and 5, 1991, indicated that the channel between the AVM transducers that measure velocity was 1.55 ft deeper and 2 ft narrower than had been previously determined from the original engineering plans and measurements of width made from the Romeoville Road bridge. Corrections for these errors were made easily for the ORE AVM data by reexecuting the computer programs developed by the USGS for estimating discharge at Romeoville, on the basis of AVM velocity and gage-height measurements. The Sarasota AVM discharge estimates were not corrected as easily because they were calculated internally by the AVM and output by the AVM to the USGS. The correction procedure used was developed on the basis of the cross-sectional average velocity computed from 15-minute and (or) daily mean discharges output by the Sarasota AVM, the estimated vertical velocity distribution, and discharge calculation by the method of horizontal slices.

Two different forms of equations for estimating discharge at Romeoville on the basis of MWRD discharge estimates at Lockport were compared. The first form was from standard multiple-linear regression between AVM discharge estimates at Romeoville and MWRD estimates of discharge at Lockport through the various outlet components--turbines, lockage, and leakage (TLL); powerhouse sluice gates; and controlling works. The error in the MWRD estimates of sluice-gate and controlling-works discharge is potentially very large. In the second form, the regression relation between MWRD estimates of TLL discharge at Lockport and AVM discharge estimates at Romeoville derived for days when the sluice gates and (or) controlling works were not in operation was used to define an error relation for the TLL discharge regime. This error relation was assumed to hold for the TLL outlets even on days when the sluice gates and (or) controlling works were active. Subtraction of the corrected TLL discharge from the AVM discharge estimates yielded corrected discharges through

the sluice gates and (or) controlling works. Regression equations were then developed between the corrected discharges and the MWRD estimates of discharge through these outlet components. The equations derived from corrected discharges performed as well as the first form of regression equations in terms of closeness of fit for the calibration period (October 2, 1986, through September 30, 1991) and estimation quality for the verification period (October 1, 1991, through May 31, 1992). The equations derived from the corrected discharges estimated the mean, standard deviation, and skewness coefficient of the daily mean discharges at Romeoville for the verification period within 0.22, 5.15, and 0.66 percent, respectively. This was considered to be an excellent verification.

The corrected discharges were used to estimate discharge at Romeoville for days when the AVM was not operational because of their excellent verification and strong physical basis. The discharge at Romeoville was estimated for 545 days on which the AVM was not operational. The corrected discharges have been entered into the discharge record for the station.

REFERENCES CITED

Chiu, C.-L., 1991, Application of entropy concept in open-channel flow study: Journal of Hydraulic Engineering, American Society of Civil Engineers, v. 117, no. 5, p. 615-628.

Espey, W.H., Barnes, H.H., Jr., and Vigander, S., 1981, Lake Michigan diversion, Findings of the Technical Committee for Review of Diversion Flow Measurements and Accounting Procedures: U.S. Army Corps of Engineers, Chicago District, 138 p.

Espey, W.H., Barnes, H.H., Jr., and Westfall, D.E., 1987, Lake Michigan diversion, Findings of the Second Technical Committee for Review of Diversion Flow Measurements and Accounting Procedures: U.S. Army Corps of Engineers, Chicago District, 206 p.

Hydrologic Engineering Center, 1990, Review of "Acoustic velocity meter regression analysis": Davis, Calif., U.S. Army Corps of Engineers, Hydrologic Engineering Center, Special Projects Memo no. 90-1, 33 p.

Keulegan, G.H., 1938, Laws of turbulent flow in open channels: Journal of Research of the National Bureau of Standards, Research Paper 1151, v. 21, p. 707-741.

Larimore, W.E., and Mehra, R.K., 1985, The problem of overfitting data: Byte, v. 10, no. 10, p. 167-180.

Prandtl, L., and Tietjens, O.G., 1934, Applied hydro- and aeromechanics: New York, Dover Publications, Inc., 311 p.

Rantz, S.E., and others, 1982, Measurement and computation of streamflow, volume 2, Computation of discharge: U.S. Geological Survey Water-Supply Paper 2175, 631 p.

Salas, J.D., Delleur, J.W., Yevjevich, V., and Lane, W.L., 1980, Applied modeling of hydrologic time series: Littleton, Colo., Water Resources Publications, 484 p.

Statware, Inc., 1990, STATIT statistics reference manual: Statware, Inc., Corvallis, Ore.

Thomann, R.V., 1982, Verification of water quality models: Journal of the Environmental Engineering Division, American Society of Civil Engineers, v. 108, no. EE5, p. 923-940.

Troutman, B.M., 1985, Errors and parameter estimation in precipitation-runoff modeling, 1, Theory: Water Resources Research, v. 21, no. 8, p. 1195-1213.

U.S. Army Corps of Engineers, 1989, Chicago Sanitary and Ship Canal at Romeoville acoustic velocity meter backup system: Chicago District, Lake Michigan Diversion Accounting Section, 117 p.

Yen, B.C., 1992, Hydraulic resistance in open channels, in Yen, B.C., ed., Channel Flow Resistance, Centennial of Manning's Formula: Littleton, Colo., Water Resources Publications, p. 1-135.

APPENDIXES

Appendix 1: History of inoperative periods for the acoustic velocity meter (AVM) at Romeoville, water years 1984-91

DATE	REMARKS
June 12, 1984	Sarasota AVM installed and operational.
November 29, 1984	Printer failed to advance paper.
March 17, 1985	Printer jammed.
February 23-24, 1985	Stage below the minimum AVM threshold value, no discharges calculated.
March 21 - April 18, 1985	Underwater cable connecting the transducers cut by a barge on March 21. The wires were reconnected and a repairman replaced the coefficients in the AVM unit, CPU timer module, and path-timer module on April 18.
April 19 - November 5, 1985	Various system problems during this period caused the AVM record to be questionable during this period. The system problems included underwater cable cut by barge on June 6-7; signal detector module failed on June 9-10; electrical power to the AVM cut off on June 27-28; AVM failed on August 1-12 because of a power surge from an electrical storm; and CPU timer module improperly set to use the last 2 minutes to calculate the 15-minute discharges.
November 6, 1985 - September 22, 1986	The CPU timer module properly reset on November 5, but operation problems resulted in low measurements of stage and velocity and, thus, in low values of computed discharge.
September 23 - October 1, 1986	On September 22, Sarasota personnel repaired the malfunctioning components of the AVM. In this period, however, the AVM was subjected to power surges.
April 24-25, 1987	AVM failed because of a power surge.
May 13-15, 1987	AVM failed because of a power surge.
December 31, 1987 - January 12, 1988	Power to the gage shut off.
January 15, 1988	ROM board replaced.
March 17 - May 24, 1988	A combination of receiver board and transducer malfunction and failure. The receiver board was replaced and reset, and all four velocity transducers and the depth transducer were replaced.

Appendix 1: History of inoperative periods for the acoustic velocity meter (AVM) at Romeoville, water years 1984-91--Continued

DATE	REMARKS
November 3-17, 1988	Installation of ORE AVM.
January 31 - February 3, 1989	AVM failed because of power surge.
June 12-14, 1989	Repair of the uplooking transducer for gage-height measurement.
August 22-23, 1989	AVM failed because of power surge.
September 26 - October 6, 1989	Underwater cables cut by a barge.
October 10-13, 1989	Transducer assembly knocked out of alignment by a barge.
November 15-21, 1989	Bad velocity measurements obtained from the transducers, and the transducers fell from their mountings into the canal.
March 16-19, 1990	Transducer assembly knocked out of alignment by a barge.
April 29 - May 1, 1990	A short circuit in the junction box caused the AVM to shut down.
May 4-8, 1990	No gage-height record because of equipment malfunction.
May 9-30, 1990	Underwater cables cut by a barge.
June 14, 1990	AVM transceiver damaged by lightning.
March 27, 1991	AVM was down because of a loss of power caused by high winds.
April 20-24, 1991	AVM was down because of the measurement section of the AVM locking up.
May 31 - June 3, 1991	AVM was down because of the measurement section of the AVM locking up.

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative

[MWRD, Metropolitain Water Reclamation District of Greater Chicago; USGS, U.S. Geological Survey; no values are shown for days with no flow through sluice gates or contolling works]

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
10/01/85	2,524	264	100			2,888	3,330
10/02/85	2,403	297	100			2,800	3,231
10/03/85	3,636	462	100	266		4,464	5,133
10/04/85	3,004	396	100			3,500	4,020
10/05/85	3,014	396	100			3,510	4,031
10/06/85	3,003	363	100			3,466	3,982
10/07/85	2,088	396	100			2,584	2,988
10/08/85	3,687	330	100			4,117	4,715
10/09/85	4,203	396	100			4,699	5,371
10/10/85	2,999	363	100			3,462	3,977
10/11/85	2,605	396	100			3,101	3,570
10/12/85	3,667	396	100			4,163	4,767
10/13/85	2,880	330	100			3,310	3,806
10/14/85	2,691	396	100			3,187	3,667
10/15/85	2,428	330	100			2,858	3,296
10/16/85	2,799	330	100			3,229	3,714
10/17/85	2,630	363	100			3,093	3,561
10/18/85	3,273	330	100	2,703	458	6,864	6,586
10/19/85	3,493	264	100	10,021	3,091	16,969	10,801
10/20/85	3,383	429	100			3,912	4,484
10/21/85	3,243	396	100			3,739	4,289
10/22/85	3,091	297	100			3,488	4,006
10/23/85	3,226	396	100	969	386	5,077	5,828
10/24/85	2,775	396	100	468		3,739	4,226
10/25/85	3,349	297	100			3,746	4,297
10/26/85	3,206	363	100			3,669	4,210
10/27/85	2,911	363	100			3,374	3,878
10/28/85	2,682	396	100			3,178	3,657
10/29/85	2,542	363	100			3,005	3,462
10/30/85	2,403	396	100			2,899	3,343
10/31/85	2,772	297	100			3,169	3,647
11/01/85	3,673	297	100	2,515		6,585	6,527
11/02/85	3,670	231	100	2,621		6,622	6,522
11/03/85	3,880	297	100			4,277	4,896
11/04/85	2,729	264	100			3,093	3,561

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
11/05/85	2,718	231	100			3,049	3,512
11/06/85	2,878	330	100			3,308	3,804
11/07/85	2,550	231	100			2,881	3,322
11/08/85	2,219	231	100			2,550	2,949
11/09/85	2,422	264	100	3,693	897	7,376	6,126
11/10/85	4,192	264	100	3,449		8,005	7,714
11/11/85	4,362	363	100	376		5,201	5,915
11/12/85	4,165	363	100	439		5,067	5,736
11/13/85	3,565	231	100	332		4,228	4,838
11/14/85	2,899	330	100			3,329	3,827
11/15/85	4,106	330	100			4,536	5,188
11/16/85	4,233	231	100	1,380		5,944	6,308
11/17/85	4,019	264	100	539		4,922	5,528
11/18/85	4,009	363	100	4,592		9,064	8,401
11/19/85	3,518	231	100	11,876	970	16,695	10,916
11/20/85	3,193	231	100	8,971		12,495	8,970
11/21/85	4,102	363	100	2,106		6,671	6,805
11/22/85	4,140	198	100	267		4,705	5,404
11/23/85	4,165	264	100			4,529	5,180
11/24/85	3,427	363	100			3,890	4,460
11/25/85	2,594	363	100			3,057	3,521
11/26/85	3,068	264	100			3,432	3,943
11/27/85	2,801	396	100			3,297	3,791
11/28/85	2,186	363	100			2,649	3,061
11/29/85	2,261	363	100			2,724	3,145
11/30/85	2,633	264	100			2,997	3,453
12/01/85	4,032	330	100	1,846		6,308	6,511
12/02/85	4,116	330	100			4,546	5,199
12/03/85	3,581	429	100			4,110	4,707
12/04/85	3,138	264	100			3,502	4,022
12/05/85	3,009	363	100			3,472	3,988
12/06/85	2,776	297	100			3,173	3,651
12/07/85	2,177	363	100			2,640	3,051
12/08/85	2,197	297	100			2,594	2,999
12/09/85	2,214	330	100			2,644	3,055

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
12/10/85	2,631	363	100			3,094	3,562
12/11/85	3,083	363	100			3,546	4,072
12/12/85	2,721	330	100			3,151	3,627
12/13/85	2,600	363	100			3,063	3,527
12/14/85	1,655	363	100			2,118	2,462
12/15/85	2,039	297	100			2,436	2,821
12/16/85	2,281	330	100			2,711	3,131
12/17/85	2,147	297	100			2,544	2,942
12/18/85	2,275	363	100			2,738	3,161
12/19/85	2,016	264	100			2,380	2,758
12/20/85	2,533	231	100			2,864	3,303
12/21/85	2,092	165	100			2,357	2,732
12/22/85	1,851	165	100			2,116	2,460
12/23/85	2,010	165	100			2,275	2,639
12/24/85	2,367	297	100			2,764	3,190
12/25/85	2,089	297	100			2,486	2,877
12/26/85	2,263	330	100			2,693	3,110
12/27/85	2,116	297	100			2,513	2,908
12/28/85	2,116	297	100			2,513	2,908
12/29/85	2,011	231	100			2,342	2,715
12/30/85	2,407	198	100			2,705	3,124
12/31/85	1,860	231	100			2,191	2,545
01/01/86	1,800	231	100			2,131	2,477
01/02/86	2,041	297	100			2,438	2,823
01/03/86	2,098	231	100			2,429	2,813
01/04/86	1,917	231	100			2,248	2,609
01/05/86	1,901	297	100			2,298	2,665
01/06/86	1,747	297	100			2,144	2,492
01/07/86	2,194	330	100			2,624	3,033
01/08/86	2,207	396	100			2,703	3,122
01/09/86	1,792	264	100			2,156	2,505
01/10/86	1,227	264	100			1,591	1,868
01/11/86	2,029	231	100			2,360	2,735
01/12/86	1,856	264	100			2,220	2,577
01/13/86	1,538	198	100			1,836	2,145

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
01/14/86	1,957	165	100			2,222	2,580
01/15/86	1,873	264	100			2,237	2,596
01/16/86	1,824	231	100			2,155	2,504
01/17/86	2,112	297	100			2,509	2,903
01/18/86	2,144	264	100			2,508	2,902
01/19/86	2,193	165	100			2,458	2,846
01/20/86	2,548	264	100			2,912	3,357
01/21/86	1,594	363	100			2,057	2,394
01/22/86	1,593	363	100			2,056	2,392
01/23/86	1,765	264	100			2,129	2,475
01/24/86	1,836	363	100			2,299	2,666
01/25/86	1,878	231	100			2,209	2,565
01/26/86	1,922	330	100			2,352	2,726
01/27/86	2,014	264	100			2,378	2,755
01/28/86	1,948	297	100			2,345	2,718
01/29/86	1,675	429	100			2,204	2,559
01/30/86	1,604	264	100			1,968	2,293
01/31/86	2,310	165	100			2,575	2,978
02/01/86	2,623	231	100			2,954	3,405
02/02/86	1,819	198	100			2,117	2,461
02/03/86	3,333	264	100			3,697	4,242
02/04/86	4,372	297	100	915		5,684	6,220
02/05/86	4,146	297	100			4,543	5,195
02/06/86	3,580	264	100			3,944	4,520
02/07/86	2,740	165	100			3,005	3,462
02/08/86	2,799	231	100			3,130	3,603
02/09/86	2,001	330	100			2,431	2,815
02/10/86	2,455	330	100			2,885	3,327
02/11/86	2,030	330	100			2,460	2,848
02/12/86	2,113	231	100			2,444	2,830
02/13/86	2,433	297	100			2,830	3,265
02/14/86	1,861	330	100			2,291	2,657
02/15/86	1,999	297	100			2,396	2,776
02/16/86	2,477	165	100			2,742	3,166
02/17/86	2,229	132	100			2,461	2,849

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
02/18/86	3,736	165	100			4,001	4,585
02/19/86	4,388	231	100			4,719	5,394
02/20/86	4,389	330	100			4,819	5,506
02/21/86	2,924	330	100			3,354	3,855
02/22/86	2,909	330	100			3,339	3,838
02/23/86	2,113	231	100			2,444	2,830
02/24/86	2,344	396	100			2,840	3,276
02/25/86	2,203	297	100			2,600	3,006
02/26/86	2,283	297	100			2,680	3,096
02/27/86	2,793	297	100			3,190	3,671
02/28/86	1,967	297	100			2,364	2,740
03/01/86	2,124	264	100			2,488	2,879
03/02/86	1,992	264	100			2,356	2,731
03/03/86	2,137	297	100			2,534	2,931
03/04/86	2,229	297	100			2,626	3,035
03/05/86	2,754	330	100			3,184	3,664
03/06/86	3,212	330	100			3,642	4,180
03/07/86	2,401	297	100			2,798	3,229
03/08/86	2,185	264	100			2,549	2,948
03/09/86	2,351	297	100			2,748	3,172
03/10/86	4,007	231	100	1,003		5,341	5,795
03/11/86	3,590	297	100			3,987	4,569
03/12/86	3,874	297	100			4,271	4,889
03/13/86	3,282	297	100			3,679	4,222
03/14/86	2,708	363	100			3,171	3,649
03/15/86	2,798	330	100			3,228	3,713
03/16/86	2,380	363	100			2,843	3,280
03/17/86	2,601	231	100			2,932	3,380
03/18/86	3,383	198	100			3,681	4,224
03/19/86	3,207	297	100			3,604	4,137
03/20/86	3,521	231	100			3,852	4,417
03/21/86	3,016	231	100			3,347	3,848
03/22/86	2,613	264	100			2,977	3,430
03/23/86	2,571	99	100			2,770	3,197
03/24/86	2,303	99	100			2,502	2,895

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
03/25/86	2,292	165	100			2,557	2,957
03/26/86	2,258	165	100			2,523	2,919
03/27/86	2,157	330	100			2,587	2,991
03/28/86	1,887	198	100			2,185	2,538
03/29/86	1,972	198	100			2,270	2,634
03/30/86	1,964	297	100			2,361	2,736
03/31/86	1,906	396	100			2,402	2,782
04/01/86	2,375	363	100			2,838	3,274
04/02/86	1,929	363	100			2,392	2,771
04/03/86	2,277	297	100			2,674	3,089
04/04/86	2,070	297	100			2,467	2,856
04/05/86	2,171	231	100			2,502	2,895
04/06/86	1,867	297	100			2,264	2,627
04/07/86	2,040	264	100			2,404	2,785
04/08/86	1,941	330	100			2,371	2,748
04/09/86	1,862	330	100			2,292	2,658
04/10/86	1,993	231	100			2,324	2,695
04/11/86	1,717	297	100			2,114	2,458
04/12/86	2,101	330	100			2,531	2,928
04/13/86	1,999	330	100			2,429	2,813
04/14/86	3,085	396	100			3,581	4,111
04/15/86	2,488	231	100			2,819	3,252
04/16/86	2,159	363	100			2,622	3,030
04/17/86	1,943	330	100			2,373	2,750
04/18/86	2,043	363	100			2,506	2,900
04/19/86	2,181	363	100			2,644	3,055
04/20/86	1,790	396	100			2,286	2,652
04/21/86	2,286	330	100			2,716	3,136
04/22/86	1,820	264	100			2,184	2,537
04/23/86	2,079	297	100			2,476	2,866
04/24/86	1,805	429	100			2,334	2,706
04/25/86	2,385	396	100			2,881	3,322
04/26/86	2,572	330	100			3,002	3,459
04/27/86	2,189	396	100			2,685	3,101
04/28/86	2,516	330	100			2,946	3,396

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
04/29/86	2,035	330	100	318		2,783	3,215
04/30/86	4,029	264	100	2,480		6,873	6,867
05/01/86	2,179	429	100			2,708	3,127
05/02/86	2,162	363	100			2,625	3,034
05/03/86	2,240	330	100			2,670	3,084
05/04/86	2,163	330	100			2,593	2,998
05/05/86	1,947	330	100			2,377	2,754
05/06/86	2,732	429	100			3,261	3,751
05/07/86	1,971	330	100			2,401	2,781
05/08/86	2,005	363	100			2,468	2,857
05/09/86	1,856	396	100			2,352	2,726
05/10/86	2,025	363	100			2,488	2,879
05/11/86	2,461	231	100			2,792	3,222
05/12/86	2,001	297	100			2,398	2,778
05/13/86	2,831	264	100			3,195	3,676
05/14/86	2,400	330	100			2,830	3,265
05/15/86	3,303	363	100			3,766	4,320
05/16/86	2,877	231	100			3,208	3,691
05/17/86	3,960	264	100	3,767	91	8,182	7,631
05/18/86	4,190	330	100	1,738		6,358	6,616
05/19/86	4,073	330	100	3,339	23	7,865	7,624
05/20/86	3,628	297	100			4,025	4,612
05/21/86	2,766	363	100			3,229	3,714
05/22/86	2,356	396	100			2,852	3,290
05/23/86	2,203	396	100			2,699	3,117
05/24/86	2,429	495	100			3,024	3,484
05/25/86	2,966	297	100			3,363	3,866
05/26/86	2,196	297	100			2,593	2,998
05/27/86	3,411	165	100	322		3,998	4,583
05/28/86	2,537	330	100			2,967	3,419
05/29/86	2,843	396	100			3,339	3,838
05/30/86	3,034	330	100			3,464	3,979
05/31/86	2,392	330	100			2,822	3,256
06/01/86	3,413	264	100			3,777	4,332
06/02/86	2,386	363	100			2,849	3,286

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
06/03/86	1,873	330	100			2,303	2,671
06/04/86	2,601	297	100			2,998	3,454
06/05/86	3,493	330	100	3,114		7,037	6,772
06/06/86	3,389	330	100			3,819	4,379
06/07/86	3,255	363	100	3,049		6,767	6,496
06/08/86	3,398	264	100			3,762	4,315
06/09/86	3,032	297	100			3,429	3,940
06/10/86	3,077	165	100	2,530		5,872	5,717
06/11/86	3,900	132	100	744		4,876	5,386
06/12/86	3,144	330	100			3,574	4,103
06/13/86	2,515	330	100			2,945	3,394
06/14/86	4,052	396	100			4,548	5,201
06/15/86	3,980	429	100			4,509	5,157
06/16/86	3,475	330	100	719		4,624	5,112
06/17/86	3,818	198	100			4,116	4,714
06/18/86	3,607	297	100			4,004	4,588
06/19/86	3,247	330	100			3,677	4,219
06/20/86	2,849	396	100			3,345	3,845
06/21/86	3,583	396	100			4,079	4,672
06/22/86	3,140	363	100			3,603	4,136
06/23/86	3,381	363	100			3,844	4,408
06/24/86	2,938	297	100			3,335	3,834
06/25/86	2,963	363	100			3,426	3,936
06/26/86	2,926	297	100			3,323	3,820
06/27/86	3,772	363	100	476		4,711	5,318
06/28/86	4,149	297	100			4,546	5,199
06/29/86	3,686	297	100			4,083	4,677
06/30/86	4,362	264	100	716		5,442	6,036
07/01/86	3,660	330	100			4,090	4,685
07/02/86	3,520	396	100			4,016	4,602
07/03/86	3,798	264	100			4,162	4,766
07/04/86	3,451	429	100			3,980	4,561
07/05/86	3,540	297	100			3,937	4,512
07/06/86	3,876	363	100			4,339	4,966
07/07/86	3,282	198	100			3,580	4,110

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
07/08/86	4,128	462	100	1,432		6,122	6,485
07/09/86	4,222	330	100	1,344	99	6,095	6,947
07/10/86	4,381	330	100			4,811	4,597
07/11/86	4,093	396	100	2,229		6,818	6,916
07/12/86	4,118	297	100	2,945		7,460	7,323
07/13/86	3,541	363	100			4,004	4,588
07/14/86	3,639	363	100			4,102	4,698
07/15/86	4,105	363	100	989		5,557	6,044
07/16/86	4,004	396	100			4,500	5,147
07/17/86	4,331	297	100			4,728	5,404
07/18/86	4,353	264	100			4,717	5,392
07/19/86	3,970	363	100			4,433	5,071
07/20/86	4,271	396	100			4,767	5,448
07/21/86	4,172	132	100			4,404	5,039
07/22/86	4,309	363	100			4,772	5,454
07/23/86	4,342	330	100			4,772	5,454
07/24/86	4,338	297	100			4,735	5,412
07/25/86	4,233	330	100	912		5,575	6,099
07/26/86	4,358	363	100			4,821	5,509
07/27/86	4,350	363	100			4,813	5,500
07/28/86	4,336	330	100	476		5,242	5,917
07/29/86	4,338	363	100			4,801	5,486
07/30/86	4,340	231	100			4,671	5,340
07/31/86	4,345	363	100	277		5,085	5,828
08/01/86	4,355	330	100			4,785	5,468
08/02/86	4,331	363	100			4,794	5,478
08/03/86	3,915	264	100			4,279	4,898
08/04/86	4,189	231	100			4,520	5,170
08/05/86	4,153	231	100			4,484	5,129
08/06/86	4,160	396	100			4,656	5,323
08/07/86	3,823	297	100			4,220	4,831
08/08/86	3,798	363	100			4,261	4,878
08/09/86	3,257	429	100			3,786	4,342
08/10/86	3,333	264	100			3,697	4,242
08/11/86	3,377	396	100			3,873	4,440

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
08/12/86	3,300	396	100			3,796	4,354
08/13/86	3,705	396	100			4,201	4,810
08/14/86	3,522	396	100			4,018	4,604
08/15/86	3,010	363	100			3,473	3,990
08/16/86	3,556	297	100			3,953	4,530
08/17/86	3,038	363	100			3,501	4,021
08/18/86	3,182	231	100			3,513	4,035
08/19/86	3,479	330	100			3,909	4,481
08/20/86	3,857	264	100			4,221	4,832
08/21/86	3,427	396	100			3,923	4,497
08/22/86	3,364	429	100			3,893	4,463
08/23/86	3,679	297	100			4,076	4,669
08/24/86	3,246	330	100			3,676	4,218
08/25/86	3,771	363	100			4,234	4,847
08/26/86	3,646	330	100	874		4,950	5,411
08/27/86	3,820	297	100			4,217	4,828
08/28/86	3,243	429	100			3,772	4,326
08/29/86	3,354	330	100			3,784	4,340
08/30/86	3,293	297	100			3,690	4,234
08/31/86	3,405	396	100			3,901	4,472
09/01/86	3,116	330	100			3,546	4,072
09/02/86	3,238	264	100			3,602	4,135
09/03/86	3,495	231	100			3,826	4,387
09/04/86	2,968	363	100			3,431	3,942
09/05/86	3,274	264	100			3,638	4,176
09/06/86	2,685	429	100			3,214	3,698
09/07/86	3,215	264	100			3,579	4,109
09/08/86	2,690	429	100			3,219	3,703
09/09/86	3,241	231	100			3,572	4,101
09/10/86	2,942	330	100			3,372	3,876
09/11/86	4,214	297	100	380		4,991	5,676
09/12/86	3,667	396	100			4,163	4,767
09/13/86	4,156	363	100			4,619	5,281
09/14/86	3,599	297	100			3,996	4,579
09/15/86	3,546	231	100			3,877	4,445

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
09/16/86	3,252	363	100			3,715	4,262
09/17/86	3,180	165	100			3,445	3,958
09/18/86	3,057	396	100			3,553	4,080
09/19/86	3,754	462	100			4,316	4,940
09/20/86	3,361	363	100			3,824	4,385
09/21/86	3,784	264	100	369		4,517	5,147
09/22/86	3,696	264	100	1,364		5,424	5,728
09/23/86	3,982	264	100	1,256		5,602	5,977
09/24/86	4,201	330	100	2,308		6,939	7,018
09/25/86	4,039	330	100	823		5,292	5,819
09/26/86	3,862	330	100	6,075	787	11,154	8,826
09/27/86	4,072	297	100	573	170	5,212	6,427
09/28/86	4,040	363	100	2,888		7,391	7,270
09/29/86	3,796	297	100	6,126	318	10,637	8,586
09/30/86	3,095	330	100	6,890		10,415	8,063
10/01/86	3,136	330	100	5,101		8,667	7,329
04/24/87	3,151	363	100			3,614	4,148
04/25/87	2,688	363	100			3,151	3,627
05/13/87	2,490	330	100			2,920	3,366
05/14/87	3,298	363	100			3,761	4,314
05/15/87	2,305	330	100			2,735	3,158
12/31/87	2,929	297	100			3,326	3,824
01/01/88	2,056	330	100			2,486	2,877
01/02/88	1,624	297	100			2,021	2,353
01/03/88	2,451	363	100			2,914	3,360
01/04/88	1,878	264	100			2,242	2,602
01/05/88	2,110	264	100			2,474	2,864
01/06/88	1,980	297	100			2,377	2,754
01/07/88	1,967	231	100			2,298	2,665
01/08/88	2,102	330	100			2,532	2,929
01/09/88	1,924	330	100			2,354	2,728
01/10/88	1,940	330	100			2,370	2,746
01/11/88	1,816	297	100			2,213	2,570
01/12/88	1,903	264	100			2,267	2,630
01/15/88	1,800	264	100			2,164	2,514

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
03/17/88	1,902	165	100			2,167	2,518
03/18/88	1,809	231	100			2,140	2,487
03/19/88	1,808	231	100			2,139	2,486
03/20/88	1,401	363	100			1,864	2,176
03/21/88	1,652	297	100			2,049	2,385
03/22/88	1,493	297	100			1,890	2,206
03/23/88	1,865	363	100			2,328	2,699
03/24/88	2,027	330	100	654		3,111	3,436
03/25/88	1,939	396	100	162		2,597	3,075
03/26/88	2,025	198	100			2,323	2,694
03/27/88	2,029	264	100			2,393	2,772
03/28/88	3,734	363	100	1,092		5,289	5,697
03/29/88	4,298	297	100	1,069		5,764	6,242
03/30/88	4,331	264	100	2,201		6,896	7,017
03/31/88	4,339	330	100			4,769	5,450
04/01/88	4,143	264	100			4,507	5,155
04/02/88	3,789	198	100	950		5,037	5,476
04/03/88	3,209	330	100			3,639	4,177
04/04/88	2,969	396	100			3,465	3,980
04/05/88	2,981	297	100	416		3,794	4,311
04/06/88	4,171	198	100	1,879		6,348	6,542
04/07/88	4,213	363	100	825		5,501	6,054
04/08/88	4,243	396	100			4,739	5,416
04/09/88	3,544	297	100			3,941	4,517
04/10/88	2,884	330	100			3,314	3,810
04/11/88	2,285	198	100			2,583	2,986
04/12/88	2,650	297	100			3,047	3,509
04/13/88	2,038	231	100			2,369	2,745
04/14/88	2,324	330	100			2,754	3,179
04/15/88	1,766	330	100			2,196	2,550
04/16/88	1,737	396	100			2,233	2,592
04/17/88	1,970	264	100			2,334	2,706
04/18/88	1,847	297	100			2,244	2,604
04/19/88	1,567	330	100			1,997	2,326
04/20/88	1,856	297	100			2,253	2,615

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
04/21/88	1,862	396	100			2,358	2,733
04/22/88	1,993	231	100			2,324	2,695
04/23/88	2,439	396	100			2,935	3,383
04/24/88	1,802	396	100			2,298	2,665
04/25/88	2,053	165	100			2,318	2,688
04/26/88	2,347	363	100			2,810	3,242
04/27/88	2,192	396	100			2,688	3,105
04/28/88	2,030	429	100			2,559	2,959
04/29/88	1,728	330	100			2,158	2,508
04/30/88	1,897	330	100			2,327	2,698
05/01/88	1,813	330	100			2,243	2,603
05/02/88	1,702	363	100			2,165	2,515
05/03/88	1,740	363	100			2,203	2,558
05/04/88	1,575	396	100			2,071	2,409
05/05/88	1,808	330	100			2,238	2,598
05/06/88	2,007	231	100			2,338	2,710
05/07/88	1,957	330	100			2,387	2,766
05/08/88	2,371	330	100			2,801	3,232
05/09/88	2,496	297	100			2,893	3,336
05/10/88	1,601	363	100			2,064	2,402
05/11/88	1,623	297	100			2,020	2,352
05/12/88	2,028	363	100			2,491	2,883
05/13/88	2,046	264	100			2,410	2,792
05/14/88	1,701	429	100			2,230	2,589
05/15/88	2,352	363	100			2,815	3,248
05/16/88	2,013	330	100			2,443	2,829
05/17/88	1,761	495	100			2,356	2,731
05/18/88	1,909	231	100			2,240	2,600
05/19/88	1,598	363	100			2,061	2,398
05/20/88	1,700	264	100			2,064	2,402
05/21/88	1,809	396	100			2,305	2,673
05/22/88	1,659	462	100			2,221	2,578
05/23/88	2,843	198	100	212		3,353	3,905
05/24/88	3,519	396	100			4,015	4,600
11/03/88	1,415	330	100			1,845	2,155

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
11/04/88	3,278	297	100	1,133		4,808	5,137
11/05/88	2,734	363	100			3,197	3,678
11/06/88	3,044	330	100			3,474	3,991
11/07/88	2,072	264	100			2,436	2,821
11/08/88	1,905	396	100			2,401	2,781
11/09/88	2,981	363	100	2,046		5,490	5,501
11/10/88	3,664	297	100	2,415		6,476	6,449
11/11/88	4,219	429	100			4,748	5,426
11/12/88	3,807	396	100			4,303	4,925
11/13/88	4,061	198	100			4,359	4,988
11/14/88	2,460	66	100			2,626	3,035
11/15/88	3,342	99	100	3,766		7,307	6,787
11/16/88	4,107	132	100	2,134		6,473	6,570
11/17/88	3,383	99	100			3,582	4,112
01/31/89	1,937	231	100			2,268	2,632
02/01/89	1,605	363	100			2,068	2,406
02/02/89	1,662	264	100			2,026	2,359
02/03/89	1,758	198	100			2,056	2,392
06/12/89	2,117	330	100	1,261		3,808	3,953
06/13/89	2,117	330	100	1,261		3,808	3,953
06/14/89	2,166	297	100	772		3,335	3,636
08/22/89	3,619	363	100			4,082	4,676
08/23/89	3,180	264	100			3,544	4,070
09/26/89	2,022	330	100			2,452	2,839
09/27/89	1,872	462	100			2,434	2,818
09/28/89	1,872	462	100			2,434	2,818
09/29/89	1,440	330	100			1,870	2,183
09/30/89	1,494	462	100			2,056	2,392
10/01/89	1,946	528	100			2,574	2,976
10/02/89	1,711	429	100			2,240	2,600
10/03/89	2,234	264	100			2,598	3,003
10/04/89	1,855	396	100			2,351	2,725
10/05/89	2,105	396	100			2,601	3,007
10/06/89	2,308	330	100			2,738	3,161
10/10/89	1,876	363	100			2,339	2,712

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
10/11/89	1,686	396	100			2,182	2,534
10/12/89	1,748	264	100			2,112	2,456
10/13/89	2,013	363	100			2,476	2,866
11/15/89	3,537	231	100	3,632		7,500	7,064
11/16/89	4,313	99	100			4,512	5,160
11/17/89	2,520	330	100			2,950	3,400
11/18/89	2,628	264	100			2,992	3,447
11/19/89	2,200	297	100			2,597	3,002
11/20/89	1,896	297	100			2,293	2,660
11/21/89	1,588	330	100			2,018	2,350
03/16/90	2,951	198	100			3,249	3,737
03/17/90	2,770	264	100			3,134	3,607
03/18/90	2,445	297	100			2,842	3,278
03/19/90	2,552	132	100			2,784	3,213
04/29/90	1,406	363	100			1,869	2,182
04/30/90	1,820	330	100			2,250	2,611
05/01/90	1,526	264	100			1,890	2,206
05/04/90	3,779	297	100	5,689	427	10,292	8,411
05/05/90	3,670	495	100			4,265	4,882
05/06/90	2,948	363	100			3,411	3,920
05/07/90	2,048	297	100			2,445	2,831
05/08/90	2,364	363	100			2,827	3,262
05/09/90	3,102	264	100	6,322	3,117	12,905	8,755
05/10/90	3,812	66	100	20,217	10,795	34,990	17,870
05/11/90	4,008	165	100	16,622	2,888	23,783	14,083
05/12/90	4,000	297	100	10,563	432	15,392	10,787
05/13/90	4,204	396	100	5,986		10,686	8,993
05/14/90	4,199	330	100	5,127		9,756	8,539
05/15/90	4,102	363	100	3,786		8,351	7,955
05/16/90	3,225	363	100	3,056		6,744	6,467
05/17/90	3,616	396	100			4,112	4,710
05/18/90	2,468	495	100			3,063	3,527
05/19/90	3,770	462	100	2,177		6,509	6,591
05/20/90	3,195	396	100			3,691	4,235
05/21/90	2,681	363	100			3,144	3,619

Appendix 2: Listing of discharges estimated for days when the acoustic velocity meter was inoperative--Continued

Date	Discharge, in cubic feet per second						
	Turbine	Lockage	Leakage	Sluice gate	Contolling works	MWRD total	USGS estimated
05/22/90	2,423	297	100			2,820	3,254
05/23/90	2,269	231	100			2,600	3,006
05/24/90	2,080	363	100			2,543	2,941
05/25/90	3,735	396	100			4,231	4,844
05/26/90	3,468	462	100			4,030	4,617
05/27/90	3,010	462	100			3,572	4,101
05/28/90	1,945	429	100			2,474	2,864
05/29/90	1,961	198	100			2,259	2,621
05/30/90	1,927	297	100			2,324	2,695
06/14/90	3,290	396	100	363		4,149	4,735
03/27/91	3,998	264	100	3,465		7,827	7,506
04/20/91	3,465	363	100			3,928	4,502
04/21/91	2,790	429	100			3,319	3,816
04/22/91	1,145	297	100			1,542	1,813
04/23/91	2,175	231	100	3,152		5,658	5,200
04/24/91	3,216	198	100			3,514	4,036
05/31/91	2,070	198	100	3,846		6,214	5,520
06/01/91	2,101	429	100	1,888		4,518	4,475
06/02/91	2,122	330	100	333		2,885	3,324
06/03/91	2,020	330	100	554		3,004	3,360

COMPARISON, ANALYSIS, AND ESTIMATION OF DISCHARGE DATA ON THE CHICAGO SANITARY AND SHIP CANAL AT ROMEOVILLE, ILLINOIS - USGS Water-Resources Investigations Report 93-4048

APPENDIX F

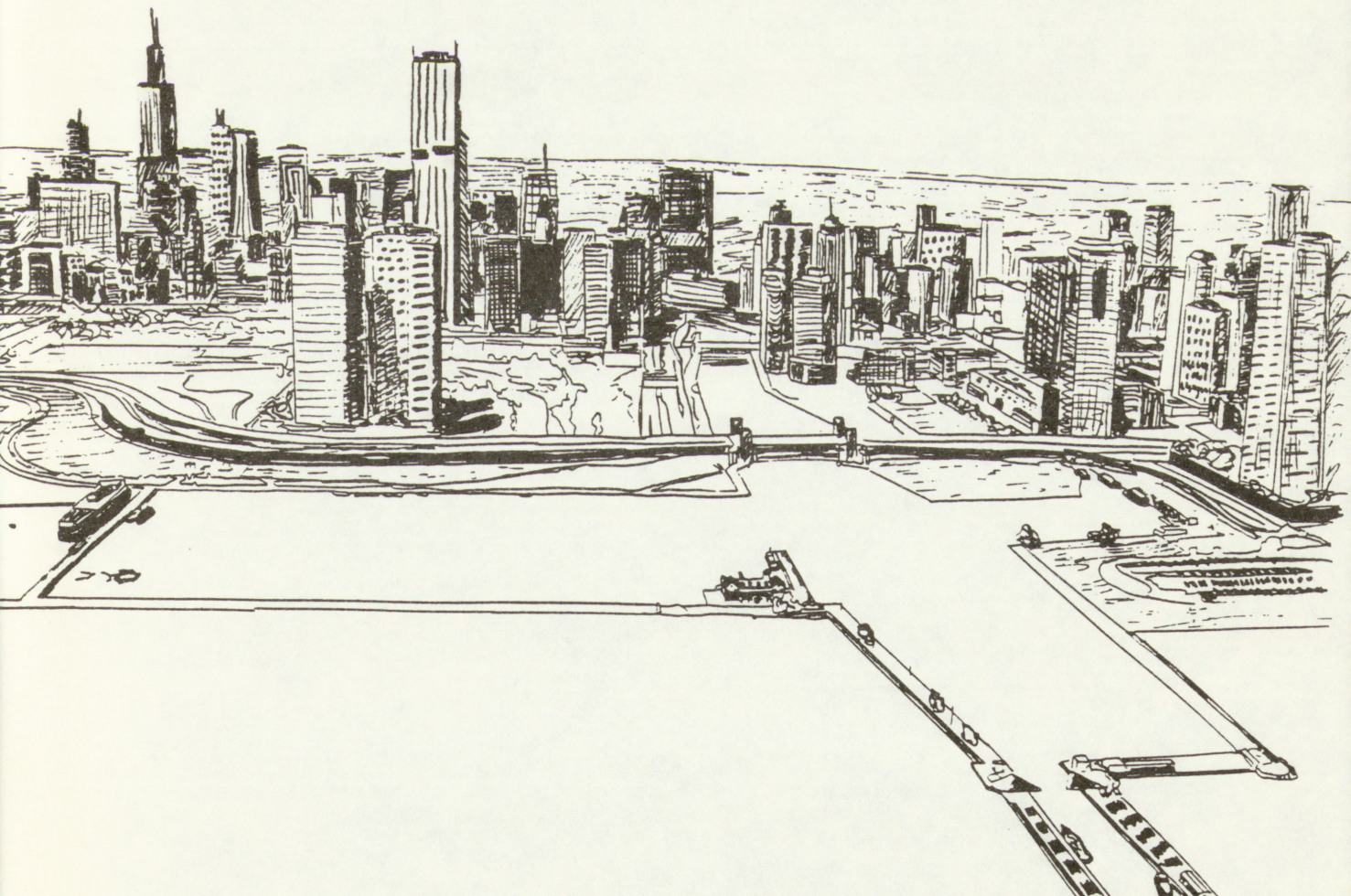
MEASUREMENTS OF LEAKAGE FROM LAKE MICHIGAN THROUGH THREE CONTROL STRUCTURES NEAR CHICAGO, ILLINOIS APRIL THROUGH OCTOBER 1993

MEASUREMENTS OF LEAKAGE FROM LAKE MICHIGAN THROUGH THREE CONTROL STRUCTURES NEAR CHICAGO, ILLINOIS, APRIL–OCTOBER 1993

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 94-4112

Prepared in cooperation with the
U.S. ARMY CORPS OF ENGINEERS, CHICAGO DISTRICT



MEASUREMENTS OF LEAKAGE FROM LAKE MICHIGAN THROUGH THREE CONTROL STRUCTURES NEAR CHICAGO, ILLINOIS, APRIL–OCTOBER 1993

by K.A. Oberg *and* A.R. Schmidt

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 94-4112

Prepared in cooperation with the

U.S. ARMY CORPS OF ENGINEERS, CHICAGO DISTRICT



Urbana, Illinois
1994

U.S. DEPARTMENT OF THE INTERIOR
BRUCE BABBITT, Secretary

U.S. GEOLOGICAL SURVEY
Gordon P. Eaton, Director

For additional information
write to:

District Chief
U.S. Geological Survey
102 E. Main St., 4th Floor
Urbana, IL 61801

Copies of this report can be
purchased from:

U.S. Geological Survey–ESIC
Open-File Reports Section
Box 25286, MS 517
Denver Federal Center
Denver, CO 80225-0046

CONTENTS

Abstract 1
Introduction 1
Purpose and Scope 4
Previous Work 4
Acknowledgments 4
Description of Control Structures 5
Chicago River Controlling Works 5
Thomas J. O'Brien Lock and Dam 5
Wilmette Pumping Station 11
Measurement Methods 11
Acoustic Doppler Current Profiler 11
Dye Dilution 17
Measurements of Leakage 23
Control Structures 24
Chicago River Controlling Works 24
Chicago Lock 24
Chicago River 31
Thomas J. O'Brien Lock and Dam 31
Wilmette Pumping Station 34
Accuracy of Measurements 34
Future Work 36
Summary and Conclusions 37
References 38
Appendixes 39
1. Summary of fluorometer calibrations, July 15-16, 1993 40
2. Summary of all dye samples collected from the Chicago Lock, both gates shut, July 15, 1993 41
3. Review of U.S. Geological Survey data collected in the Chicago area using an acoustic Doppler current profiler 43

FIGURES

1-2. Maps showing:
1. Location of study area and control structures near Chicago, Ill. 3
2. Location of the Chicago River Controlling Works, the Chicago Lock, and the Chicago River, Chicago, Ill. 6
3-4. Photographs showing:
3. Leakage through southeast harbor wall at Chicago River Controlling Works, view looking east, July 10, 1993 7
4. Chicago Lock, Chicago, Ill., looking east to west, September 1993 8
5. Diagram showing typical sluice-gate configuration at Chicago River Controlling Works and Thomas J. O'Brien Lock and Dam 9
6. Map showing location of Thomas J. O'Brien Lock and Dam, Burnham, Ill. 10
7. Diagram showing Thomas J. O'Brien Lock and Dam, Burnham, Ill. 10
8. Map and diagram showing Wilmette Pumping Station, Wilmette, Ill. 12
9. Photograph showing Wilmette Pumping Station, Wilmette, Ill., view looking northeast, September 1993 13
10. Sketch of beam pattern of acoustic Doppler current profiler showing side-lobe interference 15
11-13. Photographs showing:
11. Boat and windlass used to move boat across a measurement section 18
12. Apparatus used to inject dye mixture into the Chicago Lock, July 13-15, 1993 20
13. Dye-injection manifold in Chicago Lock, view looking north, July 13-15, 1993 21

CONTENTS

14. Generalized cross section of Chicago Lock channel showing the sample locations and bins used to calculate depth-weighted average dye concentrations 22
15. Photographs showing leakage through lake gates at Chicago Lock, view looking east, April 5, 1993 25
16. Diagram showing measurement locations, Chicago Lock, Chicago, Ill. 27
17-18. Graphs showing:
17. Mean and depth-weighted average concentrations from each vertical sampled at Chicago Lock, July 15, 1993 32
18. Sensitivity of acoustic Doppler current profiler discharge to changes in the power velocity-distribution law exponent 36

TABLES

1. Theoretical characteristics of jets from dye-injection manifold used in the Chicago Lock, July 13-15, 1993 19
2. Calibration curves for fluorometric analyses of dye samples collected in the Chicago Lock, July 13-15, 1993 23
3. Control-structure stage data for leakage measurements during April-October 1993 24
4. Acoustic Doppler current profiler data, April-October 1993 29
5. Concentrations and pumping rates for dye injection at Chicago Lock, July 13-15, 1993 31
6. Results of replicate and split samples collected at Chicago Lock, July 1993 33
7. Results of analysis of blank samples collected at Chicago Lock, July 1993 34

CONVERSION FACTORS AND VERTICAL DATUM

Multiply	By	To obtain
inch (in.)	25.4	millimeter
mile (mi)	1.609	kilometer
foot (ft)	0.3048	meter
square foot (ft^2)	0.09290	square meter
foot per second (ft/s)	0.3048	meter per second
cubic foot per second (ft^3/s)	0.02832	cubic meter per second
gallon per minute (gal/min)	0.06309	liter per second

Sea level: In this report, "sea level" refers to the National Geodetic Vertical Datum of 1929 (NGVD of 1929)—a geodetic datum derived from a general adjustment of the first-order nets of both the United States and Canada, formerly called Sea Level Datum of 1929.

Measurements of Leakage from Lake Michigan through Three Control Structures near Chicago, Illinois, April–October 1993

By K.A. Oberg *and* A.R. Schmidt

Abstract

Acoustic Doppler current profilers (ADCP's) and dye-dilution techniques were used to make 221 measurements of leakage at three control structures near Chicago, Ill. The three control structures are the Chicago River Controlling Works (CRCW), Thomas J. O'Brien Lock and Dam (O'Brien), and Wilmette Pumping Station (Wilmette). The CRCW consists of the Chicago Lock and two sets of sluice gates connected by a network of harbor walls. Lake Michigan water leaks through harbor walls, lock gates, and sluice gates at each of these control structures. The diversion of Lake Michigan water is regulated by U.S. Supreme Court decree, and the water leaking through each of these structures forms part of the diversion of Lake Michigan water by the State of Illinois.

Leakage measurements were made in April, May, July, September, and October 1993 by means of an ADCP. Dye-dilution measurements of leakage were made at the Chicago Lock in July 1993 to evaluate the discharges measured using the ADCP. The mean leakage measured by the ADCP for the Chicago Lock river gate was 133 ft^3/s (cubic feet per second); the standard deviation of the leakage measurements was 38 ft^3/s. The mean and standard deviation of the leakage measurements at CRCW were 192 and 73 ft^3/s, respectively. River-gate leakage accounted for more than half of the total leakage measured at CRCW. The mean and standard deviation of leakage measurements at O'Brien on September 17, 1993, were 21 and 10 ft^3/s, respectively. The mean leakage measured at Wilmette using the ADCP was 59 ft^3/s in April 1993; the standard deviation of the leakage measurements was 8 ft^3/s. After the pump bays at Wilmette were sealed in July 1993, the leakage decreased to less than 15 ft^3/s in September 1993.

Leakage through the river gate at the Chicago Lock was estimated by dye dilution on July 15, 1993. The discharge estimated by dye dilution was 160 ft^3/s; the standard deviation of the measurement was estimated to be 23 ft^3/s. The discharge estimated by dye dilution was within 12 ft^3/s, or 8 percent of the ADCP-measured discharge. Sensitivity analyses indicate that, for the leakages being measured, discharges are insensitive to the changes in the exponent of the power law for velocity distribution used to estimate the unmeasured flow near the channel bottom.

INTRODUCTION

Water has been diverted from Lake Michigan at Chicago into the Mississippi River Basin since the completion of the Illinois and Michigan Canal in 1848. At that time, the mean annual discharge of the diversion was about 500 ft^3/s. After construction of the Chicago Sanitary and Ship Canal (CSSC) was completed on January 17, 1900, the flow of the Chicago River was reversed and allowed to discharge into the Des Plaines and Illinois Rivers (Naujoks, 1946). On

December 5, 1901, the Secretary of War granted the State of Illinois a permit allowing a diversion of 4,167 ft^3/s through the CSSC. In October 1913, the United States filed a bill before the U.S. Supreme Court enjoining the State of Illinois from diverting more than 4,167 ft^3/s. In the ensuing years, however, the actual diversion increased, reaching a maximum of about 10,000 ft^3/s. In 1930, the U.S. Supreme Court entered a decree against the State of Illinois requiring that the diversion be reduced from a permitted 8,500 ft^3/s to 5,000 ft^3/s plus domestic pumpage by December 31, 1935, and 1,500 ft^3/s plus domestic pumpage by December 31, 1938 (Naujoks, 1946). The most recent U.S. Supreme Court decree (December 1, 1980) limits the diversion to a 40-year average of 3,200 ft^3/s. In addition, during the first 39 years, the cumulative algebraic sum of the average annual diversions minus 3,200 ft^3/s cannot exceed 2,000 ft^3/s.

The U.S. Army Corps of Engineers, Chicago District (Corps) has been charged with accounting for the amount of Lake Michigan water diverted each year by the State of Illinois. The acoustic velocity meter (AVM) on the CSSC at Romeoville, Ill. (fig. 1), is a key part of the Lake Michigan diversion-accounting procedure. Approximately 95 percent of the total diversion (including leakage) is measured at the Romeoville AVM. The U.S. Geological Survey (USGS) operates and maintains the AVM by agreement with the Corps. Prior to the installation of the AVM in 1984, the Lake Michigan diversion was measured at the Lockport Powerhouse at Lockport, Ill. Discharges were estimated by the Metropolitan Water Reclamation District of Greater Chicago (MWRD) using ratings for powerhouse turbines, powerhouse sluice gates, number of lockages, and controlling works sluice gates (located about 1 mi upstream from the powerhouse).

The Lake Michigan diversion consists of three components: direct diversion through three lakefront control structures, domestic pumpage from Lake Michigan for water supply and not returned to Lake Michigan, and stormwater runoff from the Lake Michigan watershed diverted from the lake. The direct diversion consists of four components: lockage, discretionary flow, navigation makeup flow, and leakage. Lockage is the amount of water used to lock vessels to and from Lake Michigan. Discretionary flows are used primarily for water-quality improvement in the Chicago River and CSSC. Occasionally, the water level in the Chicago River and CSSC is lowered in anticipation of a storm by increasing the discharge through the Lockport Powerhouse and Controlling Works. After the storm has passed, the sluice gates may be used to raise the water level in the Chicago River and CSSC. This component of the Lake Michigan diversion is referred to as navigation makeup. Leakage is that amount of water that leaks through or around the three control structures in an uncontrolled manner. The measurements described in this report were made to quantify this latter component of the diversion.

Recently, the Corps found that the State of Illinois has exceeded the 3,200 ft^3/s limit for each of the 1986-89 water years (U.S. Army Corps of Engineers, 1993, p. 18). (A water year is the 12-month period, October 1 to September 30, and is designated by the calendar year in which it ends.) In addition, at the end of the 1989 water year, the cumulative algebraic sum of the average annual diversions minus 3,200 ft^3/s was 2,189 ft^3/s. As a part of the accounting scheme, the Corps computes water budgets at various points in the diversion system using measured and simulated streamflow data. A rainfall-runoff model is used to estimate unmeasured diversion flows. The water budgets are used as an indication of the accuracy of diversion accounting and also indicate unmeasured or unsimulated flows. The average imbalance for water years 1986-89 was 454 ft^3/s. The Corps concluded that the annual budget imbalances were primarily a result of leakage through the lakefront control structures (U.S. Army Corps of Engineers, 1993, p. 18).

The water-surface elevation (stage) of Lake Michigan at each of the three control structures is normally higher than the stage of the Chicago and Calumet Rivers. The stage difference varies throughout the year, but it is usually largest in July when Lake Michigan is highest, and smallest in February, when Lake Michigan is lowest. Because of the stage difference at each of the lakefront control structures, a potential for leakage of water from Lake Michigan into the CSSC exists. The potential for leakage is greatest at the Chicago River Controlling Works (CRCW), which consist of the Chicago Lock and two sets of sluice gates at the mouth of the Chicago River. In the past, leakage through the lock gates at the Chicago Lock has been significant, primarily because of inadequate or missing gate seals. More recently, the gates have not been closing

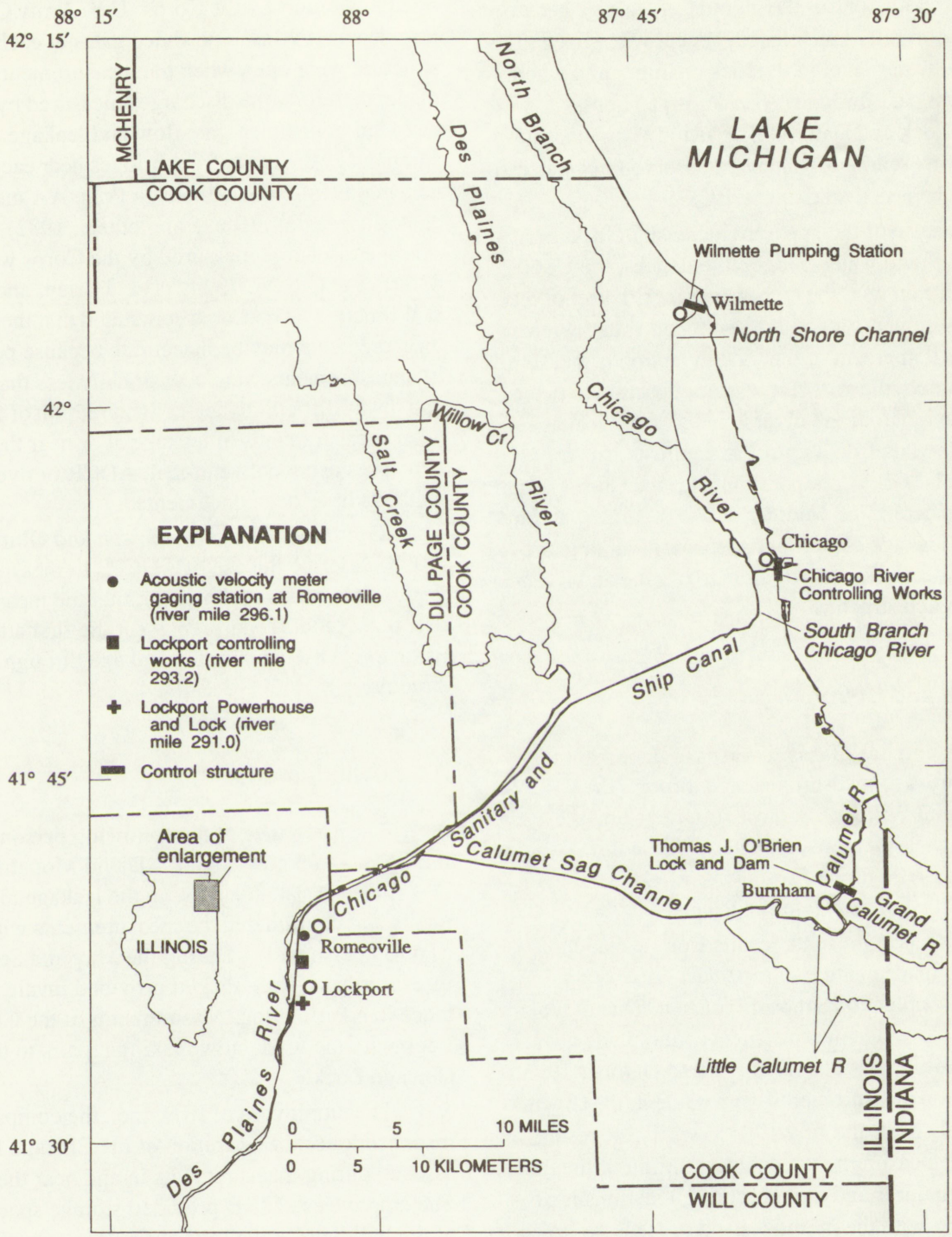


Figure 1. Location of study area and control structures near Chicago, Ill.

properly. The southeast (lake side) gate has been in especially poor condition. The opening between the two east gates, when closed as far as possible, has been as much as 24 in. The west gates (river side) were more water-tight than the east gates. The river gates normally control the amount of leakage because they are normally kept closed, except when opened to lower the water level in the lock chamber and when ships must pass through. Leakage at Thomas J. O'Brien Lock and Dam (O'Brien) and Wilmette Pumping Station (Wilmette) had been observed but was estimated to be less than that at CRCW.

Because of the apparent increase in leakage at the CRCW and water-budget imbalances, the USGS, in cooperation with the Corps, completed a set of leakage measurements at the lakefront control structures. Although 95 percent of the total diversion is measured at the Romeoville AVM, it was not possible to determine how much of the diversion was from leakage through the lakefront control structures. In April 1993, the USGS, in cooperation with the Corps, began a study to determine whether or not accurate measurements of leakage could be made and, if so, to make leakage measurements for a range of lake and river stages at each structure.

Purpose and Scope

The purpose of this report is to document the methods used to measure leakage through the CRCW, O'Brien, and Wilmette control structures and the results of measurement analysis. These three structures control the flow of water from Lake Michigan into the Chicago and Calumet Rivers.

This report presents results from leakage measurements made using two methods: use of an acoustic Doppler current profiler (ADCP) and dye dilution. Leakage measurements were made at each of the control structures from April to October 1993. Dye-dilution measurements were made in the Chicago Lock during May and July 1993 to verify the results of the ADCP measurements. A description of the three control structures and the measuring locations is provided along with the methods used to measure leakage. The accuracy of the leakage measurements also is evaluated, and the results of the two measurement methods are compared. The report also includes suggestions for future leakage measurements.

Previous Work

The Corps made discharge measurements near each of the three control structures during July 23-27, 1990. Results of these measurements are summarized in a report by the Corps (U.S. Army Corps of Engineers, 1990). The sluice gates at each control structure were open when the measurements were made; therefore, the discharges measured by the Corps included both sluice-gate flow and leakage. Measurements were made from bridges near each structure using conventional methods (a Price AA meter with cable suspension) (Rantz and others, 1982). The average discharge measured by the Corps was 830 ft^3/s at CRCW, 709 ft^3/s at O'Brien, and 225 ft^3/s at Wilmette. The Corps concluded that the July 1990 measurements may be inaccurate because point velocity measurements were occasionally less than 0.10 ft/s and it was difficult to detect the direction of flow. No other measurements of leakage at or near the control structures using conventional, ADCP, or dye-dilution methods have been documented.

Gordon (1989) and Simpson and Oltmann (1993) describe the use of ADCP's to measure streamflow. It is believed, however, that the measurements described in this report represent the first attempt at using an ADCP to measure leakage through a control structure.

Acknowledgments

The assistance of the following personnel from the Corps is acknowledged. David Moughton and David Kiel assisted in making the leakage measurements and coordinated the measurements with MWRD personnel. During the May and September measurements, Rich Pickett provided invaluable assistance in coordinating measurements at the Chicago Lock with the work crew making repairs to the Chicago Lock.

The employees of IRD, Inc., the company responsible for the operation of the Chicago Lock, assisted during measurements in and near the lock. The employees of IRD provided storage space in one of the buildings at the lock, warned boat traffic during leakage measurements, and occasionally loaned equipment. We especially acknowledge Scott Baseman, John Mammano, and Chris Boyer for their cheerful assistance provided at any time of the day or night.

Lockmaster Doyle Jennings and personnel at O'Brien are acknowledged for assisting USGS personnel during the leakage measurements at O'Brien and also for providing space for equipment storage. Jim Vey, of the MWRD, assisted by providing stage data collected during each of the measurements. MWRD dispatchers also assisted the USGS by making the required changes in gate settings at each of the three control structures.

DESCRIPTION OF CONTROL STRUCTURES

Leakage measurements were made near three different control structures: the CRCW (which includes the Chicago Lock), O'Brien, and Wilmette. A description of each of these structures is given below.

Chicago River Controlling Works

The CRCW consists of the Chicago Lock and two sets of sluice gates located in the harbor walls surrounding Chicago Harbor. The CRCW and Chicago Harbor are located just south of Navy Pier, close to the original mouth of the Chicago River (fig. 2).

The CRCW was built in 1938 as a part of the construction of the Chicago Sanitary and Ship Canal and the diversion of the Chicago River. The harbor walls were constructed of rock-filled timber cribs and, in places, the walls have metal sheet piling on one or both sides of the wall. During 1987, the tops of most harbor walls were raised and capped with concrete to reduce overtopping by waves in Lake Michigan. As a result of the original construction technique, the walls are porous. Figure 3 shows leakage through the harbor wall on the northeast side of CRCW.

The Chicago Lock is located at the far eastern end of the harbor walls, about 1,800 ft east from the Lake Shore Drive bridge (fig. 2). The lock (fig. 4) is 600 ft long and 80 ft wide and also was constructed using rock-filled timber cribs with sheet piling and a concrete facade extending below the water line. Triangle gates, located at either end of the lock, are used to seal the lock chamber. Water levels in the lock chamber are raised or lowered by gradually opening the triangle gates. The Corps is responsible for the operation and maintenance of the Chicago Lock.

Two sets of sluice gates were constructed in the harbor wall: one located near the lock and another on the south harbor wall (fig. 2). A transverse section of a typical sluice gate at CRCW is shown in figure 5. The elevation of the concrete sill at the base of both sets of sluice gates is -16.0 ft Chicago City Datum (562.48 ft above sea level). Each sluice gate at CRCW is 10.5 ft wide and 10.5 ft high. During the summer, the sluice gates along the south harbor wall are opened to allow Lake Michigan water to flow into the Chicago River and down the CSSC to improve water quality in the CSSC. The sluice gates near the lock are used only for flushing ice down the Chicago River, for flow reversals, and when the south gates are inoperable. During storms, when the water-surface elevation of the Chicago River exceeds the elevation of Lake Michigan, both sluice gates may be opened to allow flow from the Chicago River to discharge into Lake Michigan. However, the sluice gates are used only infrequently for this purpose. The MWRD is responsible for the operation and maintenance of the sluice gates.

Thomas J. O'Brien Lock and Dam

O'Brien is located at river mile 326.5 (miles above the mouth of the Illinois River) on the Calumet River (fig. 6), about 7 river miles south from Lake Michigan near Burnham, Ill. A plan view of O'Brien is shown in figure 7. O'Brien was constructed by the Corps in 1959 with techniques similar to those used at CRCW. The lock at O'Brien is 110 ft wide and 1,200 ft long and provides access from Lake Michigan to the Calumet Sag Channel, the CSSC, and the Illinois River. Streamflow is regulated at the dam by use of combinations of the four sluice gates. The sluice gates at O'Brien are 10 ft wide and 10 ft tall and are constructed similar to those at CRCW (fig. 5). The concrete sill of the sluice gates has an elevation of -13.00 ft Chicago City Datum. The sluice gates at O'Brien also have the same dual purpose as at CRCW. They are used during the summer to divert water to improve the water quality in the Calumet Sag Channel and the CSSC. The sluice gates are opened during periods of high runoff to allow water to flow toward Lake Michigan in order to minimize flood-water damage along the Calumet Sag Channel and its tributaries. The Corps of Engineers, Rock Island District, is responsible for the operation of the lock and dam.

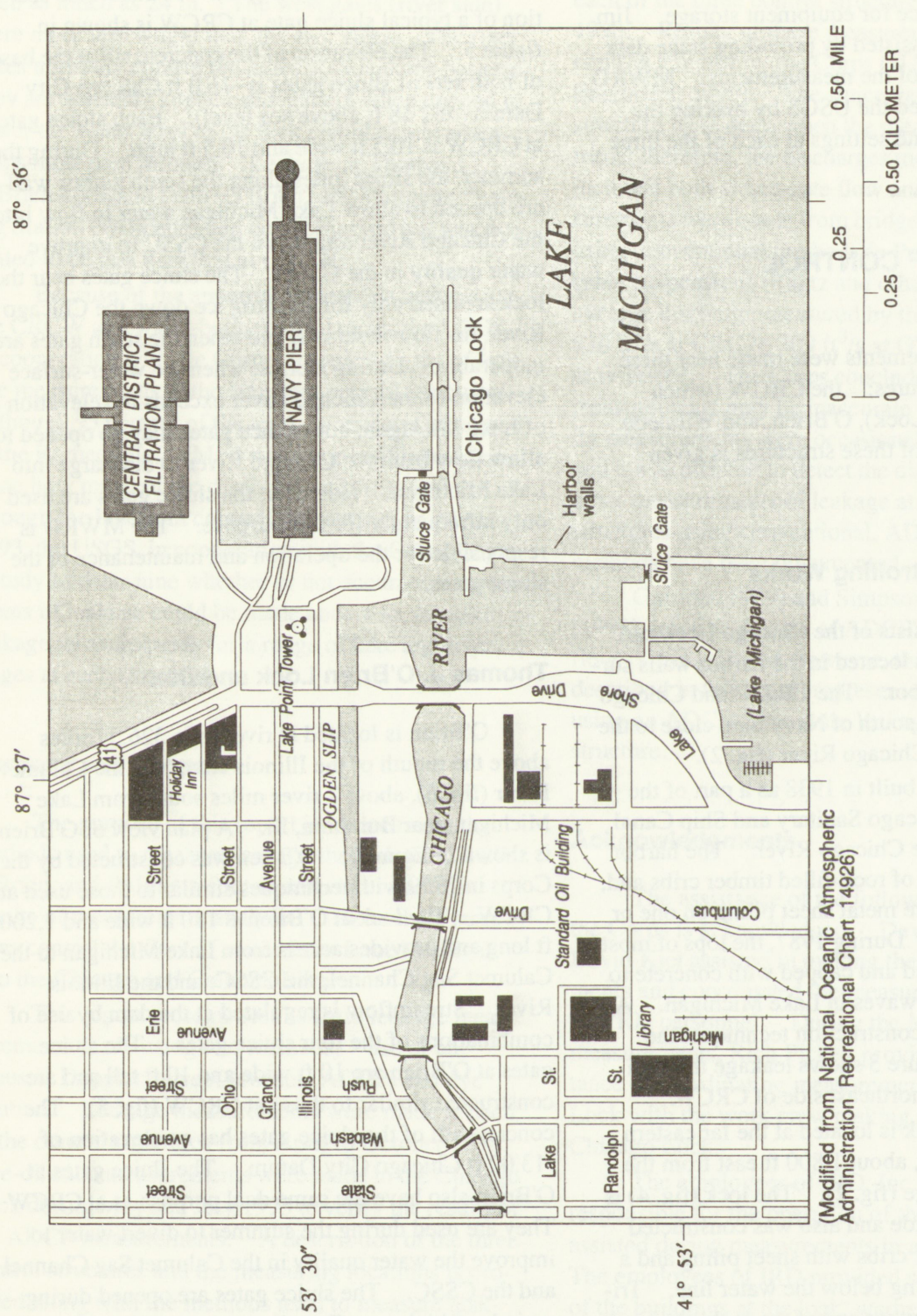


Figure 2. Location of the Chicago River Controlling Works, the Chicago Lock, and the Chicago River, Chicago, Ill.

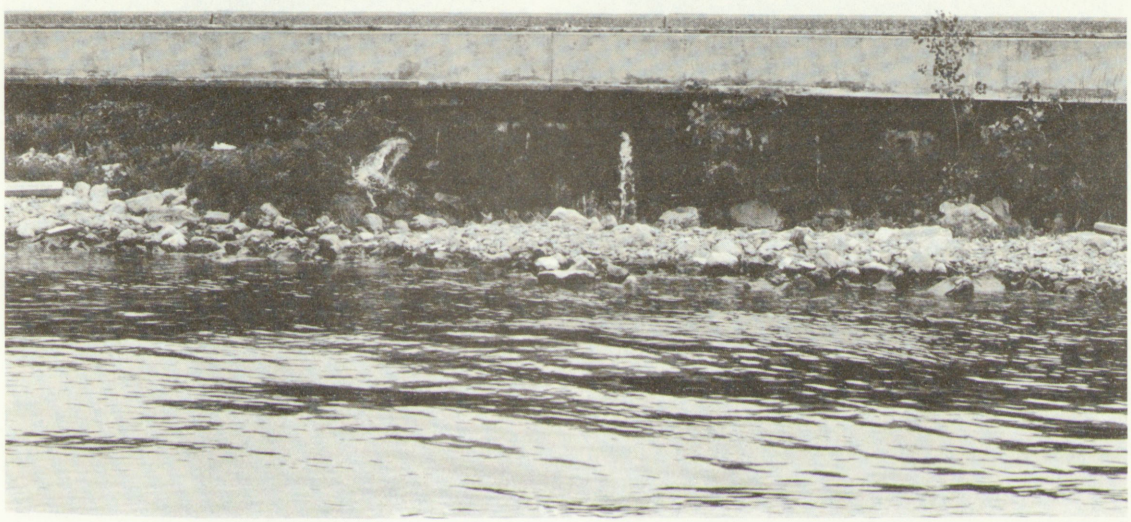


Figure 3. Leakage through southeast harbor wall at Chicago River Controlling Works, view looking east, July 10, 1993.



Figure 4. Chicago Lock, Chicago, Ill., looking east to west, September 1993.

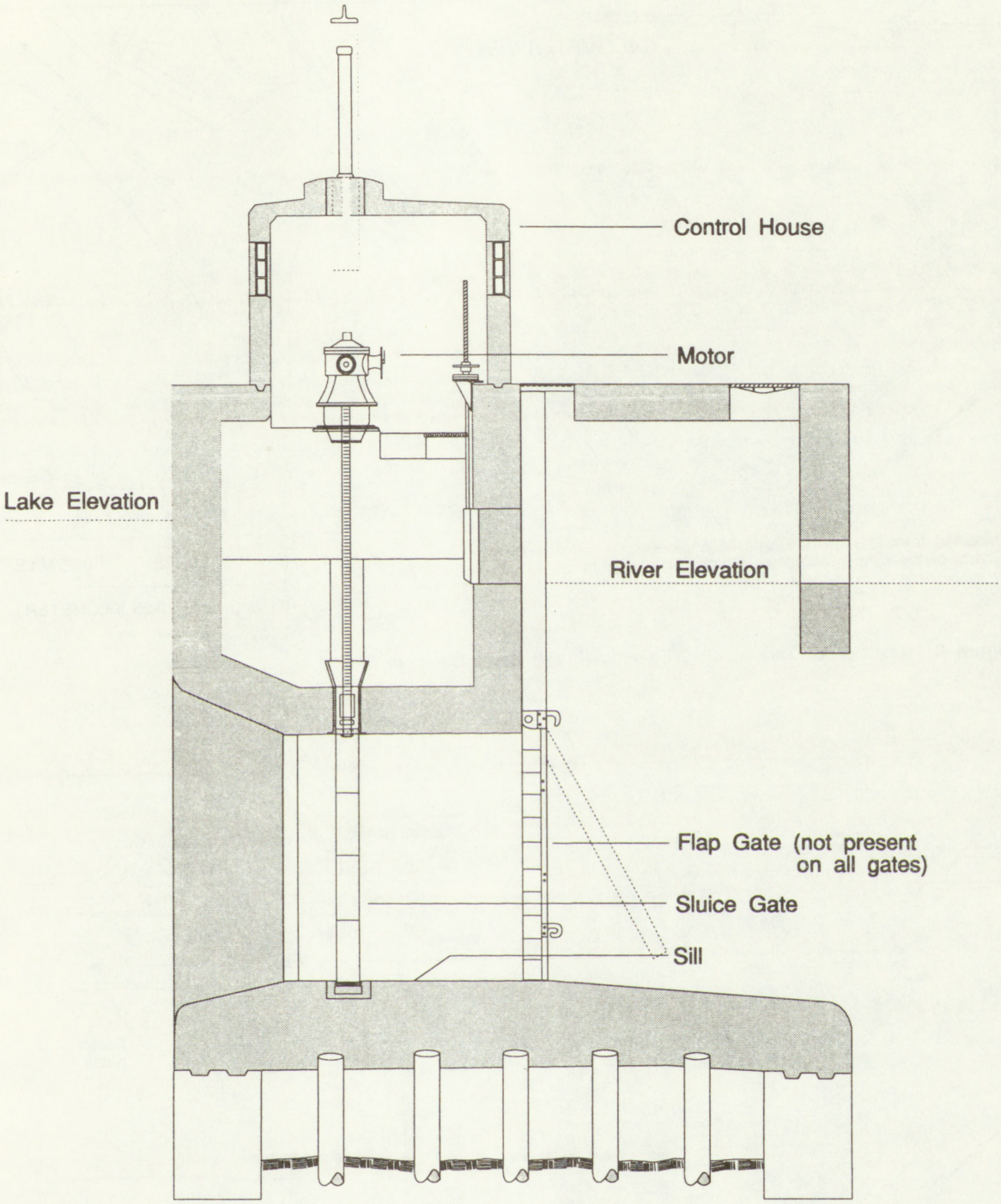


Figure 5. Typical sluice-gate configuration at Chicago River Controlling Works and Thomas J. O'Brien Lock and Dam.

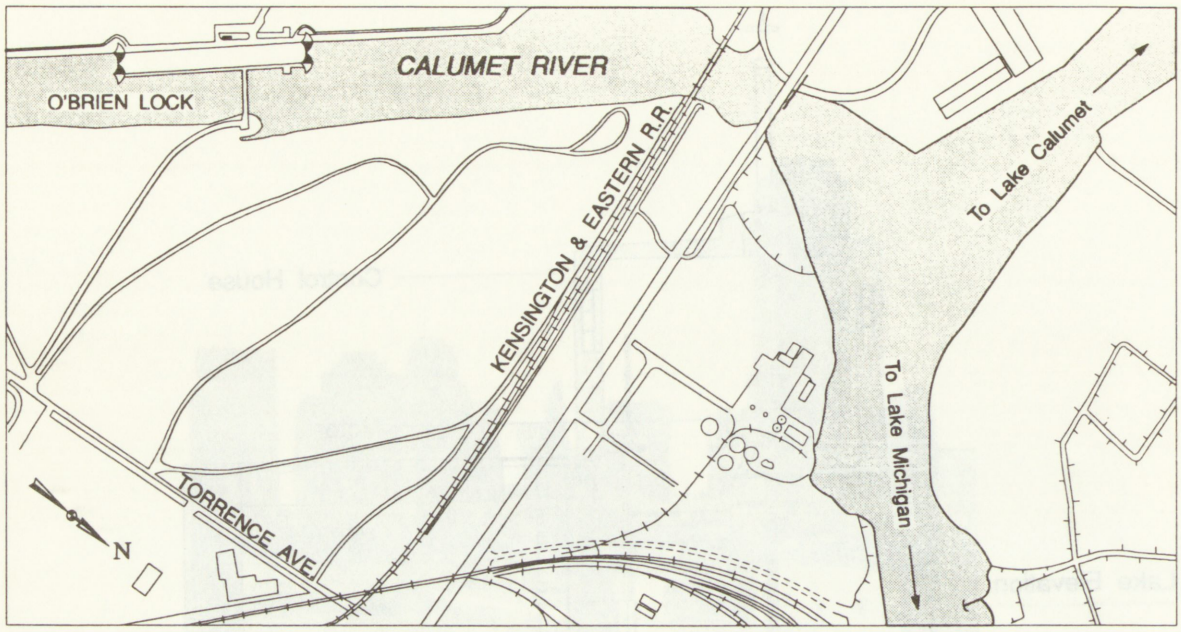


Figure 6. Location of Thomas J. O'Brien Lock and Dam, Burnham, Ill.

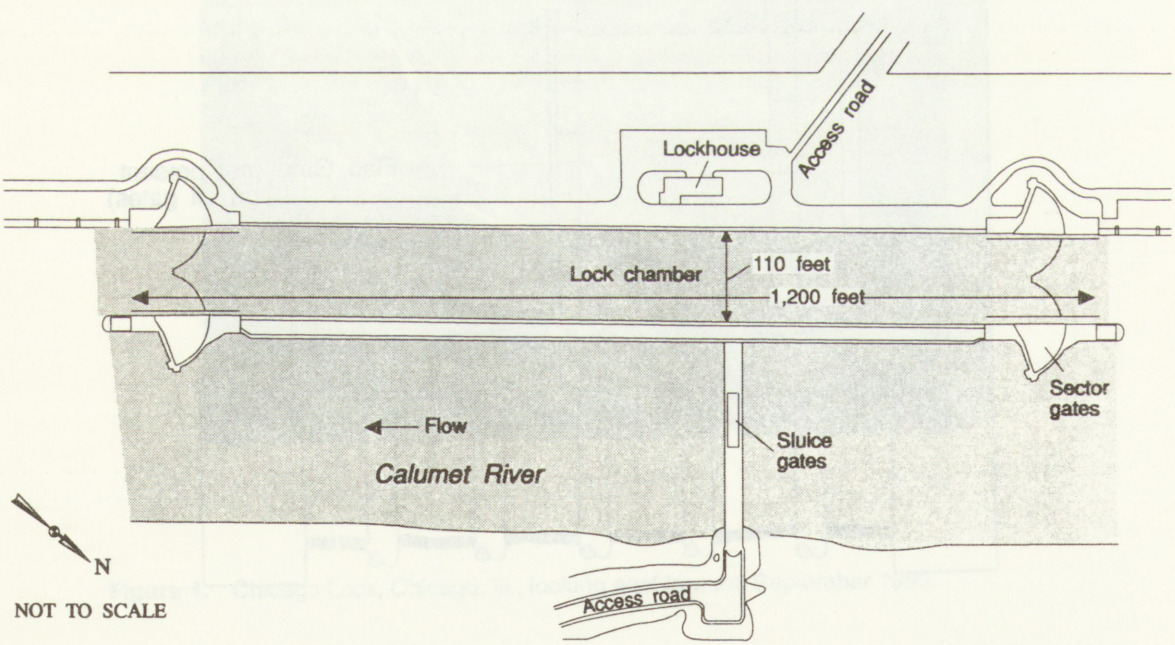


Figure 7. Thomas J. O'Brien Lock and Dam, Burnham, Ill.

The MWRD, however, has the responsibility for determining the settings for the sluice gates at O'Brien.

Wilmette Pumping Station

In 1910, the 8-mi-long North Shore Channel was constructed and connected to the CSSC (fig. 1) to carry wastewater away from north shore communities near Chicago and to provide additional Lake Michigan water for dilution of the wastewater. MWRD also constructed a pumping station and a small lock in 1910 where the North Shore Channel intersects Lake Michigan (fig. 8). At the time of construction and until the completion of CRCW in the late 1930's, the water level in the North Shore Channel was higher than Lake Michigan. The pumping station, located under the Sheridan Road bridge, was used to pump Lake Michigan water into the North Shore Channel in order to create enough head to convey wastewater down the Chicago River and the CSSC. The pumps at the pumping station were not used after the completion of the part of the Deep Tunnel project under the Chicago River (Robison, 1986). In July 1993, MWRD removed the pumps and sealed the pump bays to reduce leakage.

During the 1970's, MWRD removed the lock gates at Wilmette and installed a sluice gate at the downstream end of the lock (fig. 9). The Wilmette sluice gate is structurally somewhat different from the sluice gates at CRCW and O'Brien, but functions similarly. As at O'Brien and CRCW, the sluice gate is raised during the summer to improve water quality in the North Shore Channel and the CSSC. During periods of extremely high flow, water will sometimes overtop the lock gate at Wilmette and flow into Lake Michigan. The MWRD is responsible for the operation and maintenance of Wilmette.

MEASUREMENT METHODS

As previously mentioned, the Corps measured discharge downstream from each of the three control structures in July 1990. The discharges measured by the Corps were 352 ft^3/s greater than flows computed from MWRD sluice-gate ratings for the same period (U.S. Army Corps of Engineers, 1990, p. 25). On the basis of these measurements and other information available to the Corps, the leakage at CRCW was estimated to be between 200 and 400 ft^3/s. It is likely that the amount of leakage at CRCW and other control structures varies with the stage difference between Lake Michigan and the Chicago River. If the leakage at CRCW is assumed to be 200 ft^3/s and the cross-sectional-flow area of the Chicago River at Lake Shore Drive is approximately 5,000 ft^2, the average velocity would be 0.04 ft/s.

Because Price AA meters cannot accurately measure velocities less than 0.25 ft/s, other methods for measuring leakage were necessary. Few instruments are able to accurately measure water velocities less than 0.04 ft/s. Price AA meters have been fitted with an optic head to increase the accuracy of low-flow velocity measurements. However, optic-head meters are not rated for velocities less than 0.10 ft/s and cannot be used to determine flow direction, except at the surface. An acoustic or electromagnetic velocity meter is necessary to measure velocities less than 0.10 ft/s and to determine flow direction. However, acoustic or electromagnetic meters have the disadvantage of measuring point velocities only; therefore, measurements cannot be made any faster than with a Price AA. The ADCP was considered advantageous because it can quickly obtain measurements of discharge from a moving boat. In April 1993, therefore, trial leakage measurements were made with an ADCP. Results of the trial measurements indicated that an ADCP could be used to measure leakage at each of the three control structures.

Several dye-dilution leakage measurements were made at the Chicago Lock in order to evaluate the results from simultaneous ADCP measurements. Because of the poor measuring conditions, the dye-dilution measurements cannot provide a strict verification of ADCP-measured leakage. However, the dye-dilution measurements provide an approximate check on ADCP-measured discharges. The two methods for measuring leakage are described in the following sections.

Acoustic Doppler Current Profiler

ADCP's have been in use for more than 10 years. They have been used primarily in the study of ocean currents and estuaries. Within the last 5 years, ADCP's have been used to measure streamflow, especially in rivers or canals where conventional discharge-measurement techniques are either very expensive or impossible to apply. Recently, a more advanced ADCP has been developed to measure

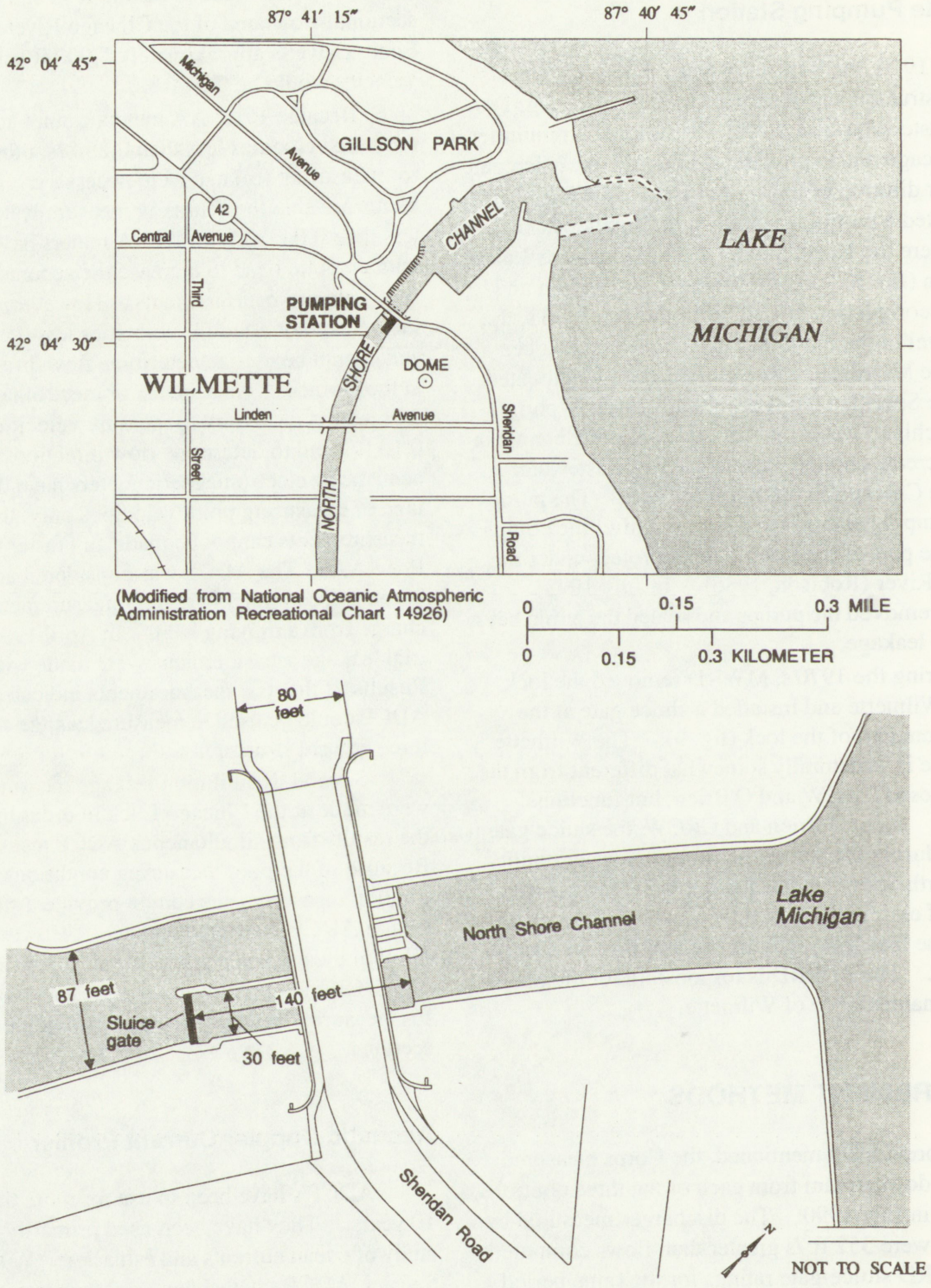


Figure 8. Wilmette Pumping Station, Wilmette, Ill.



Figure 9. Wilmette Pumping Station, Wilmette, Ill., view looking northeast, September 1993.

discharge in shallow water (as shallow as 5 ft) and with a greater vertical resolution. This type of instrument, called a Broadband ADCP (hereafter referred to as an ADCP), was used for these measurements. A brief description of how an ADCP is used to measure discharge is provided below. For a detailed description of the ADCP and its application to streamflow measurement, the reader is referred to RD Instruments (1989, 1993), Gordon (1989), and Simpson and Oltmann (1993).

An ADCP measures vertical profiles of water velocities from a moving boat. Water velocities are measured by the ADCP using transducers that transmit short, phase-encoded acoustic pulses along four narrow beams at a known, fixed frequency (from 75 to 1,200 kilohertz (kHz), depending on the transducer). These beams are positioned 90 degrees apart horizontally and at a known angle (usually 20 degrees) from the vertical. The ADCP detects and processes the echoes reflected by suspended material from successive volumes (depth cells) along the beams and determines the time-lag change and frequency shift. The time-lag change and difference in frequency (shift) between transmitted and reflected sound is proportional to the relative velocity between the ADCP and suspended material in the water that reflects the beam back to the ADCP (back scattering). This frequency shift is known as the Doppler effect. An autocovariance method is used to compute the mean value or first moment of the Doppler frequency. Using simple trigonometry and water velocities calculated from adjacent beams, the ADCP can compute water speed and direction. The size or height of the depth cells can be set by use of ADCP software parameters. Most of the measurements described in this report were made using 9.8 in. (0.25 m) depth cells.

Because water-velocity measurements are made relative to the movement of a boat, the ADCP also must measure the velocity of the boat. This process is referred to as bottom tracking. The boat velocity relative to the river bottom is computed by the ADCP using a flux-gate compass and the results of measurements of the Doppler shift of acoustic pulses reflected off of the river bottom. Bottom-tracking measurements can be made with greater accuracy than water-velocity measurements because a longer pulse is used for bottom tracking and return echoes from the bottom are much stronger than echoes from most particulates suspended in the water column. In addition to measuring boat velocity, the depth of the river is estimated by use of the amplitude of the bottom-track echoes (echoes returned from the bottom).

When the ADCP is used to measure discharge, a series of acoustic pulses known as pings are transmitted. Pings for measuring water velocities are referred to as water pings; pings for measuring the boat velocity are referred to as bottom-tracking pings. Normally, water and bottom-tracking pings are interleaved during transmission. A group of interleaved water and bottom-tracking pings is referred to as an ensemble. The number of water and bottom-tracking pings per ensemble is set by the user. An ensemble is analogous to one vertical in a conventional discharge measurement. Most leakage measurements described in this report were made with five water pings and four bottom-tracking pings. Some measurements were made with 10 water pings and 9 bottom-tracking pings. Increasing the number of pings per ensemble slows down the rate at which the ADCP makes measurements but does not necessarily increase measurement accuracy. A single discharge measurement, called a transect, is a collection of ensembles for a measuring section. A single transect typically will contain 50-60 ensembles.

The ADCP cannot measure water velocities near the top and bottom of the water column. Water velocities near the surface cannot be measured for two reasons. The ADCP must be deployed so that the transducers remain under water during the course of a measurement (fig. 10). In addition, the physical characteristics of the transducers are such that accurate velocity measurements cannot be made within at least one depth cell away from the transducer. With 9.8-in. depth cells, usually no velocity measurements are made within 20 in. of the transducers. The first depth cell for the measurements described in this report ranged from 2.4 to 5.5 ft below the water surface.

Water velocities near the bottom of the water column cannot be measured because of a phenomenon known as side-lobe interference. The return signals from particulates near the bottom are distorted by echoes from the riverbed directly below the ADCP (see figure 10). This distortion occurs because the strong bottom reflection from a weak side-lobe signal overwhelms the weak backscattered reflection of the strong main beam signal. Gordon (1989, p. 929) and Simpson and Oltmann (1993, p. 6) provide more details on side-lobe interference and its effect on water-velocity measurements. For a 1,200 kHz

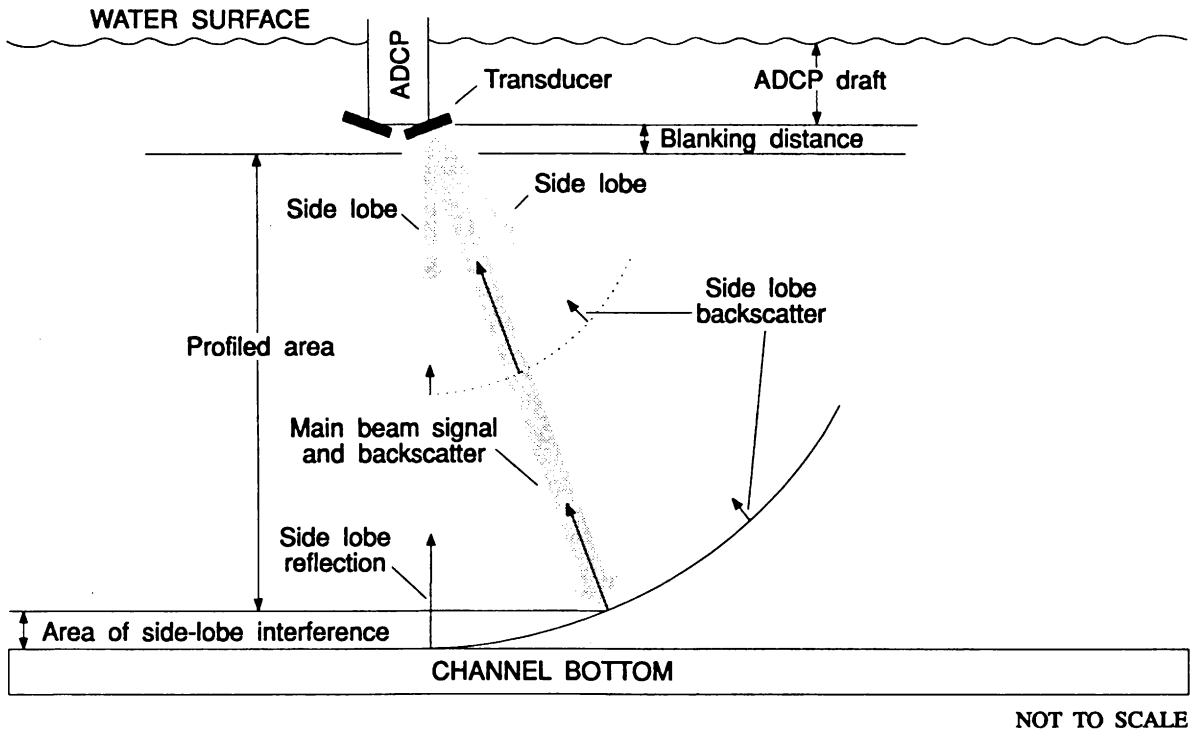


Figure 10. Beam pattern of acoustic Doppler current profiler (ADCP), showing side-lobe interference.

ADCP with a 20-degree beam angle, about 6 percent of the profiling range is lost because of side-lobe interference. The ADCP software automatically rejects water-velocity data beyond about 94 percent of the distance to the bottom.

Water velocities near the surface or near the bottom of the water column can be estimated by the constant-velocity method or the power-law method. With the constant-velocity method, the velocity at the surface or the bottom is assumed to be equal to the velocity of the first or last measured depth cell, respectively. The constant-velocity method is conceptually better for estimating velocities near the surface than the power-law method, because, in open-channel flow, the velocity approaches a constant value near the surface. However, the constant-velocity method is considered inappropriate for estimating the velocities near the bottom because typical vertical-velocity distributions for open-channel flow are not accurately represented. In open-channel flow, the velocity approaches zero near the bottom. The power-law method is based on the power law for velocity distribution presented by Chen (1989, 1991). With the power-law method, a least-squares fit of the measured water velocities is obtained using the power law for velocity distribution. The exponent of the power law can be selected by the user. The exponent is typically set to 1/6, based on the 1/6 power law suggested by Chen (1989). Chen has shown that Manning's formula is essentially the same as the 1/6 power law. The power law is then used to estimate velocities in the unmeasured parts of the water column. Conceptually, the power-law method is better than the constant-velocity method for estimating the unmeasured portion of the water column near the bottom. In this study, the constant-velocity method was used to estimate the unmeasured portion of the water column near the surface, and the power method with an exponent of 1/6 was used to estimate the unmeasured portion of the water column near the bottom.

The ADCP cannot measure water velocities near the edges of the section. This is primarily because the ADCP cannot accurately measure velocity in shallow water. ADCP water-profiling mode 2 or 4, used in this study, requires a 4.7-ft minimum depth for velocity measurement. A new water-profiling mode (currently being tested) will allow ADCP measurements in water as shallow as 3 ft. If the unmeasured discharge section is assumed to be triangular, the estimated velocity for the unmeasured section, V_e, is

estimated by Simpson and Oltmann (1993, p. 9) with the equation

$$V_e = 0.707 V_m, \quad (1)$$

where V_m is the mean velocity at the first or last ADCP-measured subsection. The assumption that the unmeasured flow area is triangular is reasonable for many river cross sections where the bottom gradually slopes upwards toward the edge of water.

Sometimes the edge of water is a vertical wall such as a sea wall. Water velocities also cannot be accurately measured by the ADCP near a vertical wall. For example, assume that the acoustic signals transmitted by the ADCP are sent out at a 20-degree angle from the vertical. As the ADCP approaches a vertical wall, the acoustic beam will impinge on the wall and cause a false bottom return. The ADCP will then calculate flow for a depth shallower than the actual depth at that location. The horizontal distance at which the acoustic beam impinges on the vertical wall depends on the depth of water near the vertical wall and the orientation of the transducers on the ADCP relative to the wall. For example, the beam will impinge on the wall sooner if two of the transducers are oriented perpendicular to the wall than if the transducers are oriented at a 45-degree angle to the wall. Most measuring locations in this study had at least one vertical wall in the measuring section. For most measurements described in this report, the transducers were oriented at 45 degrees to the wall. With this orientation, it was determined that for measurement locations, the ADCP could be brought within 8 ft of the vertical wall. Velocities for the unmeasured edge sections could then be estimated by setting $V_e = V_m$; however, this is not entirely accurate because the velocity decreases to zero near the wall. Therefore, for the measurements described in this report, velocities near vertical walls were estimated by the equation

$$V_e = 0.91 V_m. \quad (2)$$

The coefficient of 0.91 was estimated from data presented in Rantz and others (1982, p. 82) showing the relation between mean velocity in the vertical and distance from a smooth wall expressed as a ratio of the depth.

Estimates of V_e are sensitive to the value of V_m obtained from the first of last ensemble, and therefore the estimates of V_e may have unrealistic values. The result of velocity measurements from multiple ensembles may be averaged in space or time (referred to as ensemble-averaging). Sometimes the average velocity measured for the last ensemble has a sign that is opposite to the predominant flow direction. Ensemble-averaging is done so that a more consistent estimate of edge discharge can be obtained, rather than estimating the edge discharge based on the average velocity from a single ensemble. Typically, three to four ensembles were averaged, especially near the edges. For measurements made in the Chicago River, the velocity measurements from individual ensembles were averaged laterally (across the section) every 6 ft. The resulting V_m was used to estimate V_e. At all other locations, ensemble-averaging was performed every 3 ft and was used to estimate V_e. The edge discharge may then be estimated using the equation

$$Q_e = (C\ V_e\ L\ d_m), \quad (3)$$

where

Q_e = estimated edge discharge,

C = 0.707 for triangular-shaped edge section, 0.91 for vertical wall,

L = distance to the shore from the first or last ADCP-measured subsection, and

d_m = depth at the first or last ADCP-measured subsection.

The variable d_m is measured by the ADCP; L is measured using a tagline or a similar measuring device.

ADCP measurements were made at each location for a range of lake- and river-level conditions. Lake Michigan levels usually are highest during July and are lowest during February. Water levels in the Chicago and Calumet Rivers are relatively constant throughout the year. Immediately prior to a storm, river levels will decrease initially in response to increased flow at Lockport. Then, river levels will increase in response to rainfall and runoff. Occasionally, during large storms, levels in the Chicago and Calumet Rivers will be higher than in Lake Michigan. In order to measure leakage, ADCP measurements were made during dry periods when the river levels were not changing rapidly during the measurements.

Two ADCP's were used for making the leakage measurements described in this report. Both ADCP's were equipped with 1,200-kHz transducers mounted 20 degrees from the vertical. The ADCP used to

make measurements in April and May 1993 was equipped with firmware version 3.20. The ADCP used to make measurements in July, September, and October 1993 was equipped with firmware version 3.90. The main difference between firmware versions is that a new water-profiling mode was added and problems with ambiguity errors were corrected in the latter version (J. R. Marsden, RD Instruments, oral commun., 1993). The two ADCP's were otherwise identical. A metal bracket was used to suspend the ADCP from the boat and hold it steady in the water. During the April and May ADCP measurements, the bracket used contained ferrous metal. The flux-gate compass in the ADCP is affected by nearby ferrous metal. The result is that velocity directions computed by the ADCP are offset by some constant amount, provided that the ferrous metal object does not move relative to the ADCP. This, however, does not invalidate the discharges measured by the ADCP (J. R. Marsden, RD Instruments, oral commun., 1993). An aluminum bracket was used to hold the ADCP in all subsequent leakage measurements.

Normally, streamflow measurements using an ADCP are made from a moving boat without the use of a tagline. The average velocity in the Chicago River and at the Chicago Lock was estimated to be as low as 0.04 ft/s. Water velocities were presumed to be similar at Wilmette and O'Brien. Initial attempts to measure leakage at CRCW indicated that a trolling motor moved the boat too fast across the channel. In order to obtain accurate, consistent ADCP measurements, it was necessary to cross the channel with a boat speed approximately equal to the average water velocity. A tagline was used to slowly pull the boat by hand from one shore to the other in order to obtain the proper rate at which the boat crossed the channel. During measurements made in July-September 1993, a windlass winch was mounted on the boat and was used to pull the boat across the measuring section at a relatively constant speed. The windlass assembly is shown in figure 11.

Dye Dilution

Dye-dilution methods to measure discharge are based on a mass-balance calculation for the flow (Kilpatrick and Cobb, 1985). A harmless, fluorescent dye, known as rhodamine WT, was used for the dye injections described in this report. This section describes a continuous, steady-state dye injection into a steady, uniform flow at the Chicago Lock. For these conditions, the dye concentration at the sampling section will rise with time until a steady plateau concentration is reached (Kilpatrick and Cobb, 1985, p. 4-5). The time required to reach the plateau concentration depends on the dispersion in the flow. The dye concentration should remain steady once the plateau concentration is reached, provided the injection rate and concentration and the flow rate do not change. For this situation, the discharge can be calculated by use of the equation

$$Q + i = k\frac{iC_i}{C_s}, \tag{4}$$

where

Q is the discharge in the stream,

i is the injection rate of the dye,

C_i is the concentration of the injected dye solution,

C_s is the concentration of dye in the samples from the flow, and

k is a constant to convert to a consistent system of units.

As the injection rate is far less than the discharge in the stream, this equation can be simplified to

$$Q = k\frac{iC_i}{C_s}. \tag{5}$$

For i, measured in milliliters per minute, and Q, measured as cubic feet per second, the units-conversion constant

$$k = 5.885 \times 10^{-7}\ \frac{ft^3/min}{mL/s}.$$

Most applications of dye-dilution methods to measure discharge involve highly turbulent flows where mixing is not a problem. The application discussed in this report involved the use of dye-dilution methods to measure small discharges through the closed gates of the Chicago Lock between Lake Michigan and the Chicago River. An initial attempt to measure discharge by dye dilution in May 1993 proved unsuccessful because of inadequate mixing. Therefore, a manifold had to be designed to mix the dye in the flow.

The dye-injection apparatus used in this study was planned to provide continuous, uniform mixing of a small volume of dye in an open channel 80 ft wide

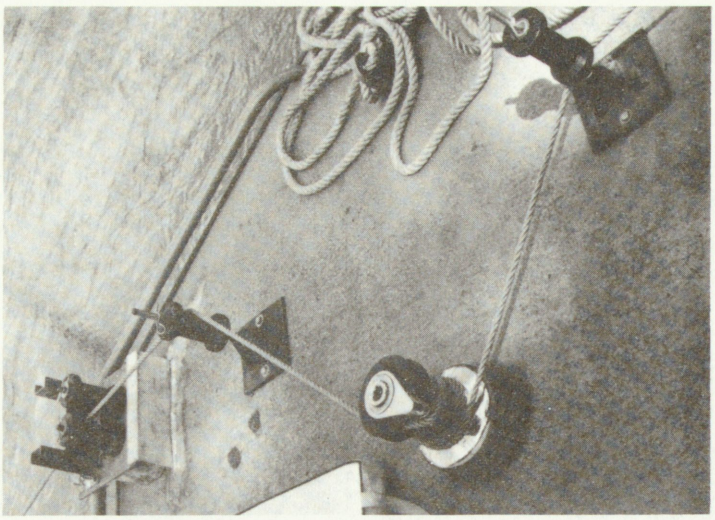




Figure 11. Boat and windlass used to move boat across a measurement section.

and 25 ft deep. The dye was metered into the outlet of a centrifugal pump with a constant-rate, positive-displacement metering pump. The centrifugal pump took water from outside the Chicago Lock and pumped it, together with the injected dye, through a manifold into the lock. The purpose of the centrifugal pump was to provide enough flow through the manifold to mix the dye solution through the entire cross section. The metering pump controlled the dye-injection rate, resulting in a steady injection of dye into the flow despite any fluctuations in the pumping rate of the centrifugal pump. The dye was pumped from a 6-L holding tank with a scale indicating the volume of dye remaining. Measurements of the volume of dye in the holding tank at intervals throughout the injection indicated the injection rate and any variance. Figure 12 shows the dye-injection apparatus.

The manifold was designed of 2-in. diameter polyvinyl-chloride pipe suspended in the lock about 50 ft west from the lake gate and about 1 ft above the bottom of the lock. The manifold had ports pointing upwards and spaced at 5-ft intervals along the length of the manifold. Theoretical mixing characteristics of each jet from the manifold are listed in table 1 and computed from Fisher and others (1979). At depths greater than 4 ft, the flow from adjacent jets should overlap. The diameter of the ports was increased with distance along the manifold to maintain a consistent flow from all ports. The total area of all the ports was 40 percent of the cross-sectional area of the pipe.

Table 1. Theoretical characteristics of jets from dye-injection manifold used in the Chicago Lock, July 13-15, 1993

[ft, feet; <, less than

Depth below water surface (ft)	Width of jet (ft)	Dilution factor	Percentage of channel width included in jet
0	6.1	260	64
2	5.6	240	63
6	4.6	190	52
10	3.6	150	40
14	2.5	110	28
18	1.5	65	17
22	.5	22	6
24	.03	0	< 1

The manifold was modified from the original design because of mechanical difficulty in joining adjacent sections of the manifold. The manifold was 45 ft long with 9 ports rather than the original at 75 ft long with 17 ports. The manifold was centered in the channel less than 1 ft above the channel bottom. Figure 13 shows the manifold suspended 2-3 ft below the water surface and the resulting jets at the water surface.

Dye standards were prepared from the dye lot used for the injection solutions. Five cuvettes were used for the fluorometric analysis for dye concentrations. All of the cuvettes were used for blank (deionized water) samples as part of calibrating the fluorometer to ensure that the fluorescence did not vary among cuvettes. The fluorometer was calibrated by analyzing six to nine standard solutions prior to analyzing any water samples. The fluorometer was recalibrated after the samples were analyzed. If the analyses extended over several hours, an extra calibration was done part way through the analyses. Fluorometer-calibration curves were estimated using ordinary-least-squares regression. The fluorometer-calibration readings are listed in appendix 1. The calibration curves are listed in table 2.

Samples were collected 500 ft downstream from the injection location at four locations across the channel and at three depths at each location. Samples were collected at depths of 6, 12, and 18 ft with a Kemmerer bottle to isolate samples from the different depths. Samples were collected 10, 30, 50, and 70 ft from the left edge of the lock (facing west). The average concentration in each vertical was calculated by two methods: (1) the mean of the three samples from that vertical and (2) the depth-weighted average of the samples. The depth-weighted average for each vertical was calculated by dividing the vertical into "bins" represented by each sample (fig. 14). The concentration of the sample is weighted by the vertical size of the bin, and the total for all samples is divided by the total depth of the vertical.

Each day, one set of samples was collected prior to starting the dye injection. These samples, referred to as background samples, were used to determine the background fluorescence of the water flowing through the lock. Samples to determine the steady dye concentration at the plateau of the concentration curve were collected at about 1/2-hour intervals starting

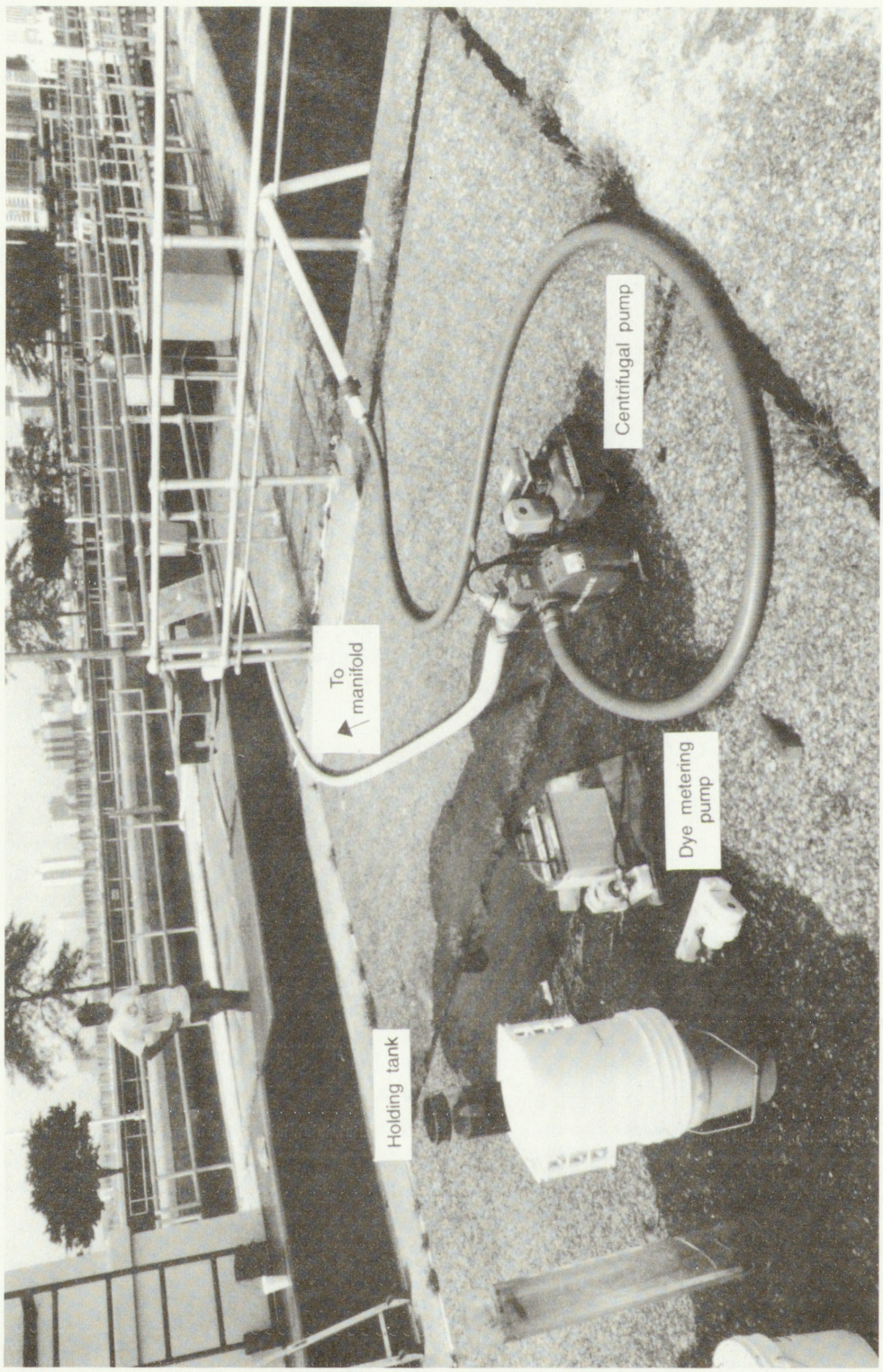


Figure 12. Apparatus used to inject dye mixture into the Chicago Lock, July 13-15, 1993.

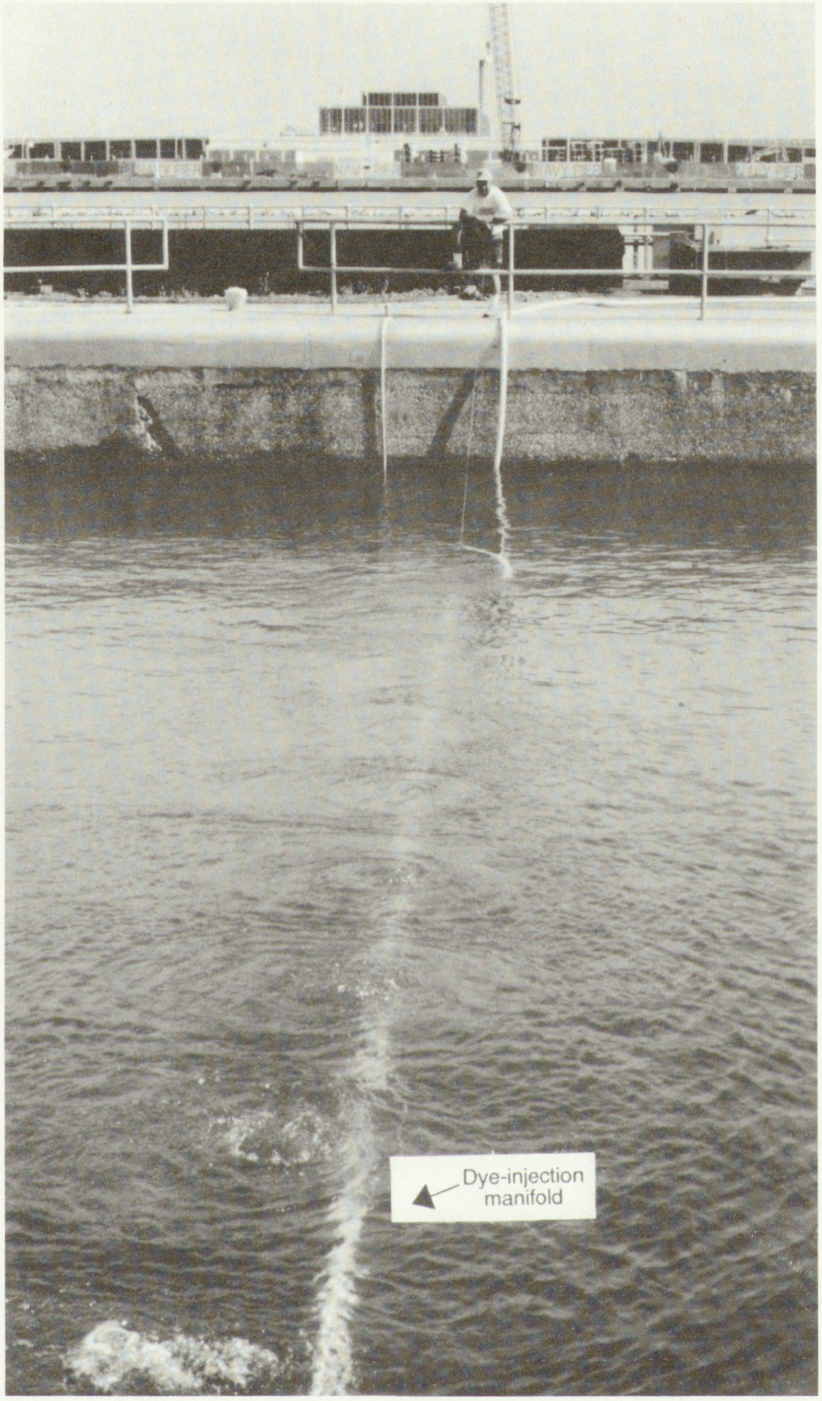


Figure 13. Dye-injection maniforld in Chicago Lock, view looking north, July 13-15, 1993.

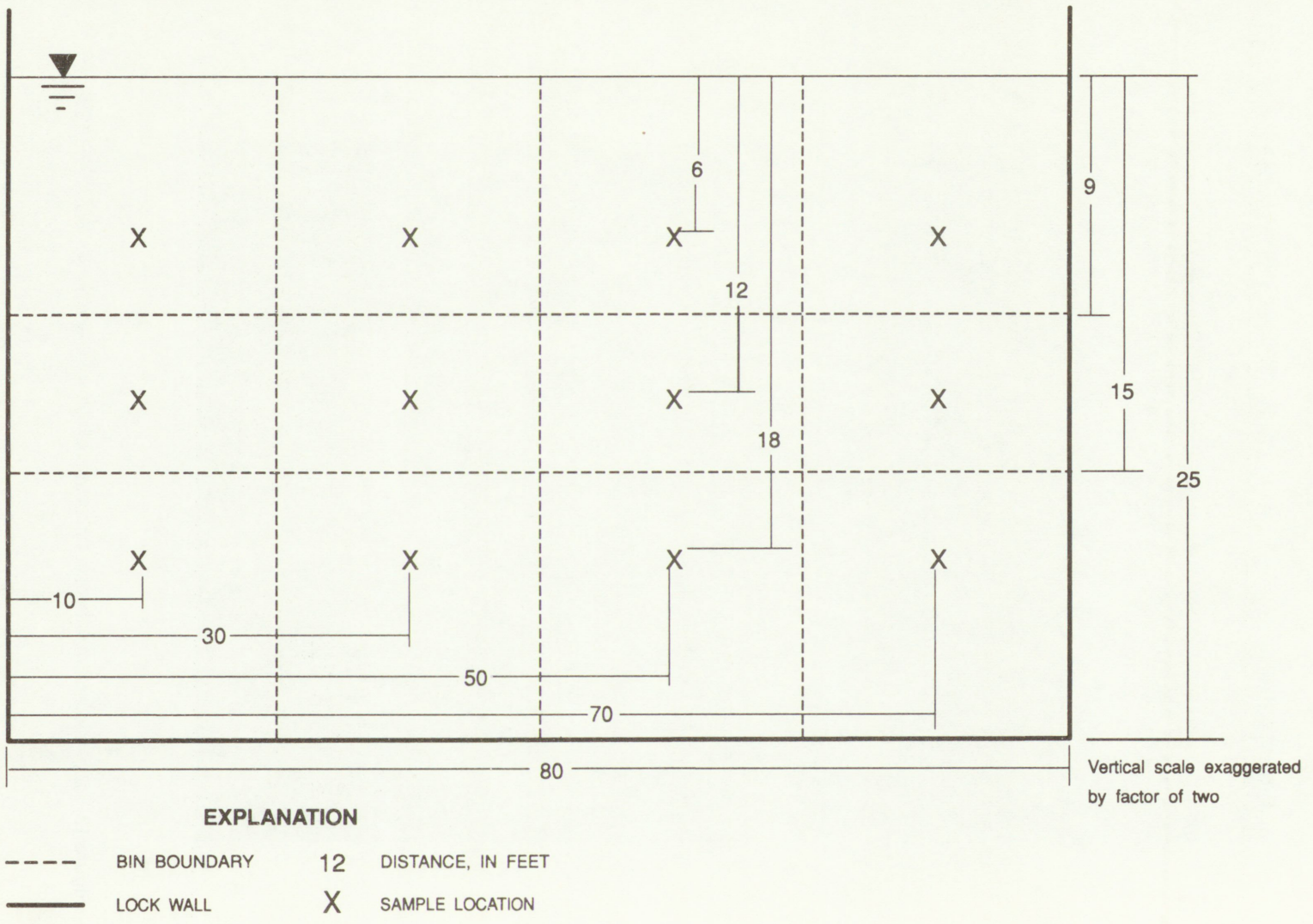


Figure 14. Generalized cross section of Chicago Lock channel showing the sample locations and bins used to calculate depth-weighted average dye concentrations.

Table 2. Calibration curves for fluorometric analyses of dye samples collected in the Chicago Lock, July 13-15, 1993

[ppb, parts per billion; ppb/unit, part per billion per unit fluorometer reading; calibrations are based on linear regression of the concentrations of standard solutions with the fluorometer readings for those standards (listed in appendix 1). These calibrations give the concentration in parts per billion for a given fluorometer reading.]

Calibration number	Calibration date	Calibration time (hours)	Intercept (ppb)	Slope (ppb/unit)	Fluorometer reading for zero concentration	Correlation coefficient	Number of samples	95-percent confidence interval for slope	
								Lower	Upper
1	July 13	1535	0.096	1.349	-0.071	0.999	9	1.321	1.378
2	July 13	1658	-.203	1.382	.147	.999	8	1.343	1.422
2.5	July 13	1818	-.177	1.389	.127	.999	8	1.354	1.423
3	July 14	1036	-.373	1.331	.280	1.00	8	1.304	1.358
3.5	July 14	1513	-.269	1.384	.194	.999	7	1.337	1.430
4	July 15	0930	-.066	1.281	.052	.999	7	1.245	1.317
5	July 16	0912	.228	1.473	-.155	.999	7	1.427	1.518
5.5	July 16	1101	-.121	1.559	.078	.999	6	1.511	1.607

about 1.5 times the estimated traveltime between the injection and sampling locations.

The dye concentration of each sample was calculated by multiplying the fluorometer reading by the slope of the regression line and adding the constant from the regression line for the calibrations before and after the sample was analyzed. The two resulting concentrations were weighted by the time between the calibration and the analysis, and the time-weighted average listed as the concentration for that sample. Dye samples are listed in appendix 2.

Quality-assurance samples were collected regularly to verify the accuracy of the sampling and analytical techniques. Split samples were collected by filling two sample bottles from the same sample of water. Split samples identify the variance in the sample analysis. Replicate samples were obtained by collecting two or more samples from the same point at about the same time. Replicate samples identify the variance in the sampling. Blank samples were collected by filling a sample bottle with distilled water and then handling the bottle exactly like the other samples. Blank samples identify contamination of the samples from handling and processing.

MEASUREMENTS OF LEAKAGE

Leakage measurements were made at CRCW, O'Brien, and Wilmette in April, May, July, September, and October 1993 by use of an ADCP. Two sets of dye-dilution measurements were made at Chicago Lock in May and July 1993. The results of all ADCP leakage measurements, the dye-dilution measurements in July, and the accuracy of the measurements are presented in the following sections. The results of the May 1993 dye-dilution measurements are not presented because the method used for dye injection did not result in complete mixing. Stages for Lake Michigan, the Chicago and Little Calumet Rivers, and the North Shore Channel are shown in table 3. The values in table 3 are the mean stages for the periods of the ADCP and dye-dilution measurements and are not daily mean stages.

Table 3. Control-structure stage data for leakage measurements during April-October 1993

[CRCW, Chicago River Controlling Works; O'Brien, Thomas J. O'Brien Lock and Dam; Wilmette, Wilmette Controlling Works; Lake, Lake Michigan; River: Chicago River (CRCW), Calumet River (O'Brien), and North Shore Channel (Wilmette); dashes indicate no data available]

	Mean stage (feet, Chicago City Datum)					
	CRCW		O'Brien		Wilmette	
Date	Lake	River	Lake	River	Lake	River
04/05/93	0.56	-1.53	--	--	--	--
04/06/93	.36	-1.46	--	--	1.22	-0.38
04/07/93	.39	-1.57	0.88	-1.77	--	--
05/04/93	1.10	-1.84	--	--	--	--
05/05/93	1.02	-1.60	--	--	--	--
05/10/93	1.07	-1.48	--	--	--	--
07/12/93	1.54	-1.40	--	--	--	--
07/13/93	1.60	-1.46	--	--	--	--
07/14/93	1.80	-1.77	--	--	--	--
07/15/93	1.74	-1.44	--	--	--	--
07/16/93	--	--	--	--	--	--
09/13/93	--	--	1.28	-2.03	--	--
09/14/93	--	--	1.59	-1.36	--	--
09/15/93	--	--	1.93	-1.12	--	--
09/16/93	--	--	1.81	-1.10	--	--
09/17/93	--	--	1.61	-1.36	--	--
09/20/93	1.35	-1.77	--	--	--	--
09/21/93	1.52	-1.49	--	--	--	--
09/23/93	--	--	--	--	1.87	-1.14
10/04/93	1.18	-1.54	--	--	--	--
10/05/93	.91	-1.29	--	--	--	--
10/06/93	.59	-1.51	--	--	--	--

Control Structures

Chicago River Controlling Works

As mentioned previously, the CRCW consists of a network of harbor walls that include the Chicago Lock and two sets of sluice gates (fig. 2). In recent years, leakage through the Chicago Lock has increased because of worn gate seals and gates not fully closing. Considerable leakage through the harbor walls can result because of the construction techniques used. In order to measure the total leakage through CRCW, ADCP measurements were made in the Chicago River at or immediately west of Lake Shore Drive (fig. 2). Separate ADCP and dye-dilution measurements were made in the Chicago Lock in order to quantify leakage through the Chicago Lock as a percentage of the total leakage at CRCW.

Chicago Lock

Recently, the Corps has had problems operating the triangle gates at Chicago Lock. One of the lake gates would not fully close, sometimes resulting in an opening as wide as 24 in. Figure 15 shows leakage through the lake gates on April 5, 1993. In May 1993, the Corps initiated emergency repairs at Chicago Lock because of problems closing the triangle gates at both the east (lake) and west (river) ends of the lock. Debris was removed from the tracks on which the gates roll, gate seals were temporarily repaired, and the gates were adjusted for better closure. After repairs were completed, the lake gates closed so that there was little or no space between the gates. Nevertheless, leakage around gate seals and through holes in one of the gates still exists. However, after these repairs the lake gates sealed better than the river gates. Further repairs were made in September 1993, when the gate rollers from the river gates were removed and repaired. The Corps plans to completely rehabilitate the Chicago Lock in the near future.

ADCP measurements were made at several locations in and near the lock (fig. 16). All ADCP leakage measurements in April 1993 were made in the lock, halfway between the lake and river gates. In May, ADCP measurements of lake-gate leakage were made outside the lock, about 90 ft east of the lake gates (fig. 16). River-gate leakage measurements were made 70 ft upstream from the river gates. In July, all ADCP measurements were made in the lock halfway between the lake and river gates (fig. 16). In April and May, measurements were made by pulling the boat with the attached ADCP across the lock with a steel tagline cable. The ADCP measurements in July were made by pulling the boat with a nylon rope and a windlass mounted to the boat deck.

Ninety-five ADCP measurements were made in April, May, and July to quantify the leakage through the river gates. Measurements of leakage through the river gates were made with the river gates closed and the lake gates open. During the April and May river-gate leakage measurements, the lake gates were left in a fully open position. During the July leakage measurements, the lake gates were opened partially or kept closed. On July 13, the leakage through the river gates of the lock was measured with the lake gates opened about 3 ft so the water level in the lock was the same as in Lake Michigan. During July 14 and 15, the leakage was measured with both the river and lock gates closed. After the repairs to both sets of gates in May, the river gates leaked more than the lake gates so that between lockages, the stage in the lock was somewhat less than Lake Michigan stage and greater than

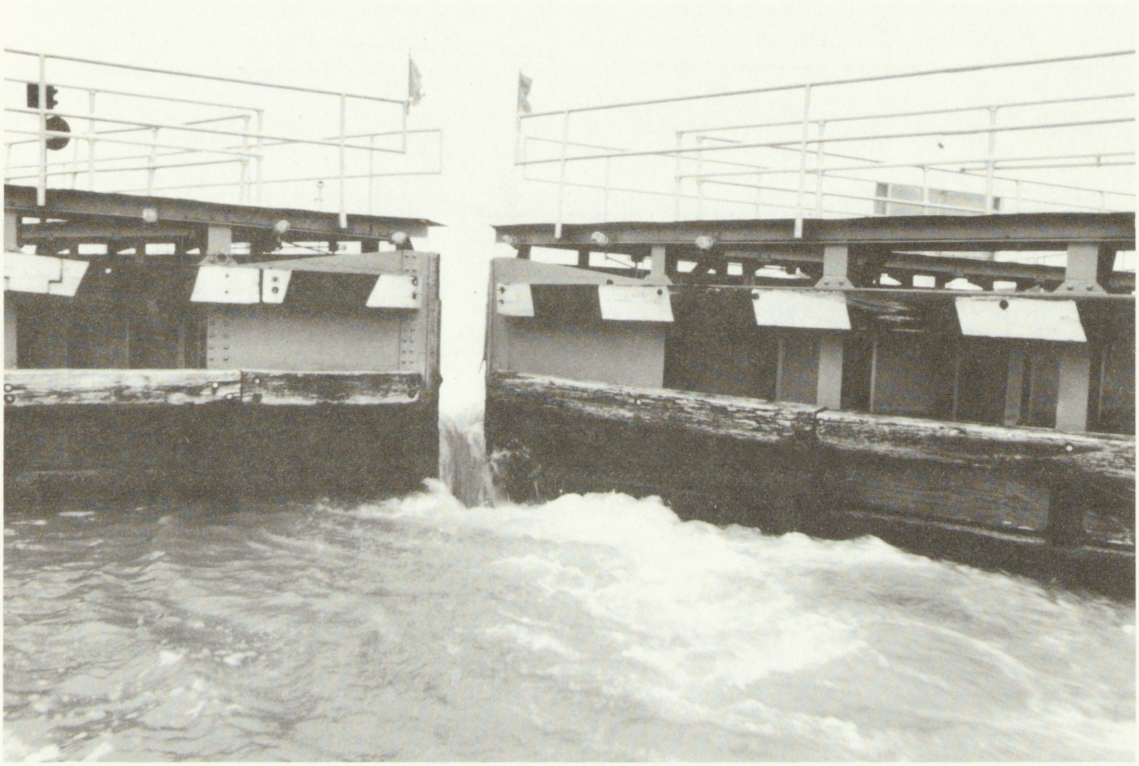


Figure 15. Leakage through lake gates at Chicago Lock, view looking east, April 5, 1993.

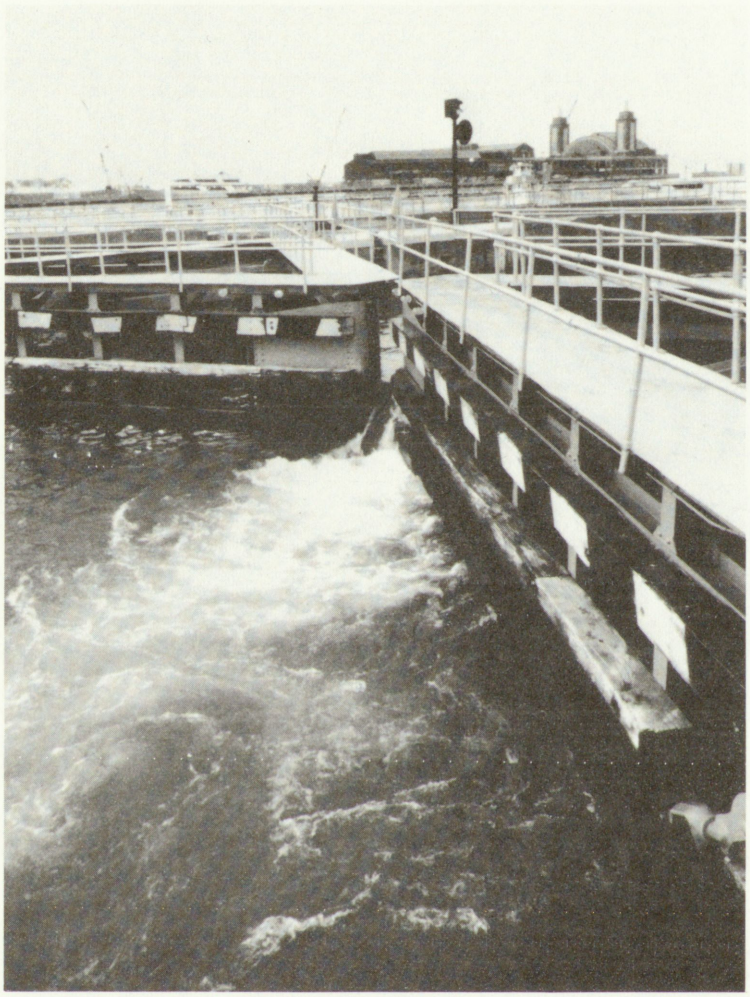


Figure 15. Continued.

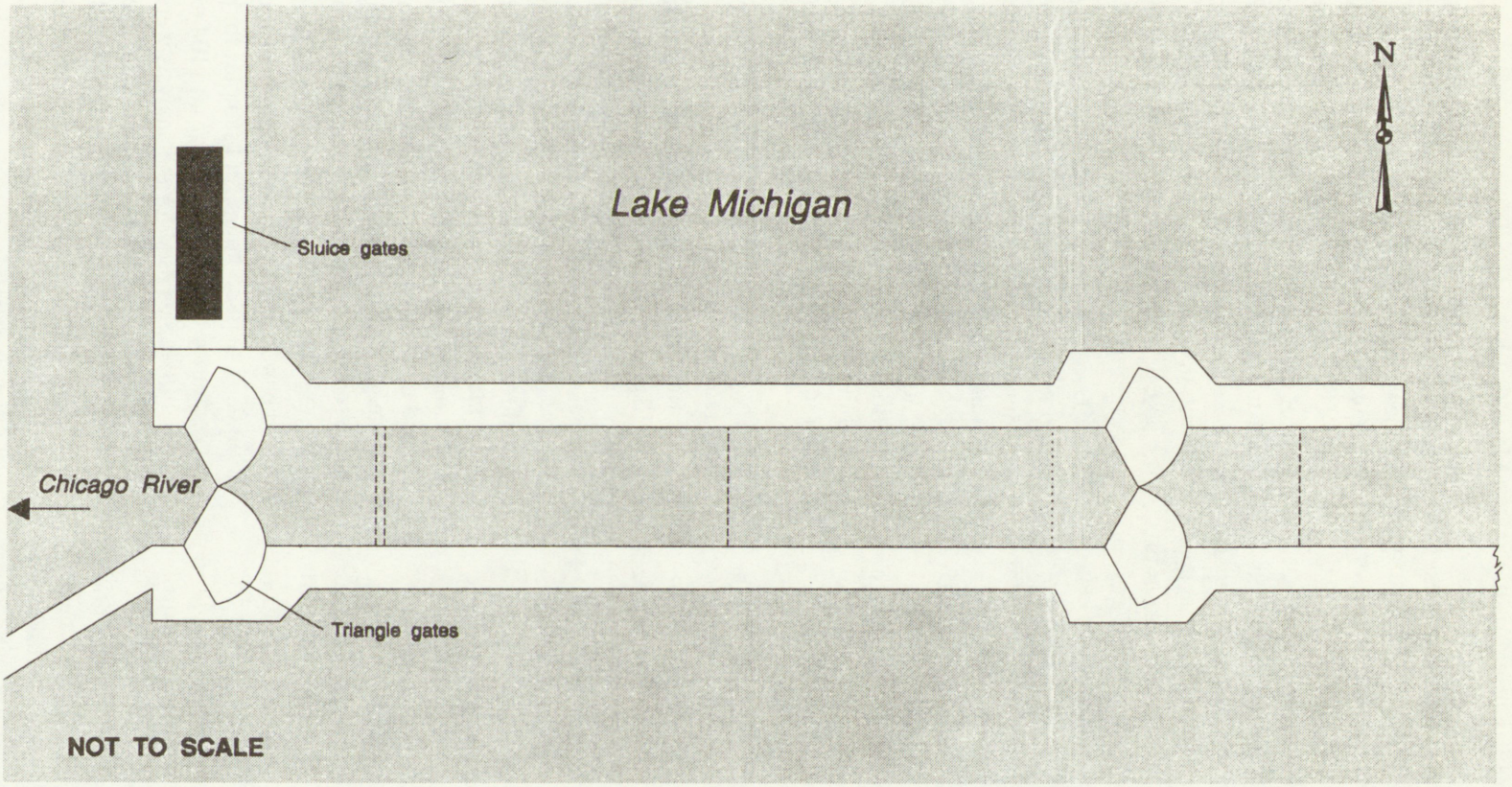


Figure 16. Measurement locations, Chicago Lock, Chicago, Ill.

the Chicago River stage. Prior to the repairs, the stage in the lock chamber normally was equal to the Lake Michigan stage. Measurements were made on July 14-15 after the water level in the lock stabilized between the lake and river levels. During April, measurement of river-gate leakage was attempted outside of the lock, about 20 ft west of the river gates. However, this measuring location was unsatisfactory, and the measurements are not included in this report. All river-gate leakage measurements described in this report were made inside the lock as described above.

The results of specific leakage measurements are shown in table 4, including the mean and standard deviation of the transects for each day. The standard deviation of the total discharge is an estimate of both measurement error and natural variability in discharge. Natural variability in flow may be caused by variation in the stage difference at the Chicago Lock (or any other structure) and other factors, such as wind speed, and direction.

The mean and standard deviation of all the river-gate leakage measurements are 133 and 38 ft^3/s, respectively. The mean river-gate leakage ranged from 102 ft^3/s on July 13, 1993, to 237 ft^3/s on May 3, 1993. Only two transects were made on May 3 because of time constraints and measurement problems caused by heavy swells from Lake Michigan. The next highest mean river-gate leakage, 148 ft^3/s, was measured on July 15.

Forty-eight ADCP measurements were made in April, May, and July to quantify the leakage through the lake gates. During these measurements, the lake gates were closed as much as possible and the river gates remained open. During the April and May leakage measurements, the south lake gate did not fully close causing the water entering the lock to flow along the inside of the south lake gate before reaching the southern lock wall. This flow pattern caused large eddies to form and move downstream. Leakage through the lake gates in July was measured by opening the gates about 20 in. to simulate the hydraulic conditions prior to the gate repairs. However, because of time constraints and difficulties in adjusting the gates, both lake gates were opened an equal amount allowing lake water to enter the center of the lock and flow parallel to the lock walls. This difference between the July lake-gate measurements and those made earlier in the year may be of significance. It also seems reasonable that this gate configuration provided less resistance to flow and, therefore, more leakage resulted. The mean and standard deviation of all the lake-gate leakage measurements are 919 and 105 ft^3/s, respectively. The mean lake-gate leakage ranged from 835 ft^3/s on April 6, 1993, to 961 ft^3/s on May 10, 1993.

Dye-dilution discharge measurements were made at the Chicago Lock on July 13-15, 1993. For all measurements, dye was injected inside the lock 50 ft west of the lake gate from the lock, and samples were collected inside the lock 50 ft east from the river gate of the lock (500 ft west of the injection point). Injecting and sampling within the lock gives the longest possible dye-mixing length. This also eliminates any dilution or dye loss from mixing with water from the lake or river, as would be possible if the injection or sampling were outside the lock.

Dye was injected with the manifold setup described earlier (see figs. 12-13). Water to mix with the dye was pumped from outside the lock (Lake Michigan water), mixed with the dye, and injected through the manifold located across the center 45 ft of the lock less than 1 ft above the bottom. The injection concentrations and dye-injection rates are listed in table 5. The volumetric pumping rate of water to mix with the dye was 23 gal/min (0.05 ft^3/s) on July 13 and about 45 gal/min (0.11 ft^3/s) on July 14 and 15. This is negligibly small compared to the flow rate measured.

Results of the dye-dilution measurements on July 13 and 14 are not presented in this report. Low pumping rate for the July 13 measurement caused poor vertical mixing of the dye in the lock. During the July 14 measurement, the injection pump stopped partway through the injection. Results from both of these measurements were poor. Only the July 15 dye-dilution measurement met all three criteria for a good measurement: (1) constant-rate injection, (2) adequate vertical and lateral mixing, and (3) adequate serial dilution of the injection solution.

On July 15, the dye was injected from 0900 to 1334 hours. Background samples were collected from 0853 to 0856 hours, and samples for plateau dye concentrations were collected from 1104 to 1337 hours. Samples used for the analysis were collected from 1253 to 1337 hours. Theoretical dye concentrations over time indicate that the dye should have reached a plateau concentration about 180 minutes after the start of dye injection. The mean and depth-weighted average concentrations from each vertical indicate that the mean concentration had reached a

Table 4. Acoustic Doppler current profiler data, April-October 1993

[For each date shown, the mean is in the first row and the standard deviation is in the second row in parentheses ( ); Top, Bottom, Left, and Right are acoustic Doppler current profiler (ADCP) estimated discharges for the respective unmeasured portion of the water column; Middle is measured ADCP discharge]

Date	Start time (hours)	End time (hours)	Number of transects	Discharge (cubic feet per second)						Transect length, in feet	Elapsed time, in seconds	Boat speed, in feet per second	Number of ensembles	Estimated discharge, in percent
				Top	Middle	Bottom	Left	Right	Total					
Chicago Lock at Chicago, Ill. –River-gate leakage														
04-05-93	1345	1835	14	24.3	90.8	6.0	4.8	6.2	132	71	235	0.34	48	30
				(14.3)	(34.2)	(2.5)	(15.8)	(13.4)	(52)	(2)	(91)	(0.12)	(28)	(9)
04-06-93	2035	2100	4	11.5	127.9	5.7	13.1	-15.5	143	69	301	0.23	73	13
				(5.6)	(105.9)	(2.4)	(2.9)	(3.8)	(109)	(7)	(17)	(0.02)	(4)	(5)
05-03-93	1745	1815	2	32.6	168.1	13.1	12.1	10.8	237	80	640	0.13	175	29
				(9.0)	(14.8)	(1.2)	(12.0)	(3.6)	(23)	(11)	(108)	(0.00)	(29)	(0)
05-04-93	0900	1630	22	15.9	87.0	8.7	10.1	9.8	131	65	465	0.14	97	34
				(6.3)	(12.8)	(2.1)	(4.5)	(3.8)	(19)	(3)	(64)	(0.02)	(25)	(6)
05-06-93	1050	1850	18	29.1	70.6	7.5	8.4	7.4	123	62	593	0.11	105	43
				(10.1)	(9.8)	(1.2)	(2.8)	(2.6)	(7)	(4)	(85)	(0.02)	(15)	(7)
07-13-93	1100	1400	10	21.6	68.4	6.3	3.4	1.8	102	65	666	0.11	105	32
				(7.7)	(15.5)	(1.4)	(6.8)	(4.4)	(25)	(3)	(164)	(0.04)	(40)	(9)
07-14-93	1020	1400	14	2.4	104.4	10.8	11.8	13.3	143	77	628	0.12	96	25
				(13.0)	(13.0)	(2.2)	(3.6)	(5.6)	(26)	(11)	(84)	(0.01)	(24)	(12)
07-15-93	1100	1400	11	33.5	87.8	5.9	11.1	9.6	148	69	596	0.13	129	40
				(4.8)	(7.4)	(0.7)	(8.7)	(8.3)	(19)	(10)	(152)	(0.05)	(33)	(11)
Chicago Lock at Chicago, Ill. –Lake-gate leakage														
04-05-93	1210	1245	7	152.1	682.0	46.8	93.6	-62.3	912	71	214	0.37	33	25
				(15.3)	(88.3)	(6.4)	(16.4)	(24.0)	(97)	(2)	(83)	(0.12)	(9)	(4)
04-06-93	1850	2020	10	101.7	654.2	52.3	31.4	-5.0	835	76	298	0.26	73	22
				(25.7)	(145.2)	(13.3)	(45.0)	(43.5)	(152)	(3)	(51)	(0.05)	(12)	(4)
05-10-93	1015	1730	27	120.4	618.1	52.1	77.3	93.3	961	63	338	0.20	74	36
				(28.7)	(51.5)	(9.7)	(5.9)	(18.5)	(40)	(2)	(79)	(0.06)	(25)	(4)
07-12-93	1735	1830	4	157.3	527.6	66.0	130.2	-19.3	862	73	215	0.34	30	37
				(25.2)	(74.0)	(12.1)	(170.5)	(102.3)	(166)	(8)	(26)	(0.02)	(4)	(15)
Chicago River at Lake Shore Drive, Chicago, Ill.														
04-07-93	0830	0830	5	69.0	88.8	7.6	6.5	0.0	172	232	327	0.72	79	52
				(11.2)	(51.7)	(4.4)	(2.7)	(7.3)	(61)	(3)	(47)	(0.12)	(11)	(16)
05-10-93	0615	0700	4	9.7	136.6	16.4	-1.6	6.2	167	194	576	0.34	96	13
				(21.4)	(29.4)	(3.9)	(5.8)	(8.0)	(60)	(1)	(76)	(0.05)	(14)	(22)

Table 4. Acoustic Doppler current profiler data, April-October 1993—Continued

Date	Start time (hours)	End time (hours)	Number of transects	Discharge (cubic feet per second) Top	Middle	Bottom	Left	Right	Total	Transect length, in feet	Elapsed time, in seconds	Boat speed, in feet per second	Number of ensembles	Estimated discharge, in percent
Chicago River at Lake Shore Drive, Chicago, Ill.—Continued														
07-14-93	0430	0630	5	126.4	100.9	14.9	-0.4	9.4	251	238	836	0.29	116	63
				(15.9)	(65.8)	(8.6)	(12.5)	(9.6)	(81)	(17)	(167)	(0.04)	(23)	(14)
07-15-93	0425	0610	4	26.4	128.7	17.7	6.5	2.6	182	249	955	0.27	131	30
				(12.6)	(62.8)	(9.2)	(10.8)	(7.0)	(87)	(31)	(269)	(0.05)	(36)	(2)
07-16-93	0455	0530	2	29.7	96.6	7.5	11.5	0.4	145	194	1,237	0.16	284	33
				(25.8)	(9.7)	(0.6)	(2.7)	(6.4)	(32)	(2)	(72)	(0.01)	(1)	(8)
09-20-93	0935	1130	6	59.6	139.1	11.3	18.7	-1.5	227	243	803	0.31	183	40
				(22.1)	(63.9)	(5.6)	(17.7)	(17.1)	(84)	(6)	(146)	(0.07)	(24)	(7)
09-21-93	0810	1010	7	-68.6	257.6	20.4	1.2	-3.1	208	258	832	0.31	190	-31
				(27.3)	(60.6)	(5.1)	(22.6)	(9.8)	(79)	(8)	(120)	(0.04)	(28)	(25)
10-05-93	1400	1425	2	22.2	113.9	8.7	2.7	5.8	153	195	770	0.25	203	26
				(20.4)	(18.7)	(1.5)	(0.9)	(5.6)	(6)	(9)	(10)	(0.02)	(2)	(9)
10-06-93	0800	0915	4	29.1	103.1	8.0	2.9	2.1	145	238	1,050	0.23	276	29
				(7.3)	(34.5)	(2.4)	(4.9)	(1.4)	(45)	(14)	(174)	(0.05)	(45)	(4)
Thomas J. O'Brien Lock and Dam near Burnham, Ill.														
09-14-93	1045	1245	10	1.4	17.0	1.5	4.1	-3.6	20	99	254	0.39	58	-55
				(17.8)	(13.6)	(1.2)	(6.7)	(7.7)	(31)	(9)	(38)	(0.02)	(8)	(358)
09-17-93	0915	1240	10	3.4	14.7	1.4	-0.5	2.3	21	104	958	0.11	218	21
				(5.5)	(7.8)	(0.7)	(5.2)	(3.7)	(10)	(9)	(205)	(0.02)	(47)	(47)
Wilmette Pumping Station at Wilmette, Ill.														
04-06-93	1145	1320	12	20.6	26.9	4.9	-1.4	7.9	59	73	405	0.19	110	54
				(4.8)	(3.4)	(0.6)	(1.3)	(1.6)	(8)	(1)	(81)	(0.04)	(22)	(5)
09-22-93	1520	1610	3	-1.2	8.8	1.6	-4.3	-2.1	3	97	871	0.11	363	71
				(1.1)	(0.9)	(0.2)	(1.8)	(11.1)	(12)	(19)	(224)	(0.01)	(95)	(101)
09-23-93	1200	1300	4	-1.0	-0.2	-2.2	26.3	-16.9	6	89	533	0.21	210	89
				(4.9)	(4.0)	(4.1)	(11.7)	(22.6)	(21)	(20)	(319)	(0.12)	(127)	(14)

Table 5. Concentrations and pumping rates for dye injection at Chicago Lock, July 13-15, 1993
[L, liters; mL, milliliters; min, minutes; mL/min, milliliters per minute]

Date	Volumes of dye and water in injection mixture (L)		Injection concentration (parts per billion)		Volume of mix injected (mL)	Injection time (min)	Injection rate (mL/min)	
	Dye	Water	Mean	Variance			Mean	Variance
July 13	5.4	2.2	1.41×10^8	3.61×10^{12}	3,722	250	14.89	1.64
July 14	5.8	0	2.38×10^8	0	2,247	110	20.43	4.18
July 15	5.0	0	2.38×10^8	0	4,732	276	17.14	5.63

plateau by the time the samples were collected at 1253 hours (fig. 17) and that the dye was fairly well mixed throughout the cross section. The discharge estimated for this dye-dilution measurement is 160 ft^3/s.

Results of quality-assurance samples are listed in tables 6 and 7. Analysis of variance (ANOVA) of normalized concentrations from replicate and split samples indicated that the variance from sampling and analysis was significantly smaller (p less than 0.0001) than the variance across the channel. Concentrations were normalized by dividing by the average of the four depth-averaged concentrations (one from each vertical) from the same sampling transect that the quality-assurance samples were from. These normalized concentrations allowed comparison of samples from days and times with different average dye concentrations in the ANOVA.

Results from blank samples indicated that any contamination was negligibly small compared to the concentration in the river. The mean and standard deviation of the blank samples were 0.06 and 0.09 parts per billion, respectively. In contrast, the mean and standard deviation of the background samples were 0.187 and 0.128, respectively. The two-way Mann-Whitney test (nonparametric t-test) was used to test if the equivalent concentrations of the blank samples were significantly different (α=0.05) from equivalent background concentrations. Results from this test and the observed difference in the means indicated that the equivalent background concentrations were significantly higher than the equivalent concentrations (p equal 0.01) from the blank samples.

Chicago River

The measurements in the Chicago Lock were made to determine the amount of leakage through the lock only. Therefore, the ADCP measurements on the Chicago River were made to determine the total leakage through CRCW into the Chicago River. Most of the Chicago River measurements were made about 20 ft west from the Lake Shore Drive bridge (fig. 2). Several measurements made in April, however, were made at the Columbus Drive bridge and halfway between Columbus Drive and Lake Shore Drive. During the April measurements, the boat was powered across the channel using an outboard or electric trolling motor.

Thirty-nine ADCP measurements were made on the Chicago River in April, July, September, and October 1993 (table 4). The mean and standard deviation of all Chicago River measurements are 192 and 73 ft^3/s, respectively. The highest mean leakage measured was 251 ft^3/s on July 14, and the lowest mean leakage measured was 145 ft^3/s on July 16 and October 6, 1993.

Thomas J. O'Brien Lock and Dam

The leakage through O'Brien is considerably less than at CRCW. While there is noticeable leakage between the sector gates and gate seals in the lock, leakage through the four sluice gates is not visually apparent. Attempts to measure leakage through the sluice gates in September were unsuccessful. Any leakage through the sluice gates was too small to be accurately measured with the ADCP.

Four leakage measurements were made at O'Brien in April 1993. The measurements were made about 100 ft upstream from the lock (Lake Michigan side). An outboard motor was used to power the boat across the Calumet River for each of the four measurements. Because the leakage through the lock and sluice gates was so small and the boat velocity was much larger than the water velocity, accurate measurements were not possible. Therefore, the results of these measurements are not shown in table 4.

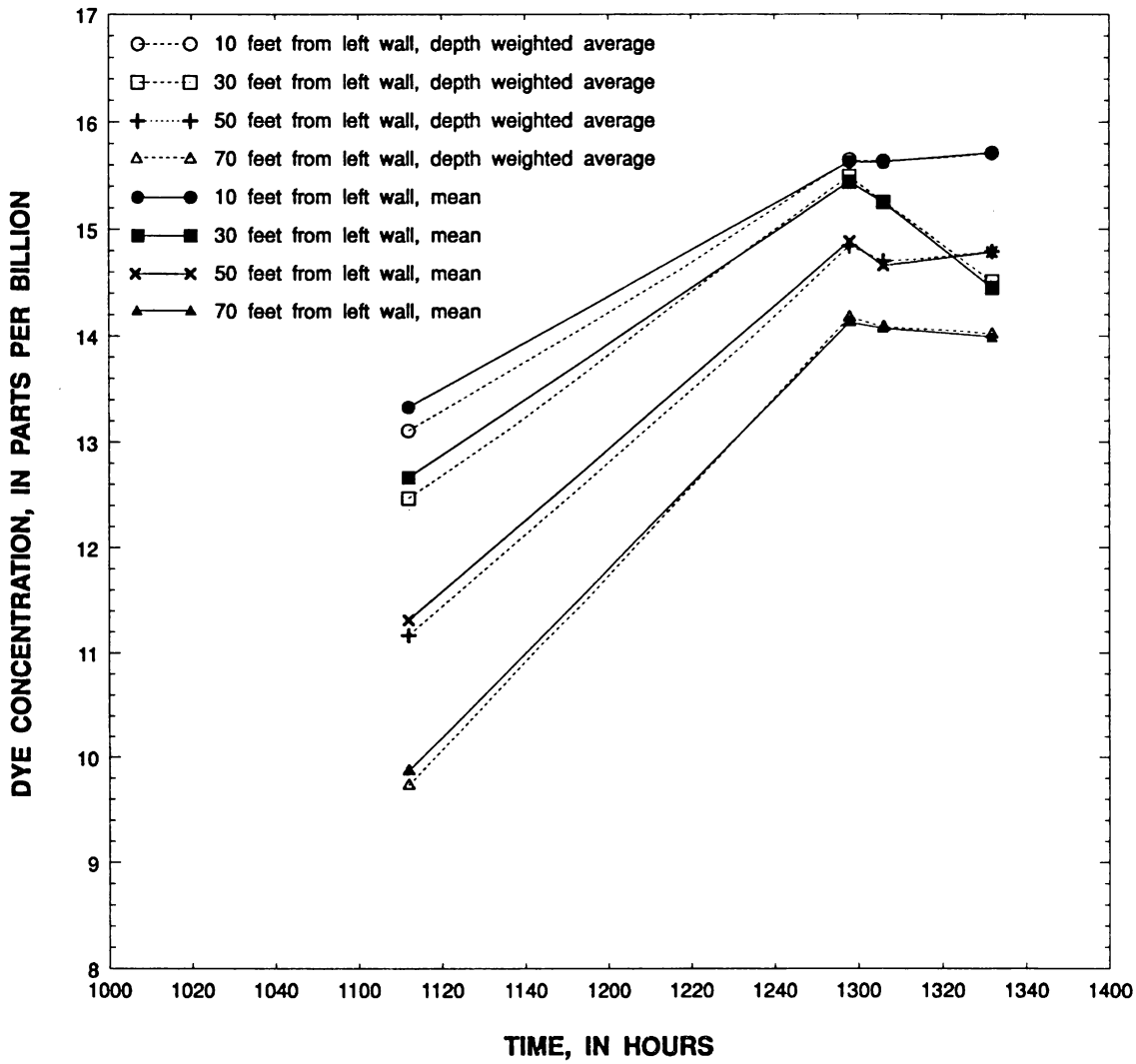


Figure 17. Mean and depth-weighted average concentrations from each vertical sampled at Chicago Lock, July 15, 1993.

Table 6. Results of replicate and split samples collected at Chicago Lock, July 1993

[ft, feet; ppb, parts per billion; calibration number refers to table 2; concentrations were normalized by dividing by the average of the depth-averaged concentrations for all four verticals from the transect that the sample was taken]

Sample number	Sample date	Sample time (hours)	Distance from left edge of lock (ft)	Depth (ft)	Sample type	Calibration number	Fluorometer reading	Mean concentration in transect (ppb)	Equivalent dye concentration (ppb)	Normalized concentration
25	July 13	1138	50	12	Sample	1	2.2	3.24	2.96	0.911
26	July 13	1139	50	12	Replicate of 25	1	2.6	3.24	3.46	1.066
27	July 13	1140	50	12	Replicate of 25	1	2.3	3.24	3.05	.952
39	July 13	1227	30	12	Sample	1	2.1	3.83	2.79	.729
40	July 13	1227	30	12	Replicate of 39	1	2.0	3.83	2.72	.710
48	July 13	1232	70	18	Sample	1	4.5	3.83	6.06	1.582
49	July 13	1233	70	18	Replicate of 49	1	4.0	3.83	5.37	1.402
59	July 13	1311	10	6	Sample	2	1.9	3.66	2.43	.665
60	July 13	1311	10	6	Split of 59	2	1.8	3.66	2.29	.628
61	July 13	1312	10	6	Replicate of 59	2	1.7	3.66	2.16	.590
62	July 13	1312	10	6	Split of 61	2	1.7	3.66	2.18	.598
136	July 15	1256	50	12	Sample	5	9.9	15.02	15.01	.999
147	July 15	1256	50	12	Split of 136	5	10.0	15.02	15.16	1.009
151	July 15	1259	30	18	Sample	5	10.5	15.02	15.95	1.062
152	July 15	1259	30	18	Split of 151	5	10.5	15.02	15.95	1.062
153	July 15	1301	10	6	Sample	5	10.0	15.02	15.21	1.012
154	July 15	1301	10	6	Replicate of 153	5	10.0	15.02	15.21	1.013

Table 7. Results of analysis of blank samples collected at Chicago Lock, July 1993
[ft, feet; ppb, parts per billion; calibration number refers to table 2; dashes indicate no data]

Sample number	Sample date	Sample time (hours)	Distance from left edge of lock (ft)	Depth (ft)	Calibration number	Fluorometer reading	Equivalent dye concentration (ppb)
456	July 14	--	--	--	1	0.2	-0.010
457	July 14	--	--	--	1	.2	-.010
93	July 14	1149	10	0	3	.3	.070
106	July 14	1251	80	0	3	.25	.027
146	July 14	1347	80	0	3	.2	.002
175	July 15	1334	--	--	5	.19	.227
179	July 15	1338	--	--	5	.15	.141

Measurements of the leakage through the lock at O'Brien were made with the ADCP on September 14 and 17, 1993. ADCP measurements were made in the lock about 200 ft away from the downstream sector gates with the windlass assembly to power the boat across the lock. The traverse times across the lock for measurements made on September 14 averaged 254 seconds. These times were considerably faster than traverse times for the measurements made on September 17 because of a malfunction in the windlass. The mean and standard deviation of ADCP measurements made on September 14, 1993, are 20 and 31 ft^3/s, respectively. The mean and standard deviation of ADCP measurements made on September 17, 1993, are 21 and 10 ft^3/s, respectively.

Wilmette Pumping Station

Most of the leakage at Wilmette is through the pump bays in the pumphouse. Little leakage through the sluice gate has been observed. As previously mentioned, in July 1993, MWRD removed the pumps and sealed the pump bays at Wilmette. Prior to the sealing of the pump bays, the leakage through the bays was readily apparent. After the sealing, little or no leakage could be observed.

Three sets of leakage measurements were made at Wilmette with the ADCP: one set on April 6, 1993, and the others on September 22-23, 1993. A tagline was used to pull the boat across the North Shore Channel at Wilmette during the April 1993 measurements. The September 1993 measurements were made with the windlass assembly to power the boat across the channel. A total of 19 transects were made at Wilmette, 12 of which were made on April 6, 1993. The mean and standard deviation of the April 6 ADCP measurements are 59 and 8 ft^3/s, respectively. Leakage during the September measurements was so small that these ADCP measurements are questionable. It is likely that the leakage is less than 15 ft^3/s at Wilmette.

Accuracy of Measurements

The accuracy of the measurements made with the Broadband ADCP cannot be calculated in a rigorous statistical manner at the present time. Although sources of error for narrowband ADCP's have been identified and discussed by a number of investigators, including Simpson and Oltmann (1993), little work has been done to document the errors of Broadband ADCP's.

Uncertainty in ADCP measurements may be both random and systematic. Random errors can be reduced by data averaging; systematic errors cannot be reduced. Random errors may be caused by side-lobe interference, self-noise, or errors in the signal-processing algorithm used in the ADCP (M.R. Simpson, U.S. Geological Survey, written commun., 1993). Theoretical equations for describing the random uncertainty of the Broadband ADCP have not yet been released by the manufacturer. Systematic ADCP errors have been reduced with the introduction of the Broadband ADCP's used in this study because of wider bandwidth in the acoustic signals and a better signal-processing algorithm (M.R. Simpson, U.S. Geological Survey, written commun., 1993). The most significant systematic errors are because of potential misalignment in the beam angles and pitch and roll offsets. The manufacturer has instituted procedures that minimize potential for error in transducer

alignment and provides a rigorous test of each ADCP prior to delivery (J. R. Marsden, RD Instruments, oral commun., 1993). Therefore, systematic errors are believed to be less than 3 percent.

Generally, the accuracy of the discharges measured using the ADCP tended to improve with time. Most of this increase in accuracy can be attributed to improvement in measurement technique over time. For the measurements made in April and May 1993, average boat velocities tended to be higher and the number of ensembles lower than those for measurements made in July-October 1993. This is particularly noticeable for measurements made in the Chicago River (see table 4).

The accuracy of the measured ADCP discharges also is reflected in the standard deviations of measured discharges and the sample sizes shown in table 4. The standard deviation of the Chicago River measurements tended to be higher than the standard deviation of measurements elsewhere. This, however, is not only an estimate of measurement uncertainty, but it also may be an indication of the natural variability in flow. At such low velocities, the potential for oscillations in flow and velocity is considerable, particularly at sites such as the Chicago River.

The percentage of the total ADCP discharge that is estimated, including the top and bottom of the water column and the two edges, is also shown in table 4. The percentage estimate shown in table 4 was computed by (1) summing the unmeasured (estimated) discharge (top, bottom, left, and right discharge) and dividing by the total discharge and (2) computing the mean for each transect. The percentage of the discharge estimated (not measured by the ADCP) ranged from -55 percent for O'Brien measurements to 89 percent for Wilmette measurements. For most measurements, however, the discharge estimated with the ADCP ranged from 30 to 40 percent. If the percentage of the discharge estimated is negative, it indicates that one or more of the estimated discharges were negative. For example, on September 21, 1993, a strong wind was blowing from west to east at CRCW. This caused bi-directional flow, with the top layer flowing upstream (eastward) and the rest of the water column flowing downstream (westward). The top discharge shown in table 4 is negative and, therefore, the percentage of discharge estimated is -31 percent.

The effect of the techniques for extrapolating the velocity profile measured by the ADCP was evaluated with sensitivity analysis. The extrapolation technique and the value of the exponent were varied for selected ADCP measurements. The exponent was allowed to vary between 0.11 (1/9) and 0.20 (1/5), representing a range of exponents suggested by various investigators, and the resulting discharge was computed. Results of this sensitivity analysis indicate that the measured discharges are relatively insensitive to changes in the exponent. Figure 18 shows the results of the sensitivity analysis for several transects at CRCW.

The error in the dye-dilution discharges was estimated using a first-order analysis (Ang and Tang, 1975, p. 199) to estimate the effect of error in each measured factor on the error in the calculated discharge assuming that the factors are independent. The equation used to determine the error is

$$S^2(\hat{Q}) = \left(\frac{\partial(\hat{Q})}{\partial(i)}\right)^2 S^2(i) + \left(\frac{\partial(\hat{Q})}{\partial(C_i)}\right)^2 S^2(C_i) + \left(\frac{\partial(\hat{Q})}{\partial(C_s)}\right)^2 S^2(C_s), \quad (6)$$

where

$S^2(\bullet)$ is the variance of the term in parentheses,

$\hat{Q}$ is the estimated discharge, and

$\partial(\hat{Q})/\partial(\bullet)$ is the partial derivative of the discharge with respect to the term $\bullet$.

The variance of the injection rate was determined from the samples of volume remaining in the dye container at different times. The variance of the injection concentration was determined from analysis of multiple samples of the injection solution. For days that raw dye was injected, the variance in injection concentration was set to zero. The variance in the sampled concentration was determined from the average concentration from each vertical sampled.

The error analysis describes the variance in the estimated discharge for the conditions measured. The sensitivity of the estimated discharge to each term varies as the injection rate, injected concentration, and sampled concentration vary. The variance, expressed as the standard deviation of the estimated discharge on July 15 and computed using equation 6, is 23 ft^3/s. This is a conservative estimate of the variance because

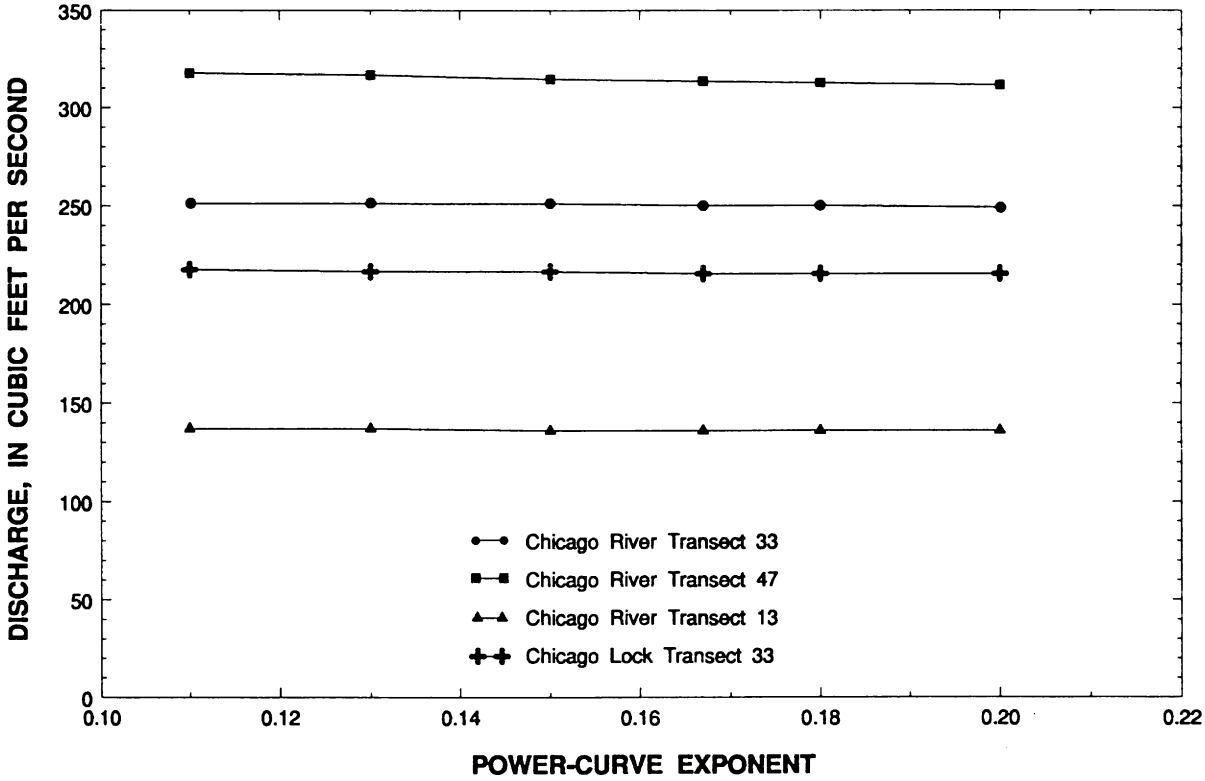


Figure 18. Sensitivity of acoustic Doppler current profiler discharge to changes in the power velocity-distribution law exponent.

it was computed using the variance of the point-sample data rather than the variance of the cross-section averages for the injection rate, injected concentration, and sampled concentration.

The dye-dilution measurements were made because ADCP's have not previously been used to measure such low discharges. Comparison of discharges estimated by dye dilution with those measured with the ADCP show that on July 15, 1993, the dye-dilution estimate of discharge was within 8 percent (12 ft^3/s) of the ADCP-measured discharge. The difference between the two discharges is less than the standard deviation of the dye-dilution discharge estimate.

FUTURE WORK

Study results indicate that the ADCP and dye-dilution leakages should be used with caution. Measurements described in this report give an indication of the order of magnitude of the leakage through each control structure, rather than precise values of leakage. Use of these leakage values to develop a method for predicting leakage at CRCW, O'Brien, and Wilmette for a range of stage differences is not recommended at this time. Estimates of leakage may be more sensitive to factors other than the stage difference at the control structure. For example, the gates at Chicago Lock did not always close to exactly the same position with each use. The effective gate opening is likely to have a much larger effect on the leakage than the head differences normally observed for Lake Michigan and the Chicago River.

The measurement results indicate that additional leakage measurements are needed at each of these locations if more accurate values are desired and if those values are to be used for estimating leakage. Specifically, it is desirable to obtain leakage measurements for a complete range of head conditions at each control structure. The accuracy of the measured leakage may also be improved by increasing the number of transects. Most measurements made in the Chicago River consisted of only 3-5 transects, for example. Alternative techniques for measuring discharge at the Chicago River, such as a Neil-Brown acoustic velocity meter, might be explored. Use of an alternative technique for measuring discharge in the Chicago River would provide greater confidence in

the ADCP measurements at this location and improve the possibility of developing a means to estimate leakage.

Dye-dilution measurements can also be done for O'Brien. A continuous injection over a long time period (1 week) should allow for more reliable estimates of leakage at both of these locations. This can be achieved without requiring that the lock be closed to river traffic.

SUMMARY AND CONCLUSIONS

Measurements of leakage were made at three control structures near Chicago, Ill., using acoustic Doppler current profilers (ADCP's) and dye-dilution techniques. The leakage results from the seepage of Lake Michigan water through sea walls and through and around lock gates and sluice gates. The water leaking through each of these structures forms part of the diversion of Lake Michigan water by the State of Illinois. The amount of the Lake Michigan diversion is regulated by U.S. Supreme Court decree.

Leakage measurements were made at the Chicago River Controlling Works (CRCW) (including both the Chicago Lock and the Chicago River), Thomas J. O'Brien Lock and Dam (O'Brien), and Wilmette Pumping Station (Wilmette) in April, May, July, September, and October 1993 using an ADCP. ADCP's are used to measure vertical profiles (ensembles) of water velocities from a moving boat. Water velocities are measured by transmitting short, phase-encoded acoustic pulses along four narrow beams at a known frequency. The ADCP detects and processes the echoes reflected from successive volumes along the beams and determines the time-lag and difference in frequency (frequency shift). The time-lag change and frequency shift between transmitted and reflected sound is proportional to the relative velocity between the ADCP and the suspended material in the water column. The ADCP cannot profile near the water surface, near the channel bottom, near vertical walls, or in water less than 5 ft deep. Velocities near the surface were assumed to be constant and equal to the shallowest water velocity measured by the ADCP. Velocities near the bottom were estimated using an approximation of the 1/6 power velocity-distribution law. Velocities in the unmeasured edges of a channel were estimated using the average velocity of ensembles near each edge.

Dye-dilution measurements of leakage were made at the Chicago Lock in July 1993 to evaluate the discharges measured using the ADCP. Dye-dilution methods to measure discharge are based on a mass-balance calculation for the flow. Most applications of dye-dilution methods to measure discharge are for highly turbulent flows where mixing is not a problem. In the application used in the study and described in this report, an injection manifold was designed for mixing of the dye in the flow because leakages were too small for adequate mixing.

A total of 221 discharge measurements were made with the ADCP. The greatest mean leakage measured for the lake gate at Chicago Lock was 961 ft^3/s. However, because the river gate is normally closed, the leakage through that gate is a better estimate of the normal leakage through the Chicago Lock. The mean and standard deviation of leakage measured by the ADCP for the Chicago Lock river gate were 133 and 38 ft^3/s, respectively. The mean and standard deviation of ADCP leakage measurements at CRCW were 192 and 73 ft^3/s, respectively. River-gate leakage accounted for more than half of the total leakage measured at CRCW. The mean and standard deviation of leakage measured at O'Brien on September 17, 1993, were 21 and 10 ft^3/s, respectively. Leakage measurements at O'Brien on September 14, 1993, were less accurate; the mean and standard deviation were 20 and 31 ft^3/s, respectively. The mean and standard deviation of April 1993 leakages measured at Wilmette using the ADCP were 59 and 8 ft^3/s, respectively. After the Metropolitan Water Reclamation District of Greater Chicago sealed the pump bays at Wilmette in July 1993, the leakage dropped to less than 15 ft^3/s in September 1993.

Discharges were estimated by dye dilution at Chicago Lock on July 13-15, 1993. The measurements made on July 13-14 did not meet the criteria for a good dye-dilution measurement and, therefore, were not used for further analysis. On July 15, the leakage through the Chicago Lock river gates was estimated by dye dilution to be 160 ft^3/s or within 8 percent of the ADCP-measured discharge.

The sensitivity of ADCP-measured discharges to changes in the power velocity-distribution law exponent was evaluated. The exponent was varied from 0.11 (1/9) to 0.20 (1/5) for a number of transects near CRCW. Results indicate that for the leakages being

measured, discharges are insensitive to the changes in the exponent.

ADCP measurements made in April and May 1993 are less accurate than those made in July, September, and October 1993. The improved accuracy obtained for measurements after May 1993 is primarily due to improvements in ADCP measuring techniques. Results of the measurements reported here should be used with caution and should probably not be used to estimate leakage based only on stage differences at the three control structures. Although the results of the ADCP measurements are, in general, acceptable, improvements in measurement methods can be made. Alternative methods for measuring discharge at the Chicago River, such as the use of an acoustic velocity meter, might provide more reliable leakage information at this location. Similarly, dye-dilution measurements could be made at O'Brien and Wilmette if more reliable estimates of leakage are necessary.

REFERENCES

Ang, A.H.S., and Tang, W.H., 1975, Probability concepts in engineering planning and design; Volume I, Basic principles: New York, Wiley, 409 p.

Chen, Cheng-lung, 1989, Power law of flow resistance in open channels–Manning's formula revisited, in International Conference on Channel Flow and Catchment Runoff: Centennial of Manning's Formula and Kuichling's Rational Formula, Charlottesville, Virginia, May 22-26, 1989, Proceedings, p. 817-848.

———1991, Unified theory of power laws for flow resistance: Journal of Hydraulic Engineering, American Society of Civil Engineers, v. 117, no. 3, p. 371-389.

Fisher, H.B., List, E.J., Koh, R.C.Y., Imberger, Jörg, and Brooks, N.H., 1979, Mixing in inland and coastal Waters: New York, Academic Press, Inc., 483 p.

Gordon, R.L., 1989, Acoustic measurement of river discharge: Journal of Hydraulic Engineering, American Society of Civil Engineers, v. 115, no. 7, p. 925-936.

Kilpatrick, F.A., and Cobb, E.D., 1985, Measurement of discharge using tracers: U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chap. A16, 52 p.

Naujoks, H.H., 1946, The Chicago water diversion controversy: Marquette Law Review, Marquette School of Law, v. 30, no. 3, p. 147-176.

Rantz, S.E., and others, 1982, Measurement and computation of streamflow, Volume 1, Measurement of discharge: U.S. Geological Survey Water-Supply Paper 2175, 631 p.

RD Instruments, 1989, Acoustic Doppler current profilers, Principles of operation: A practical primer: San Diego, RD Instruments, 36 p.

———1993, Direct-reading broadband acoustic Doppler current profiler technical manual: San Diego, RD Instruments, 52 p.

Robison, R., 1986, The tunnel that cleaned up Chicago: Civil Engineering, American Society of Civil Engineers, July 1986, p. 9-13.

Simpson, M.R., and Oltmann, R.N., 1993, Discharge-measurement system using an acoustic Doppler current profiler with applications to large rivers and estuaries: U.S. Geological Survey Water-Supply Paper 2395, 32 p.

U.S. Army Corps of Engineers, 1974, Charts of the Illinois Waterway: Chicago District, 77 p.

———1990, Current-meter measurements of discretionary and leakage flows at the Chicago River Controlling Works, O'Brien Lock and Dam, and the Wilmette Controlling Works: Chicago District, Lake Michigan Diversion Accounting Section, 25 p.

———1993, Lake Michigan diversion accounting annual report, water years 1990-1992: Chicago District, Lake Michigan Diversion Accounting Section, 19 p.

U.S. Department of Commerce, 1990, Recreational chart 14926, Chicago and south shore Lake Michigan: National Oceanic and Atmospheric Administration, National Ocean Service, 29 p.

APPENDIXES

APPENDIX 1. SUMMARY OF FLUOROMETER CALIBRATIONS, JULY 15-16, 1993

[ppb, parts per billion]

Date	Time (hours)	Standard concentration (ppb)	Fluorometer scale	Fluorometer reading
		Calibration 4		
July 15	0925	0.5	3	0.63
July 15	0927	1	3	1.23
July 15	0929	5	10	4.30
July 15	0930	20	30	15.0
July 15	0932	25	30	18.5
July 15	0934	50.	100	40
July 15	0935	100	100	78
		Calibration 5		
July 16	0906	.5	3	.55
July 16	0907	1	3	1
July 16	0909	5	10	3.7
July 16	0911	20	30	13.
July 16	0913	25	30	15.5
July 16	0915	50	100	34
July 16	0917	100	100	68
		Calibration 5.5		
July 16	1057	1	3	.90
July 16	1059	5	10	3.5
July 16	1100	20	30	12.5
July 16	1102	25	30	15.5
July 16	1103	50	100	33
July 16	1105	100	100	64

APPENDIX 2. SUMMARY OF ALL DYE SAMPLES COLLECTED FROM THE CHICAGO LOCK, BOTH GATES SHUT, JULY 15, 1993

[ft, feet; ppb, parts per billion; ±, plus or minus; <, less than; calibration number refers to table 5; shaded lines indicate background samples; the suffix 's' on the sample number indicates a split sample; dashes indicate no data]

Sample number	Sample date	Sample time (hours)	Distance from left edge of water (ft)	Sample depth (ft)	Analysis date	Analysis time (hours)	Fluorometer readings	Calibration number	Dye concentration (ppb)	95-percent confidence interval for dye concentration (ppb)
113	July 15	0853	70	12	July 15	0910	±0.35	4	0.38	±0.01
114	July 15	0854	50	12	July 15	0912	±.40	4	.45	±.01
115	July 15	0855	30	12	July 15	0914	±.32	4	.34	±.01
116	July 15	0856	10	12	July 15	0916	±.30	4	.32	±.01
117	July 15	1104	70	6	July 16	0920	7.00	5	10.56	±.33
118	July 15	1105	70	12	July 16	0922	7.00	5	10.56	±.33
119	July 15	1108	70	18	July 16	0923	5.60	5	8.49	±.26
120	July 15	1110	50	6	July 16	0925	9.10	5	13.68	±.42
121	July 15	1111	50	12	July 16	0926	7.70	5	11.61	±.36
122	July 15	1112	50	18	July 16	0927	5.70	5	8.64	±.27
123	July 15	1114	30	6	July 16	0929	9.70	5	14.59	±.45
124	July 15	1115	30	12	July 16	0931	8.90	5	13.41	±.41
125	July 15	1116	30	18	July 16	0933	6.60	5	9.99	±.31
126	July 15	1117	10	6	July 16	0935	9.50	5	14.32	±.44
127	July 15	1118	10	12	July 16	0936	9.60	5	14.47	±.45
128	July 15	1119	10	18	July 16	0938	7.40	5	11.20	±.34
129	July 15	1211	40	6	July 16	0940	11.00	5	16.58	±.51
130	July 15	1213	40	12	July 16	0943	11.00	5	16.60	±.51
131	July 15	1214	40	18	July 16	0944	10.00	5	15.11	±.47
132	July 15	1253	70	6	July 16	0946	9.10	5	13.77	±.42
133	July 15	1254	70	12	July 16	0947	9.20	5	13.92	±.43
134	July 15	1254	70	18	July 16	0950	9.70	5	14.69	±.45
135	July 15	1255	50	6	July 16	0953	10.00	5	15.15	±.47
136	July 15	1256	50	12	July 16	0955	9.90	5	15.01	±.46
147	July 15	1256	50	12	July 16	0954	10.00	5	15.15	±.47
148	July 15	1257	50	18	July 16	0958	9.50	5	14.42	±.44
149	July 15	1258	30	6	July 16	1000	10.00	5	15.18	±.47
150	July 15	1258	30	12	July 16	1001	10.00	5	15.19	±.47
151	July 15	1259	30	18	July 16	1002	10.50	5	15.95	±.49
152	July 15	1259	30	18	July 16	1003	10.50	5	15.95	±.49
153	July 15	1301	10	6	July 16	1005	10.00	5	15.21	±.47
154	July 15	1301	10	6	July 16	1006	10.00	5	15.21	±.47
155	July 15	1302	10	12	July 16	1008	10.30	5	15.68	±.48
156	July 15	1302	10	18	July 16	1011	10.50	5	15.99	±.49
157	July 15	1304	30	6	July 16	1013	10.00	5	15.24	±.47
158	July 15	1304	30	12	July 16	1014	10.00	5	15.25	±.47
159	July 15	1305	30	18	July 16	1015	10.00	5	15.25	±.47
160	July 15	1307	50	6	July 16	1016	9.70	5	14.80	±.45
161	July 15	1307	50	12	July 16	1018	9.40	5	14.35	±.44
162	July 15	1308	50	18	July 16	1020	9.70	5	14.82	±.45

APPENDIX 2. SUMMARY OF ALL DYE SAMPLES COLLECTED FROM THE CHICAGO LOCK, BOTH GATES SHUT, JULY 15, 1993—Continued

Sample number	Sample date	Sample time (hours)	Distance from left edge of water (ft)	Sample depth (ft)	Analysis date	Analysis time (hours)	Fluorometer readings	Calibration number	Dye concentration (ppb)	95-percent confidence interval for dye concentration (ppb)
163	July 15	1309	70	6	July 16	1021	9.00	5	13.75	±0 .42
164	July 15	1310	70	12	July 16	1023	9.20	5	14.07	± .43
165	July 15	1311	70	18	July 16	1025	9.40	5	14.38	± .44
166	July 15	1327	70	6	July 16	1027	8.90	5	13.62	± .41
167	July 15	1328	70	12	July 16	1029	9.10	5	13.94	± .42
168	July 15	1329	70	18	July 16	1032	9.40	5	14.41	± .44
169	July 15	1330	50	6	July 16	1034	9.70	5	14.88	± .45
170	July 15	1330	50	12	July 16	1036	9.60	5	14.74	± .45
171	July 15	1331	50	18	July 16	1037	9.60	5	14.74	± .45
172	July 15	1333	30	6	July 16	1038	9.60	5	14.74	± .45
173	July 15	1333	30	12	July 16	1041	9.10	5	13.99	± .42
174	July 15	1334	30	18	July 16	1043	9.50	5	14.61	± .44
176	July 15	1336	10	6	July 16	1045	10.20	5	15.70	± .47
177	July 15	1337	10	12	July 16	1048	10.20	5	15.72	± .47

APPENDIX 3. REVIEW OF U.S. GEOLOGICAL SURVEY DATA COLLECTED IN THE CHICAGO AREA USING AN ACOUSTIC DOPPLER CURRENT PROFILER

Prepared by

James R. Marsden, Ph.D.
RD Instruments

9855 Businesspark Ave., San Diego, CA 92131 USA, Phone: 619-693-1178, Fax: 619-695-1459

Review of U.S. Geological Survey Data collected in the Chicago area using an Acoustic Doppler Current Profiler.

Discharge measurements at several sites in the Chicago area were collected by the U.S. Geological Survey between May and October 1993. The data were acquired using a Acoustic Doppler Current Profiler (ADCP). The sites are on the Chicago River and in or near control structures that allow water to pass from Lake Michigan into the Chicago River. Of particular interest for this review are those measurements collected when all sluice gates and/or locks were in their closed positions.

In the following, an overall review of the general quality of the data is given and some specific measurements are examined in detail. The emphasis is on determining the accuracy of the data relative to the expected performance of the instrument and the technique used to compute the discharge.

An ADCP measures a vertical profile of the horizontal velocity of the water. The velocity is determined in a number of discrete bins known as 'depth cells' (see Figure 1). The ADCP also measures its own velocity relative to the bottom of the channel. This velocity is then subtracted from the water's velocity relative to the ADCP to determine the velocity of the water relative to the earth. The total discharge through the section of the water actually measured by the ADCP is the time integral of the cross product of the ADCP's velocity and the water's velocity. This computation is done using software supplied with the ADCP.

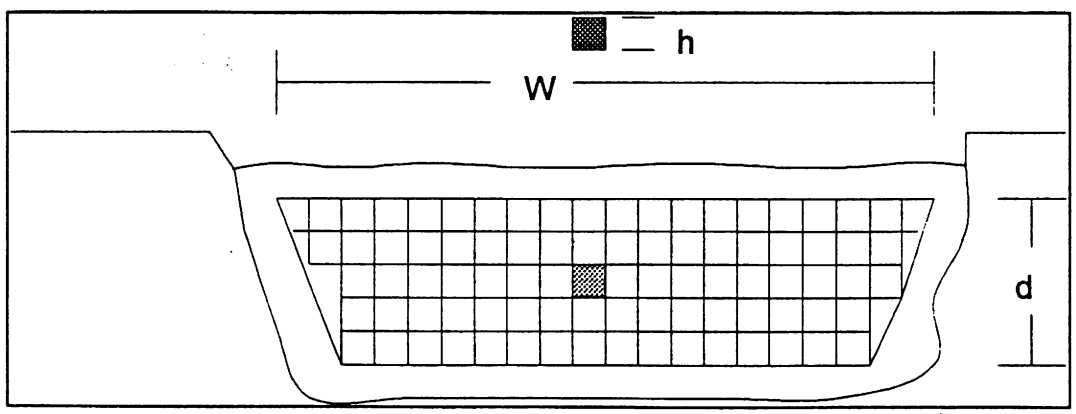


Figure 1
Channel section actually measured with an ADCP

It is important to note that the ADCP does not measure the entire water column. There are layers on the top and bottom that are not measured; likewise for the edges. The discharge in these unmeasured sections is extrapolated from the data actually acquired with the ADCP. The user has some control over the method of extrapolation.

The error of a discharge measurement has a number of possible sources. The prominent contributions are:

- the ADCP's inherent measurement errors
- extrapolation of the unmeasured discharge
- temporal fluctuations in flow

For the ADCP used in these measurements, it can be shown that the instrument's contribution to error in the discharge calculated for the section directly measured by the ADCP is given by

$$\Delta Q_{ADCP} = \sigma_v \sqrt{Whdv_b t}$$

where v_b is the average velocity of the boat,
σ_v is the single ping standard deviation of the ADCP,
t is the time for an individual ping.

The expected error in the top layer extrapolation (extrapolating the uppermost depth cell velocity to the surface, known as constant extrapolation) is given by

$$\Delta Q_{top} = \sigma_v l \sqrt{Wv_b t}$$

where l is the thickness of the extrapolated layer.

The error predicted for one side's discharge extrapolation is given by

$$\Delta Q_{side} = \sigma_v \frac{.707 L d_m}{2} \sqrt{\frac{h}{dw_p}}$$

where d_m is the actual depth of the vertical section nearest the shore
w_p is the number of pings in the vertical section.

Note that the section referred to above may have comprise many individual pings or ensembles of pings averaged together. The total side extrapolation error will include the error from both sides of the channel. The .707 factor may change depending on the geometry of the channel banks.

The three equations above allow the total measurement error to be estimated. There are other less significant errors; these will be ignored for the following discussion.

Case Study: Chicago River at Lake Shore Drive

As an example to analyze, consider the measurement on 9/21/93 in file RIV1043R.000. The approximate parameters for this run are

W	72 meters	L_1	2.5 meters
d	6 meters	L_2	5.0 meters
v_b	0.1 meters/sec	d_{m1}	3.5 meters
v_w	0.015 meters/sec	d_{m2}	5.0 meters
h	0.25 meters	t	1.0 sec
σ_v	0.10 meters/sec.	l	1.5 meters
w_p	5 pings		

The 0.707 factor used to estimate edge discharge was changed to 0.91 for this channel

From these parameters, the various components of the measurement error are estimated to be

$$\Delta Q_{ADCP} = 0.33 \text{ m}^3/\text{s} = 12 \text{ cfs}$$
$$\Delta Q_{top} = 0.40 \text{ m}^3/\text{s} = 14 \text{ cfs}$$
$$\Delta Q_{side1} = 0.07 \text{ m}^3/\text{s} = 2.5 \text{ cfs}$$
$$\Delta Q_{side2} = 0.15 \text{ m}^3/\text{s} = 5.1 \text{ cfs}$$

In addition to these errors, the other errors not accounted for in detail here (such as turbulence or temporal variations in the flow) will be, in aggregate, on the order of the top layer error. This gives a total estimated error of roughly 48 cfs for this measurement.

A total of 7 discharge measurements were made at this site on 9/21/93 under similar conditions. For those seven measurements, the calculated standard deviation is 67 cfs. This is in fair agreement with the error predicted above. The slightly higher than estimated error may be due to a shear in the vertical velocity profile that was present during these runs. The top few feet were flowing east while the underlying water was flowing west. This shear condition will add an error term that is not accounted for above.

With this one exception, the data from these measurements are consistent with the error that should be expected for the flow conditions

present. The overall quality of the data is quite good. The ADCP data itself exhibits good backscatter strength (for a good signal to noise ratio) and nearly ideal correlation (one of the ADCP's measures of the quality of the data). There are no BIT (built in test) errors in the data, so that based on the self test routines that occur on every ping the instrument appears to be functioning correctly.

Overview of the data at all three sites

Chicago River at Lake Shore Drive

Only data collected with the lock gates and sluice gates all closed are considered here. In particular the data from 9/20, 9/21, 10/5 and 10/6, 1993 are examined. For all four sets of data, the average flow velocity, and river cross section are similar. Based on the ADCP's indicators of data quality, the raw data are all valid. The average discharge (except for the edges) is 210 ± 85, 220 ± 70, 145 ± 10, and 140 ± 44 cfs for the three days respectively. And for each set of data, the edge distances and hence the edge contributions will be about the same. From the case study above, we know the edge discharge is about 10 cfs. The final values, which include edge discharge, given in Table 4 of the body of the report (Oberg and Schmidt, 1994) are reasonable and accurate values.

Thomas O'Brien Lock and Dam

Measurements of leakage through the lock were collected on two days, 9/14 and 9/17, 1993. On 9/14, the time taken to cross the channel was typically 3 to 4 minutes. while on 9/17, the time was between 15 and 20 minutes. The longer data acquisition times on 9/17 yielded much better quality data. Neglecting the edge areas, the discharge results were 19.5 ± 21.7 cfs and 22.1 ± 7.4 cfs on 9/14 and 9/17 respectively. Using the earlier formula for the error contributions, we predict an error of 2.7 cfs from the ADCP, and a top layer error of 4.0 cfs, for a total of 6.7 cfs from those sources for the data of 9/17. This is in excellent agreement with the actual data. For the data of 9/14, the same errors are 5.9 cfs and 8.9 cfs for a total of 14.8 cfs, also consistent with the actual results.

Data files OBR1093R.000 and OBR1099R.000 we examined in detail. All quality indications from the ADCP were good, and the instrument registered no self test errors. Using the average measured velocity and the edge distances for the runs of 9/17, the total edge discharge is 2.5 cfs. So, the total average leakage through the O'Brien Lock is roughly 24 ± 7.5 cfs on 9/17/93.

Wilmette Pumping Station at Wilmette, IL.

Again, only data collected with all gates closed is analyzed. For the data collected on 9/22/93, the flow was organized but very slow, with an average velocity of 0.023 ft/sec. The average discharge (without the edges) is 8.3 ± 1.3 cfs. This is consistent with a predicted error of 1.5 cfs for the channel parameters. With edge estimates, the flow is 9.0 ± 1.5 cfs. For the data collected on 9/23/93, the dominant flow feature is an eddy structure that fills the volume measured. The discharge measured with the ADCP is 3.1 ± 1.1 cfs. However, for this case, the edge estimates may dominate the total flow: this is because the highest velocities are at the edges of the flow rather than in the middle. An average velocity cannot be used to extrapolate the flow in these regions.

The data from files WLM1004R.000 and WLM1028R.000 were examined in detail. All the data were within acceptable limits by all criteria that the ADCP makes available. The speed of the boat was sufficiently slow to keep the measurement technique effects from dominating the data.

This site is the most difficult of the three because of its shallow depth and the very low water velocity. The data collected on 9/22/93 are the more reliable of the two data sets because the flow was more uniform without the large scale circulation evident in the data of 9/23/93.

Summary

All of the data that were reviewed appear to be of high quality. The ADCP had sufficient signal, the pulse-to-pulse correlation coefficient was within acceptable bounds, and the ADCP reported no BIT errors during the data sets reviewed in detail.

The statistical error of the data is in good agreement with theoretically predicted statistical error for the channel geometry and flow velocities involved at the three sites (when the edge discharge estimates are neglected).

An important parameter in achieving these results was the extremely slow boat speeds that were used on many of the measurement runs. The average boat speed was 2 cm/sec for many of these channel transects. This allowed a sufficiently large number of individual data points to be gathered to bring the statistical errors down low enough so as not to dominate the very low velocities encountered at these sites. These discharge measurements are the lowest values ever achieved with such good statistical reproducibility.

References

Oberg, K. A. and Schmidt, A. R., *Measurements of Leakage from Lake Michigan through Three Control Structures near Chicago, Illinois, April - October, 1993* (1994).

APPENDIX G

USGS LETTER REPORT ON SLUICE GATE MEASUREMENTS AT THREE LAKEFRONT CONTROL STRUCTURES NEAR CHICAGO, ILLIINOIS SEPTEMBER THROUGH OCTOBER 1993

United States Department of the Interior

U.S. GEOLOGICAL SURVEY

Water Resources Division
102 E. Main Street, 4th Floor
Urbana, IL 61801

July 21, 1994

Mr. Thomas Fogarty, P.E.
Chief, Hydrology and Hydraulics Branch
U.S. Army Corps of Engineers,
Chicago District,
111 N. Canal Street,
Chicago, IL 60606

Dear Tom:

Enclosed please find a summary table of the measurements of sluice-gate flows made by the U.S. Geological Survey (USGS) at the Chicago River Controlling Works (CRCW), Thomas J. O'Brien Lock and Dam (O'Brien), and Wilmette Pumping Station (Wilmette) in September and October 1993. An acoustic Doppler current profiler (ADCP) was used to make the discharge measurements. The use of ADCP's for streamflow measurement is discussed by Gordon (1989), Simpson and Oltmann (1993), and Oberg and Schmidt (1994).

The techniques used to measure sluice-gate flows were much the same as those described by Oberg and Schmidt (1994). Measurements were made immediately downstream of the sluice gates at O'Brien and Wilmette. Measurements of sluice-gate flows at CRCW were made at Lake Shore Drive. Prior to making any measurements, USGS personnel telephoned the Metropolitan Water Reclamation District of Greater Chicago (MWRD) dispatcher for gate settings and the time of the gate settings, if appropriate. Flow was allowed to stabilize, if necessary, and then a series of measurements were made.

Table 1 shows the date and time of each set of discharge measurements, the mean and standard deviation of the ADCP-measured discharge, and the number of ADCP measurements used to compute the mean and standard deviation. The total gate opening, gate configuration, and the mean head were obtained from data supplied by the MWRD during or shortly after the measurements. The rated discharges shown were computed using the mean head, total gate opening, and the rating published by the U.S. Army Corps of Engineers (1990, pages E1-E4). The percent difference was computed by subtracting the measured from the rated discharges and dividing by the rated discharge. It is important to note that flows measured at all three structures include leakage. This is especially important at CRCW. If possible, it would be better to

measure only the flow passing through the sluice gates at CRCW. I believe that this can be done using the ADCP.

A USGS report summarizing the results of the sluice-gate measurements is in preparation. The report will receive peer review prior to publication. Until the report is published, the ADCP-measured discharges must be considered preliminary. In addition, the rated discharges have not been checked by MWRD and were computed using the mean head for all ADCP measurements made that day (both leakage-only and sluice-gate measurements). The data shown in table 1 for O'Brien need to be reviewed further. More detailed head data are needed for all measurements before accurate comparisons between rated and measured discharges can be made.

If you have any questions about this letter and the enclosed table, please call me at (217) 398-5366.

Sincerely yours,

Kevin A. Oberg
Chief, Network Operations

Enclosure

REFERENCES

Gordon, R.L., 1989, Acoustic measurement of river discharge: Journal of Hydraulic Engineering, American Society of Civil Engineers, v. 115, no. 7, p. 925-936.

Oberg, K.A., and Schmidt, A.R., 1994, Measurements of leakage from Lake Michigan through three control structures near Chicago, Illinois, April - October 1993: U.S. Geological Survey Water-Resources Investigation Report 94-4112, 48 p.

Simpson, M.R., and Oltmann, R.N., 1993, Discharge-measurement system using an acoustic Doppler current profiler with applications to large rivers and estuaries: U.S. Geological Survey Water-Supply Paper 2395, 32 p.

U.S. Army Corps of Engineers, 1990, Current-meter measurements of discretionary and leakage flows at the Chicago River Controlling Works, O'Brien Lock and Dam, and the Wilmette Controlling Works": Chicago District, Lake Michigan Diversion Accounting Section, 25 p.

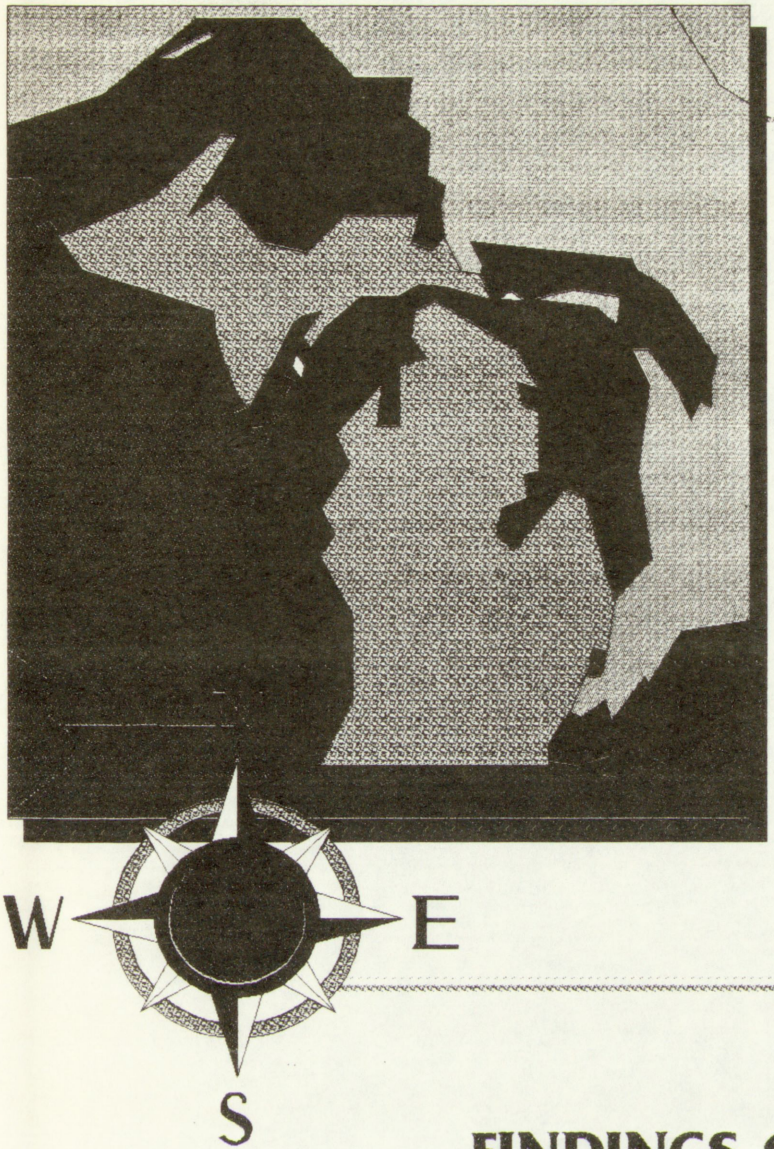
Table 1.-- Measurements of Sluice-gate Discharge and Leakage at Lake Michigan Control Structures near Chicago, Ill.

[Rated Discharge and Rated Leakage, determined from sluice-gate ratings published by U.S. Army Corps of Engineers, 1990, pages E1-E4; cfs, cubic feet per second; Measured Discharge, mean of discharge measurements; Standard Deviation, standard deviation of measured discharge; Difference, Total Rated Discharge minus Measured Discharge]

Date	Start Time	End Time	Total Gate Opening, (ft)	Description	Mean Head (ft)	Rated Discharge (cfs)	Rated Leakage (cfs)	Total Rated Discharge (cfs)	Measured Discharge (cfs)	Standard Deviation (cfs)	Difference (cfs)	Difference (percent)	Number of measurements
Chicago River Controlling Works													
09-21-93	12:58	14:15	2	Two gates at 1 ft. each	3.01	167	35.3	202	412	106	-209	103	5
09-21-93	08:40	11:01	3	Two gates at 1.5 ft. each	3.01	251	35.3	286	412	70	-125	44	10
10-04-93	14:37	15:46	6	Two gates at 3 ft. each	2.72	476	33.5	510	708	83	-198	39	4
10-05-93	07:50	09:47	8	Two gates at 4 ft. each	2.20	572	30.1	602	785	54	-183	30	7
10-05-93	15:24	16:10	10	Two gates at 5 ft. each	2.20	715	30.1	745	1027	53	-282	38	4
10-06-93	09:35	10:36	10	Two gates at 5 ft. each	2.10	699	29.4	728	940	67	-211	29	5
Thomas J. O'Brien Lock and Dam													
09-13-93	13:16	13:25	6	Three gates at 2 ft. each	3.31	674	23.3	697	770	471	-73	10	2
09-15-93	09:44	14:23	6	Gate settings unknown	3.05	648	22.4	670	611	165	59	-9	19
09-16-93	10:07	10:49	6	Two gates at 3 ft. each	2.91	631	21.8	653	535	80	118	-18	4
09-16-93	11:08	12:32	6	One gate at 6 ft.	2.91	631	21.8	653	737	200	-84	13	8
09-16-93	13:02	14:35	6	Three gates at 2 ft. each	2.91	631	21.8	653	730	188	-77	12	5
09-16-93	----	----	6	Average for 6 ft. opening	2.91	631	21.8	653	687	188	-34	5	17
09-13-93	13:34	14:17	10	Two gates at 4 ft., One gate at 2 ft.	3.31	1,123	23.3	1,146	953	350	194	-17	5
09-13-93	14:19	14:30	16	Two gates at 5 ft, One gate at 6 ft.	3.31	1,796	23.3	1,819	1510	85	309	-17	3
Wilmette Pumping Station													
09-23-93	10:32	11:59	0.5	One gate at 0.5 ft.	3.01	118	5.3	123	71	46	52	-42	13

APPENDIX H

FINDINGS OF THE THIRD TECHNICAL COMMITTEE FOR REVIEW OF DIVERSION FLOW MEASUREMENTS AND ACCOUNTING PROCEDURES



LAKE MICHIGAN DIVERSION

FINDINGS OF THE THIRD TECHNICAL COMMITTEE FOR REVIEW OF DIVERSION FLOW MEASUREMENTS AND ACCOUNTING PROCEDURES

AUGUST 5, 1994

Committee Members:
Dr. W. H. Espey, Jr., Chairman
Mr. O. G. Lara
Dr. R. L. Barkau

Prepared for:
U.S. Army Corps of Engineers
Chicago District
Chicago, Illinois

TABLE OF CONTENTS

Section			Page
	List of Tables		i
	List of Figures		ii
	List of Appendices		iv
	Acknowledgements		v
	Executive Summary		vi
1.0	INTRODUCTION		1
	1.1	Purpose and Scope	1
		1.1.1 First Technical Committee	1
		1.1.2 Second Technical Committee	1
		1.1.3 Third Technical Committee	2
	1.2	History of Lake Michigan Diversion	3
	1.3	Components of Diversion	6
	1.4	Committee Findings	9
		1.4.1 Review of First Committee Report	9
		1.4.2 Review of Second Committee Report	10
		1.4.2.1 First Technical Committee Report (1981)	10
		1.4.2.2 Diversion Accounting Reports WY 1981-84	10
		1.4.2.3 Diversion Accounting Procedures	11
		1.4.2.4 Acoustical Velocity Meter - Romeoville	12
		1.4.2.5 Lockport Lock and Dam Facilities	12
		1.4.2.6 TARP	13
2.0	LAKE MICHIGAN DIVERSION ACCOUNTING PERIOD 1981-1989		14
	2.1	First and Second Technical Committees' Findings Accounting Period 1981-1989	14
		2.1.1 1983 Annual Report	14
		2.1.2 1985 Annual Report	15
		2.1.3 1986 Annual Report	15
		2.1.4 Summary of First and Second Technical Committees' Findings	16

2.2 Third Technical Committee Review and Recommendations Accounting Period 1987-1989 17

2.2.1 1987 Annual Report (No Accounting Diversion) 20
2.2.2 1988 Annual Report (No Accounting Diversion) 21
2.2.3 1989 Annual Report (Accounting Years 1984-1985) 22
2.2.4 1990-1992 Annual Report (Accounting Years 1986, 1987, 1988, and 1989) 23
2.2.5 Summary of Lake Michigan Diversion Accounting 24
2.2.6 Technical Committee's Findings and Recommendations . . . 25

3.0 REVIEW OF CURRENT ACCOUNTING SYSTEM 29

3.1 Accounting Report Table 30

3.2 Accounting Hydrologic Modeling 38

3.2.1 Modeling Approach 39
3.2.1.1 Precipitation Network 44
3.2.1.2 HSPF Model 50
3.2.1.3 Interception Storage 50
3.2.1.4 Impervious Area Runoff 50
3.2.1.5 Infiltration 50
3.2.1.6 Upper Zone Storage 51
3.2.1.7 Overland Flow 51
3.2.1.8 Lower Zone 51
3.2.1.9 Interflow 51
3.2.1.10 Groundwater and Deep Percolation 51
3.2.1.11 Evaportranspiration 52

3.2.2 SCALP 52

3.2.3 TNET 54

3.2.4 Comments 55

4.0 AVM SYSTEM AT ROMEOVILLE 66

4.1 AVM System 67

4.2 Development of a Discharge Rating 68

4.2.1 Rating Velocity 70

4.2.2 Rating Area 70

4.3 Romeoville AVM Station 71

4.4 Review of the Romeoville AVM Ratings 74

4.4.1 Methods for Daily Discharge Completion 76

4.4.2 Discharge Equations 76

4.4.2.1 Measured Discharge vs Sarasota AVM Discharge . . 81
4.4.2.2 Measured Discharge vs. ORE AVM Discharge 82

4.5 Backup System Regression Equations 84

4.5.1 Index Velocity Method 86

4.5.2 Area Rating 87

5.0 LEAKAGE AT LAKE MICHIGAN CONTROL STRUCTURES 92

6.0 TURBINE DISCHARGE RATING 101

6.1 Third Committee 104

6.2 Recommendations 105

7.0 SUMMARY RECOMMENDATIONS 119

7.1 First and Second Committee Recommendations 119

7.2 Master Plan 119

7.3 Quality Assurance Plan 120

7.4 AVM System Recommendations 120

7.5 Lakefront Measurements 121

7.6 Lockport/Romeoville Regression Equations 122

7.7 Annual Diversion Report 122

7.8 Des Plaines Pumping Station 122

7.9 Grand Calumet River 122

8.0 REFERENCES 124

9.0 APPENDICES 126

LIST OF TABLES

Table		Page Number
2.1	Chronological Summary of Lake Michigan Diversion Events	18
2.2	Status of the State of Illinois' Diversion from Lake Michigan Under the 1980 Modified U.S. Supreme Court Decree	24
3.1	Description of Diversion Accounting Columns	31
3.2	Lake Michigan Diversion Accounting, Water Year 1989	32
3.3	Cumulative Corrections and Percent Changes for Gages 1987 Water Year	41
3.4	WY 1989 Summary of Simulation Statistics	56
3.5	Upper Des Plaines Pumping Station Updated Model Simulated Flow	58
3.6	Sanitary Flow Used in the SCALP Program for Selected Watershed	59
3.7	Sewer System Evaluation Survey	65
4.1	Summary of Discharge Measurements for Chicago Sanitary and Ship Canal at Romeoville, Illinois	75
4.2	Discharge Equations for the AVM at Romeoville, Illinois	77
4.3	Discharge Equations for the AVM at Romeoville, Illinois for the Period November 17, 1988 to November 2, 1990.	81
4.4	Discharge Equations for the AVM at Romeoville, Illinois for the Period November 2, 1990 to May 16, 1991.	89
4.5	Discharge Equations for the AVM at Romeoville, Illinois for the Period May 16, 1991 to Present.	90
5.1	Preliminary Results of Measurements of Leakage through Chicago Lakefront Control Structure (1993)	96
5.2	Summary of the Average Extreme Levels - Lake Michigan	101
6.1	Revised WY 1981, 1982, and 1983	104
6.2	Mean Daily Discharges for Romeoville AVM and Lockport, December 1990	112
6.3	Mean Daily Discharges for Romeoville AVM and Lockport, July - September 1991	114
6.4	Selected Mean Daily Lockport and Romeoville Discharges August - October, 1992	118

LIST OF FIGURES

Figure		Page Number
1.1	Canal System - Before and After	4
1.2	Lake Michigan Diversion at Chicago	7
1.3	Component Breakdown of Illinois' Lake Michigan Diversion 1989	8
2.1	WY 81-89 Pumpage, Runoff, Diversion, and Rainfall Trends	28
3.1	Rain Gage Network	42
3.2	Precipitation Patterns, 1984, 1985, 1987	43
3.3	Water Year 1987 Adjusted Precipitation Pattern	45
3.4	Chicago Diversion Precipitation Network	46
3.5	Precipitation Patterns for Water Year 1990	47
3.6	Precipitation Patterns for Water Year 1991	48
3.7	Precipitation Patterns for Water Year 1992	49
3.8	Budget 8 - 1985 Des Plaines	61
3.9	Budget 8 - 1986 Des Plaines	62
3.10	Budget 8 - 1987 Des Plaines	63
3.11	Budget 8 - 1988 Des Plaines	64
4.1	Sketch of Acoustic Velocity Meter, Chicago Sanitary and Ship Canal at Romeoville, Illinois	73
4.2	Diagram of Channel at Romeoville, Illinois	79
4.3	Comparison Between AVM and Measured Discharge - Sarasota AVM (1-41)	80
4.4	Comparison Between AVM and Measured Discharges - ORE AVM	83
5.1	Chicago River Controlling Works	93
5.2	O'Brien Lock and Dam	94
5.3	Longterm Fluctuations in the Water Level of Lake Michigan 1950-90	99
6.1a	MSD and AVM - Adjusted Discharge	107

LIST OF FIGURES

Figure		Page Number
6.1b	MSD and AVM - Adjusted Discharge	108
6.1c	MSD and AVM - Adjusted Discharge	109
6.1d	MSD and AVM - Adjusted Discharge	110
6.2	Flow Difference (AVM-MSD)	111
6.3	Difference in Mean Daily Lockport Discharge and Romeoville AVM Discharge October - December 1990	116
6.4	Difference in Mean Daily Lockport Discharge and Romeoville AVM Discharge July - September 1991	117

APPENDIX

Appendix 1 Diversion Accounting Table Columns 4-13

Appendix 2 Description of the Diversion Accounting Computational Budgets

Appendix 3 USGS's Romeoville AVM Station Description

Appendix 4 USGS's Romeoville AVM Station Analysis 1989-1992

Appendix 5 Stability of the Romeoville Measuring Section

Appendix 6 Comments on Technical Committee Finds Regarding the Romeoville AVM Stage-area Rating

Appendix 7 Romeoville AVM Quality Assurance Plan

Appendix 8 Committee's Workshop Agendas

ACKNOWLEDGEMENTS

During the course of the Committee's work, a number of individuals and organizations were extremely helpful in assisting the Committee to accomplish its tasks. The Committee would like to express its appreciation for the assistance of the U.S. Army Corps of Engineers (USACE), Chicago District, and specifically, Mr. David Kiel and Mr. Thomas Fogarty for their contribution in providing numerous reports and data, and arranging and assisting the Committee in the various workshops and field trips. The Committee also acknowledges the assistance of Mr. Daniel Injerd, Illinois Department of Transportation (IDOT). Additionally, the Committee acknowledges the invaluable assistance from the U.S. Geological Survey (USGS) and especially from Mr. Kevin A. Oberg. The Committee acknowledges Ms. Mary Ellen Brown for her invaluable assistance in coordinating the preparation of this report and Mr. Harmon C. Miller for graphics support.

EXECUTIVE SUMMARY

The Third Technical Committee was appointed by the Corps of Engineers to conduct an assessment and evaluation of the accounting procedure and methodology used in the determination of the diversion from Lake Michigan and ascertain whether or not the methods are in accordance with the best current engineering practice and scientific knowledge, as stipulated in the 1980 modified Decree of the U.S. Supreme Court. This review is to be performed by a Technical Committee every five years, and a report evaluating the accounting and operational procedures is to be presented to the U.S. Army Corps of Engineers (USACE) and to other interested parties.

The Third Technical Committee was appointed by the Corps of Engineers in July of 1992 and convened in February of 1993. The Committee was appointed to conduct a comprehensive review of the current diversion accounting procedures. This task would consist of a review of the First and Second Committees' reports. Work would also include review of the automated diversion accounting system, the current diversion related measurement techniques at the Acoustical Velocity Meter Site, the Lockport control structure, the precipitation gage network, and other measurement locations to determine that these existing accounting measurement techniques and procedures are in accordance with the stipulation of the 1980 Supreme Court Decree. The specific objectives of the Committee were to (1) analyze of current diversion related measurement techniques and accounting procedures; (2) evaluate these techniques and procedures as to whether they are using the best current engineering practice and scientific knowledge; (3) recommend appropriate revisions within the legal constraints of the Decree; and (4) prepare draft and final reports.

The Third Technical Committee is in agreement with the findings of the 1987 report of the Second Technical Committee (Espey, et al, 1987). Considerable progress has been made in improvement of the accuracy of the current Lake Michigan diversion accounting procedures and measurements. Both the First and Second Technical Committees' recommendations have in general been initiated and/or completed. Significant progress has been made in upgrading the overall diversion program since the First Technical report in 1981.

With the incorporation of more sophisticated computer hydrologic models (HSPF, TNET, SCALP, etc.) in the accounting procedure, considerable effort has been expended to update basic model input and parameters as recommended by the Second Technical Committee. The Committee applauds the initiative of the Corps of Engineers for developing a better understanding of the complex hydrologic aspects of the accounting process with regards to flow diversion from Lake Michigan.

After confronting innumerable problems with the Sarasota AVM system which became operational in June 12, 1984, a new system (ORE, Inc.) was installed in November 1988, and reliable and more accurate flow records were being recorded by December 1, 1988. Analysis of these new data indicated the present flow diversion exceeds the limits set by the Court. The Committee concluded that the use of the more reliable AVM system contributed significantly to the improvement in accuracy of the accounting procedures.

The 1990-92 Annual Reports (USACE), which include Water Years 1986 through 1989 diversion, results indicate that the State of Illinois has exceeded the cumulative deviation (2,000 cfs) from the annual average diversion of 3,200 cfs by 2,189 cfs for Water Years 1981-1989. The Committee received (late July 1994) a draft copy of the Lake

Michigan Diversion Accounting Water Year 1993 Annual Report which includes 1990 diversion. This report indicates that the State of Illinois diverted 3,531 (Water Year 1990) and that the cumulative diversion has increased to 2,520 cfs years (Water Years 1981-1990).

The Committee's review and evaluation of Lockport and AVM records further support the conclusion that the earlier records at Lockport for low flow conditions were under reported, which represent primarily turbine flow. Since flow through the turbines on an annual basis represents approximately three-quarters of the amount of Lake Michigan diversion, the Committee recommends that Water Years 1981 through 1984 be re-evaluated with regards to the amount of diversion. The Committee recognizes the position of the Corps as stated in their 1988 annual report with respect to certification and the "moving target" issue related to management of Lake Michigan diversion. The Committee has concluded that the primary reason for the Lake Michigan diversion exceeding the flow limits of the Court is the improved accuracy of the accounting procedures and measurements. The major part of this improved accuracy can be attributed to the AVM system. The Committee has concluded, because of the significance of the 1990-92 Annual Report (Water Years 1986-89) and the draft 1993 Annual Report (Water Year 1990) indicating that the State of Illinois has exceeded the allowable cumulative diversion amount, that these diversion results will probably result in re-evaluation of the amount of Lake Michigan diversion. If this is the case, a re-evaluation of the entire record, beginning with 1980, would allow a more informed evaluation for possible future modifications of the amount of Lake Michigan diversion. In the Committee's opinion, more timely release of the Annual Reports of Lake Michigan

diversion to the Court and all interested parties is important for timely and proper water management of Lake Michigan.

CHAPTER 1

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The modified U.S. Supreme Court Decree for the Lake Michigan Diversion at Chicago, Illinois, adopted by the Court on December 1, 1980, provides the U.S. Army Corps of Engineers (USACE) convene a three-member Technical Committee at least every five years to review and report on the methods of flow measurement and procedures for diversion accounting. The Committee review is to include an evaluation of the current procedures used for the measurement and accounting of diversion in accordance with the best current engineering practice and scientific knowledge and recommendations for any appropriate changes to those procedures.

1.1.1 First Technical Committee

The first three-member Technical Committee was appointed in 1981 to review the diversion accounting procedures employed at that time. Their review and recommendations were reported (October 1981), and presented to the USACE for its use in providing supervision and direction of flow measurements, computations, and the accounting of diversion. As a result of review and comments by USACE, IDOT, MSD, and various other interested parties, an Addendum to the report was issued on March 30, 1982.

1.1.2 Second Technical Committee

In July 1986, the second three-member Technical Committee was appointed by the USACE to conduct its review of the diversion measurement and

accounting procedures. Two of the three members of the Second Committee were also members of the First Committee. The purpose of the Second Committee review was twofold. First, to review the First Committee recommendations and the subsequent changes to the diversion measurements and accounting procedures during the past five years. Second, to review the current measurement techniques and accounting procedures used for computing diversion as being in accordance with the best current engineering practice and scientific knowledge. The basic tasks included: 1) analysis of current diversion-related measurement techniques and accounting procedures, 2) evaluation of these techniques and procedures as to whether the best current engineering practice and scientific knowledge are used, 3) recommendation for appropriate revisions within the legal constraints of the decree, and 4) preparation of draft and final reports.

1.1.3 Third Technical Committee

The third three-member Technical Committee was appointed by the USACE in July 1992, and convened in February of 1993, to conduct a review of the first and second Committees' reports and a comprehensive review of the current diversion accounting procedures. The Chairman, Dr. William H. Espey, Jr., has served in that capacity for both the first and second committees. Mr. Oscar Lara and Dr. Robert L. Barkau are new members.

More specifically, the work included the review of the automated diversion accounting system as well as the review of current diversion related measurement

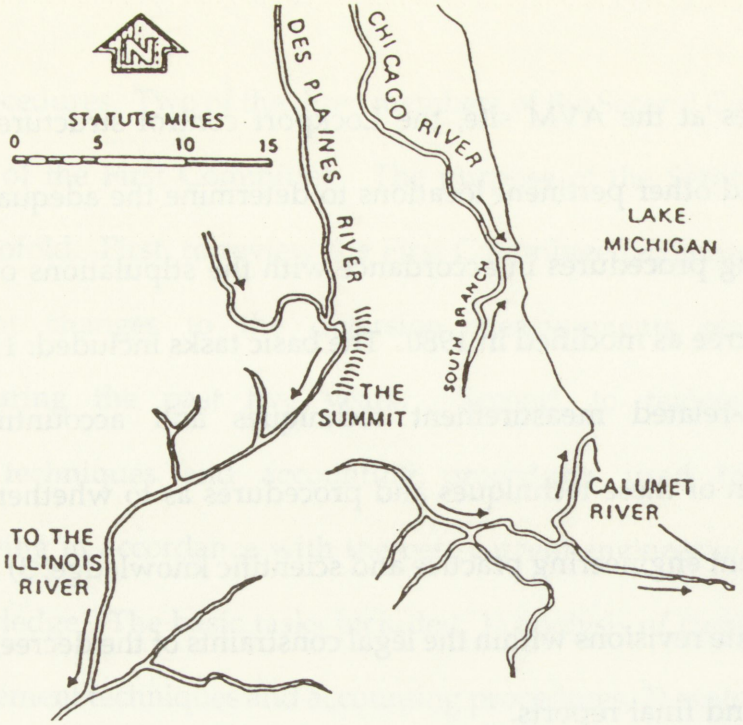
techniques at the AVM site, the Lockport control structures, the precipitation gages, and other pertinent locations to determine the adequacy of these existing accounting procedures in accordance with the stipulations of the 1967 Supreme Court decree as modified in 1980. The basic tasks included: 1) analysis of current diversion-related measurement techniques and accounting procedures, 2) evaluation of these techniques and procedures as to whether they are using the best current engineering practice and scientific knowledge, 3) recommendation of appropriate revisions within the legal constraints of the decree, and 4) preparation of draft and final reports.

1.2 History of Lake Michigan Diversion

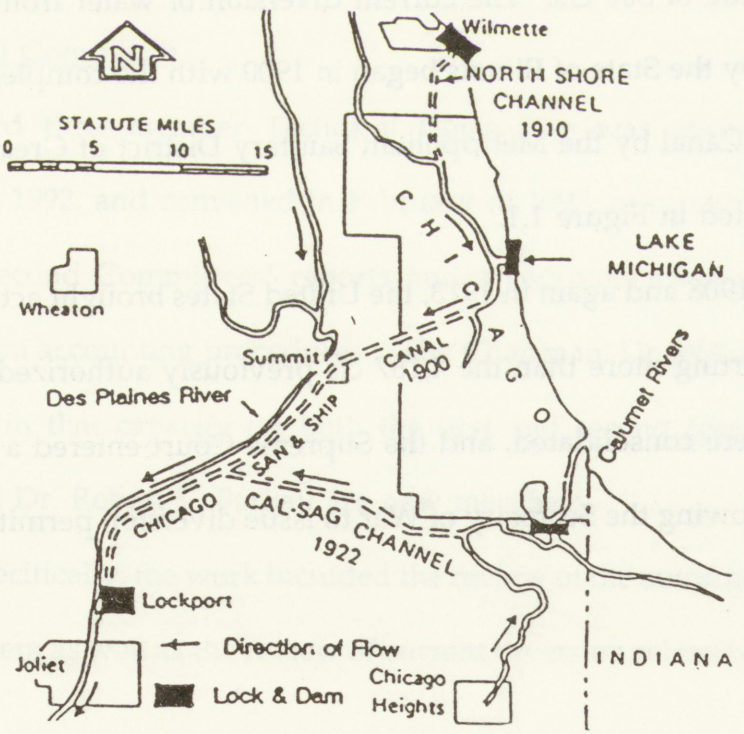
Water diversion from Lake Michigan at Chicago into the Mississippi River basin began in 1848 upon the completion of the Illinois and Michigan Canal at an average rate of 500 cfs. The current diversion of water from Lake Michigan at Chicago by the State of Illinois began in 1900 with the completion of the Sanitary and Ship Canal by the Metropolitan Sanitary District of Greater Chicago (MSD) as illustrated in Figure 1.1.

In 1908 and again in 1913, the United States brought actions to enjoin MSD from diverting more than the 4,167 cfs previously authorized in 1901. The two actions were consolidated, and the Supreme Court entered a decree on January 5, 1925 allowing the Secretary of War to issue diversion permits. In March of the

FIGURE 1.1



Rivers in Chicago Region Before 1848



Diversion in Canal System

same year, a permit was issued to divert 8,500 cfs which was about the average then being used.

In 1922, 1925, and finally in 1926, several Great Lakes states filed similar original actions in the U.S. Supreme Court seeking to restrict diversion at Chicago. A Special Master, appointed by the Court to hear the combined three suits, found the 1925 permit to be valid and recommended dismissal of the action. However, the Supreme Court reversed his finding. Subsequently, the Court instructed the Special Master to determine the steps necessary for Illinois and MSD to reduce diversion. Consequently, a 1930 Decree reduced the allowable diversion (in addition to domestic pumpage) in three steps: 6,500 cfs, after July 1, 1930; 5,000 cfs after December 30, 1935; and 1,500 cfs after December 31, 1938.

The 1967 Decree limited the diversion, including domestic pumpage, to an average of 3,200 cfs over a five-year running accounting period. The first accounting period began March 1, 1970 and ended to February 28, 1975. During this period, the average diversion was 3,183 cfs. The sixth and latest accounting period began March 1, 1975 and ended February 29, 1980. During this period, the average diversion was 3,044 cfs.

The U.S. Supreme Court amended its 1967 Decree on December 1, 1980. The amendment changes, in part, the provisions of the 1967 Decree which prevented the State of Illinois from effectively utilizing and managing the 3,200 cfs of Lake Michigan water which had been allocated previously by the Court.

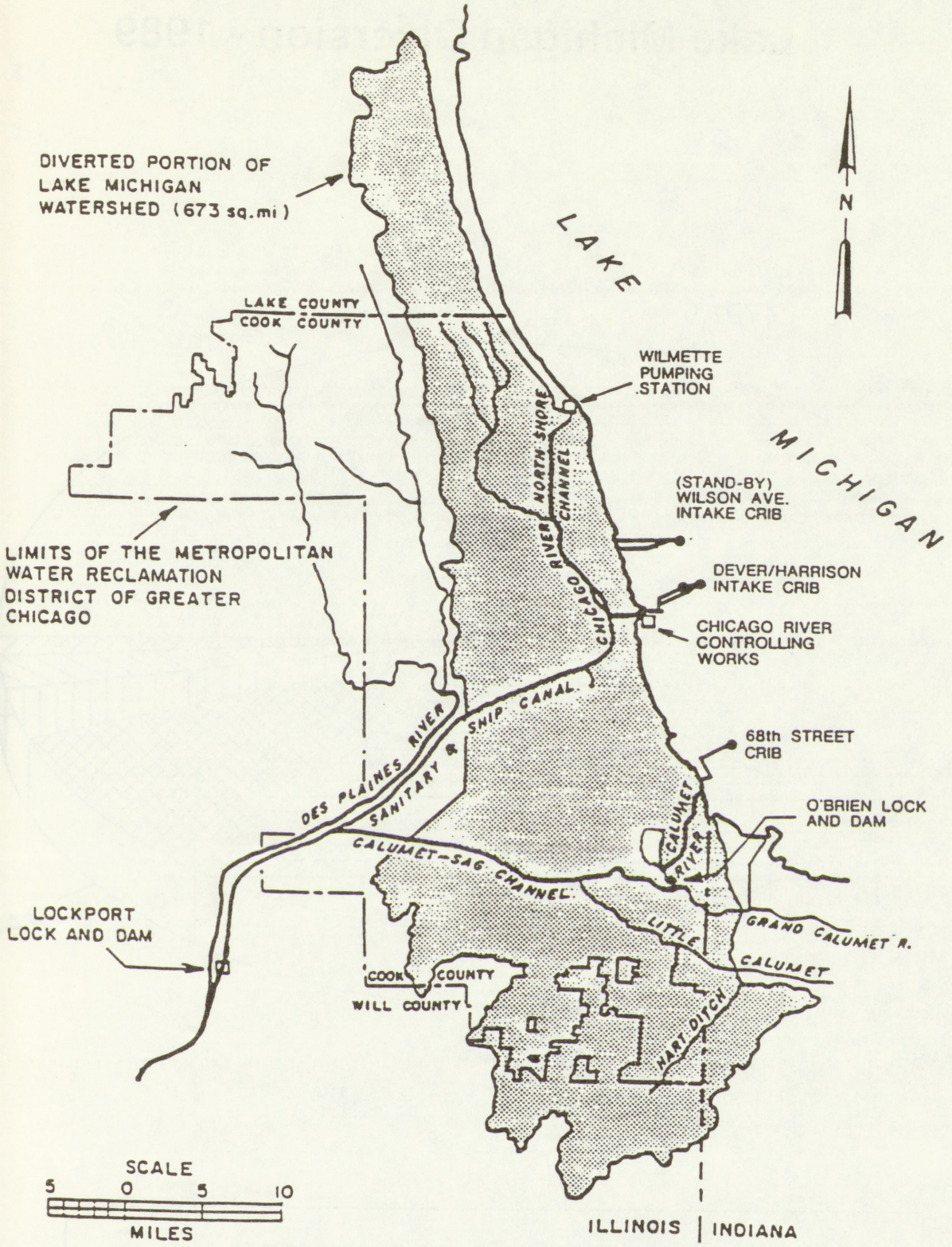
This amendment forms the current diversion criteria this report addresses. These criteria can be summarized as follows:

1. An increase in the period for determining compliance with the diversion rate limit from a 5-year running average to a 40-year running average;
2. Changing the beginning of the accounting year from March 1 to October 1;
3. A limit on the average diversion in any accounting year to 3,680 cfs, except for an average diversion of 3,840 cfs in any two accounting years within a 40-year period; and
4. A limit on the cumulative algebraic sum of the average annual diversions minus 3,200 cfs during the first 39 years to 2,000 cfs-years.

1.3 COMPONENTS OF DIVERSION

The geographic area of concern is illustrated in Figure 1.2 which shows part of Lake Michigan, the diverted watershed, the canal system, and the location of the major hydraulic structures. The primary components of the Lake Michigan diversion are illustrated in Figure 1.3. The contribution shown for each water supply for Accounting Year 1989 from Lake Michigan intake cribs and discharged into the river and canal system in the greater Chicago area as treated sewage is 61.1 percent.

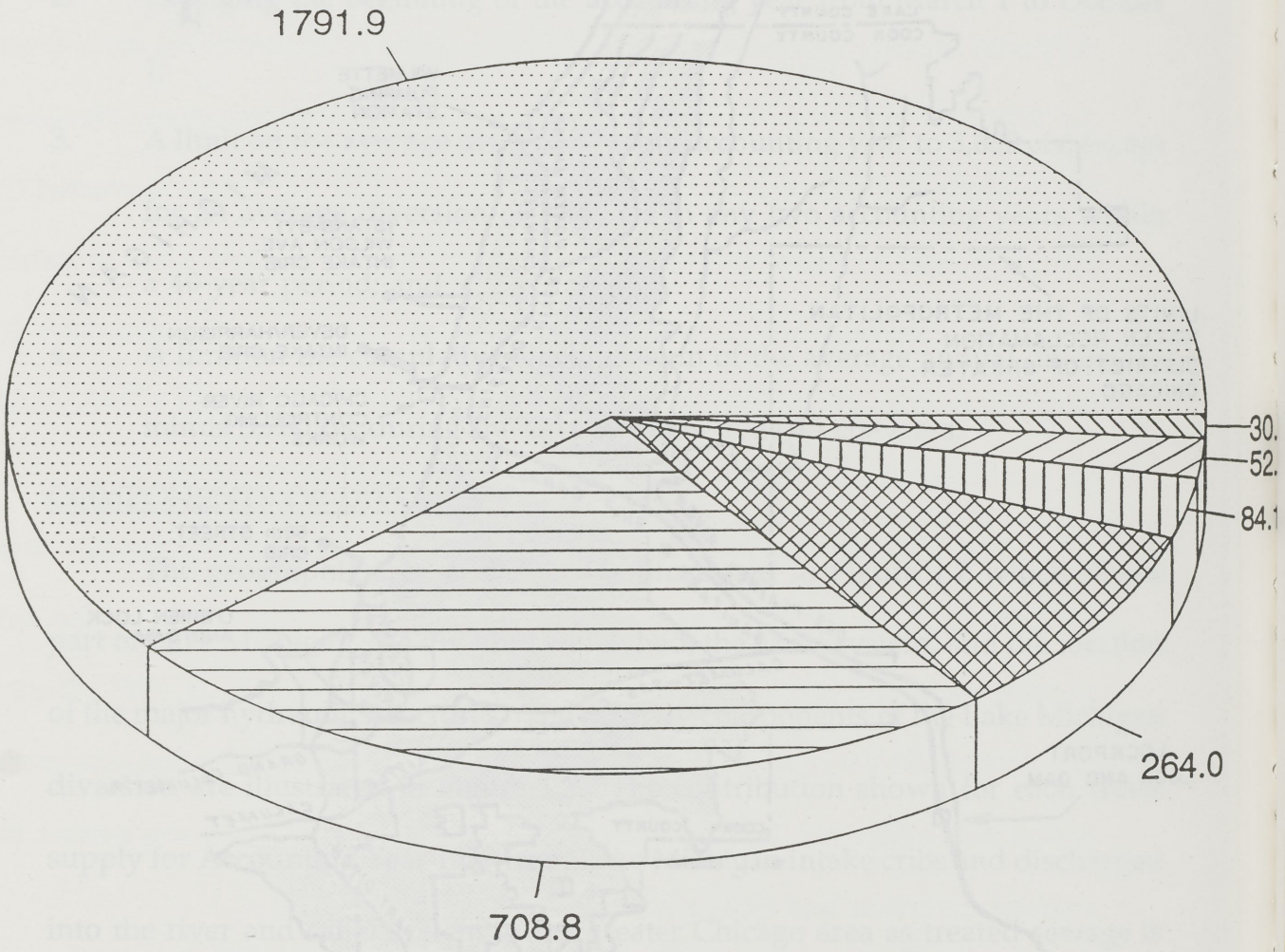
FIGURE 1.2



LOCATION PLAN-LAKE MICHIGAN DIVERSION AT CHICAGO

FIGURE 1.3

Component Breakdown of Illinois' Lake Michigan Diversion - 1989



DIVERSION COMPONENTS

WATER SUPPLY	61.1 %	RUNOFF	24.2 %
DISCRETIONARY	9.0 %	LOCKAGES	2.9 %
NAV MAKEUP	1.8 %	LEAKAGES	1.0 %

1.4 COMMITTEE FINDINGS

1.4.1 Review of First Committee Report

The report of the first three-member Technical Committee, dated October 1981, provided a discussion of the history of diversion, the components of diversion, and the various flow measurements and computations used to determine diversion. The First Committee found virtually every aspect of the program to account for diversion of Lake Michigan to be in need of improvement. The diversion accounting process lacked credibility. The measurement of flow at Lockport, the cornerstone for diversion accounting, at that time, was found to be lacking with respect to best current engineering practice and scientific knowledge and deficient in terms of techniques and reliability.

The First Committee report was reviewed to establish a base of reference for the evaluation of diversion activities initiated since 1981. The following brief summary of recommendations made by the First Committee was useful for that purpose:

- Preparation of a Master Plan for Diversion Accounting;
- Establishment of a Quality Assurance Program including an Operational Procedure Manual;
- Consideration of Alternatives to Measurements at Lockport Facilities; and
- Modifications and improvements to Flow Measurement Practices for Facilities including:

- Lockport turbines
- Lockport powerhouse sluice gates
- Lockport controlling works

The Committee also recommended modifications to flow measurements practices for Lockport lock leakage.

- Monitoring of Flow Measurement Devices for Industrial Diversion
- Modifications for Flow Measurement Practices for Domestic Pumpage
- Modifications to Flow Measurement Practices for Storm Runoff and Infiltration from Illinois Watershed
- Revaluation of Upper Des Plaines Pumping Station Data

1.4.2 Review of Second Committee Report

The following is a brief summary of the major conclusions and recommendations of the Second Committee.

1.4.2.1 First Technical Committee Report (1981)

1. The Second Committee was in general agreement with the findings and recommendations made by the First Committee (1981).
2. The Master Plan for diversion accounting and the Quality Assurance program are essential elements of the diversion accounting program.

1.4.2.2 Diversion Accounting Reports Water Years 1981 -1984

1. The diversion accounting certification report should provide the reader a narrative description of the facts which support the certification evaluation.

2. At some appropriate time, probably no earlier than after the completion of the 1987 Water Year, the diversion records for water years after 1980, should be reviewed, and if appropriate, revised as necessary to account for the apparent errors in the Lockport discharge rating used during the 1981-84 Water Years.

1.4.2.3 Diversion Accounting Procedures

1. Columns 7 and 9, representing the so-called sewer induced groundwater inflow should be withdrawn from the diversion accounting format.
2. Columns 4 and 15 of the Diversion Accounting Report should be discontinued.
3. Action should be initiated to address the deficiencies in the data bases for parameter values and model calibration, verification, and simulation, especially as they pertain to those drainage areas used directly in computing diversion (e.g., Column 8 deductions).
4. Examine the constancy of the relation between water supply pumpage and sewage treatment plan inflows and its applications for the purpose of estimating infiltration and inflow deductions for the Des Plaines watershed.
5. Reconsider the alternatives (modeling, etc.) for estimating the annual runoff from the Lake Michigan watershed.
6. Redefine or restate the goals and technical objectives for the simulation modeling.

7. Evaluate the simulation modeling requirements within the frame of reference provided by recommendations 2-6 above.

8. Measures should be taken, depending on the response to the broader recommendations given above, to deal with the unreported inflows perceived to be derived from the waterways and reported in Columns 8 and 14.

1.4.2.4 Acoustical Velocity Meter - Romeoville

1. The efforts by the USGS to establish written guidelines to promote improvement in the quality of the AVM records should be continued.

2. The current regressions of the daily discharges for the AVM and MSD Lockport, used for the AVM back-up, should be reconsidered specifically giving attention to the actual Lockport operating configurations.

3. A technical review of the AVM flow records should be conducted annually by the participating agencies.

4. The flow records for the AVM and MSD Lockport should be reviewed and compared for consistency on an annual basis.

5. The mean bed elevation for the canal in the reach defined by the transducer locations should be determined, as well as along the transducer paths.

1.4.2.5 Lockport Lock and Dam Facilities

1. The Lockport facilities of MSDGC and USACE should be used for the back-up to the AVM system at Romeoville.

2. Execute a set of field measurements designed to verify the WES ratings for both the Lockport Powerhouse sluice gates and the Controlling Works.

1.4.2.6 TARP

1. Infiltration and inflow of groundwater into the TARP tunnels should be treated as a deduction to the flows measured at Lockport. It should be computed by using the inflow rate of 0.05 MGD per mile of tunnel.
2. The runoff to the TARP system from the Lower Des Plaines combined sewer system should be determined and included in Column 8 as a deduction.

2.0 REVIEW OF LAKE MICHIGAN DIVERSION ACCOUNTING PERIOD 1981-1989

2.1 FIRST AND SECOND TECHNICAL COMMITTEES FINDINGS - ACCOUNTING PERIOD 1981-1986

Based in part on the recommendations (by the First and Second Technical Committees) to diversion accounting procedures along with the modifications to diversion accounting as contained in the amended Decree, numerous changes were made to the Lake Michigan accounting procedures for diversion during the past nine years. To logically review and evaluate these changes in chronological order, a discussion will be presented of each annual report prepared by the USACE. The three annual reports on Lake Michigan diversion reviewed by the First Technical Committee provide a detailed accounting of diversion flow for Water Years 1981-83 and diversion-related activities through Water Year 1986. Under the terms of the modified Decree, the USACE is required to report annually to the parties of the litigation on the measurement and computation of diversion by the State of Illinois.

2.1.1 1983 Annual Report

The report of the diversion for Water Years 1981 and 1982 is dated September 1983, some 16 months after completion of the First Technical Committee's work. The Corps certification of the 1981-82 diversion accounting is without qualifications. The First Technical Committee questioned the

unqualified certification of the diversion record by USACE in light of deficiencies in the Lockport measurement system.

2.1.2 1985 Annual Report

The 1985 Annual Report contains IDOT's computation of diversion flow for Water Year 1983, as well as a summary of significant hydrologic and related events through Water Year 1985. The diversion flow of Water Year 1983 is based on a new accounting procedure. The procedure, proposed by IDOT, utilizes a NIPC-developed version of the Hydrocomp computer simulation model. The technical merits of the new process are examined in detail in Section 3.0.

The IDOT report of the 1983 Water Year diversion is appended to the Corps' 1985 Annual Report. Several problems encountered in the first year's use of the new accounting process were discussed by IDOT. The USACE review of the 1983 Water Year diversion flows addresses the accounting process in some detail but does certify the diversion record without qualification. The process used to certify the 1983 Water Year diversion is somewhat paradoxical because the process, a comparison of similar trends for AVM vs. MSD Lockport and MSD Lockport simulated vs. recorded flows, clearly suggests errors in the MSD Lockport recorded flows, both high and low.

2.1.3 1986 Annual Report

Section 1142 of the Water Resources Development Act of 1986, Public Law 99-662, provides that beginning on October 1, 1987, the Secretary of the Army in

cooperation with the State of Illinois shall carry out measurements and make necessary computations required by the Supreme Court decree.

2.1.4 Summary of First and Second Technical Committees' Findings

The review of the USACE Annual Report is in essence a review of the USACE diversion related activities. The annual reports should reflect the direction, activity, and evaluation of the diversion accounting process. The presentation should be objective, substantive, and informative enough for the interested parties and the Court to fully understand and appreciate the technical considerations of the accounting process. The evaluation of the annual diversion record should include narrative statements which provide the reader with an understanding of the process. The narrative should provide information describing the quality of the basic records and the analytical techniques in sufficient detail to support whatever diversion record certification is made. For example: the certification of the diversion records for 1981-82 Water Years should have been, at least, certified with the condition that the 1981 Technical Committee found cause to suspect inaccuracies, perhaps even significant errors, in the Lockport flow record for prior years. Similarly, the 1983 Water Year record certification should have been conditional because of the questionable ratings for the Lockport turbines. Certainly, the certification statement for all three years should have provided for a subsequent review and revision of the diversion records if found to be necessary. At some appropriate time, probably no earlier than after completion of the 1987 Water Year, the diversion records for the period

after 1980 should be reviewed, and if appropriate, revised to account for the apparent errors in the Lockport ratings used during the 1981-84 Water Years.

2.2 Third Technical Committee Review, Findings and Recommendations - Accounting Period 1986-1989

As part of the U.S. Corps of Engineers responsibilities as stated in the modified U.S. Supreme Court Decree (1980) for Lake Michigan diversion at Chicago, Illinois, adopted by the Court on December 1, 1980, is the issuing of annual reports on the measurements and computations of Lake Michigan diversion at Chicago, Illinois. Summarized in Table 2.1a and 2.1b is a Chronological history of major technical events regarding the Lake Michigan diversion program with respect to the Technical Committees.

TABLE 2.1a

CHRONOLOGICAL SUMMARY OF LAKE MICHIGAN DIVERSION EVENTS		
First Technical Committee Convened June 1981, Final Report - October 1981 Reviewed Status of Diversion Computation as Stipulated by the 1980, Modified Decree of the U.S. Supreme Court		
Second Technical Committee		
Annual Report	**Water Year Diversion Results**	**Remarks**
1981, 1982 Annual Report 11/83 - Released	1981/82	First Technical Committee convenes June 1981 Lockport Measurement Site - First Committee Report (October 1981 Harza report proposed new diversion accounting system WY 81-82 Diversion certified despite Technical Committee (1981) concerns regarding Lockport rating
1983, 1984, 1985 Annual Report 2/86 - Released	1983	New Accounting System (NIPC) Use hydrologic computer models. WES Report (1985) Powerhouse and Controlling Works sluice gate - new rating resulted in a reduced diversion (180 cfs) for 1988 WY AVM installation (March 18-23, 1984), AVM data suggest Lockport Turbine/low flows consistently low. 1983 diversion certified despite concerns on Lockport rating (Technical Committee, 1981) findings
1986 Annual Report 3/87 - Released	X	TARP - Began new accounting system, development of a computerized water budget, HEC analysis of Hydrologic Simulation procedures Second Committee convenes
Legend: WY = Diversion Calculation X = No diversion results		

CHRONOLOGICAL SUMMARY OF LAKE MICHIGAN DIVERSION EVENTS

Third Technical Committee

Annual Report	Water Year Diversion Results	Remarks
1987 Annual Report 9/88 - Released	X	The Water Resource Development Act of 1986 gave USCE responsibility for the computation of diversion flow. (effective October 1987) Second Technical Committee draft report (in review)
1988 Annual Report 3/89 - Released	X	Continuing problems with AVM - new system; new AVM system to be installed Diversion Accounting certification suspended in FY 88 pending revision of modeling parameters Second Technical Committee Final Report (November 1987) Released
1989 Annual Report 11/93 - Released	1984-85	November 1988 - ORE AVM installed First Annual Report that USCE assumes responsibility for the compilation of diversion Diversion Account Report developed by NIPC, reviewed and updated by USCE USCE updated model parameters and revised 1984-85 flows based on AVM records
1990-92 Annual Report 1/94 - Released	1986-87 1988-89	New Regression Equations (USGS)(WY 86, 87, 88, 89) — 1986 problem with AVM Modeling update - TARP — 1987 AVM - little missing record Lakefront measurements — 1988-89 Solar Radiation Correction New rain gage network - installed (October 1990) Grand Calumet River West Branch gage established Diversion results indicated State of Illinois exceeded allowable diversion - 1988
1993 - Annual Report+ 8/94 - Draft	1990	Incorporate the 25-gage precipitation network into the WY 1990 diversion accounting procedures and computations. In addition, updated combined sewer special contributing areas and land-use/land-cover to reflect the revised precipitation network and land cover assignments.
Future		Des Plaines Pumping Station Investigation Lakefront measurements/analysis Additional ADCP measurements at Romeoville Grand Calumet River Gage first used WY 1991
Legend: WY = Diversion Calculation X = No diversion results		

+Committee received the Draft Report in late July 1994, during final preparation of the Committee's report; therefore, time did not permit a detailed review of this report.

2.2.1 1987 Annual Report (no diversion accounting report)

The 1987 Annual Report was published in July of 1988 and did not include any diversion accounting for Water Year 1987. The report is a summary of events occurring during the 1987 Water Year. During this reporting period, the 1984 Water Year accounting report, which had been submitted by the State of Illinois, was under review. Major events during this period included the performance and evaluation of the Sarasota acoustical velocity meter to measure flow at Romeoville and the refinement of the new diversion accounting system and progress made by the Second Technical Committee. Based on the Water Resources Development Act of 1986, the Corps assumed, effective October 1, 1987, total responsibility for the measurements and computations necessary to account for the amount of water diverted from Lake Michigan at Chicago. Based in part on a recommendation by the First Technical Committee, a master plan for Lake Michigan diversion monitoring activities was developed and presented in the 1986 Annual Report. The purpose of this master plan was to define the responsibilities of the Corps of Engineers with regards to Lake Michigan diversion, establish routine annual goals and objectives, establish a generic annual schedule of activities and provide specific short- and long-range objectives for this program. An important part of this master plan was a series of standard operating procedures. A major element of the Corps' activity during 1987 was assisting the Second Technical Committee in prosecution of its work. The Second Technical Committee was convened in July 1986. The first two workshops were convened

during the 1986 Water Year. The work of the Committee continued throughout the course of the 1987 accounting year with two additional workshops. The third workshop was held in Chicago, October 21 through 24, 1986. The fourth workshop was also held in Chicago from January 27 through 30, 1987. The initial draft of the Committee's report was provided to the Corps on June 9, 1987, and a second draft on August 3, 1987. The final Committee report was submitted to the Corps in November 1987.

2.2.2 1988 Annual Report Review (no diversion accounting report)

The 1988 accounting year represents a transition to the USACE of primary responsibility for diversion computations. The 1988 Annual Report discussed modifications, based on the Second Committee's findings, to the hydrologic modeling parameters used in the diversion accounting. The Chicago District, in cooperation with the U.S. Geological Survey, contracted with ORE, Inc. for installation of a new AVM measurement flow at Romeoville because of the continuing problems with the Sarasota AVM. The Chicago District released a draft report "Acoustic Velocity Meter Regression Analysis" (August 1988) for an AVM backup system, as recommended by the Second Technical Committee. Mainstream Tunnel and Reservoir Plan (TARP) was brought on line at the beginning of FY 1986. The TARP impact and diversity will be a simulated and incorporated into the accounting diversion procedure prior to the FY 1986 accounting year.

In summary, the Lake Michigan diversion accounting program experienced significant modifications of the technical methodology and procedures used in computing the total flow at Lockport and in the computations of diversion deductions. The 1988 Annual Report was released March 1989.

2.2.3 1989 Annual Report Review (Water Year 1984 and Water Year 1985 accounting reports)

The 1989 accounting year was marked the first year in which the Chicago District Corps of Engineers initiated compiling the diversion accounting data and calculating Lake Michigan diversion flows. The Second Technical Committee completed its report November 1987. Major activities described in this Annual Report included revision of the diversion accounting modeling parameters development of a backup system to the AVM system, and installation of precipitation gage network. The 1989 Annual Report was released in November 1993. This report presented the accounting computation for Lake Michigan diversion for Water Year 1984-85, the first published since the First Technical Committee Report. The 1984-85 Water Year accounting flows developed by NIPC were recalculated based on revised modeling parameters and based on AVM flows rather than Lockport flows for water year 1984. The Corps updated the modeling parameters and calculated AVM flows for Water Year 1984 using Lockport flows and the regression equation AVM backup system. The Corps recalculated the diversions for both Water Year 1984 and Water Year 1985 using the updated modeling parameters and Romeoville AVM flows. Water Year 1984

was the first year in which the AVM (installed March 1984 and began reporting June 12, 1984) was used to measure the flow in the canal system. In Water Year 1984, the AVM measured flow was 314 cfs greater than the Lockport flow, and in Water Year 1985, the AVM flow was 229 cfs greater than the Lockport flow.

2.2.4 The 1990 - 1992 Annual Report Review (diversion accounting years 1986, 1987, 1988, and 1989)

The 1990-1992 Annual Report was released in draft form in November 1993, and the final in January 1994. The 1986 Water Year accounting was a joint effort with NIPC and the Corps of Engineers. Water Years 1987, 1988, and 1989 were calculated by the Corps of Engineers. This represents approximately a seven-year delay in releasing the diversion computations for Water Year 1986. Major accomplishments during this reporting period include the following:

1. Hydrologic simulation models were revised and updated to reflect the addition and impact of the TARP system.
2. The Illinois State Water Survey initiated and collected data from a precipitation network which included 25 gages.
3. New regression equations were developed for the Romeoville AVM gage for calculating flows when the AVM is out of operation. The USGS revised the regression equations, and the updated flows have been included in the accounting report for Water Year 1986 through 1989. The 1990-1992 Annual Report reflects a comprehensive presentation of the U.S. Corps of Engineers diversion related activities. The report reflects a detailed

discussion of the various activities, computational and evaluation processes of the diversion accounting process. This Annual Report provides the basis for interested parties and the Court to evaluate the technical aspects of the accounting process.

2.2.5 Summary of Lake Michigan Diversion Accounting

Lake Michigan diversion flow data are summarized in accounting reports prepared on an annual basis as flows are certified. Table 2.2 presents certified accounting flows for Water Years 1981-1989, the running average flows, and the cumulative deviation from the allowable diversion of 3,200 cfs.

Table 2.2

Status of the State of Illinois' Diversion from Lake Michigan Under the 1980 Modified U.S. Supreme Court Decree

Accounting Year	Certified Flow (cfs)	Running Average (cfs)	Cumulative Deviation (cfs)
1981	3,106	3,106	94
1982	3,087	3,097	207
1983	3,613	3,269	-206
1984	3,432	3,310	-438
1985	3,472	3,342	-710
1986	3,751	3,410	-1,261
1987	3,774	3,462	-1,835
1988	3,376	3,451	-2,011
1989	3,378	3,443	-2,189

The running average diversion for the period Water Year 1981 through Water Year 1989 is 3,443 cfs, 236 cfs greater than the 3,200 cfs, 40-year average diversion specified by the modified decree. The annual flow has twice exceeded 3,680, the maximum number of occurrences allowed under the decree, and the annual flows have not exceeded the two-year upper limit of 3,840 cfs. The cumulative deviation, the sum of the annual average flows over the allowed 3,200 cfs, is -2,189 cfs-years. The negative cumulative deviation indicates a flow debt against the maximum debt allowed of 2,000 cfs-years over the first 39 years of the 40-year averaging period.

2.2.6 Technical Committee's Findings and Recommendations

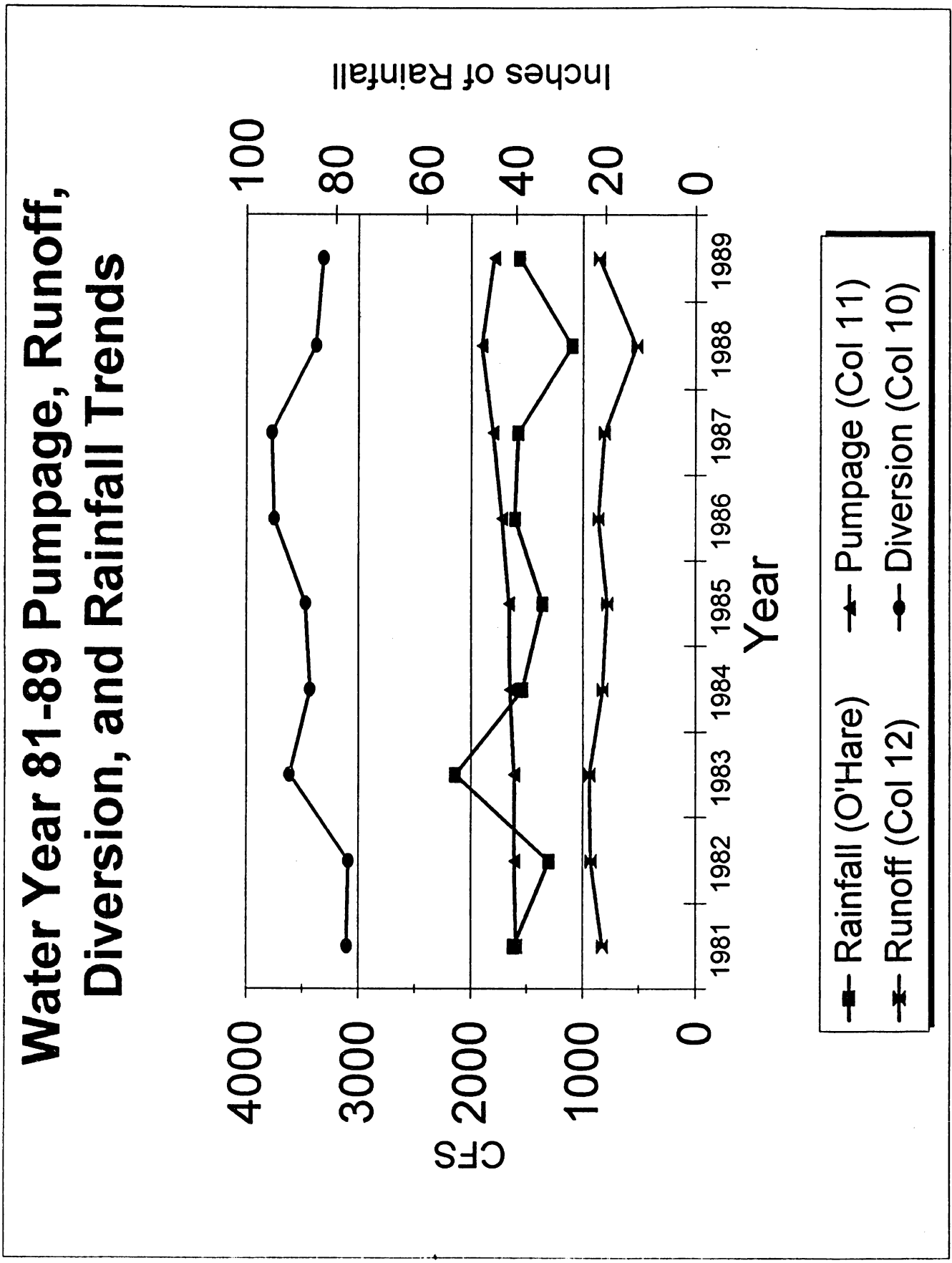
The modified U.S. Supreme Court Decree (1980) for Lake Michigan diversion at Chicago, Illinois required that an annual report on the measurement and computations of Lake Michigan diversion be issued by the Corps of Engineers. This annual report was the instrument to release information and the results of the Lake Michigan diversion computations. These annual reports, therefore, provide the basis for interested parties and Court to understand and evaluate the progress and status of the Lake Michigan diversion. The Second Technical Committee recommended these annual reports include narrative statements which would aid the reviewer in understanding the Lake Michigan diversion process. These narratives should provide information describing the quality of the basic record and the analytical techniques in sufficient detail to support whatever diversion record certification is made by the Corps of Engineers. The Second Committee took exception to the Corps' certification of the diversion record for 1981 and 1982 Water

Years in light of the findings of the 1981 Technical Committee with regards to apparent errors in the Lockport flow. Similarly, the Second Technical Committee further questioned the certification of the 1983 Water Year with regards to questionable rating for the Lockport turbines. The 1990-92 annual report is a significant improvement toward providing interested parties with a comprehensive, orderly presentation of the components of diversion with regards to methodology, technical approach, and the various issues with regards to the determination of the amount of Lake Michigan diversion. The 1990-92 Annual Report represents the first combination of multiple year (three) annual reports finally released in January 1994. This combined annual report contains diversion results for 1986 through 1989 Water Years. The release of this 1990-92 Annual Report reflects considerable delay in the release of the annual diversion computations. The reasons for delay are combinations of various technical issues not being timely resolved combined with the lack of resources and appropriate priorities. Because of the obvious major issue concerning the State of Illinois exceeding the Court's cumulative diversion of 2,000 cfs, a more timely release of this information could have allowed a more timely response by the various parties and possible action by the State of Illinois with regards to management alternatives.

Analysis of historical trends in the Lake Michigan data (Figure 2.1) suggest that basically pumpage and runoff have not increased significantly while Lake Michigan diversion displayed a marked increase in 1983. Water Year 1983 was a "wet" year and also marked the first year the AVM was used to compute Lake Michigan diversion. The Committee has concluded that the primary reason for the Lake Michigan diversion

exceeding the flow limits of the Court is the improved accuracy of the accounting procedures. The major part of this improved accuracy can be attributed to the AVM system.

FIGURE 2.1


Water Year 81-89 Pumpage, Runoff, Diversion, and Rainfall Trends
Inches of Rainfall
100
80
60
40
20
0
CFS
4000
3000
2000
1000
0
1981
1982
1983
1984
1985
1986
1987
1988
1989
Year
Rainfall (O'Hare)
Pumpage (Col 11)
Runoff (Col 12)
Diversion (Col 10)

3.0 REVIEW OF CURRENT ACCOUNTING SYSTEM

There are three components to the Lake Michigan diversion: direct diversion through three lakefront structures; pumpage from the lake for domestic water supply which is not returned to the lake; and stormwater runoff from the Lake Michigan watershed that is diverted from the lake. Direct diversion occurs at the Chicago River Controlling Works, the O'Brien Lock and Dam, and the Wilmette Controlling Works and consists of four components: lockage, leakage, discretionary flow, and navigation makeup flow. Discretionary flow is that which the MWRDGC (note changed from MSD) releases to dilute the effluent from sewage discharges into the canal. Navigation makeup flow is used by the MWRDGC to raise the level of the canal after it has been drawn down in anticipation of a rain event. Domestic pumpage is used to provide Chicago and suburbs with its water supply. The effluent resulting from that water supply is discharged into Water Reclamation's plants and eventually discharged to either the canal system or the Des Plaines River and its tributaries. Stormwater runoff that previously drained into Lake Michigan now drains into the Chicago Sanitary and Ship Canal or the Calumet Sag Channel (SCCS).

The amount of diversion is calculated using an acoustic velocity meter (AVM) which has been installed at Romeoville, just upstream from Lockport. The AVM measures the total flow of water in the CSSC. Most of the Lake Michigan diversion flows, and some non-Lake Michigan flows, flow by the AVM site. The diversion accounting system takes the total flow measured at Romeoville and

subtracts from it, flows not attributable to the diversion. Diversion flows which by pass Romeoville are added to the figure derived from the AVM, yielding the net computed diversion of water from Lake Michigan.

Deductions from Romeoville include groundwater supply effluent discharged into the canal, Lake Michigan pumpage by Federal facilities that discharge to the CSSC; runoff from the Des Plaines River watershed discharged into the canal, domestic pumpage from all sources by Indiana and Wisconsin the effluent of which reaches the CSSC, and any water diverted by Illinois into the Lake from outside the Lake Michigan watershed. Any amount of domestic pumpage by Illinois, the effluent of which bypasses the canal and instead reaches the Illinois waterway is then added to the amount at Romeoville.

3.1 Accounting Report

The format of the diversion accounting tables have been revised for Water Year 1986 due to the streamlining of the computational process and to make the results easier to interpret. The diversion accounting results are presented as a series of columns that are listed in Table 3.1 and Table 3.2 (Example Water Year 1989). Appendix 1 describes the various components of Columns 4-13. Column 1 through 3 compute the total flow in the Sanitary and Ship Canal. Column 4 through Column 7 presents the deductions from the Canal system flows with the total deduction in Column 8. Column 9 presents the additions to the Canal system record. Column 10 is the computed Lake Michigan diversion accountable to Illinois and is equal to the canal system flow minus the deductions plus the

additions. Columns 11 through 13 are independent flow estimates for the three sources of diversion: water supply pumpage from Lake Michigan, runoff from the diverted Lake Michigan Watershed, and direct diversion through the lakefront structures. Column 11 through Column 13 are not used in the diversion calculation but are included as another estimate of the diversion for comparison of the diversion flows in Column 10. The sum of Column 11 through Column 13 should theoretically equal the flow in Column 10.

TABLE 3.1

Description of Diversion Accounting Columns

COLUMN	DESCRIPTION
1	Chicago Sanitary and Ship Canal (CSSC) at Romeoville, USGS AVM Gage Record
2	Diversions from the CSSC above the Gage
3	Total Flow Through the CSSC
4	Groundwater Pumpage Discharge into the CSSC and Adjoining Channels
5	Water Supply Pumpage from Indiana Reaching the CSSC
6	Runoff from the Des Plaines River Watershed Reaching the CSSC
7	Lake Michigan Pumpage by Federal Facilities Which Discharge to the CSSC
8	Total Deduction from the CSSC Romeoville Gage Record
9	Lake Michigan Pumpage not Discharged into the CSSC
10	Total Diversion Accountable to the State of Illinois
11	Pumpage from Lake Michigan Accountable to the State of Illinois
12	Runoff from the Diverted Lake Michigan Watershed
13	Direct Diversion Through Lake Front Control Structures Accountable to the State of Illinois

Lake Michigan Diversion Accounting - WY 1989

Summary of Diversion Flows (cfs)

LAKE MICHIGAN DIVERSION ACCOUNTING WY 1989	ROMEOVILLE GAGE RECORD	DIVERSIONS ABOVE THE GAGE	TOTAL FLOW THROUGH THE CANAL	GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL	WATER SUPPLY PUMPAGE FROM INDIANA REACHING THE CANAL	RUNOFF FROM THE DES PLAINES RIVER WATERSHED REACHING THE CANAL	LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGED TO THE CANAL	TOTAL DEDUCTION FROM THE ROMEOVILLE GAGE RECORD	LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO THE CANAL	TOTAL DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS	PUMPAGE FROM LAKE MICHIGAN ACCOUNTABLE TO THE STATE OF ILLINOIS	RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED	DIRECT DIVERSION ACCOUNTABLE TO THE STATE OF ILLINOIS
MONTH	1	2	3	4	5	6	7	8	9	10	11	12	13
OCT 88	3235.5	1.2	3236.7	79.5	27.8	68.2	2.2	177.7	103.6	3162.6	1745.9	478.0	552.4
NOV 88	3828.4	0.6	3829.0	110.5	27.3	262.8	2.6	402.3	99.5	3626.2	1672.1	1366.6	180.9
DEC 88	2782.2	1.1	2783.3	80.9	27.2	154.6	2.6	265.3	103.0	2619.9	1663.7	567.9	144.6
JAN 89	2743.2	2.1	2745.3	77.1	27.3	151.3	1.8	257.5	86.8	2584.7	1674.0	694.4	76.3
FEB 89	2346.8	1.6	2348.4	83.8	27.2	102.7	2.2	196.0	83.8	2246.2	1664.0	376.8	69.9
MAR 89	3099.1	1.0	3100.1	94.5	27.4	185.0	2.7	309.6	82.7	2883.2	1652.5	805.8	66.8
APR 89	2617.5	1.6	2619.1	81.3	27.5	104.7	2.1	195.7	95.8	2519.4	1674.1	432.0	100.8
MAY 89	2813.8	0.6	2814.4	67.3	26.0	56.1	1.5	152.8	112.0	2773.6	1796.4	378.4	182.8
JUN 89	4483.4	1.2	4484.5	80.6	28.4	152.5	1.5	262.9	137.2	4356.9	1989.0	1186.1	402.2
JUL 89	4317.1	0.9	4318.0	86.9	29.6	93.8	1.5	211.7	139.5	4245.8	2215.4	636.9	777.5
AUG 89	4736.7	3.4	4742.1	97.2	28.3	111.2	2.0	238.7	118.5	4619.9	1947.5	715.5	1166.8
SEP 89	5027.4	0.6	5028.0	83.2	27.9	178.5	1.8	291.3	107.2	4843.9	1794.7	866.3	1422.7
AVERAGES	3515.2	1.3	3516.5	82.0	27.8	134.9	2.0	246.8	108.2	3377.9	1791.9	708.8	430.7

COMPUTATIONS:

1. COLUMN 3 EQUALS THE SUM OF COLUMN 1 AND COLUMN 2.
2. COLUMN 8 EQUALS THE SUM OF COLUMN 4 THROUGH COLUMN 7.
3. COLUMN 10 EQUALS COLUMN 3 MINUS COLUMN 8 PLUS COLUMN 9.

DEDUCTIONS FROM THE ROMEOVILLE GAGE RECORD

ADDITIONS TO THE ROMEOVILLE GAGE RECORD

NOTES:

1. ALL VALUES ARE ROUNDED TO THE NEAREST TENTH.
2. MATHEMATICAL COMPUTATIONS BETWEEN COLUMNS UTILIZE UNROUNDED VALUES.
3. AVERAGE VALUES FOR WY89 WERE COMPUTED USING DAILY VALUES.

Thirteen computational budgets are prepared as input to the diversion calculation and to estimate flows that cannot be measured. A summary of these budgets is presented in Appendix 2. Budgets 1 and 2 are summations of critical water supply pumpage data. Budget 3 through Budget 6 partition stream gage records into runoff and sanitary/industrial discharge components to estimate a portion of the runoff from the diverted watershed that is used as input to Column 13, (Runoff from the Diverted Lake Michigan Watershed.) Budget 7 through Budget 13 compare simulated to measured flows at MWRDGC facilities. These budgets are for verification of the diversion accounting procedures and give an indication of the accuracy of the diversion accounting. Budget 14 compares canal system inflows and outflows. The following is a brief narrative description of each column:

Column 1: Chicago Sanitary and Ship Canal (CSSC) at Romeoville (USGS - AVM Gage)

Column 1 represents the discharge at the Romeoville gage located on the CSSC approximately 5.2 miles upstream of the Lockport Powerhouse. Records are computed by the U.S. Geological Survey using the acoustical velocity meter gage at this station location. Initially, records were based on the Sarasota AVM from the 12th of June 1984 to the 3rd of November 1988. A new AVM manufactured by ORE became operational on the 17th of November 1988.

Column 2: Diversion From the CSSC Above the Gage

This column represents diversions from the CSSC flow, either municipal or industrial. Argonne National Laboratories and Uno-Ven Corporation presently are the only entities

diverting flow from the CSSC upstream of the Romeoville gage. The average withdrawal for Water Year 1989 was 1.3 cfs.

Column 3: Total Flow Through the CSSC

Column 3 represents the Column of 1 and 2 and represents the total flow entering the canal system.

Column 4: Groundwater Discharge to the CSSC and Adjoining Canals

Column 4 represents the effluent whose source is groundwater supply, pumpage by communities, industrial users, and other private users as reported by the Illinois State Water Survey (ISWS). Groundwater seepage into the TARP system that is discharged to the canal is included. Groundwater discharge is determined by summing all reported groundwater sources in the area tributary to the canal and the estimated groundwater seepage into the Mainstream and Calumet TARP systems. This total flow is then adjusted by subtracting the groundwater normally tributary to the canal that is contained in the combined sewer overflows which discharge to the Des Plaines River and other water courses not tributary to the CSSC. Groundwater seepage into the mainstream TARP system was determined through simulation and pumpage records. Groundwater constituent of combined sewer outflow is determined entirely through simulation. Groundwater pumpage from the Lake Michigan watershed whose effluent is discharged to the canal is a deduction.

Column 5: Water Supply Pumpage From Indiana Reaching and the Chicago Sanitary and Ship Canal

Column 5 is the water supply pumpage by the State of Indiana which reaches the canal in the form of effluent. This water is not charged to Illinois' allotment. It is a deduction

from the flow measurement at Lockport. This column is the same as Column 6 of the previous accounting format. Computation of the Indiana water supply reaching the canal through the Grand Calumet and Little Calumet River represents a complex drainage system with respect to Little Calumet River and Hart Ditch. Grand Calumet River has a summit. On one side of the summit, the flow is towards Lake Michigan. On the other side, the flow is towards the Calumet Sag Canal. However, this location of the summit is variable and highly influenced by Lake Michigan levels (USGS 1984). Thus the calculation of the Indiana deduction from the Romeoville record is influenced by Lake Michigan levels. Because of no stream gaging station on the Grand Calumet River to measure westward flow into Illinois, the flow is based on statistical relationships and computed from regression equations of which the principal independent variable is lake level. The flow in the Grand Calumet is estimated to be in excess of 90 percent sanitary effluent. It is therefore assumed the portion of this flow which is attributable to domestic water supply is equal to the sum of the daily water supply for East Chicago, Hammond, and Whiting, Indiana unless the sum is greater than the flow in the Grand Calumet River. If the combined water supply from these communities is in excess of the flow in the Grand Calumet, it is assumed that the flow consists entirely of effluent that originates from water supply. The USGS has established a gaging station on the Grand Calumet River to better define the variable flow conditions. The results of this gage will be used for the Water Year 1991 accounting.

Column 6: Runoff from the Des Plaines River Watershed (DPW) Reaching the Chicago Sanitary and Ship Canal

This column is made up of the following flow components:

1. Infiltration and inflow from the DPW discharged to the Water Reclamation's plants;
2. Infiltration and inflow from the DPW reaching the canal through the combined sewer; and overflows
3. Runoff from the Lower Des Plaines and Summit conduit areas.

This flow deduction is mainly determined by computer simulation. But, it is also influenced by the O'Hare basin flow transfer.

Column 7: Lake Michigan Pumpage by Federal Facilities Which Discharge to Chicago Sanitary and Ship Canal

Column 7 represents Lake Michigan diversion by federal facilities not chargeable to the State of Illinois allocation. Federal facilities represented by this column are as follows:

- Hines VA Hospital
- Fort Sheridan
- Glenview Naval Air Station
- U.S. Army Corps Emergency Navigation Makeup

Column 8: Total Deductions from Chicago Sanitary and Ship Canal at Romeoville Gage Records

Column 8 is the sum of columns 4, 5, 6, and 7 and represents the total deductions from Romeoville record.

Column 9, Lake Michigan Pumpage not discharged to the Chicago Sanitary and Ship Canal.

This column represents water supply pumpage from Lake Michigan that is not discharged to the canal. The water supply pumpage not discharged to the canal has two basic components: (1) water supply used by communities serviced by water reclamation facilities that do not discharge to the CSSC; and (2) the sanitary portion of combined sewer overflows that do not discharge to the CSSC which is attributable to Lake Michigan domestic water supply.

Column 10: Total Diversion

Column 10 is equal to Column 3 minus Column 8 and plus Column 9. Columns 11, 12, and 13 are not used in the computation of diversion but represent the flow estimate of the three basic components of Lake Michigan diversion.

Column 11:

Column 11 is the summarization of Lake Michigan pumpage for which Illinois is accountable. This is the same as Column 13 in the previous accounting report.

Column 12:

Column 12 represents the simulated runoff from the Lake Michigan watershed and includes infiltration and inflow entering the storm sewer system. This runoff is estimated using the computer simulation hydrologic models.

Column 13:

Column 13 represents the total direct diversion of Lake Michigan water into the diverted rivers systems through the controlling structure at Wilmette, the Chicago River Controlling Works, and the O'Brien Lock.

Because the diversion estimated from Columns 11 through 13 is based on computer simulation, questionable rating of the lakefront structures and simple flow separation techniques, the estimate is not expected to be as accurate as the AVM based calculations.

3.2 ACCOUNTING - HYDROLOGIC MODELING

The flow record at the AVM at Romeoville represents the majority of volume of water diverted from Lake Michigan. But in addition to the Chicago Sanitary and Ship Canal flow, the record also contains the following deductions:

- Runoff from the Des Plaines River watershed that was diverted into the canal system by the network of sewers.
- Groundwater water supply pumpage from outside the basin that was treated and discharged from sewage treatment plants into the canal system.
- Water supply pumpage from the State of Indiana, entering from the Calumet River.

- Water supply pumpage from Federal Facilities that is discharged into the canal is deducted from the Romeoville flow.

The hydrologic model computes the runoff from the entire watershed and routes the water through the network of sewers and the TARP system. The water is not routed through the canal system because of the short travel time. **The primary goal of the hydrologic runoff modeling and the hydraulic sewer routing models is to estimate the volume of water from the Des Plaines River watershed that enters the canal.** The contributing Des Plaines River watershed is about 217 square miles, about 35 percent of the 673 squares miles that the canal drains. The secondary goal for the hydrologic and hydraulic modeling is to compute the entire hydrologic budget for the Chicago Canal System. From these budgets, considerable information can be determined. The total diverted runoff from the Chicago River can be calculated. Additionally, the budgets are used as a verification of simulated flows and to indicate problems associated with the simulated or recorded flows used in computing the diversion.

3.2.1 Modeling Approach

The hydrology of the basin is simulated on a continuous basis. The Hydrocomp Simulation Program Fortran (HSPF) model is used to simulate the hydrology. The product of the HSPF model is runoff in inches. The runoff is applied to the SCALP model which converts the runoff into sewer discharge and routes the flow through the sewer networks either to sewage treatment plants or

to the drop shafts of the TARP system. The TARP model, which uses TNET, routes the flow through the TARP tunnels from which the wastewater is pumped into the Water Reclamation's Stickney and Calumet Treatment plants. The outfalls from the sewage treatment plants flow into the Chicago Sanitary and Ship Canal. There is no routing through the Sanitary and Ship Canal. Routing of flow through the Sanitary and Ship Canal is not simulated in the HSPF model applied in the diversion accounting.

3.2.1.1 Precipitation Network

Prior to Water Year 1990, a network of 13 precipitation gages was used to collect rainfall data over the watersheds. Figure 3.1 shows this network. This network had problems. The gages were maintained by MWRD, the City of Chicago, and the National Weather Service. Each of these agencies had different procedures for maintaining the gages and for collecting and reducing the data. The gages were of different types. Many of the gages had poor exposure being overshadowed by rooftops and trees. Moreover, the distribution of gages was uneven causing inconsistent sampling of precipitation. The problems resulted in unusual precipitation patterns as shown in Figure 3.2. The spacial variation in rainfall from 20 to 40 inches was unreasonable for a basin of this size. The Corps of Engineers retained the Illinois State Water Survey to adjust the data. The rain gage at O'Hare Field was determined to be accurate, and the rainfall was adjusted by a complex procedure which analyzed each individual storm. The adjustments to each rain gage for the Water Year 1987 is shown in Table 3.3. The maximum

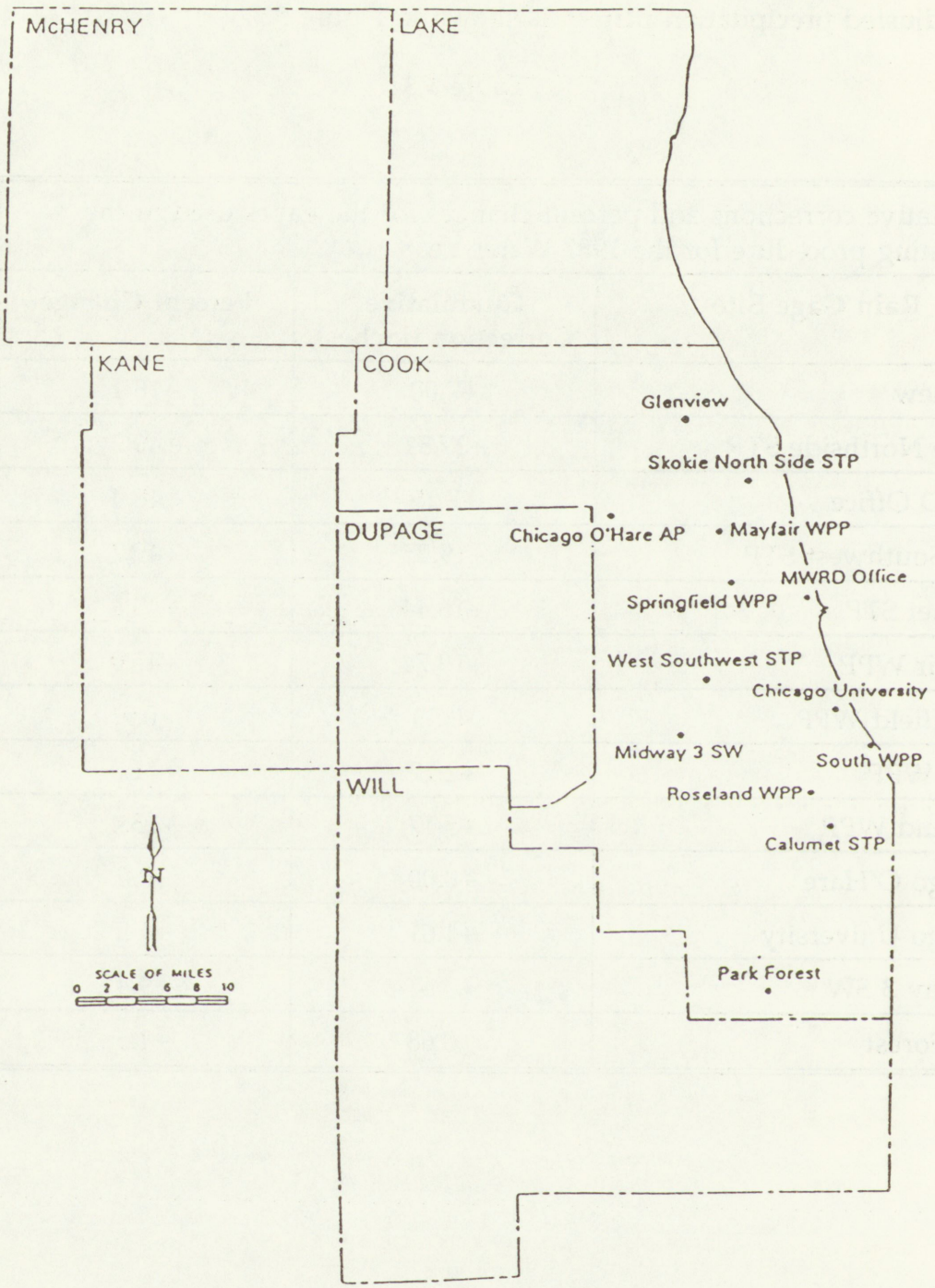
adjustment is almost 50% at the West Southwest Sewage Treatment Plant. The adjusted precipitation pattern is shown in Figure 3.3.

Table 3.3

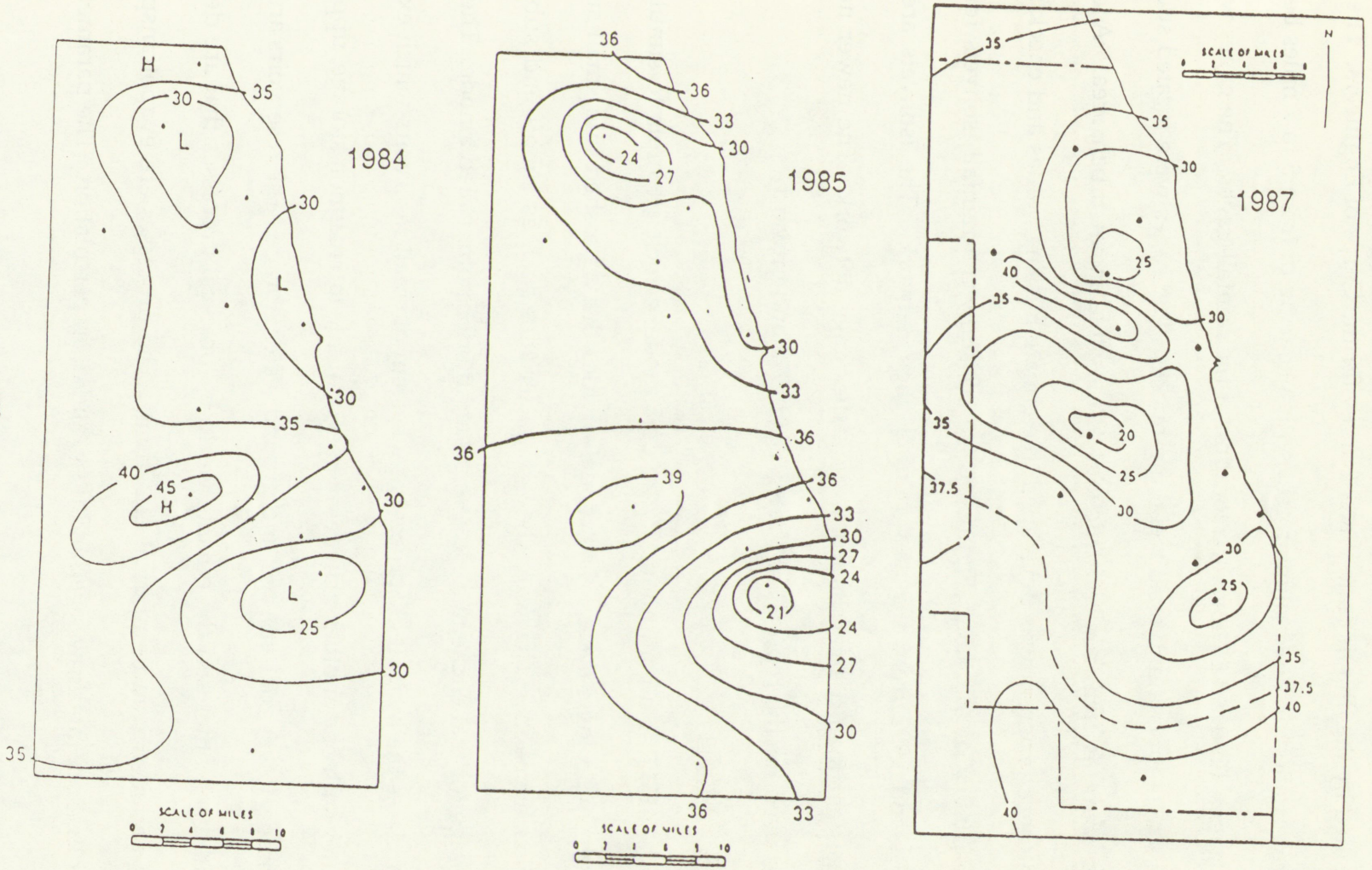
Cumulative corrections and percent changes for the gages used in the accounting procedure for the 1987 Water Year

Rain Gage Site	Cumulative Correction (inches)	Percent Change
Glenview	+7.00	+18.1
Skokie Northside STP	+17.51	+40.7
MWRD Office	+7.15	+18.4
West Southwest STP	+19.25	+49.7
Calumet STP	+16.44	+43.4
Mayfair WPP	+19.74	+45.0
Springfield WPP	-0.39	-0.9
South WPP	+4.17	+11.3
Roseland WPP	+5.17	+13.4
Chicago O'Hare	+0.00	+0.0
Chicago University	+1.61	+4.5
Midway 3 SW	+3.67	+9.4
Park Forest	+0.68	+1.5

FIGURE 3.1



NWS, MWRDGC, and City of Chicago Rain Gage Sites in Cook County



Precipitation patterns from original records (inches)

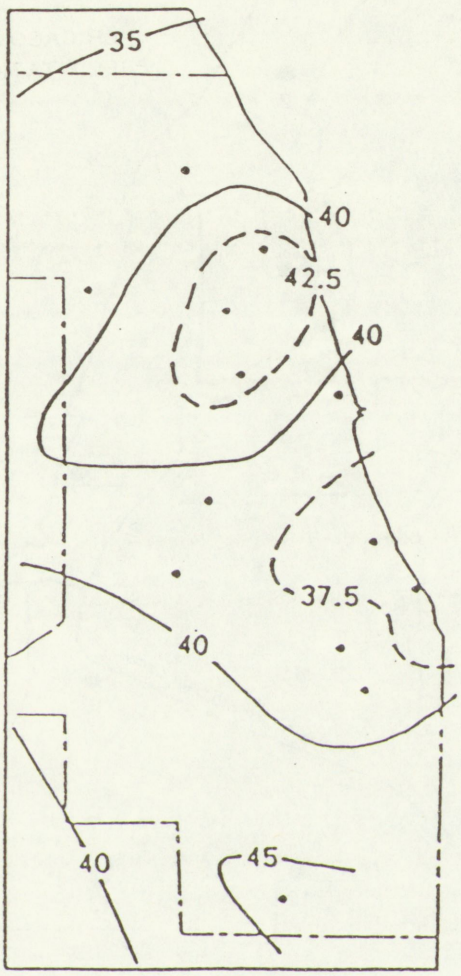
FIGURE 3.2

In the Summer of 1989, a new network of 25 precipitation gages was installed by the Illinois State Water Survey for the Corps of Engineers. The gages were installed on a rectangular grid with a spacing of from 5 to 7 miles between the gages. Figure 3.4 shows the location of the rainfall gages. The gages were all of a single type, a universal weighing bucket. The gages were located such that they were as free from obstructions as possible in an urban area. A quality assurance program was developed to estimate missing values and check for the consistency of the data. Figures 3.5 to 3.7 show the rainfall isohyets for 1990, 1991, and 1992 from the new rainfall gage network. The isohyets are more consistent than the results from the earlier gage network. The newer network was used for the reports for Water Year 1990 and onward.

3.2.1.2 HSPF Model

HSPF is a continuous hydrologic model which attempts to simulate the entire hydrologic cycle. The model divides the watershed into small subareas called elements. The watershed characteristics such as runoff and subsurface storage for that element are known as an interior point called a node. The model simulates the hydrologic process as a system of small reservoirs which exchange water with one another. The reservoirs simulate interception storage, upper zone storage, lower zone storage, and ground water storage. The reservoirs are linked with each other and the outside world by physical processes that are described by parametric and empirical equations. The processes are evapotranspiration, infiltration, interflow, overland flow, and deep percolation. The parameters for

FIGURE 3.3



Water Year 1987 Adjusted Precipitation Pattern

FIGURE 3.4

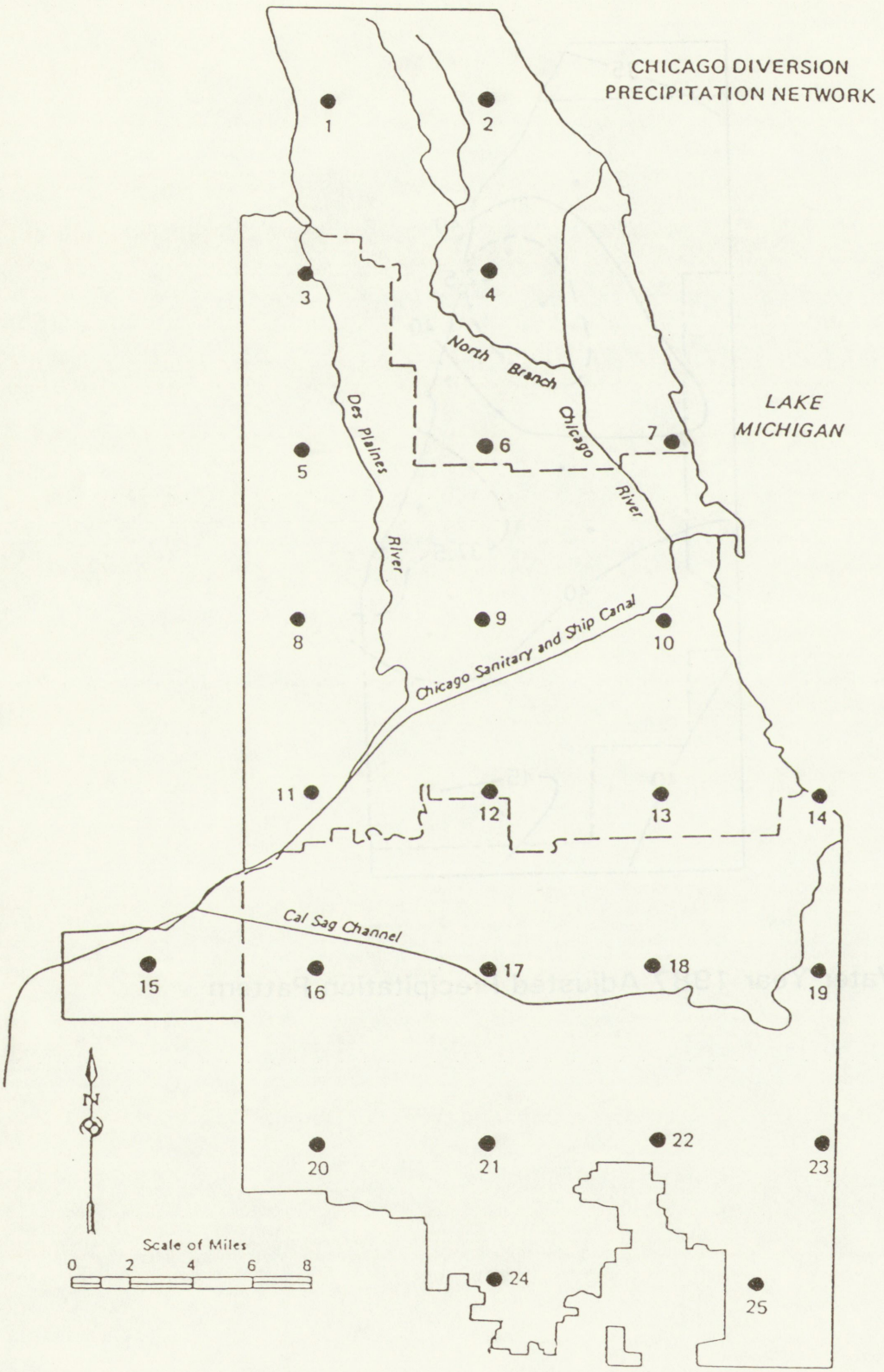
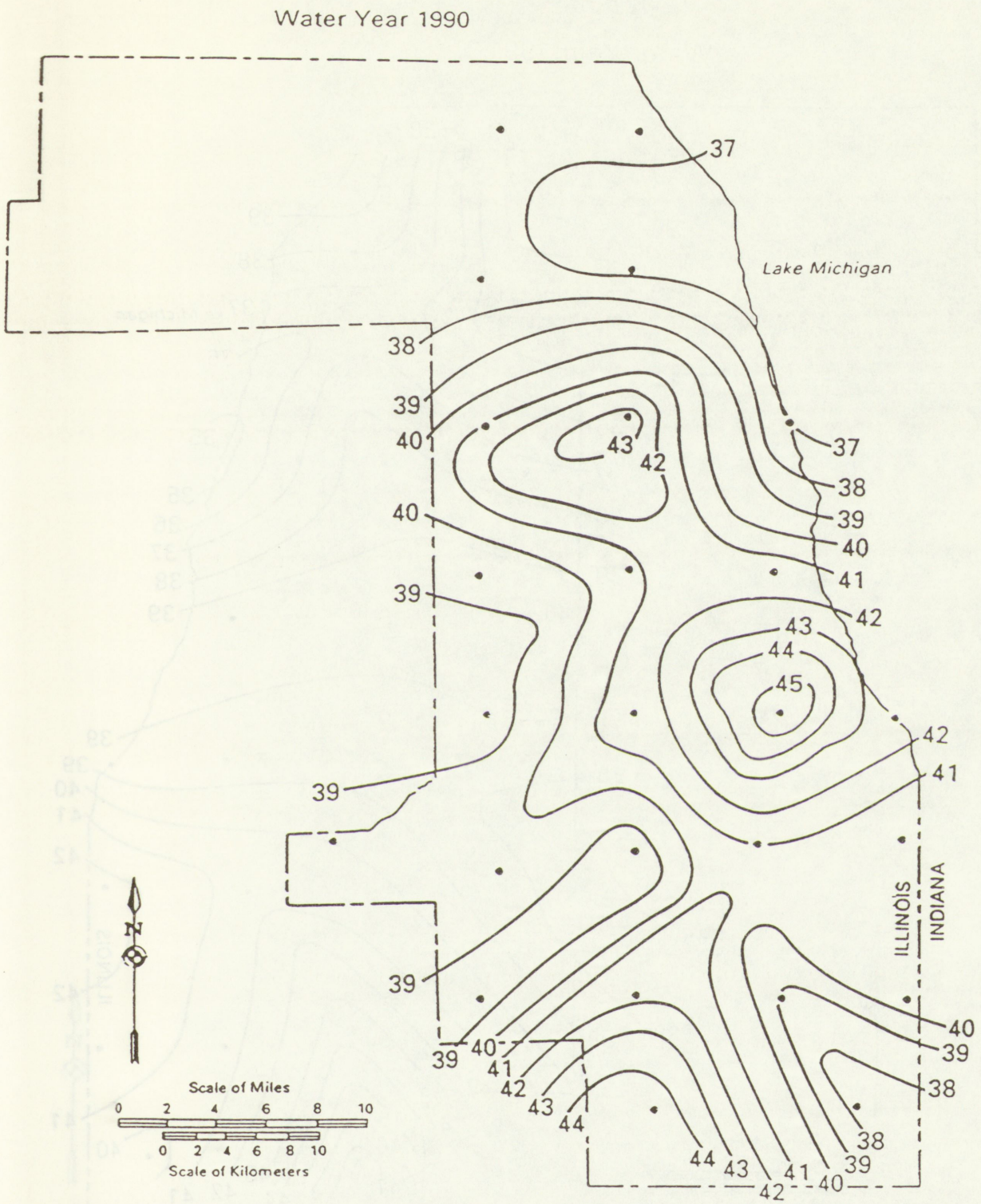

CHICAGO DIVERSION
PRECIPITATION NETWORK
LAKE
MICHIGAN
North Branch
Chicago
River
Des Plaines
River
Chicago Sanitary and Ship Canal
Cal Sag Channel
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
N
Scale of Miles
0 2 4 6 8

FIGURE 3.5



Precipitation pattern (inches) for Water Year 1990. Dots indicate network sites.

FIGURE 3.6

Water Year 1991

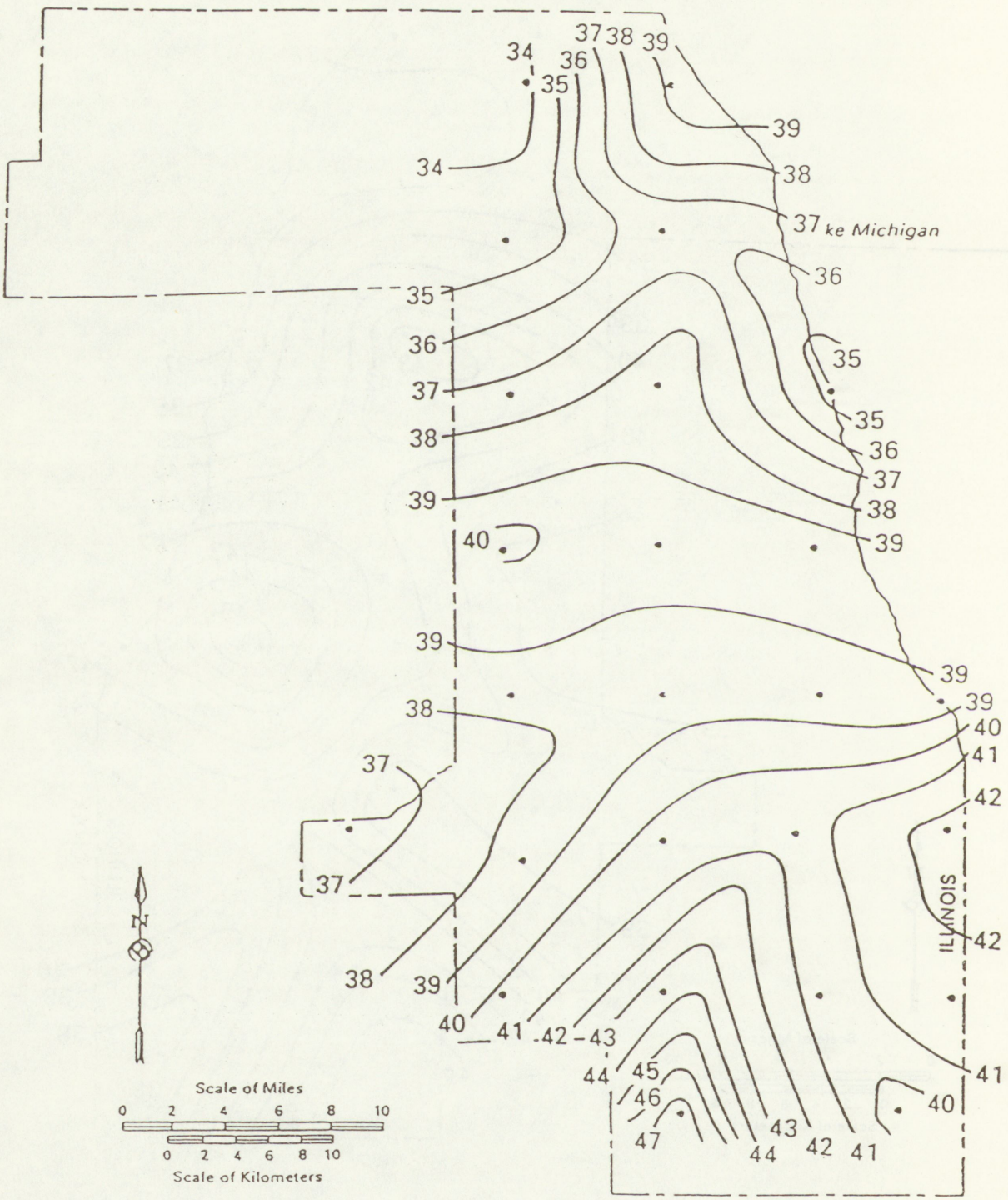
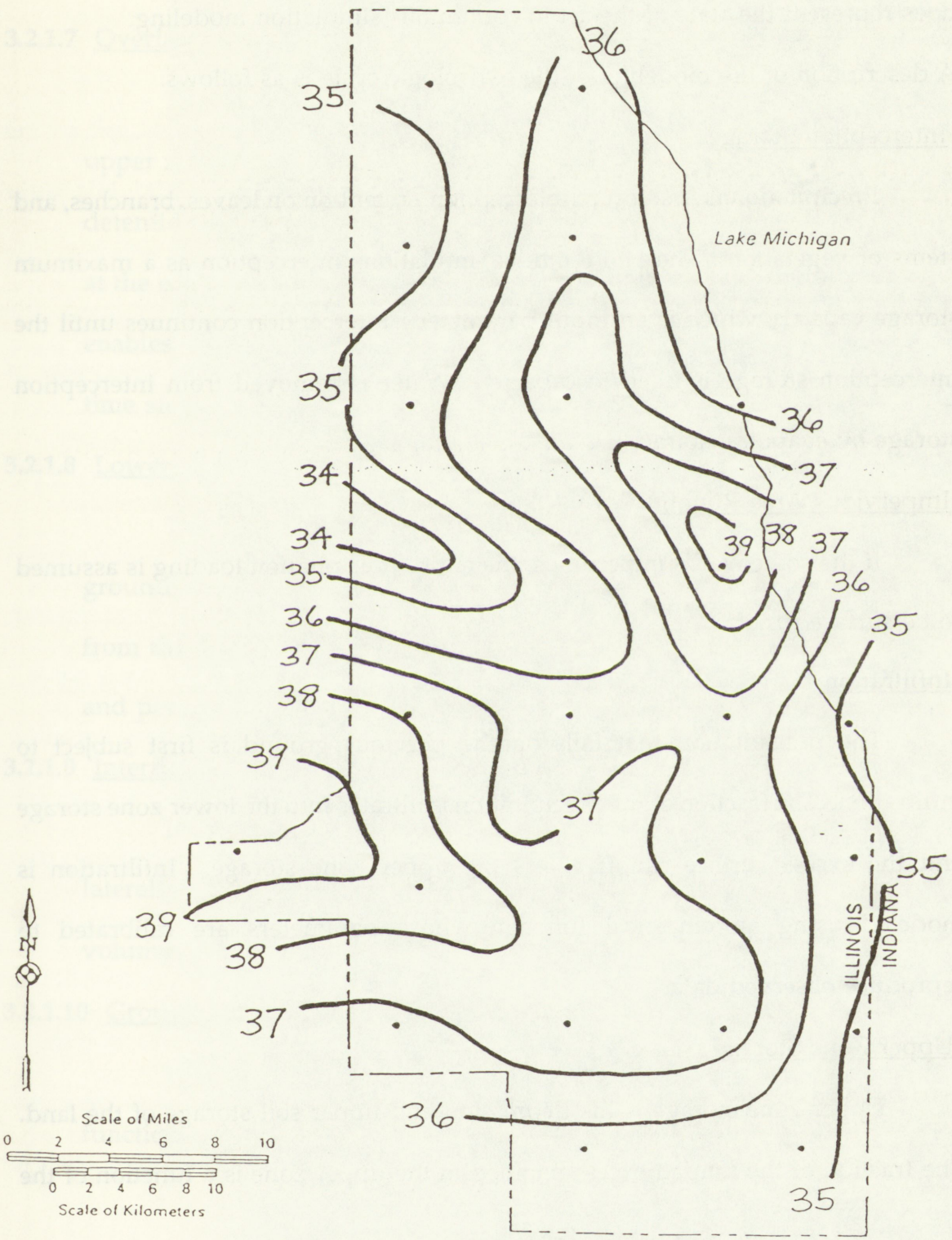


FIGURE 3.7

Water Year 1992



the equations are determined by calibration to observed data. The program, which has its roots in the Stanford Watershed Model, is an old concept, but it does represent the state-of-the-art in continuous simulation modeling.

A description of the modeling of the hydrologic cycle is as follows:

3.2.1.3 Interception Storage

Precipitation is first lost to interception - retention on leaves, branches, and stems of vegetation. The HSPF model simulations interception as a maximum storage capacity which is an input parameter. Interception continues until the interception storage is filled to capacity. Water is removed from interception storage by evapotranspiration.

3.2.1.4 Impervious Area Runoff

If the soil cover is impervious, the entire precipitation loading is assumed to be surface runoff.

3.2.1.5 Infiltration

The precipitation that falls on the pervious ground is first subject to infiltration. The fraction of the precipitation infiltrates into the lower zone storage and the excess surface runoff enters the upper zone storage. Infiltration is modeled using an empirical function whose parameters are calibrated to reproduce observed data.

3.2.1.6 Upper Zone Storage

Upper zone storage is the depression and upper soil storage of the land. The fraction of the rainfall excess retained in the upper zone is a function of the

upper soil moisture and the nominal storage capacity of the upper zone. Water is lost to the upper zone through evapotranspiration, interflow, and infiltration to the lower zone.

3.2.1.7 Overland Flow

The excess precipitation after losses to interception, infiltration, and the upper zone enters overland flow detention. Water is routed from overland flow detention by an empirical relation. Water remaining in overland flow detention at the end of a time step is added to the precipitation for the next time step. This enables the residual detention volume to contribute to infiltration for the later time steps.

3.2.1.8 Lower Zone

The lower zone extends from the upper zone down to the top of the groundwater table. Water enters the lower zone as infiltration from runoff or from the upper zone. Water leaves the lower zone through evapotransporation, and percolation to the deep groundwater storage.

3.2.1.9 Interflow

Interflow is the process where flow leaves the upper zone and flows laterally to the stream channel. Interflow is assumed to be a function of the volume of storage in the upper zone.

3.2.1.10 Groundwater and Deep Percolation

Water enters groundwater storage from the lower zone. The flow is a function of the moisture level and nominal storage capacity in the lower zone.

A portion of this inflow can be diverted into deep groundwater storage where the water is completely lost to the system. Water in groundwater storage returns to the river channel according to an exponential recession.

3.2.1.11 Evapotranspiration

Evapotranspiration is the loss of moisture from plants and soil to the atmosphere. HSPF models evapotranspiration as an assumed function. For water in depression storage, the evapotranspiration is the potential rate which is rate from class A pan evaporation records. For water in the lower zone, the evapotranspiration is a function of the water in storage and an assumed index of vegetation density.

3.2.2 SCALP

The SCALP program converts the unit runoff from the HSPF model computed for each segment into flow and routes the flow through lateral, submain, and main sewers. The Chicago sewers are divided into two types - separate and combined sewers. Separately sewered areas use a linear routing technique. In the Chicago model, the main sewers are called interceptor sewers. When the capacity of an any sewer is exceeded, that sewer either overflows into the Chicago Canal System or overflows into a drop shaft of the Chicago TARP system. The SCALP model was specifically designed to calculate these overflows.

The SCALP model reads the three HSPF unit runoff files: SUBRO - subsurface runoff; IMPRO - impervious runoff; and OLFRO - overland pervious surface runoff. For each subarea, the surface runoff for each time step is

computed by multiplying the pervious and impervious drainage area times the pervious and impervious unit runoff. Subsurface runoff is computed by multiplying the pervious drainage area times the unit subsurface runoff. Sanitary flow is determined by multiplying per capita loading by a population equivalent for each subarea. For both types of sewer systems, combined and separate, sanitary flow, surface runoff, and subsurface runoff is routed through the sewers.

Infiltration is the groundwater which seeps into the sewer system through joints and fissures. Inflows are unregulated connections to the sewer system. Two examples of inflows are the discharge from gutter downspouts and the discharge from basement sump pumps. For combined sewer areas, 100 percent of the infiltration (subsurface runoff) and inflow (surface runoff) is estimated as entering the sewers. For the separately sewered areas, Burke recommended (1990) that the sum of 100% of the subsurface runoff and 5% of impervious flow be assigned as infiltration and inflow.

The SCALP uses a simplified hydrologic routing technique to route flow through the sewers. Each sewer line is viewed as a small reservoir and a system of sewers is viewed as a series of cascading reservoirs. The outflow from the reservoir is a linear function of storage,

$$Q=\frac{1}{K}S \tag{3.1}$$

in which Q is the outflow; K is the linear routing factor and S is storage. This type of model does not simulate the hydraulics of the sewer lines, but, since, the goal is yearly runoff totals the model is adequate.

The output from SCALP is the routed outflow which is the inflow to the sewage treatment plant and the overflows from the sewers which are the inflow to the TNET model. The sewer flow and the overflows are both written to the Time Series Storage (TSS) database.

3.2.3 TNET

The Tunnel and Reservoir Plan consists of a dendritic network of deep tunnels underlying the City of Chicago. The tunnels collect sanitary and stormwater runoff that overflow from the interceptor sewers up to the storage capacity of the tunnels. The flow into the tunnel is controlled by gates on the drop shafts. After the gates on the drop shafts, wastewater overflows into the canal system. After the storm event, the storage of the tunnel is pumped to the Stickney Water Reclamation Plant and the Calumet Water Reclamation Plant and then discharged to the canal system. At present, the Mainstem and Calumet Tunnel Systems are in operation, and the Des Plaines tunnel system is under construction. A reservoir is being designed for the downstream end of the Mainstem Tunnel to increase the available storage.

Both tunnels are being simulated by the TNET model. The TNET program simulates both open channel and pressure flow using the open channel flow equations. The open channel flow equations are "tricked" to simulate pressure flow through the Preissmann slot (Cunge, 1980). The Preissmann slot is a slot of

very small width at the top of the tunnel. The width of the slot is set such that the celerity of the waves inside the tunnel are the same as the celerity of a pressure wave. The width of the slot is generally about .001' in width which produces a wave celerity of about 4,900 fps.

The overflows, which were computed by SCALP, are input to the drop shafts. The TNET program sets the drop shaft gates according to the volume of storage in the tunnel. When the gates are closed, the overflow, which is dumped to the canal, is written to DSS.

3.2.4 Comments

Insufficient Volume at Lower Des Plaines and Other Sites

Table 3.4 shows the simulated and recorded mean flow at the gaging stations within the basin. The mean flow is the average for the entire year; therefore, errors in the temporal distribution of rainfall and routing are washed out by the lumping of the flow data. The simulated volumes for the Northside WRP and the Des Plaines PS are consistently low. The volume deficiency is of particular concern for the Des Plaines PS which, in theory, acts as is a calibration point for the Des Plaines watershed. This flow deficiency requires resolution before this site can be used as a calibration point for the Des Plaines watershed models that impact the simulated Des Plaines watershed runoff deduction contained in Calumet of the accounting report.

The Calumet WRP also shows a similar deficiency, but in the final year, 1989, the deficiency disappears. The COE analyzed the Calumet basin and

TABLE 3.4

WY 1989 SUMMARY OF SIMULATION STATISTICS

Budget No.->	7	8	9	10	11	12	13	14
Description	Northside WRP (1)	Upper Des Plaines Pump Station (1),(3)	Mainstream TARP Pump Station (2)	Stickney WRP (1)	Calumet TARP Pump Station (2)	Calumet WRP (1)	Lemont WRP (1)	Chicago Canal System Balance (1)
Mean Recorded Flow, cfs	422.0	82.3	86.6	1152.3	31.9	421.3	2.3	3043.5
Max. Recorded Flow, cfs	682.8	179.7	359.9	2134.0	122.3	709.9	6.5	13410.0
Min. Recorded Flow, cfs	330.3	21.3	14.2	688.6	0.0	296.7	0.8	1671.7
Mean Simulated Flow, cfs	407.2	67.8	89.1	1187.7	27.5	419.0	1.8	3523.7
Max. Simulated Flow, cfs	680.8	186.0	186.8	2589.8	89.4	784.7	4.9	13754.0
Min. Simulated Flow, cfs	307.0	43.3	38.8	819.1	6.5	303.9	1.2	1877.5
Mean S/R	0.97	0.82	1.03	1.03	0.86	0.99	0.78	1.16
Max. S/R	1.42	3.79	7.30	1.66	6.14	1.53	3.67	1.56
Min. S/R	0.65	0.43	0.25	0.56	0.27	0.66	0.36	0.78
Correlation	0.80	0.47	0.55	0.74	0.68	0.72	0.72	0.95

(1) Based on daily values.
(2) Based on weekly values.
(3) Does not include days with missing records.

determined that the per capita sanitary loadings were too small. The loadings were revised and the S/R ratio was increased to its current value of 0.99.

The Second Committee also noted the deficit in flow in their report. Their concern was the infiltration and inflow was too low in the separately sewered areas. As a result, the COE retained Christopher B. Burke Engineering to review the parameters in SCALP which determine inflow and infiltration. Burke's Report (1990) was based primarily on the flow records at the Des Plaines Pumping Station. Special sewer studies at other locations were found to have significant errors, and the results of these sewer flow measurements were unusable.

Calibrating to the Des Plaines Pumping Station data, Burke suggested that the infiltration be increased to 100% of the SUBRO (subsurface unit runoff), and the inflow be increased to 5% of the IMPRO (impervious unit runoff). With the Burke parameter assignments, the simulated flow was only increased from 80% to 85% of the recorded flow for 1985, and Table 3.5 demonstrates similar results for succeeding years. These recommendations were implemented for the 1986, 1987, 1988, and 1989 diversion accounting reports for all separately sewered areas. Figures 3.8 to 3.11 compare simulated and recorded flow at the Des Plaines Pumping Station for Water Years 1986 to 1989. The hydrography show a consistent deficiency at low flow. The reproduction of high flow, while not perfect, is still acceptable. Clearly, the volume deficiency is from an inadequate reproduction during low flow periods. The COE has concluded that the deficiency in flow cannot be resolved until the measurement problems at the

Table 3.5

Upper Des Plaines Pumping Station (UDPPS)
Updated Model Simulated Flow (cfs)
(after Burke, 1990)

		WY 83	WY 84	WY 85
TOTAL	Rainfall (inches)[1]	53.5	38.6	43.1
	Recorded[2]	88.5	88.5	81.2
	Total	82.2	72.9	69.8
	Runoff	9.8	8.1	7.1
	Infiltration	28.4	20.0	17.9
	Sanitary	44.0	44.8	44.8
SEPARATE SEWER AREA	Total	33.7	28.9	27.2
	Runoff	1.2	1.0	.09
	Infiltration	18.1	13.0	11.4
	Sanitary	14.4	14.9	14.9
COMBINED SEWER AREA	Total	48.5	44.0	42.6
	Runoff	8.6	7.1	6.2
	Infiltration	10.3	7.0	6.5
	Sanitary	29.6	29.2	29.9

[1]Recorded Rainfall at O'Hare International Airport.
[2]Record flow has 176 days of missing average daily flow data.

pump station are resolved. During high flow on the Des Plaines River, the flow can be diverted around the pumping station into the Des Plaines River. The deficiency in flow may be caused by two factors: 1) sanitary inflow is underestimated, and 2) the infiltration and inflow is still underestimated. The sanitary inflow is input a per capita loading and a population equivalent into the SCALP model. Since this watershed was developed 40 to 50 years ago, the land use has changed little over the past 15 years and the population would be

relatively constant. Therefore, the primary focus would be the per capita loading. Table 3.6 compares the loading factors for the Des Plaines watershed and other basins in the system. The Committee cannot specify new loading factors; instead, the committee recommends that the basin be studied and that the loading factors be adjusted, if warranted.

The Burke infiltration and inflow study raises more questions than answers. The Burke recommendation of assigning 100% of the subsurface runoff to infiltration clearly violates physical reality. This leads to a dilemma: Either the subsurface flow from HSPF is too small or the conceptual model of sewer infiltration using HSPF and SCALP is wrong.

TABLE 3.6

Sanitary Flow Used in the SCALP Program for Selected Watersheds

Subarea	Type	Sanitary Flow in CFS x .0001 per Person
Upper Des Plaines	Combined	2.22
	Separate	2.22
Chicago contributing to Stickney STP	Combined	4.88
Des Plaines watershed inflowing into Tunnel 13a	Combined	1.92
	Separate	1.92
Inflow to future Des Plaines Tunnel	Combined	1.93
Lower Des Plaines	Combined	4.88
	Separate	2.08
Suburban Lake Michigan watershed contributing to Northside STP	Combined	2.55
Chicago watershed contributing to Northside STP	Combined	2.47

The subsurface component can be increased by either increasing the inflow through infiltration from the surface or by reducing the evapotranspiration and percolation to deep groundwater. It is unlikely that one can adjust these parameters sufficiently to compensate for the 15% shortfall. Is there any other source of inflow into the lower zone? Infiltration into the lower zone can only come from pervious areas. Can impervious areas be reassigned to the pervious category, thereby increasing the inflow into the subsurface areas? Are rooftops considered impervious area? Roofs drain through gutters onto yards which is a pervious area. Do impervious areas contribute to the lower zone through cracks in the pavement and through exfiltration from sewers under surcharge? The subsurface component of HSPF was designed to simulate the flow of water from soil storage into stream channels - thus creating base flow. This concept is similar but different from the infiltration into sewers. The infiltration into sewers is caused by a head of water forcing water through joints into the pipe. Is a different conceptual model which would simulate the pressure component needed?

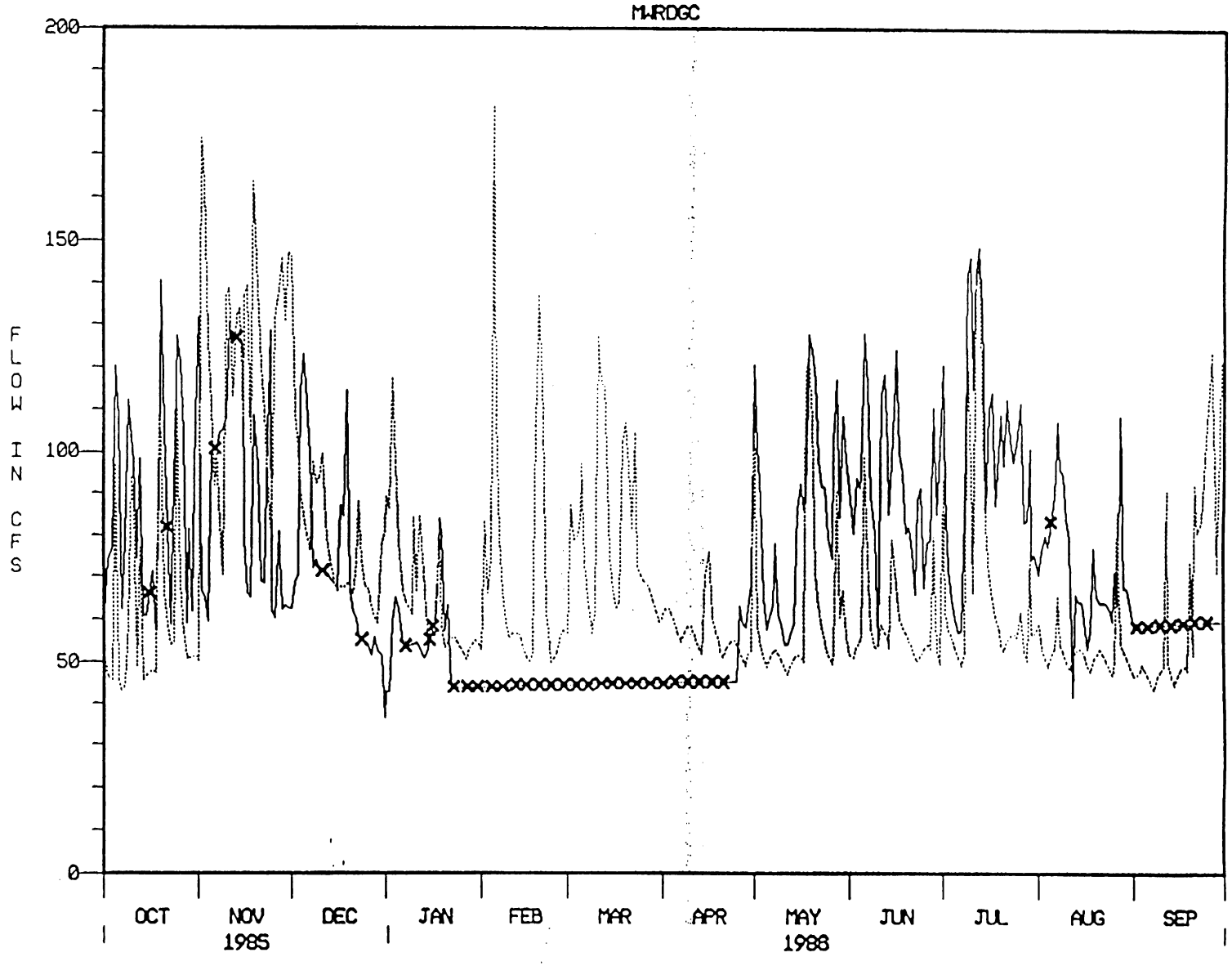


FIGURE 3.8

Budget 8 - Simulation of the MWRDGC Upper Des Plaines Pump Station

FIGURE 3.9

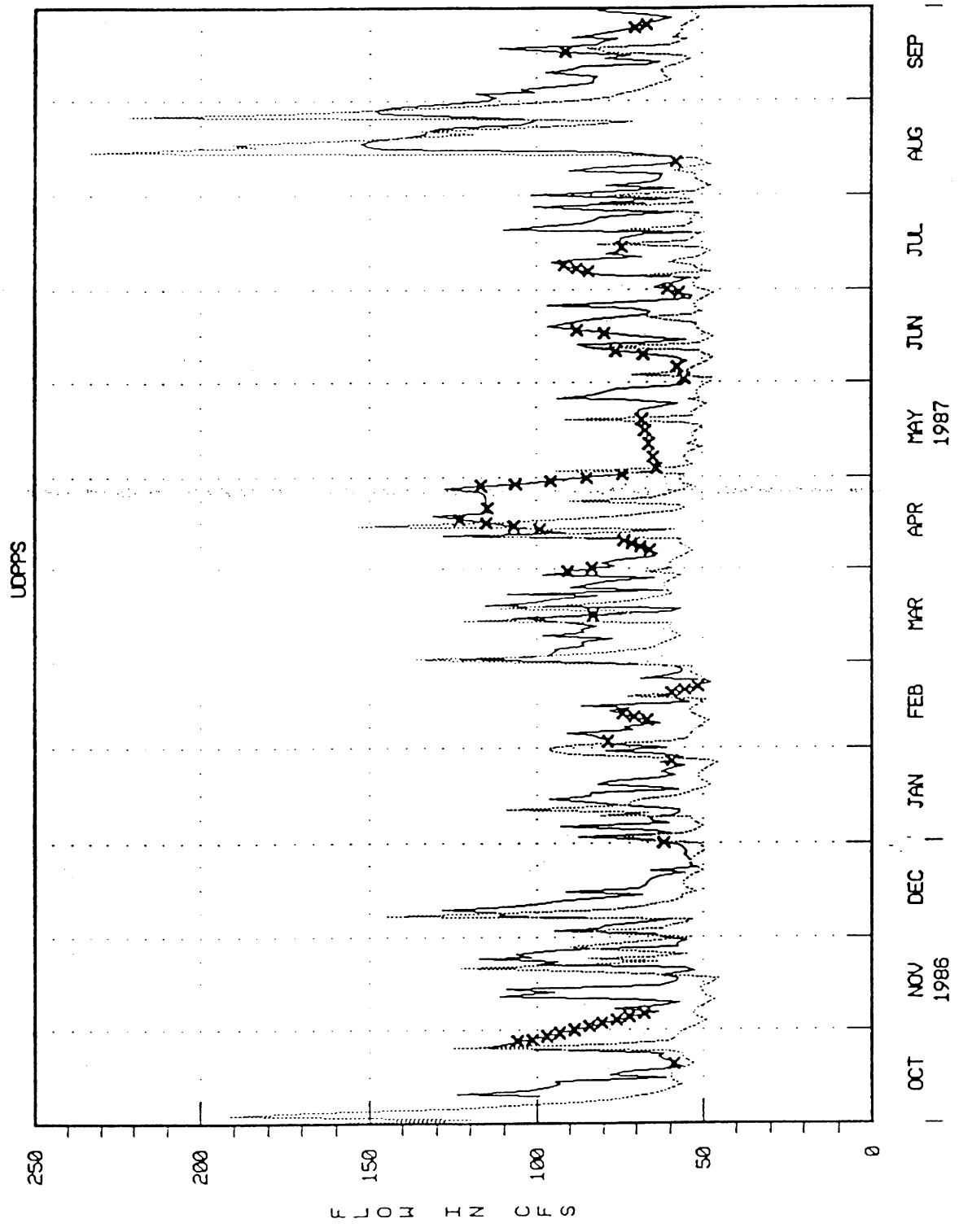
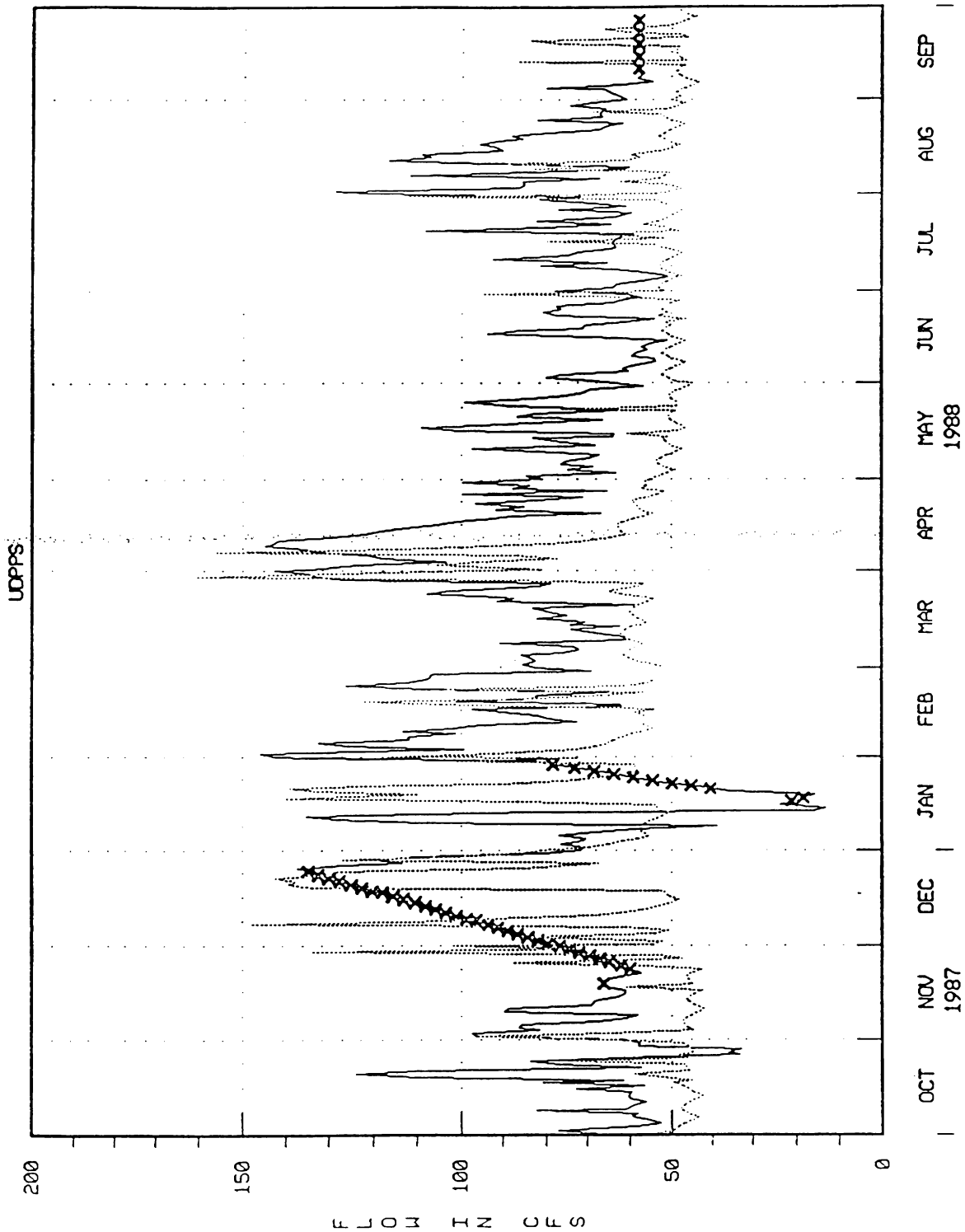

UDPPS
250
200
150
100
50
0
FLOW IN CFS
OCT
NOV
1986
DEC
JAN
FEB
MAR
APR
MAY
1987
JUN
JUL
AUG
SEP
TOTAL PLANT FLOW OBSERVED FLOW
TOTAL PLANT FLOW SIMULATED FLOW
Budget 8 - Simulation of the MWRDGC ' ner Des Plaines Pump Station

FIGURE 3.10



UPPER DES PLAINES PUMP STATION OBSERVED FLOW
UPPER DES PLAINES PUMP STATION SIMULATED FLOW

Budget 8 - Simulation of the MWRDGC Upper Des Plaines Pump Station

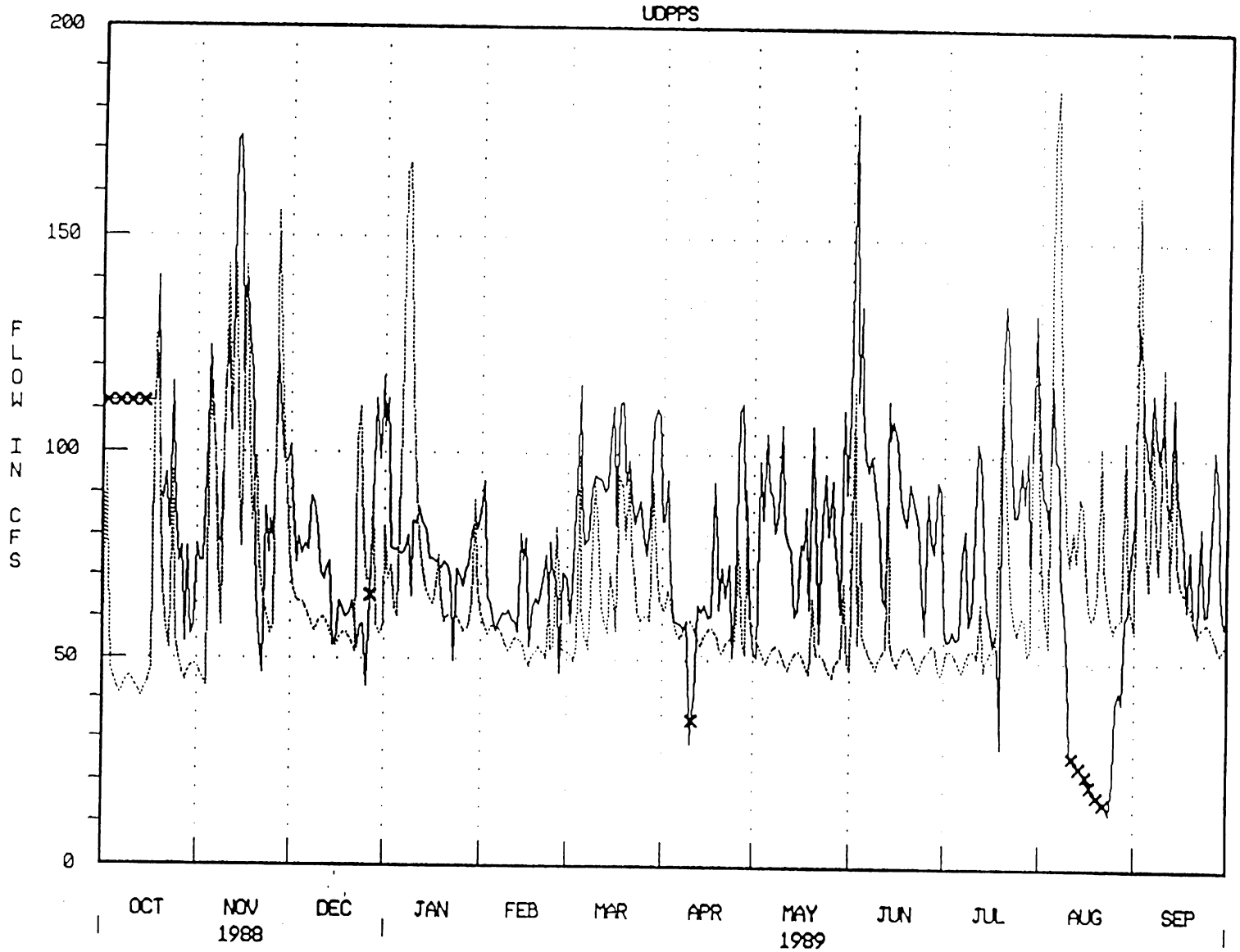


FIGURE 3.11

UPPER DES PLAINES PUMP STATION OBSERVED FLOW
UPPER DES PLAINES PUMP STATION SIMULATED FLOW

Budget 8 - Simulation of the MWRDGC Upper Des Plaines Pump Station

TABLE 7

SEWER SYSTEM EVALUATION SURVEY (S.S.E.S.)

After Burke, 1990

UPPER DES PLAINES BASIN

	COMMUNITY	STUDY DATE	CONTRACTOR	POPULATION EQUIVALENT	STUDY AREA (acres)	SANITARY SEWER LENGTH (miles)	SANITARY SEWER LENGTH (ln.-mi.)	SANITARY (gpcpd)	SANITARY (mgd)	FLOW INFILTRATION (gpcpd)	FLOW INFILTRATION (mgd)	INFLOW (gpcpd)	INFLOW (mgd)
1)	DES PLAINES	2/87	RJN	23,677	4,922	104.64	1033.75	88	2.04	121	2.86	1234	29.22
2)	FRANKLIN PARK	1/87	KUDRNA	16,401	800	22.90	262.1	150	2.46	61	1.00	456	7.48
3	HARWOOD HEIGHTS	4/89	MCCLURE	4,363	230	4.45	61.13	166	0.72	22	0.10	446	1.95
4)	LEYDEN TWSHP			11,800	1,260	30.46	352.89	60	0.71	64	0.76	339	4.00
5)	MELROSE PARK	11/87	SSE	8,848	768	11.87	121.92	142	1.26	111	0.98	261	2.31
6)	NORTHLAKE			21,071	1,866	35.6	397.05	203	4.28	133	2.80	532	11.21
7)	NORRIDGE	1/89	KUDRNA	15,954	883	26.97	272.75	80	1.26	53	0.85	871	13.90
8)	PARK RIDGE	1/87	HARZA	291	30	0.87	7.02	113	0.03	28	0.01	684	0.20
9)	ROSEMONT	3/88	BURKE	13,524	768	15.3	142.35	70	0.95	23	0.31	471	6.37
10)	SCHILLER PARK	11/87	SSE	11,103	960	20.97	192.66	96	1.07	111	1.23	166	1.84
11)	STONE PARK			4,815	218	5.8	54.71	107	0.52	40	0.19	0	0.00
	TOTAL			131,847	12,747	279.91	2898.35		15.30	23.67	11.09	17.16	79.49
	AVERAGE			11,986	1,159	25.45	263.49		1.39		1.01		7.14

4.0 AVM SYSTEM AT ROMEOVILLE

After a number of studies and detailed investigations, the first and the second Technical Committees concluded that the flow control structures and devices (i.e. turbines, sluice gates, and the upstream controlling works) at the Lockport Powerhouse are inadequate to provide acceptable and accurate accounting of the flows through the Lockport facilities.

In response to the Committee's concerns, the USACE in cooperation with the USGS installed an acoustic velocity meter (AVM), manufactured by Sarasota Automation, at Romeoville, five miles upstream from Lockport. The AVM is an accurate flow meter, which when properly calibrated, will provide more reliable and accurate accounting of the flows than those reported by the MWRDGC through the Lockport facilities. This AVM site is operated and maintained by the Illinois District of the U.S. Geological Survey and became operational on June 12, 1984 (USGS's Romeoville AVM Station Description - Appendix 3). A short time afterwards in March 1985, the AVM was damaged by barge traffic, which cut the submarine cables and rendered the AVM inoperative. Many problems were encountered while repairing the AVM including the inability to get replacement parts or service from the manufacturer. Eventually the AVM was repaired and put into operation in May 24 1988 (Station Analysis, 1989-1992; Appendix 4).

Because of the problems encountered with the AVM and the many problems in getting it repaired, a replacement system was bought from another manufacturer, ORE, Inc. This new AVM was installed in November 1988, and reliable flow records were being recorded by December 1, 1988.

To estimate flows during periods of malfunction of the AVM, a number of regression equations, relating Lockport flows to AVM flows, were developed by the Corps of Engineers, the USGS, Harza Engineering Company, and the second technical committee. A detailed discussion of the characteristics of these regression equations can be found in a report titled "Chicago Sanitary and Ship Canal at Romeoville Acoustic Velocity Meter Backup System." (USACE,1989). The regression equations that were ultimately selected and used to estimate the missing AVM flows from Water Year 1986 through Water Year 1991 were developed by the USGS. These equations are discussed in detail in a preliminary report titled "Comparison, Analysis, and Estimation of Discharge Data on the Chicago Sanitary and Ship Canal at Romeoville Illinois." This report will review the most important aspects of the AVM operation.

4.1 AVM SYSTEM

It has been demonstrated that acoustic velocity meters are particularly useful and successful in obtaining a continuous record of the discharge of rivers and canals, in sites such as Romeoville, where a simple stage-discharge relation cannot be applied satisfactorily.

When rating an AVM system a relation needs to be developed, not only between stage and cross-sectional area, but also between the line velocity measured by the AVM system and the mean velocity in the stream. At river locations with significant range in stage, a relation can be obtained also between stage and K-coefficient (the ratio of mean velocity to line velocity).

At this time, the U.S. Geological Survey does not have official guidelines for defining these relations. Nevertheless usable and pertinent information is available in papers and articles written by U.S. Geological Survey hydrologists. Particularly informative and useful among these papers is one titled "Defining the Relation Between Mean Velocity and the Line Velocity of an Acoustic Velocity Meter" prepared by A. Laenen (1992), USGS, preliminary draft copy.

This paper provides guidelines for defining a mean velocity to AVM velocity rating, explains the theory of the mean-velocity to line velocity (K-factor) relation, discusses various environmental variables that affect the AVM rating and a method to incorporate this variables into the rating, and describes velocity measurement methods that can be used to calibrate a rating.

Pertinent guidelines and information contained in the Laenen report that may be applicable to the Romeoville site have been gleaned and included in the following paragraphs. The Romeoville site will be reviewed based on these guidelines and information.

4.2 DEVELOPMENT OF A DISCHARGE RATING

Ratings for the K-factor, the mean velocity at the channel, and the cross-sectional area need to be established in order to compute discharge at an AVM location. The K-factor (ratio of mean channel velocity to AVM path velocity) is related to variables other than stage. Some of the variables are more significant than others and become more significant because of changing hydraulic or environmental conditions. Additionally, the shape of the vertical-velocity curve

may vary with the magnitude of the flow and a separate velocity rating may be required. Cross-sectional area is related to the stage and this relation is subject to change due to scour and fill. Scour and fill are, however unlikely in the Sanitary and Ship Canal at Romeoville because at this location sediment loads are small and the canal is carved into bedrock.

Rating environmental variables affecting the AVM rating include ice, wind, bed roughness, and aquatic biota. One way of allowing for the influence of environmental factors is to apply a shift based on measurements of discharge. However, several of these environmental variables can be measured accurately on a real-time basis. The K-factor can be defined as a function of stage and any number of environmental factors. These relations can be summarized in tables or represented by equations defined by regression analyses:

$$K = k_1 + k_2 s + k_3 s^2 + k_4 v_1 + k_5 v_2 \ldots\ldots + k_n v_{n-3} \quad (3.1)$$

or

$$K = k_1 * s^{x_1} * v^{x_2} * v^{x_3} \ldots\ldots v^{x_n} \quad (3.2)$$

where:

K = line velocity to mean velocity ratio related to stage and environmental factors.
K_n = coefficient defined by regression analysis or other curve fitting technique.
xn = exponents determined by regression analysis or other curve fitting technique.
s = stage
v^{xn} = environmental variable

4.2.1 Rating Velocity

A velocity rating can be developed as a function of the magnitude of the AVM line velocity. A velocity rating can either be tabular or an equation defined by regression analyses:

$$V = k1 + k2\ Vavm + k3\ (Vavm)^2 \qquad (3.3)$$

When developing a velocity rating, it is important that the vertical-velocity profile be defined as accurately as possible. The best way of defining accurate velocity distribution along the vertical is by making additional measurements of point velocities, thus defining individual vertical velocity profiles (for a range of flow conditions) that will establish relative bed roughness, shear velocity, and define local momentum changes.

4.2.2 Rating Area

Area can be a straightforward rating if the cross section remains stable or changes little. Area can be defined by depths measured when making current meter measurements of discharge or by bathymetric survey. For locations where channel fill and scour make changes in the cross-section continually, provisions need to be made to define these changes. Periodic measurements can provide a shift which can be applied to the rating. Area ratings can be defined by measurements of depth and be presented in tabular form or equations defined by regression analyses:

$$A = k_1 + k_2 s + k_3 s^2 + k_4 d \quad (3.4)$$

or

$$A = k_1 * s^{x_1} + k_2 * d^{x_2} \quad (3.5)$$

where:
A = area rating
d = depth change from channel fill or scour.

Discharge is computed by multiplying K, V and A ratings, which have been defined by an adequate number of discharge measurements:

$$Q = K A V \quad (3.6)$$

where Q = discharge in cfs.

4.3 ROMEOVILLE AVM STATION

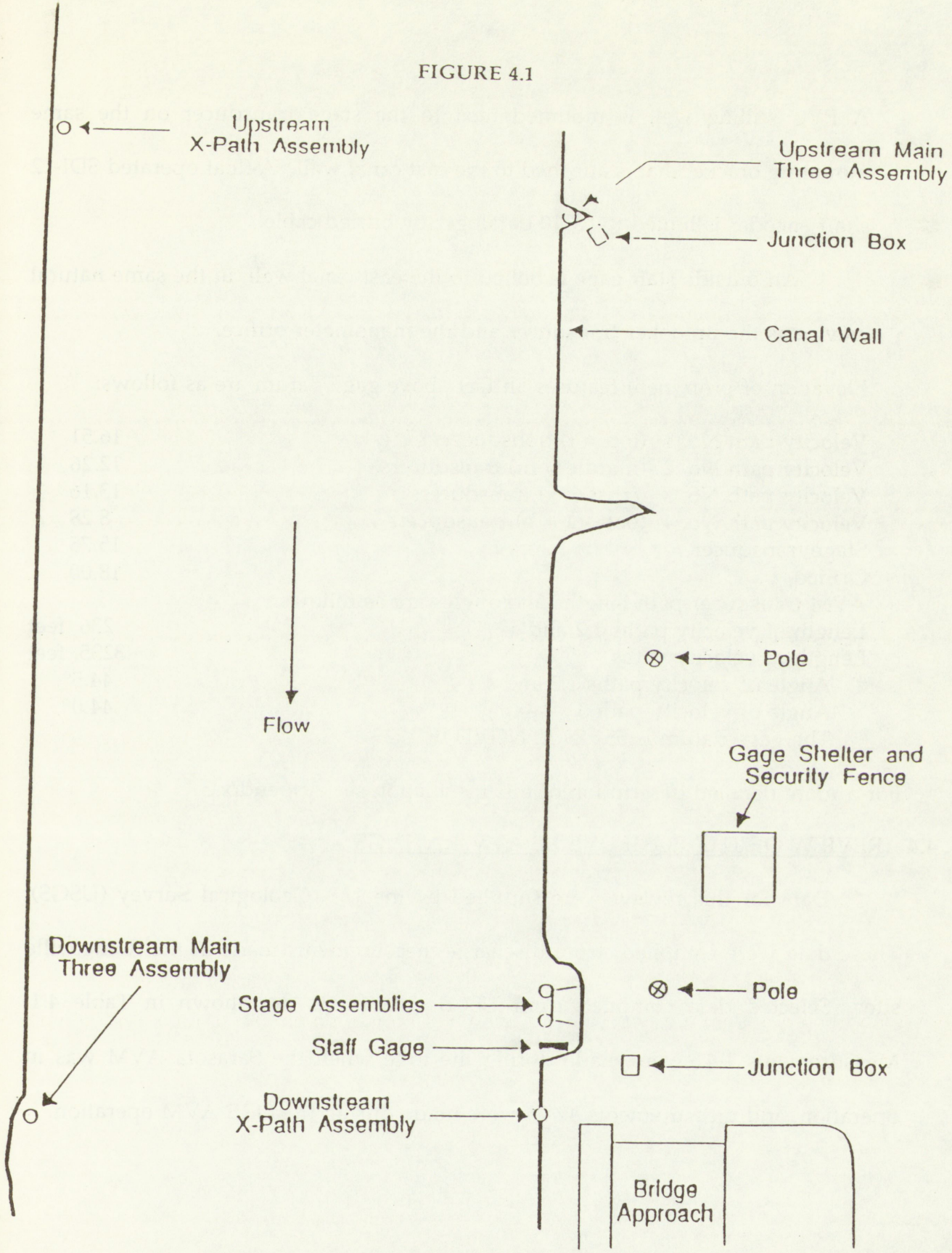
The gage is located on the left bank 40 feet upstream from the Romeoville Road (135th Street) bridge over the canal at Romeoville, Illinois 5.2 miles upstream from Lockport Lock and Dam.

The instrumentation includes a four-path Ferranti ORE 7410 Acoustical Velocity Meter (AVM), DecWriter printer, Campbell Scientific CR-10 dataloger with a temperature thermocouple, Handar SDI-12 shaft encoder, a manometer, equipped with a 10-turn potentiometer, and a telemetry system within an 8 ft x 8 ft x 8 ft concrete block shelter.

Eight velocity transducers are installed at four locations along the canal walls with two locations on each bank approximately 170 apart and 162 feet across from the other bank (Figure 4.1). The velocity transducers are installed in or attached to 3 or 6 inch plastic pipes. One set of three transducers (upper,

middle, and lower paths) is located along the east (left) bank approximately 130 feet upstream of the gage. The other set of three transducers is located along the west (right) bank near the upstream side of the bridge. The remaining pair of transducers make up the cross-path. The upstream cross-path transducer is located approximately 170 feet upstream from the bridge on the west bank and the downstream transducer is located on the east bank near the upstream side of the bridge. Each pair (path) of transducers is at approximately the same elevation across the canal from one another at approximately a 45 degree angle to the canal sides, and at different vertical locations within the cross-section. Each pair forms a velocity path. One stage transducer is attached to the east canal wall near the gage house and is pointed at the water surface. All transducers are linked to the AVM by individual electrical wires.

FIGURE 4.1



Sketch of Acoustic Velocity Meter, Chicago Sanitary and Ship Canal at Romeoville, Il.

A PVC stilling well is mounted next to the stage transducer on the same mounting bracket that is attached to the east canal wall. A float operated SDI-12 shaft encoder is linked to CR10 dataloger by buried cable.

An outside staff gage is bolted to the east canal wall, in the same natural crevice as the uplooker transducer and the manometer orifice.

Elevation of prominent features, in feet above gage datum are as follows:

Velocity path No. 1 (top = t) transducers	16.51
Velocity path No. 2 (middle = m) transducers	12.26
Velocity path No. 3 (cross = x) transducers	13.16
Velocity path No. 4 (bottom = b) transducers	8.28
Stage transducer	15.75
Orifice	18.00
AVM transducer path lengths and angles are as follows:	
Length of velocity paths 1,2 and 4	236. feet
Length of velocity paths	3235. feet
Angle of velocity paths 1,2 and 4	44.5°
Angle of velocity path 3	44.0°
The gage datum is 551.89 ft. NGVD of 1929.	

For a more detailed description of this installation see Appendix 3.

4.4 REVIEW OF THE ROMEOVILLE AVM RATINGS

Data for this review were supplied by the U.S. Geological Survey (USGS). These data were compiled from discharge measurements made at the Romeoville site. Selected data compiled from USGS data forms are shown in Table 4.1. Measurements 1-41 were made during the time when the Sarasota AVM was in operation, and measurements 42-73 were made during the ORE AVM operation.

TABLE 4.1

Discharge Measurement Data at Romeoville

Meas No.	Date	Width	Gage Height	Area	Measured Discharge	AVM Discharge	Observed Velocity
1	22-Mar-84	163	25.50	4190	6400	6310	1.53
2	12-Jun-84	164	25.20	4170	3710	3591	0.89
3	29-Jun-84	164	24.60	4088	3400	2436	0.83
4	16-Oct-84	164	22.20	3660	7670	8477	2.10
5	25-Feb-85	164	23.40	3830	5770	6117	1.51
6	04-Mar-85	164	22.20	3640	17700	17908	4.86
7	04-Apr-85	164	22.40	3680	17900	17842	4.86
8	29-Aug-85	164	24.90	4110	3460	2685	0.84
9	17-Oct-85	164	25.00	4090	2550	1486	0.62
10	19-Nov-85	164	23.20	3800	6660	6383	1.75
11	20-Nov-85	163	22.20	3650	8510	8230	2.33
12	20-Dec-85	164	25.10	4160	2920	2675	0.70
13	10-Jan-86	164	25.00	4280	1360	944	0.32
14	04-Apr-86	164	25.10	4210	1880	1392	0.45
15	01-Jul-86	164	24.30	3970	5100	4827	1.28
16	14-Jul-86	164	24.70	4150	3050	2678	0.73
17	07-Aug-86	164	24.70	4090	4720	4134	1.15
18	22-Sep-86	164	24.70	4150	4400	2541	1.06
19	25-Sep-86	164	24.80	4080	4460	4389	1.09
20	03-Oct-86	164	21.80	3660	15800	15725	4.32
21	12-Dec-86	164	25.00	4170	3430	3275	0.82
22	26-Jan-87	164	25.00	4190	2850	2903	0.68
23	Mar/16/87	164	25.10	4180	1950	2180	0.47
24	07-Apr-87	164	25.00	4170	2430	2383	0.58
25	15-May-87	164	25.10	4190	3120	3114	0.74
26	19-Jun-87	164	24.30	4080	7300	6838	1.79
27	24-Jul-87	164	24.70	4110	3970	3992	0.97
28	17-Aug-87	164	20.90	3450	14700	14325	4.26
29	09-Oct-87	164	24.90	4140	2670	2689	0.64
30	15-Oct-87	164	24.80	4160	2930	2837	0.70
31	15-Oct-87	164	24.90	4170	2460	2116	0.59
32	15-Oct-87	164	25.10	4160	2660	2271	0.64
33	19-Oct-87	164	25.20	4180	2320	1983	0.56
34	19-Oct-87	164	25.20	4170	2550	1983	0.61
35	03-Nov-87	164	25.10	4150	2020	1993	0.49
36	23-Nov-87	164	24.90	4150	2810	2847	0.68
37	30-Dec-87	164	25.00	4140	2840	3803	0.69
38	12-Feb-88	164	25.50	4220	2080	2181	0.49
39	10-Jun-88	164	25.00	4160	4110	4181	0.99
40	27-Jul-88	164	25.20	4150	3010	2942	0.73
41	15-Sep-88	164	24.80	4100	5230	5228	1.28
42	18-Nov-88	164	25.10	4210	4510	3810	1.07
43	13-Jan-89	164	25.30	4210	2580	2677	0.61
44	03-Mar-89	164	25.50	4220	2250	2401	0.53
45	05-Apr-89	164	25.60	4230	2560	2634	0.61
46	03-May-89	164	25.40	4140	2070	2233	0.50
47	01-Jun-89	164	21.20	3530	14400	14323	4.08
48	01-Jun-89	164	21.10	3550	15100	14209	4.25
49	18-Jul-89	164	24.70	4140	5480	5330	1.32
50	04-Aug-89	164	25.10	4140	4040	4090	0.98
51	12-Sep-89	164	24.70	4200	6010	5932	1.43
52	21-Nov-89	163	25.60	4170	2690	2804	0.65
53	21-Nov-89	163	25.60	4190	2890	2705	0.69
54	11-Jan-90	164	25.20	4200	2070	2319	0.49
55	11-Jan-90	164	25.10	4140	1990	2241	0.48
56	06-Mar-90	164	25.60	4260	2270	2046	0.53
57	06-Mar-90	164	25.50	4250	2700	2929	0.64
58	24-Apr-90	164	25.70	4220	2730	2746	0.65
59	11-Jul-90	164	25.60	4160	5450	5467	1.31
60	11-Jul-90	164	25.40	4180	5310	5362	1.27
61	20-Jul-90	164	20.70	3400	14400	13942	4.24
62	22-Aug-90	164	25.30	4150	5250	5082	1.27
63	14-Nov-90	164	25.60	4360	1920	2068	0.44
64	04-Dec-90	164	25.40	4380	4330	4848	0.99
65	06-Feb-91	164	25.50	4270	2560	2596	0.60
66	11-Mar-91	164	25.60	4390	2210	2152	0.50
67	15-Apr-91	176	21.90	4410	16900	16706	3.83
68	20-May-91	162	25.60	4430	2370	2542	0.53
69	18-Jul-91	162	25.20	4440	3780	3879	0.85
70	17-Oct-91	162	25.40	4460	2700	2772	0.61
71	17-Oct-91	162	25.50	4460	2590	1997	0.58
72	13-Jan-92	162	25.50	4460	2300	2289	0.52
73	23-Mar-92	162	25.50	4470	2620	2738	0.59

Measurements 1 through 62 were made from the upstream side of the Romeoville Road bridge. Measurements 63 through 73 were made at a section half way between the upstream and downstream transducers. Measurement 67 was made 4.3 miles upstream at Lemont Road.

4.4.1 Methods for Daily Discharge Computations.

Three methods for the computation of daily discharges, developed by the U.S. Geological Survey, were used during the 1986-1991 period:

1. During periods of normal AVM operation discharges were computed using AVM velocity records and manometer gage height records and a set of discharge equations developed for this purpose.
2. For periods of missing AVM record discharges were estimated from the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) records for Lockport which are adjusted using regression equations developed by the U.S. Geological Survey.
3. Beginning with the 1991 water year, daily discharges for periods of normal AVM operation, are computed using the Index Velocity method through the Automatic Data Processing System (ADAPS). The system consists of a velocity-velocity rating, and a stage-area rating.

4.4.2 Discharge Equations

The original set of discharge equations developed for use during periods of normal AVM operation are listed in Table 4.2.

TABLE 4.2

Discharge Equations for the AVM at Romeoville

Paths Working	EQUATION
b,m,x,t	Q = 1697*Vb + 205*Vm + 335*Vx + (Gh-14.67)*164*Vt
b,m,x	Q = 1697*Vb + 205*Vm + (Gh-12.63)*164*Vt
b,m, t	Q = 1697*Vb + 515*Vm + (Gh-14.52)*164*Vt
b, x,t	Q = 1722*Vb + 515*Vx + (Gh-14.67)*164*Vt
m,x,t	Q = 1867*Vm + 335*Vx + (Gh-14.67)*164*Vt
b,m	Q = 1697*Vb + (Gh-11.38)*164*Vm
b, x	Q = 1722*Vb + (Gh-11.53)*164*Vx
b, t	Q = 2032*Vb + (Gh-13.42)*164*Vt
m,x	Q = 1867*Vm + (Gh-12.63)*164*Vx
m, t	Q = 2177*Vm + (Gh-14.52)*164*Vt
x,t	Q = 2196*Vx + (Gh-14.67)*164*Vt
b	Q = 674*Vb + (Gh-5.14)*164*Vb
m	Q = 819*Vm + (Gh-6.24)*164*Vm
x	Q = 838*Vx + (Gh-6.39)*164*Vx
t	Q = 1086*Vt + (Gh-8.28)*164*Vt

These equations assumed that the channel was 164 feet wide and that the bottom was at elevation 0.0 foot. However, based on a bathymetric survey made in June 1991, the width was revised to 162 feet and the bottom elevation to -1.55 feet. A diagram of the revised channel is shown in Figure 4.2 and the revised equations are listed in Table 4.3. The revised equation listed in Table 4.3 were used to recompute the discharge record from November 18, 1988 to November 2, 1990. After November 2, 1990, the index velocity method was applied to estimate discharge of Romeoville,

whereas prior to November 3, 1988 discharge at Romeoville was calculated internally by the Sarasota AVM.

FIGURE 4.2

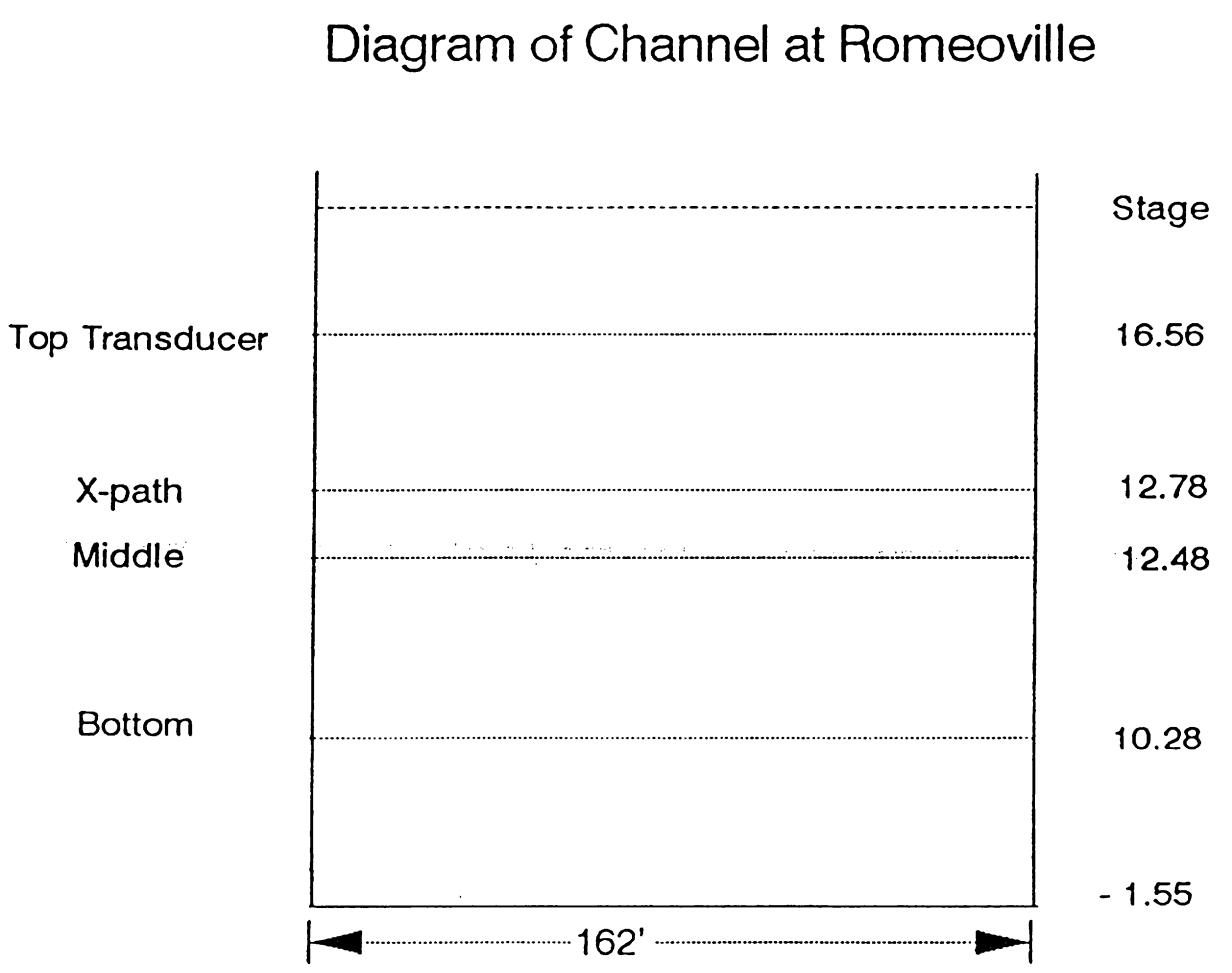

Diagram of Channel at Romeoville
Stage
Top Transducer
16.56
X-path
12.78
Middle
12.48
Bottom
10.28
- 1.55
162'

FIGURE 4.3

Comparison Between AVM and
Measured Discharges (1-41) - Sarasota AVM

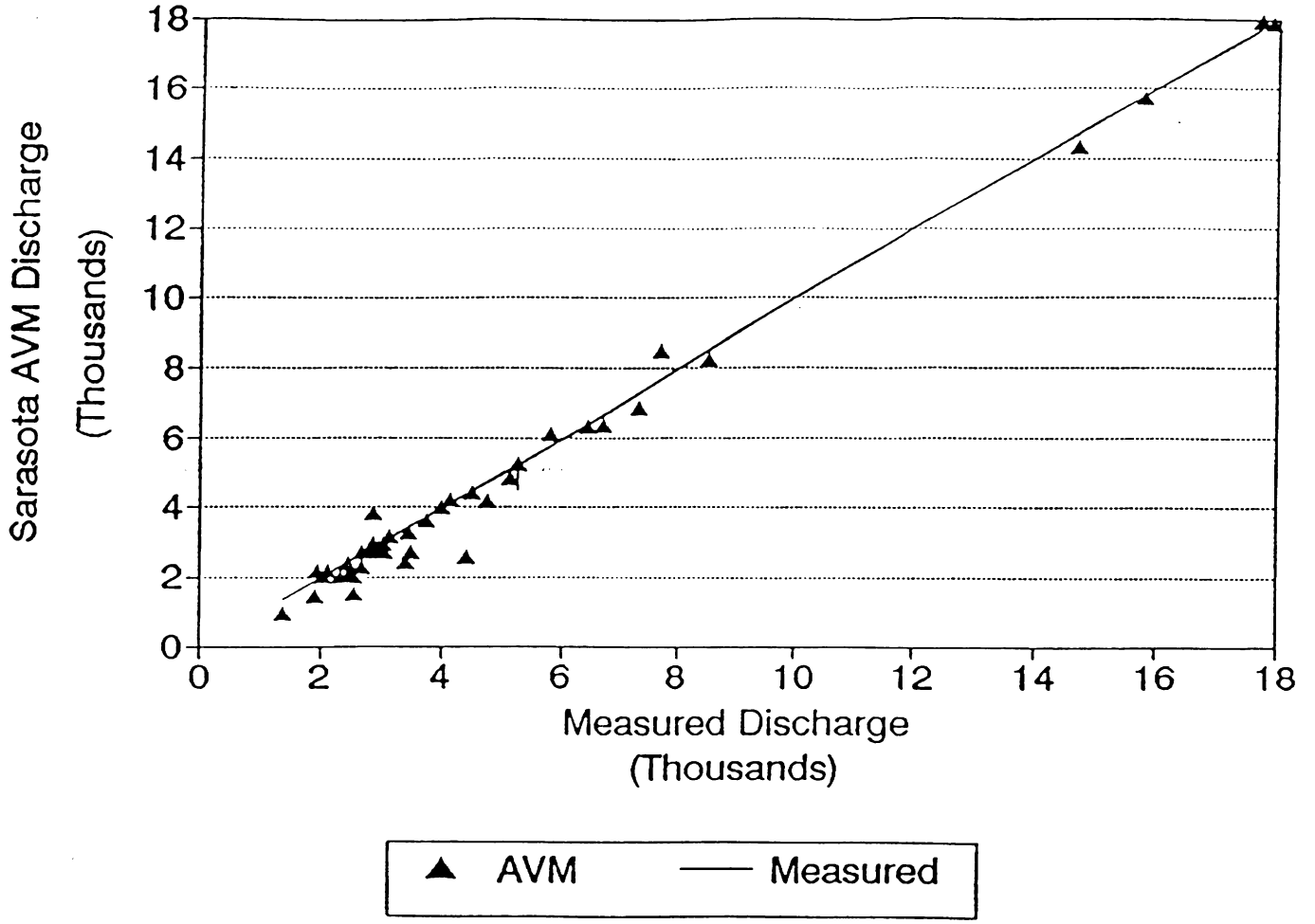


TABLE 4.3

Discharge Equations for the AVM at Romeoville

For the Period November 17, 1989 to November 2, 1990

Paths Working	EQUATION
b,m,x,t	Q = 1903*Vb + 202*Vm + 330*Vx + (Gh-14.67)*162*Vt
b,m,x	Q = 1903*Vb + 202*Vm + (Gh-12.63)*162*Vx
b,m, t	Q = 1903*Vb + 509*Vm + (Gh-14.52)*162*Vt
b, x,t	Q = 1927*Vb + 509*Vx + (Gh-14.67)*162*Vt
m,x,t	Q = 2070*Vm + 330*Vx + (Gh-14.67)*162*Vt
b,m	Q = 1903*Vb + (Gh-11.38)*162*Vm
b, x	Q = 1927*Vb + (Gh-11.53)*162*Vx
b, t	Q = 2234*Vb + (Gh-13.42)*162*Vt
m,x	Q = 2070*Vm + (Gh-12.63)*162*Vx
m, t	Q = 2376*Vm + (Gh-14.52)*162*Vt
x,t	Q = 2396*Vx + (Gh-14.67)*162*Vt
b	Q = 767*Vb + (Gh-4.36)*162*Vb
m	Q = 909*Vm + (Gh-5.46)*162*Vm
x	Q = 929*Vx + (Gh-5.62)*162*Vx
t	Q = 1174*Vt + (Gh-7.50)*162*Vt

4.4.2.1 Measured Discharge Vs. Sarasota AVM Discharge

Measurements 1 through 41 were made to verify the Sarasota AVM calibration. The AVM discharge data as well as the measured discharges are listed in Table 4.1. A plot of these data, Figure 4.3, shows that during the period when the AVM was working properly, the calibration remain reasonable constant throughout the period, and the AVM discharge records compared fairly well with the measured discharges.

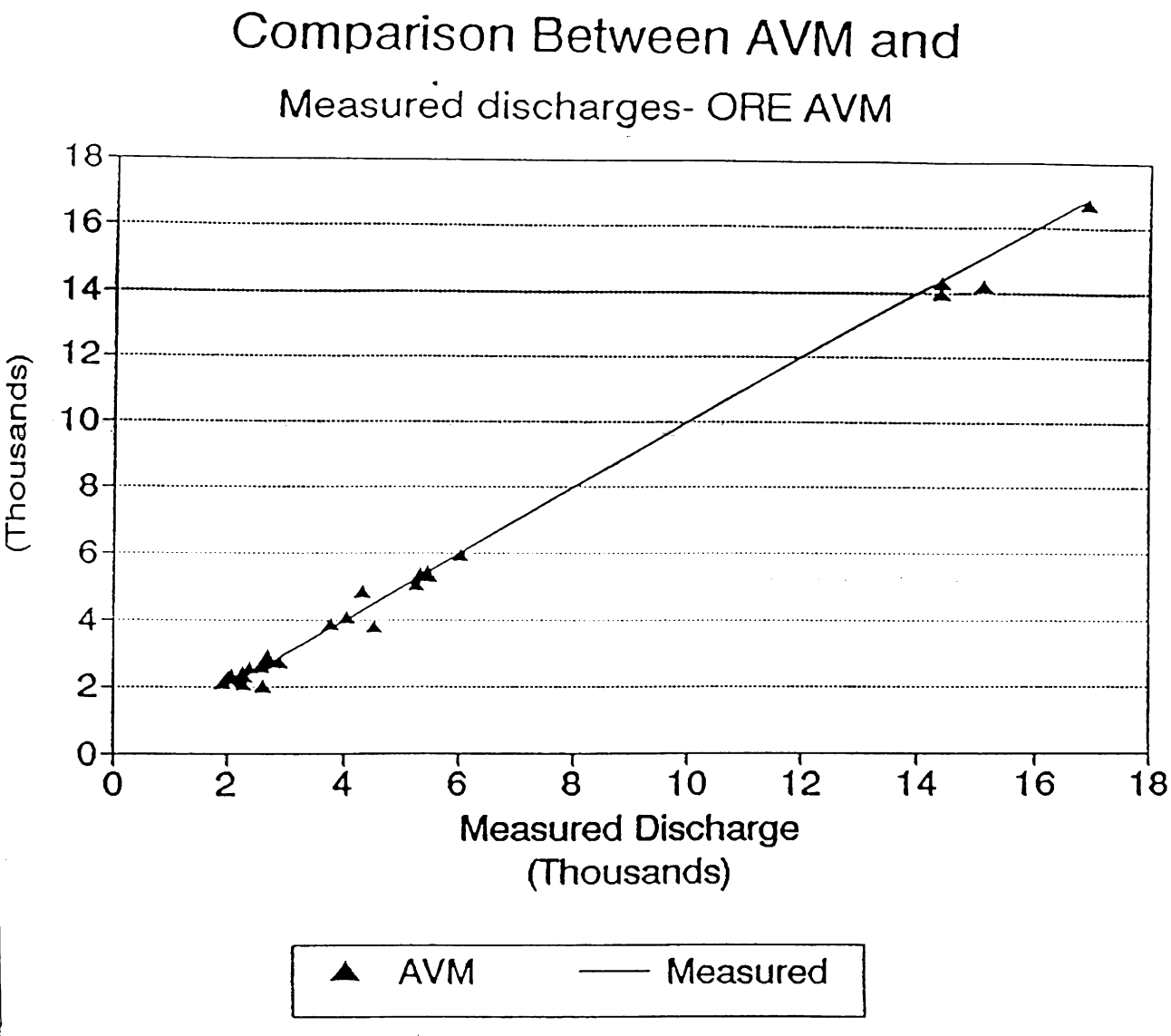
4.4.2.2 Measured Discharge Vs. ORE AVM Discharge

Measurements 42 to 73 were made to verify the calibration of the ORE AVM. Pertinent data are listed in Table 4.1, and for the purpose of comparison are plotted in Figure 4.4. The data in Table 4.1, as well as the plot in Figure 4.4, show that during periods of normal operation the ORE AVM discharge records compare reasonably well with the measured discharges.

Based on the above observations it is evident that the equations yield acceptable results. However, evaluating the equations according to guidelines (Laenen, 1992), the equations are theoretically incomplete, because the velocities are not adjusted as a function of stage. The guidelines suggest that including K factor, as defined by equations 1 or 2, in the discharge equations, would provide a tool for developing ratings where stage is a significant rating factor.

It appears that the reason why K is not included in the discharge equations, is because not enough data is available to develop a sound relationship between K and stage. The data that is needed for developing reliable values of K, are well defined velocity profiles. Collecting these type of data by current conventional methods is time consuming and expensive. However, the U.S. Geological Survey is now using a BroadBand Acoustic Doppler Current Profiler (ADCP) which is a new generation instrument that is able to measure with considerable improved precision. This improved precision allows finer vertical velocity resolution. Furthermore, measurements can be completed in minutes instead of hours. It is expected that the availability of this new technology will improve considerably the reliability and accuracy of the discharge computations at Romeoville.

FIGURE 4.4


Comparison Between AVM and
Measured discharges- ORE AVM
(Thousands)
18
16
14
12
10
8
6
4
2
0
0
2
4
6
8
10
12
14
16
18
Measured Discharge
(Thousands)
AVM
Measured

4.5 BACKUP SYSTEM Regression Equations.

The need for developing a backup system for estimation of discharges at times when the AVM is inoperative has been well documented, justified, and recommended by the second technical committee. The task of developing such a system was assigned to the Illinois District of the U.S. Geological Survey. A description of the analysis and documentation of this system was made available to the committee in a report titled "Comparison, Analysis, and Evaluation of Discharge Data on the Chicago Sanitary and Ship Canal at Romeoville, Illinois".

Briefly, the analysis involved: (1) checking the consistency of the discharges estimated by two different AVMs installed at Romeoville for consecutive time periods by statistical and regression analyses; (2) adjusting the discharge record to account for corrections to the width and depth of the canal determined by field measurements; and, (3) development of equations for estimating discharges on days when the AVM was inoperative using discharge estimates made by the Metropolitan Water Reclamation District of Greater Chicago at the lock, powerhouse, and controlling works at Lockport, Illinois.

The Committee reviewed this report with particular emphasis on the quality of the basic data, on the statistical techniques used to develop the model, and on the efficiency and reliability of the regression equations. Based on this review, the committee reached the following conclusion:

a. The data compiled from October 1, 1986 through May 31, 1992 is of acceptable quality. U.S Geological Survey hydrologists reviewed the validity and consistency of discharge estimates by the Sarasota and ORE

AVMs and found no significant difference between these two sets of data. This conclusion was reached based on statistical as well as hydrological considerations. The Committee agrees with this assessment of the data.

b. The estimating equations were developed using current and accepted statistical methods. The resulting equations exhibit excellent regression coefficients and explain over 95% of the total variance.

c. The report received a comprehensive technical review by highly qualified personnel at the Office of Surface Water, Water Resources Division, of the U.S. Geological Survey and has been published in a report titled "Comparison, Analysis, and Estimation of Discharge Data from Two Acoustic Velocity Meters on the Chicago Sanitary and Ship Canal at Romeoville, Illinois", U.S. Geological Survey, Water-Resources Investigations Report 93-4048.

The Committee is gratified to see this project is commanding the attention and support that is appropriate not only at the District level but at the Division level as well. The report constitutes a valuable and useful tool for computation of discharges for periods of AVM failure.

The estimating equations published in the report are:

When flows at Lockport are through the turbines only,

$$Q_{AVM} = (1.1270 \times Q_{TLL}) + 75.48 \quad (3.7)$$

When flows at Lockport are through the turbines and powerhouse sluice gates with sluice gate flow less than 5,000 cfs.

$$Q_{AVM} = (1.1270 \times Q_{TLL}) + (0.6842 \times Q_{PHSG}) + 219.7 \quad (3.8)$$

When flows at Lockport are through the turbines, powerhouse sluice gates, and controlling works or when flow through the powerhouse sluice gates is greater than 5,000 cfs.

$$Q_{AVM} = (1.1270 \times Q_{TLL}) + (0.4361 \times Q_{PHSG}) + (0.3228 \times Q_{CW}) + 1086 \quad (3.9)$$

Where:

Q_{AVM} = Estimated AVM discharge
Q_{TLL} = MWRDGC reported flow through the Lockport turbines and locks plus leakage.
Q_{PHSG} = MWRDGC reported flow through the Lockport powerhouse sluice gates.
Q_{CW} = MWRDGC reported flow through the Lockport controlling works.

4.5.1 Index Velocity Method

Beginning with the 1991 water year the discharge records are computed using the Index-Velocity method. This method consists of a velocity-velocity rating and stage area rating, given by the following equations:

Channel Flow Area = (Gh * 162) + 251 **(3.10)**
Mean Channel Velocity = 0.92*(Mean AVM velocity) **(3.11)**

Where:

Gh = Gage height in feet
Mean AVM velocity = (Vel_t+ Vel_m + Vel_x + Vel_b)/4 if all paths are working

If all paths are not working, then the mean AVM velocity is computed by first correcting each working path velocity by the appropriate (Mean AVM Vel)/(Path velocity) ratio and then taking their arithmetic mean. These corrections ratios are listed below:

(Mean AVM Vel)/(Vel_t) = 0.964
(Mean AVM Vel)/(Vel_m) = 0.989
(Mean AVM Vel)/(Vel_x) = 1.000
(Mean AVM Vel)/(Vel_b) = 1.053

The mean AVM velocity computed using the above procedure appears to be a reliable index of the mean channel velocity.

4.5.2 Area Rating

Analysis of discharge measurements made at the upstream side of bridge from 1984-1990 indicated that the mean bottom elevation of the channel at the upstream side of the bridge fluctuated between about -0.20 feet to -0.9 feet below the zero of the gage.

This fluctuation appears to be primarily due to varying amounts of debris that from time to time accumulated and later transported away from the vicinity of the measuring section.

Beginning with discharge measurement #63 (November 14, 1990), the measuring site was moved to its present location, midway between the upstream and downstream transducers. Based on the results of a bathymetric survey of this new site, made in June 1991, the mean bottom elevation of the channel was found to be at 1.55 feet below the zero of the gage. It seems that the mean bottom elevation at this location remained nearly constant through the 1992 water year.

Based on the results obtained from studying the stability of the mean bottom elevation at the upstream side of the bridge (Appendix 5), the Committee was concerned about the stability of the mean bottom elevation of the channel at this new location. In response to this concern, and through a series of discussions and written communication, the U.S. Geological Survey informed the Committee (Appendix 6) that they plan to measure the cross section annually, and if needed, the stage-area rating will be revised or shifted as necessary to account for any significant changes in cross-section that may be detected.

To check the validity of the discharges computed using the index-velocity method, the U.S. Geological Survey maintains an updated table of equations for the AVM. These updated equations are compiled in Tables 4.4 and 4.5.

TABLE 4.4

Discharge Equations for the AVM at Romeoville

For the Period November 2, 1990 to May 16, 1991

Paths Working	EQUATION
b,m,x,t	Q = 1756*Vb + 364*Vm + 344*Vx + (Gh-14.64)*162*Vt
b,m,x	Q = 1756*Vb + 364*Vm + (Gh-12.52)*162*Vx
b,m, t	Q = 1756*Vb + 667*Vm + (Gh-14.38)*162*Vt
b, x,t	Q = 1798*Vb + 667*Vx + (Gh-14.64)*162*Vt
m,x,t	Q = 2056*Vm + 344*Vx + (Gh-14.64)*162*Vt
b,m	Q = 1756*Vb + (Gh-10.27)*162*Vm
b, x	Q = 1798*Vb + (Gh-10.53)*162*Vx
b, t	Q = 2100*Vb + (Gh-12.40)*162*Vt
m,x	Q = 2056*Vm + (Gh-12.52)*162*Vx
m, t	Q = 2358*Vm + (Gh-14.38)*162*Vt
x,t	Q = 2391*Vx + (Gh-14.64)*162*Vt
b	Q = 637*Vb + (Gh-3.36)*162*Vb
m	Q = 895*Vm + (Gh-5.36)*162*Vm
x	Q = 929*Vx + (Gh-5.62)*162*Vx
t	Q = 1170*Vt + (Gh-7.48)*162*Vt

TABLE 4.5

Discharge Equations for the AVM at Romeoville

For the Period May 16, 1991 to Present

Paths Working	EQUATION
b,m,x,t	Q = 1756*Vb + 395*Vm + 344*Vx + (Gh-14.84)*162*Vt
b,m,x	Q = 1756*Vb + 395*Vm + (Gh-12.71)*162*Vx
b,m, t	Q = 1756*Vb + 667*Vm + (Gh-14.38)*162*Vt
b, x,t	Q = 1828*Vb + 667*Vx + (Gh-14.84)*162*Vt
m,x,t	Q = 2086*Vm + 344*Vx + (Gh-14.84)*162*Vt
b,m	Q = 1756*Vb + (Gh-10.27)*162*Vm
b, x	Q = 1828*Vb + (Gh-10.72)*162*Vx
b, t	Q = 2100*Vb + (Gh-12.40)*162*Vt
m,x	Q = 2086*Vm + (Gh-12.71)*162*Vx
m, t	Q = 2358*Vm + (Gh-14.38)*162*Vt
x,t	Q = 2416*Vx + (Gh-14.84)*162*Vt
b	Q = 637*Vb + (Gh-3.36)*162*Vb
m	Q = 895*Vm + (Gh-5.36)*162*Vm
x	Q = 953*Vx + (Gh-5.80)*162*Vx
t	Q = 1170*Vt + (Gh-7.48)*162*Vt

A general evaluation of the work concerning the direct measurement of discharge can be summarized by saying that the U.S. Geological Survey has done a commendable work. They have acquired and are using state-of-the-art equipment and instrumentation for measuring stream flow, have developed and continue to improved computational techniques which conform with the latest scientific knowledge and engineering practice. They have developed a reliable backup computational procedure to be used at times of

AVM failure, and maintain and updated set of AVM discharge equations to be used for quality control of the discharges computed using the current Index-Velocity method.

5.0 LEAKAGE AT LAKE MICHIGAN CONTROL STRUCTURES

The State of Illinois has requested leakage through two federally owned and operated lakefront facilities at Chicago River Controlling Works (CRCW) (Figure 5.1) and O'Brien Lock and Dam (O'Brien) (Figure 5.2) be considered a deduction from the total flow charged against the State of Illinois diversion. The Corps of Engineers requested the U.S. Geological Survey to make field measurements to determine the amount of leakage at each of the three lakefront structures (CRCW and O'Brien), including Wilmette Pumping Station (WPS). The Chicago District requested that the Committee observe the field measurements and review the preliminary results. The U.S. Geological Survey used two techniques for measuring leakage through the lakefront control structures. The first technique used was a BroadBand Acoustic Doppler Current Profiler (ADCP) from a moving boat. The second was a dye-dilution measuring technique, which was used to verify the measurements made with the ADCP at CRCW. ADCPs are state-of-the-art instruments which measure current velocity profiles. Recently, the U.S. Geological Survey began to use the ADCP system (a system developed by RD Instruments, Inc. and funded partially by the U.S. Geological Survey) to measure streamflow, especially in rivers or canals where standard discharge measurement techniques were either very difficult because of low velocities or very expensive. The use of an ADCP system to measure streamflow is described by Gordon (1989), and further documented by Simpson (1986), and Simpson and Oltman (1991).

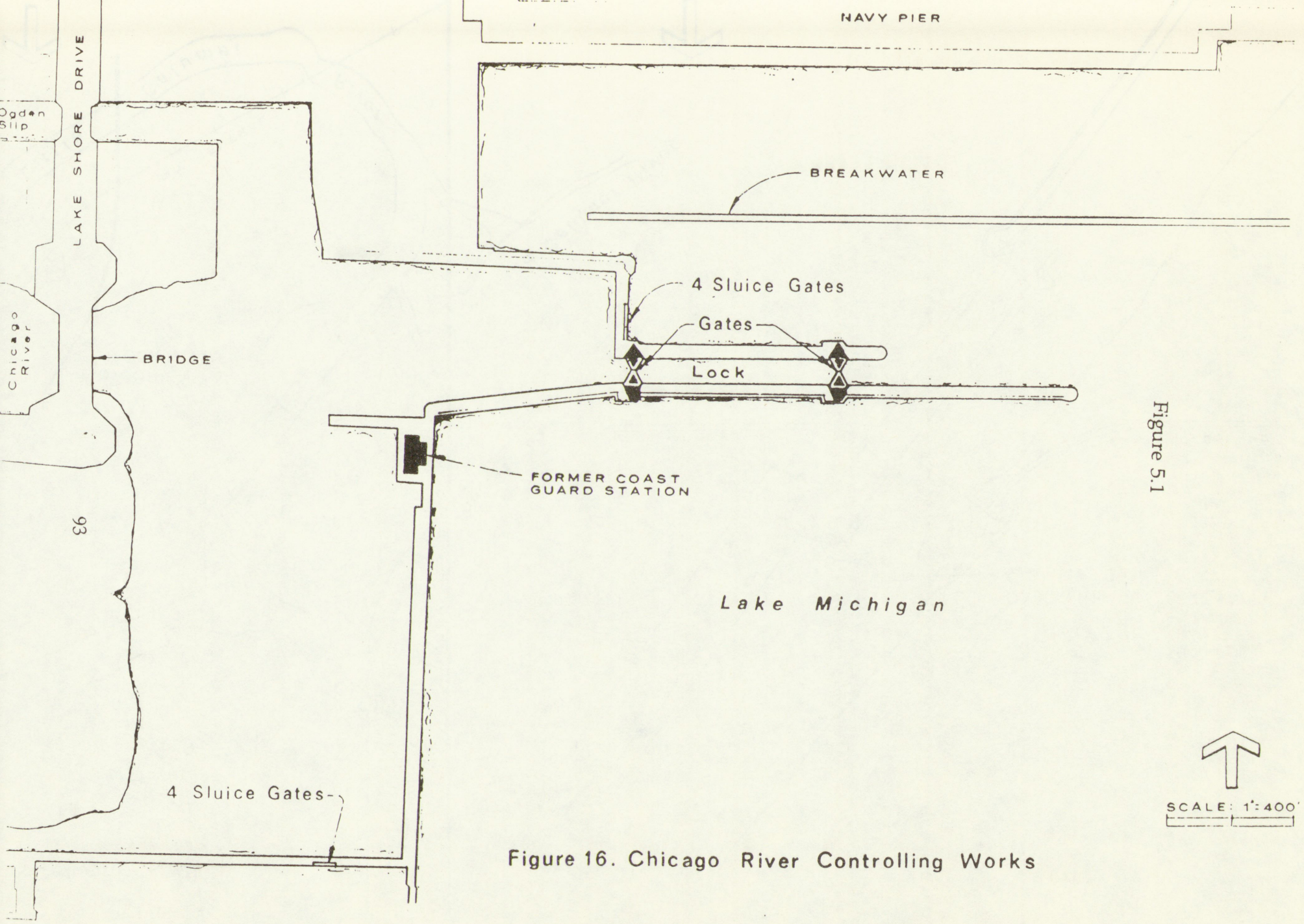


Figure 5.1

Figure 16. Chicago River Controlling Works

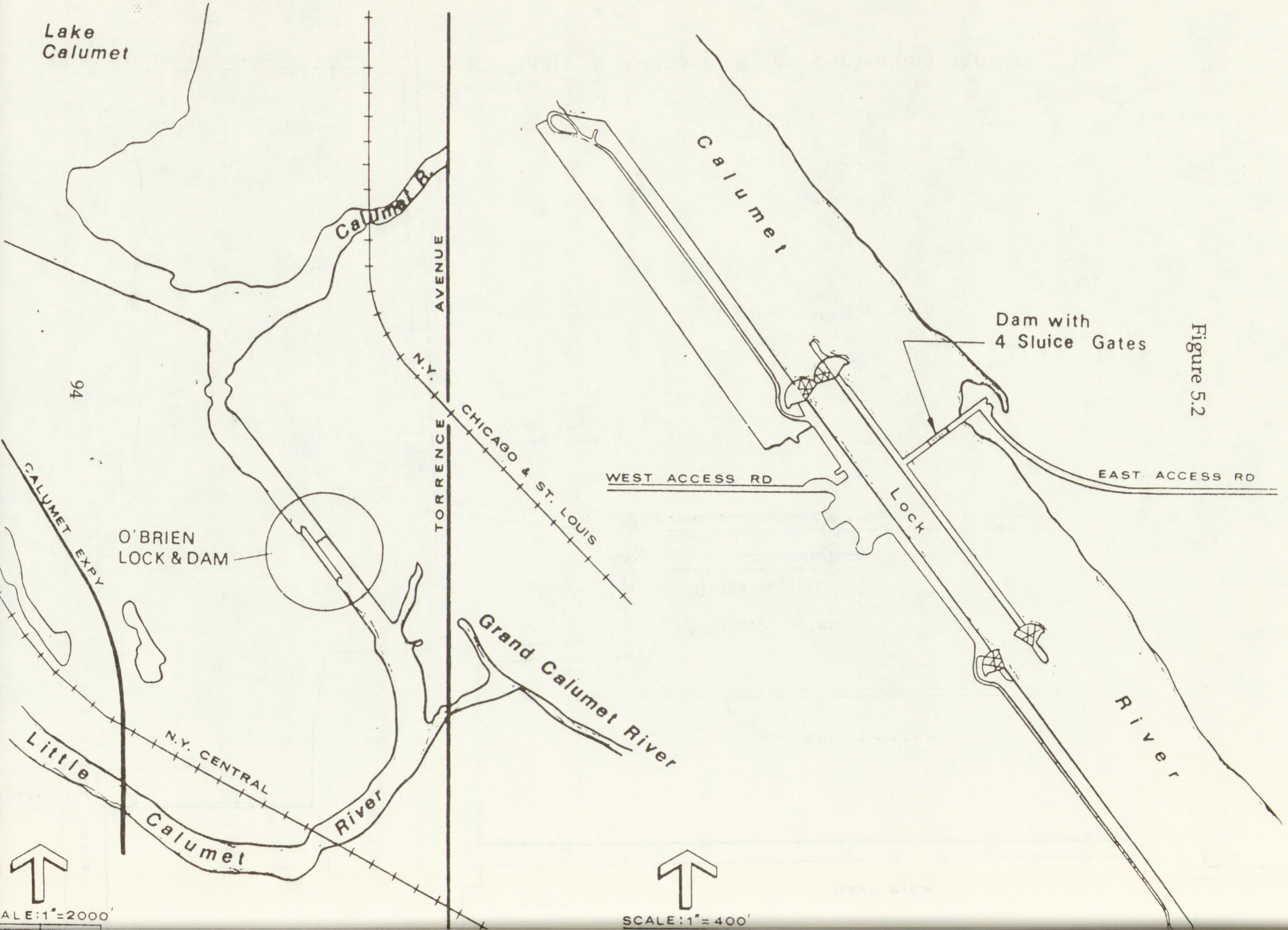

Lake
Calumet
Calumet R.
TORRENCE AVENUE
N.Y. CHICAGO & ST. LOUIS
94
CALUMET EXPY
O'BRIEN
LOCK & DAM
N.Y. CENTRAL
Little Calumet River
Grand Calumet River
Calumet
Dam with
4 Sluice Gates
WEST ACCESS RD
EAST ACCESS RD
Lock
River
SCALE: 1"=2000'
SCALE: 1"=400'

Figure 5.2

Dye-dilution discharge measuring techniques are described by Kilpatrick and Cobb (1985), Rantz, and others (1982). This technique was used to verify ADCP measurements. Dye-dilution and ADCP measurements were made simultaneously.

The U.S. Geological Survey completed a series of measurements at the Chicago River Controlling Works (CRCW), Wilmette Pumping Station (WPS), and O'Brien Lock and Dam (O'Brien). Table 5.1 summarizes the dates during which measurements were made at each location, the number of measurements made that day, and the mean measured discharge for a given location and date.

A summary description of these measurements and a discussion of the limitations of the discharge measurements are contained in a preliminary progress report letter (October 16, 1993) from the U.S. Geological Survey prepared by Kevin Oberg. Key portions of this progress report are summarized in this section.

Accurate measurements of very low velocities (such as in this study) require the speed of the boat as it moves across the section being measured be similar to the flow velocity. This concept was not fully appreciated in the April/May 1993 measurements. Even in the later measurements, the combination of water and boat velocities being measured are so small that the precision limits of the ADCP are approached. Several dye-dilution measurements were also made in order to compare results with those of the ADCP. The final results of the ADCP and dye-dilution measurements will be documented in a final U.S.

TABLE 5.1

Preliminary Results of Measurements of Leakage Through Chicago Lakefront Control Structures (Months) 1993

Month (Date, N, Discharge)		Chicago Lock						Chicago River at Lake Shore Drive			WSP	O'Brien Lock
		East-gate			West-gate							
April	Date	6	-		5	-	-	7	-	-	6	7
	N	19	-		5	-	-	5	-	-	12	4
	Discharge	825	-		106	-	-	194	-	-	58	(*)
May	Date	5	-		4	6	-	10	-	-	-	-
	N	6	-		23	18	-	4	-	-	-	-
	Discharge	867	-		142	127	-	178	-	-	-	-
July	Date	-	-		13	14	-	14	15	26	-	-
	N	-	-		10	4	-	5	4	2	-	-
	Discharge	-	-		140	195	-	355	213	200	-	-
September	Date	-	-		-	-	-	20	21	-	22	17
	N	-	-		-	-	-	6	7	-	5	10
	Discharge	-	-		-	-	-	359	334	-	(*)	53.2
October	Date	-	-		-	-	-	4	5	6	-	-
	N	-	-		-	-	-	3	2	4	-	-
	Discharge	-	-		-	-	-	310	210	204	-	-

Date = Day of measurement
N = Number of measurements
Discharge = Mean of all discharge measurements
WPS = Wilmette Pumping Station

Geological Survey report that is in preparation. Because the data are preliminary, they should be used with caution. The mean discharges shown are, in some cases, based on only a few measurements. Additional quality assurance review of the measurements will be made by U.S. Geological Survey and RD Instruments, Inc. prior to publication of the final report.

The following information regarding the limitations of the data should also be considered. During May 1993, both gates at the Chicago Lock underwent extensive repair by the Corps of Engineers. As a result, leakage through these gates has been changed considerably. In addition, it appears the Metropolitan Water Reclamation District sealed off the pump bays at the WSP sometime during the Summer of 1993. This could be the reason no leakage could be measured there in September. The April and May ADCP measurements in the Chicago River are probably less accurate than those made later in the year because of the following: (1) the April measurements were intended as reconnaissance measurements only and are not as accurate as later measurements; (2) U.S. Geological Survey measurement techniques and procedures improved during the later measurements as a result of the experience gained during the April and May measurements.

Tentative results of the preliminary measurements of leakage are encouraging, primarily because it appears the U.S. Geological Survey is on the right track to solving the difficult problem of measuring low velocity streams where unsteady flow patterns may be present or where the flow may be affected by rapidly changing tide or backwater conditions.

The U.S. Geological Survey's objective to validate the ADCP measurements using dye-dilution as an alternative method was good. However, after evaluating the physical characteristics of the site and taking into account the accuracy with which the major components of the dye-dilution need to be measured, the Committee believes that using the dye-dilution technique at this particular location may be more difficult, costly, and possibly less accurate than using the ADCP system. After reviewing the final report on ADCP and dye-dilution measurements for lake front leakage, which currently is being prepared by the U.S. Geological Survey, the Committee will be able to comment on this issue.

It is reasonable to assume the rate of leakage flow does not remain constant for long periods of time, but increases and decreases as the gates are damaged and repaired, and the seals deteriorate and are replaced. For a given set of conditions, the rate of leakage also changes as a function of the surface elevation of Lake Michigan. This will become a significant factor, particularly in those years within the accounting period, where the elevation of Lake Michigan reached record height.

From late 1984 to about February 1987, Lake Michigan exceeded its previously recorded high levels (Figure 5.3). The highest annual average level in 90 years of record (1903-1993) was reached in 1986. In fact, every monthly mean in 1986 was the highest in record, except for January which reached its highest average level in 1987. The highest monthly average elevation for the period of record was 582.38 feet (October 1986). During 1986, the monthly mean water levels of Lake Michigan were more than 2 feet above their respective long-term

Long-term fluctuation in the

water levels of Lake Michigan 1950-90

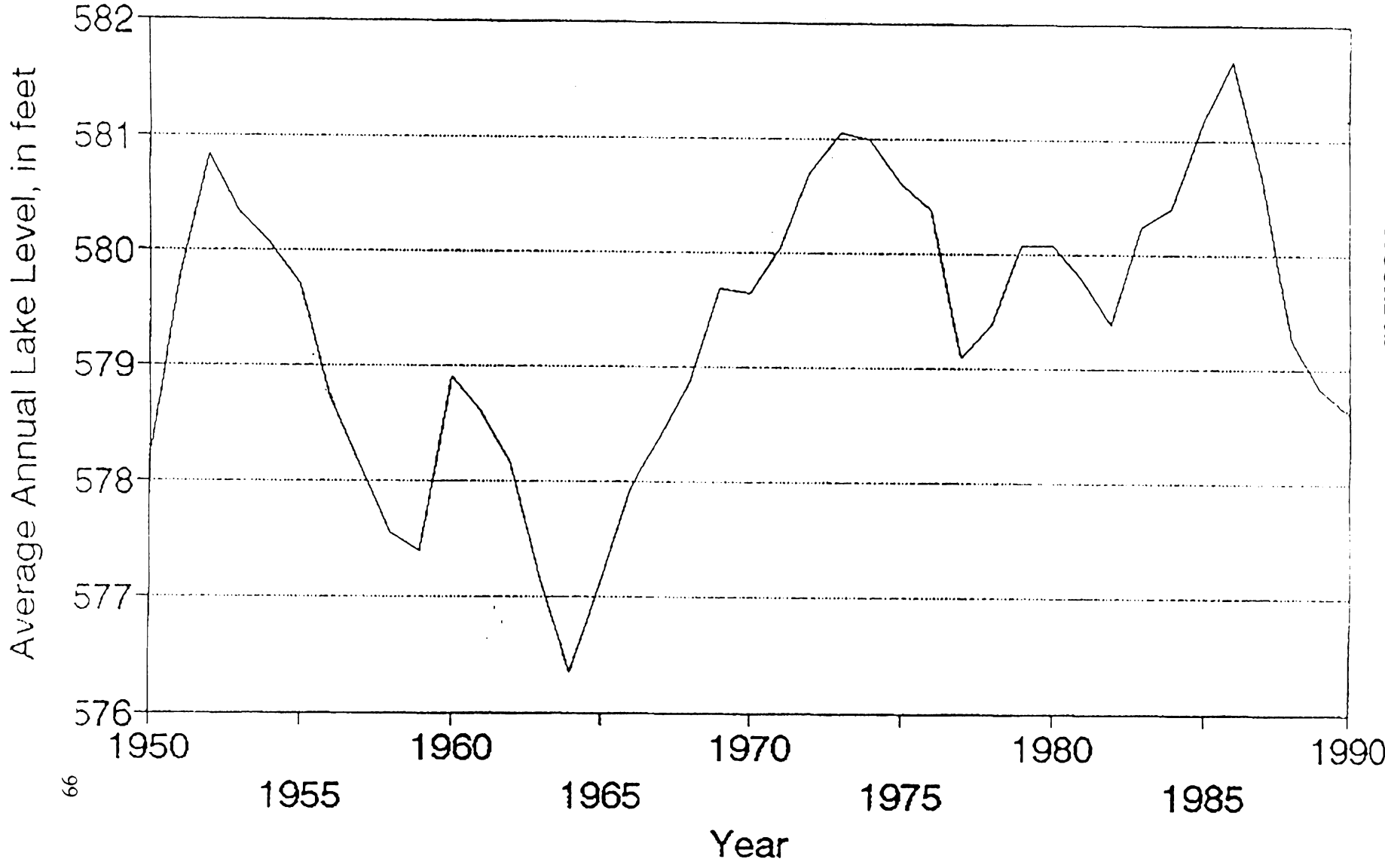


FIGURE 5.3

averages and over 3 feet during October and November (Table 5.2). For this reason, it is important to define, if at all possible, a relation between the rate of leakage flow vs. lake elevation.

Table 5.2

Summary of Average and Extreme Levels - Lake Michigan

	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Annual
	Monthly and Annual Average lake Levels for Period 1903 - 1990												
	578.41	578.44	578.57	578.83	579.16	579.36	579.42	579.33	579.16	578.93	578.70	578.54	578.93
	Highest Monthly and Annual Lake Levels for Period 1903 - 1990												
	581.56	581.26	581.16	581.62	581.72	581.89	582.02	582.02	581.98	582.38	581.79	581.46	581.69
	2.85	2.82	2.59	2.79	2.56	2.53	2.59	2.69	2.82	3.45	3.08	2.92	2.76
Year	1987	1986	1986	1986	1986	1986	1986	1986	1986	1986	1986	1986	1986

U.S. Department of Commerce - Lake Water Levels

6.0 TURBINE DISCHARGE RATING

Water Year 1984 was the first year in which the AVM was used to measure the flow in the canal system. As reported in the 1989 Annual Report, for Water Year 1984 the AVM measured flow was 314 cfs greater than the Lockport measured flow and in Water Year 1985, the AVM flow was 229 cfs greater than the Lockport flow. The First Technical Committee in evaluating the various measurement elements at the Lockport gage estimated the probable error of the turbines as a minus 15 percent. Corresponding relative of error percentage was 71.5 percent with respect to diversion computation. Based on the annual flow at Lockport for the period 1961 through 1969, turbine flow represent an average of approximately 78 percent of the average annual flow at Lockport (Espey et al, 1982). The Sarasota AVM system was installed March 18 - 23, 1984. This was the second AVM system, the first being an experimental Westinghouse system which was installed in the channel early in the 1970s and subsequently abandoned. The diversion accounting for Water Years 1984 and 1985 were the first water years which utilized the Sarasota AVM system. The 1981, 1982, and 1983 Water Years were still based on Lockport measurements. The Second Technical Committee analyzed the difference between the AVM and Lockport flows and determined the difference in the AVM and Lockport flow scan be attributed to a series of errors in the rating for one or more of the various Lockport flow components, that is, turbines, lockage, leakage, powerhouse, service gates, and controlling work gates. The Second Technical Committee reported (Espey et al, 1987) that on March 4, 1985 a high rate of flow was sustained for a period of about 9 hours

when all power house and controlling work gates were opened. During this 9-hour period, the discharge at the AVM site in Romeoville varied gradually from about 18,000 to 16,500 cfs while the reported MSD discharge varied from 35,000 to 35,200 cfs. The MSD reported discharge included about 12,000 cfs through the controlling work and about 21,000 cfs through the Lockport power house sluice gates. It was concluded the discharge through the controlling works was more likely to have been 5,000 to 6,000 cfs than the reported 12,000 cfs. The Second Technical Committee concluded that the MSD flows versus AVM flows in the lower flow range which is normally controlled by the turbines, lock operations, and leakage, the flow reported at Lockport was consistently under reported (Figure 6.1c - 6.1d and Figure 6.2). Conversely, the MSD flows are over reported when all gates for the powerhouse and controlling works are open. Controlling works flow are significantly affected by downstream submergence which leads to over reporting of the flow with five or more gates are opened.

The Second Technical Committee developed regression equations for MSD Lockport flow records for the periods June 12, 1984 - March 20, 1985 and September 23 - December 31, 1986 for turbine only flow conditions only:

$$Q_{AVM} = 1.10\ Q_{MSD} + 84. \qquad (6.1)$$

The flow record used to develop Equation 1 contains a period of record for which MSD turbine-only flow is unique because the navigation lack was closed for repairs from July 7 to September 28, 1984. Consequently, both lock leakage and

daily lockage flow were zero, and all flow was discharged through the turbines. A regression of the flow data for this period is:

$$Q_{AVM} = 1.08\ Q_{MSD} + 207. \qquad (6.2)$$

for MSD flows ranging from 2,457 to 4,221 cfs. Discharges estimated using Equations 6.1 and 6.2 agree within 2 percent.

Assuming Equation 6.2 is adequate for correcting under reported turbine flow and that approximately 78 percent of historical reported diversion is turbine flow, the "adjusted diversion flows" for 1981, 1982, and 1983 are presented in Table 6.1.

Table 6.1

Revised Water Years 1981, 1982, and 1983

Year	Reported	Corrected	Increase
1981	3106	3506	+400
1982	3087	3487	+400
1983	3612	4045	+433

6.1 THIRD COMMITTEE

The data available to the Third Technical Committee is longer and reliable. In addition, the MWRDGC installed an ORE AVM system at the Lockport Powerhouse turbines and began operating on March 19, 1992. This is a significant step towards achieving reliable technical quality in the direct measurement of

diversion flow at the turbines at Lockport. This information will be very valuable in establishing correct turbine flows for historical record. Future regression equations will use the turbine AVM flows as one independent variable.

A sample of the data now available is listed in Table 6.2 and 6.3, these data were supplied by the U.S. Geological Survey, and the tables contain typical mean daily discharges for Lockport (turbines, lockage, and leakage) and concurrent AVM discharges recorded at Romeoville for selected periods in 1990 and 1991. These data show the Lockport estimates are consistently low by 400 cfs on the average for the sample period in 1990 (Figure 6.3) and about 373 cfs for the period in 1991 (Figure 6.4) roughly an 11 percent bias overall. The data in Table 6.2 is a typical sample mean daily discharges recorded concurrently at AVMs in Lockport and Romeoville. It should be pointed out the Lockport discharges do not include the lockage discharge. The Lockport discharges, adjusted for the number of reported lockages are listed in the next to last column of Table 6.4. These adjusted AVM discharges recorded at Lockport compare reasonably well with the Romeoville AVM discharges.

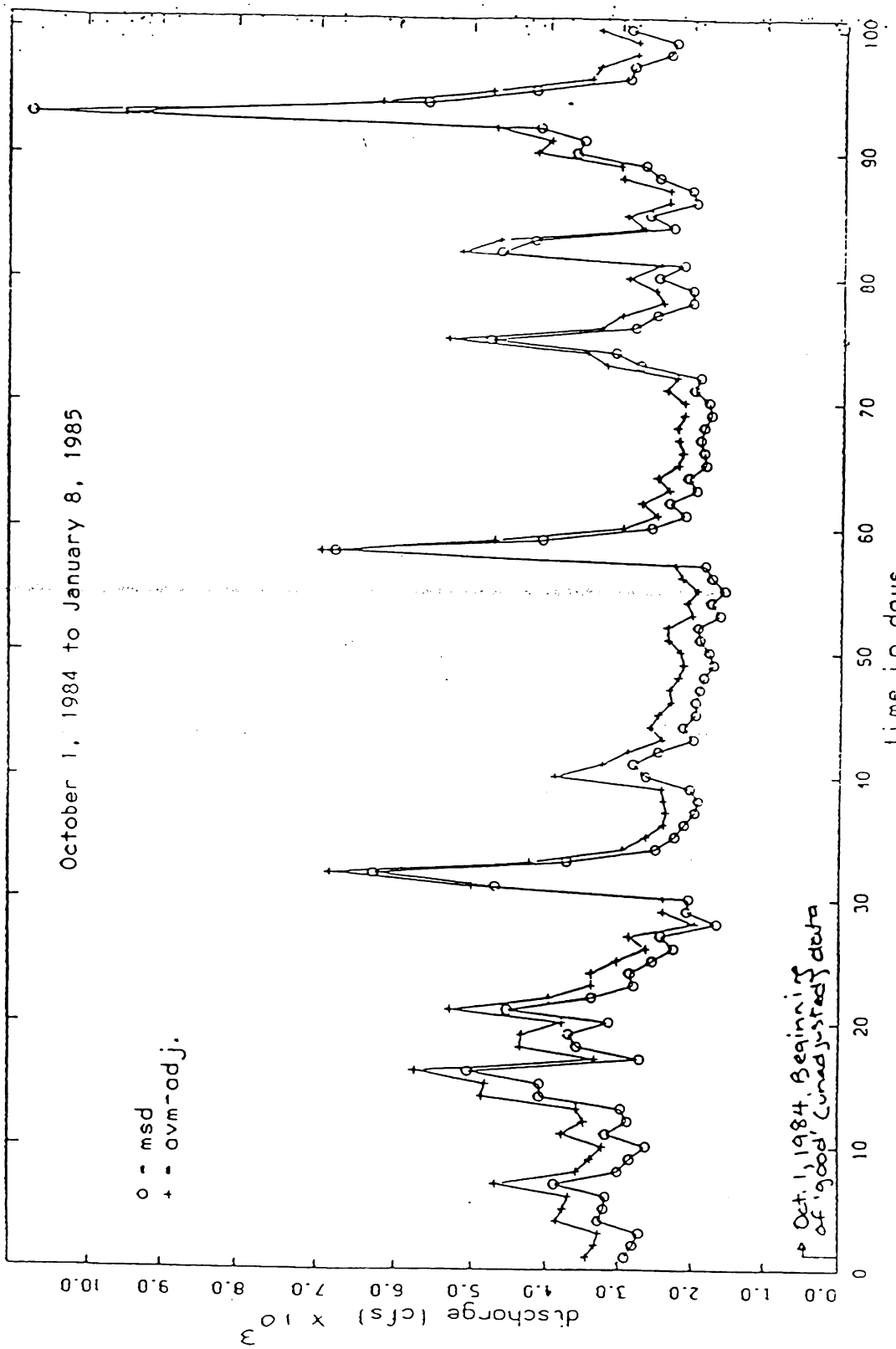
6.2 Recommendation

It has been shown, by the First and Second Technical Committees, and confirmed by current data, that turbine flow components at the Lockport facility are biased. For this reason, this Committee supports the recommendation made by the Second Technical Committee that the reports for Water Years after 1980 should be revised as necessary, in order to account for the errors in the Lockport discharge ratings used during the 1981-85 Water Years. The Third Technical

Committee believes that revising past records to reflect present knowledge and standards would remove a potential source of controversy, and most importantly would bring undeniable credibility to the diversion accounting process.

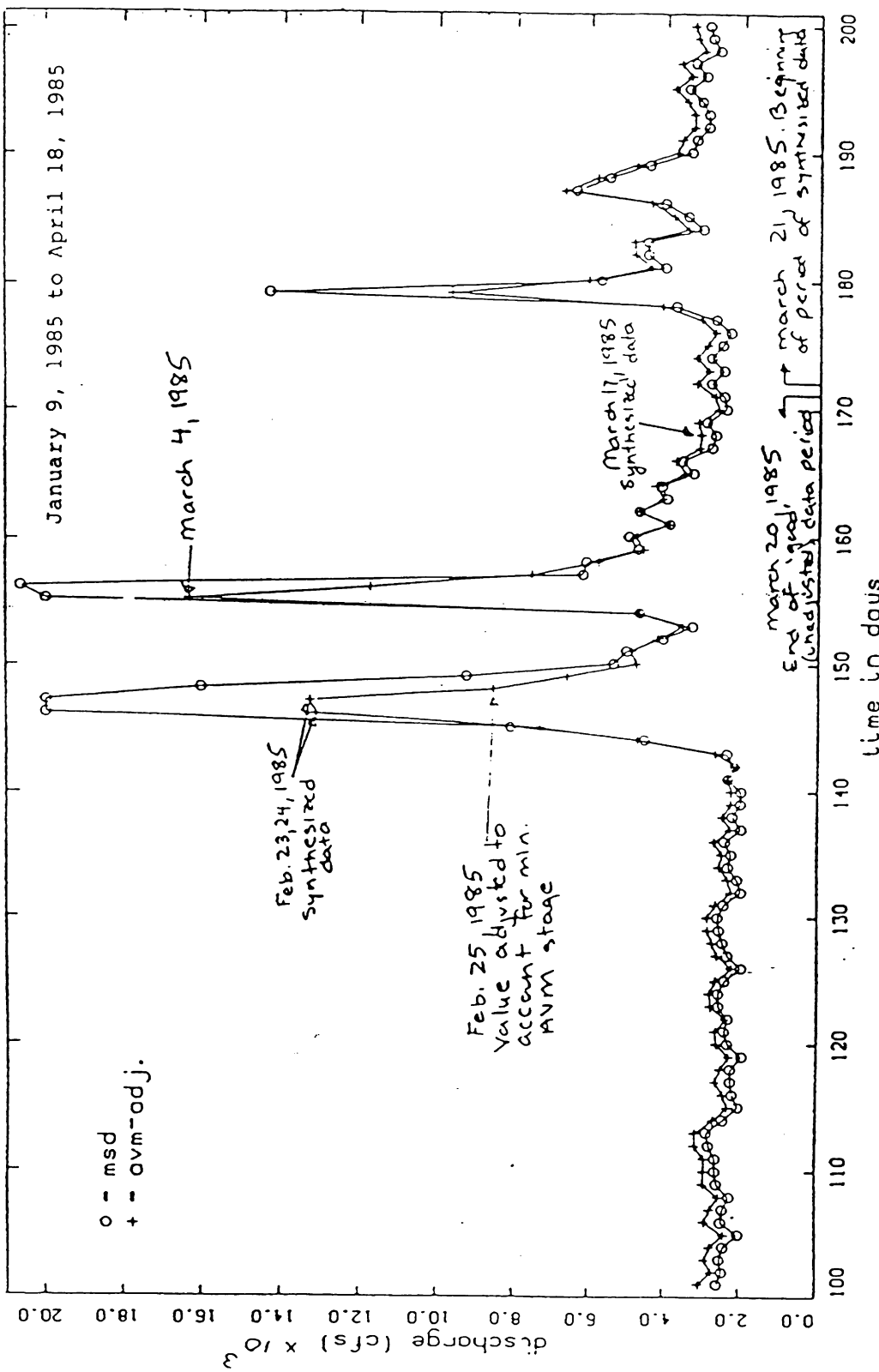
The Third Technical Committee commends the State of Illinois, the MWRDGC, and the U.S. Army Corps of Engineers for acquiring state-of-the-art equipment for the direct measurement of the diversion flow.

FIGURE 6.1a



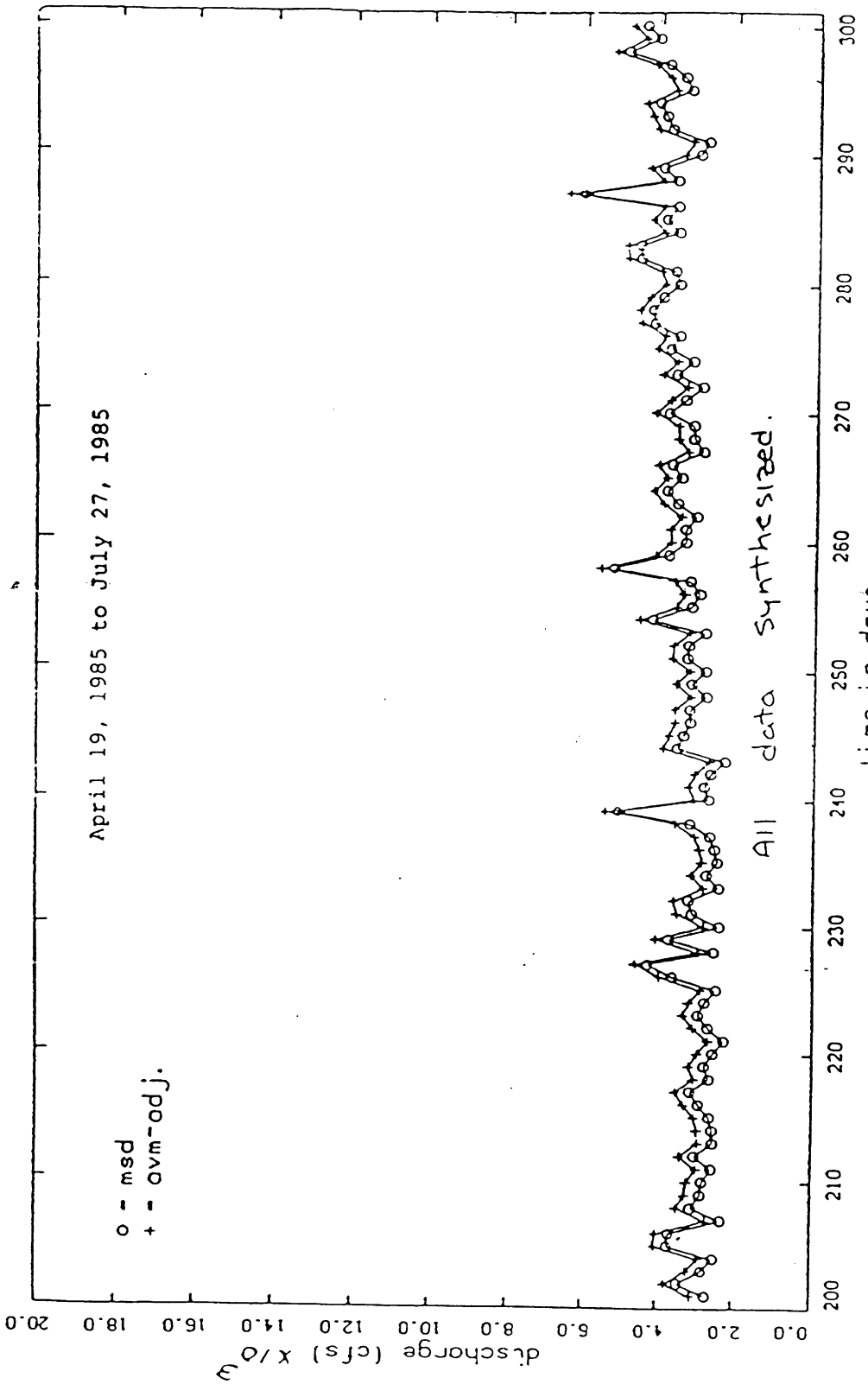
MSD and AVM - Adjusted discharge.

FIGURE 6.1b



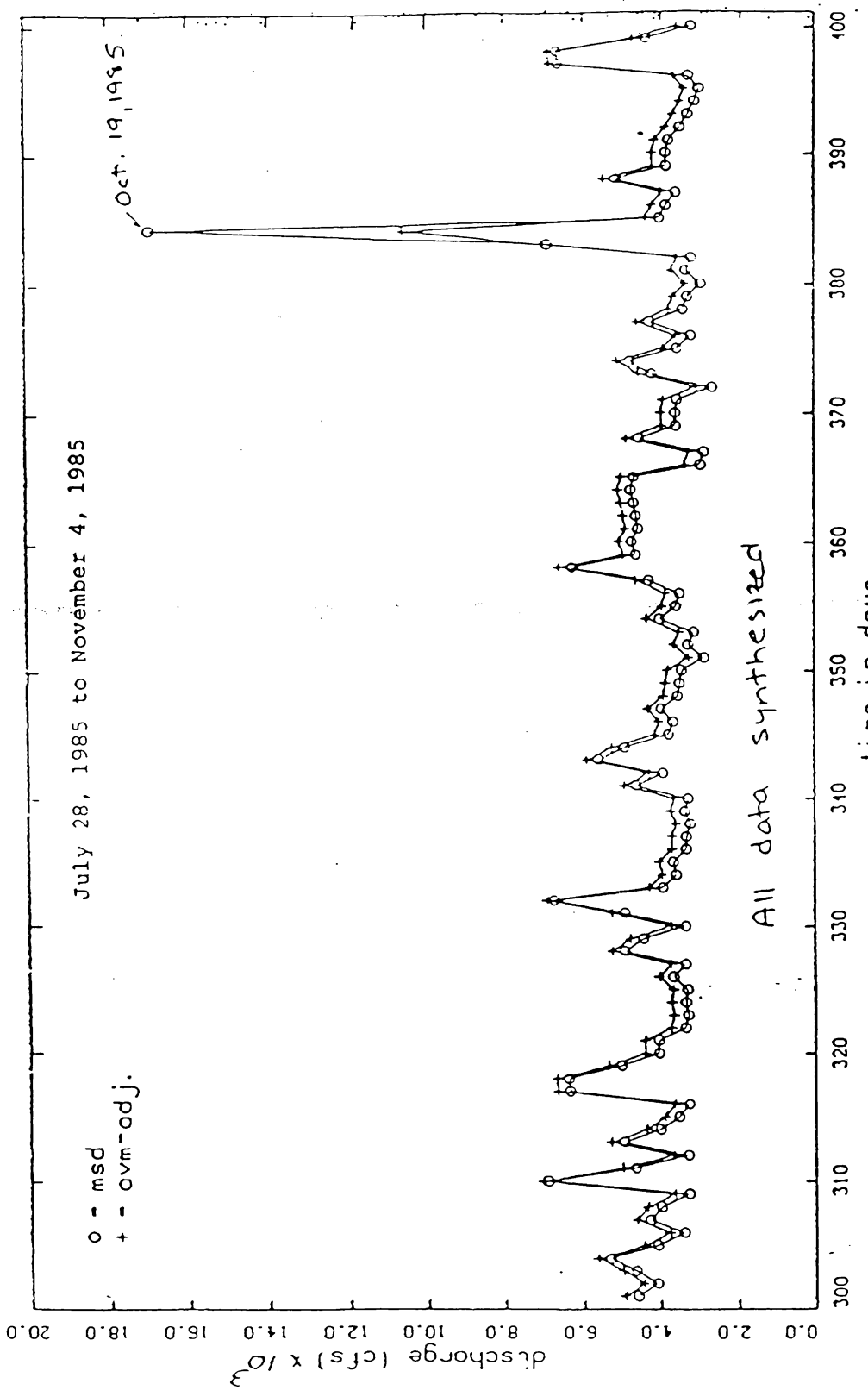
MSD and AVM - Adjusted discharge (cont'd)

FIGURE 6.1c



MSD and AVM - Adjusted discharge (cont'd)

FIGURE 6.1d



MSD and AVM - Adjusted discharge (cont'd)

FIGURE 6.2

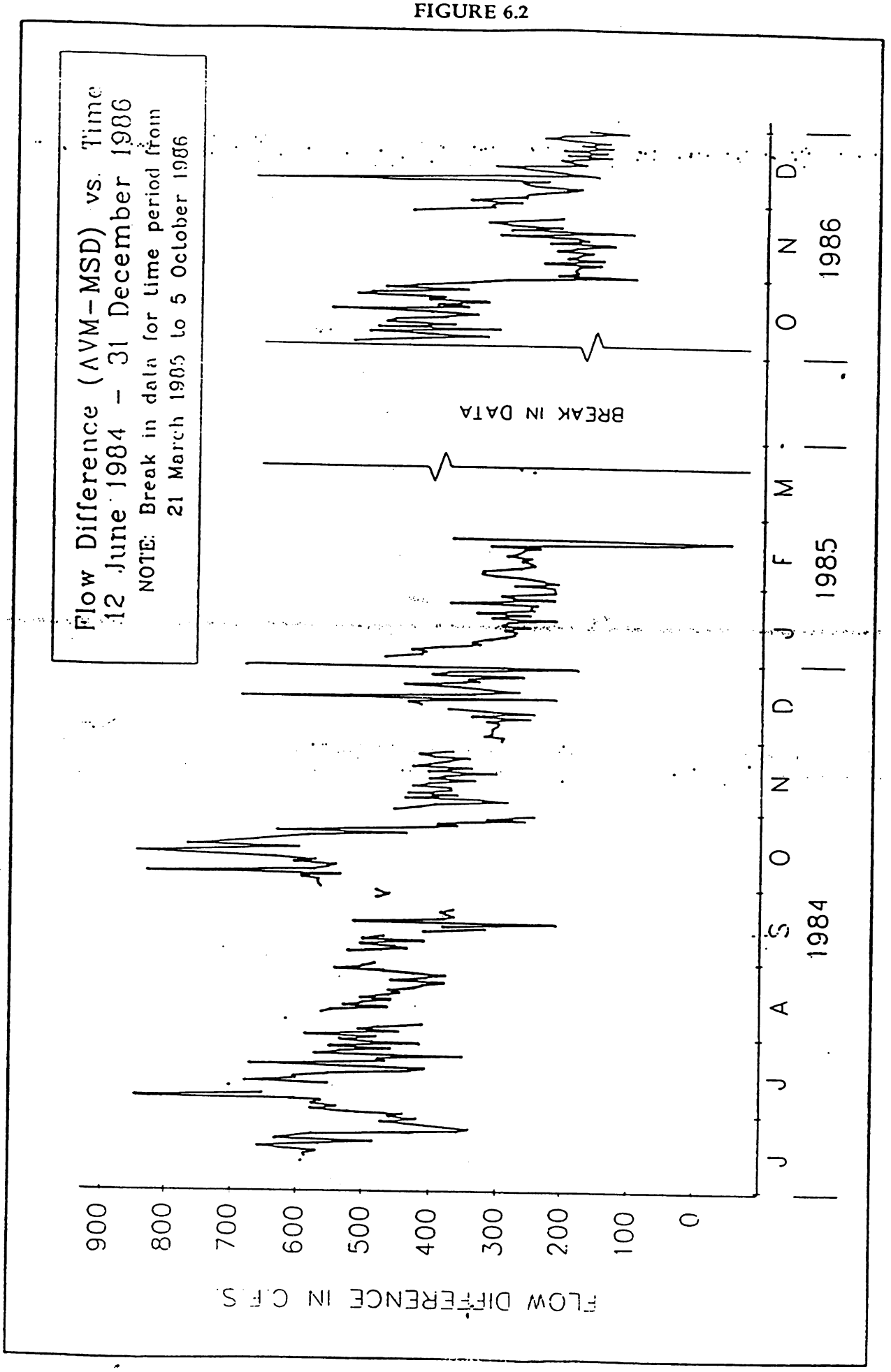

Flow Difference (AVM-MSD) vs. Time
12 June 1984 - 31 December 1986
NOTE: Break in data for time period from
21 March 1985 to 5 October 1986
BREAK IN DATA
FLOW DIFFERENCE IN C.F.S.
900
800
700
600
500
400
300
200
100
0
J
A
S
O
N
D
J
F
M
A
M
J
J
A
S
O
N
D
1984
1985
1986

TABLE 6.2

Mean daily discharges for Romeoville AVM and Lockport. October - December, 1990

Date	Mean daily discharge, in cubic feet per second								
	Turbine	Lockage	Leakage	Sluice gate	Control works	Lockport total	Romeoville AVM	Absolute Difference	Percent Difference
10/1/90	1235	396	100	0	0	1731	2063	332	16.09%
10/2/90	1330	330	100	0	0	1760	2157	397	18.41%
10/3/90	2248	297	100	0	0	2645	2949	304	10.31%
10/4/90	2757	297	100	0	0	3154	3466	312	9.00%
10/5/90	1649	330	100	0	0	2079	2458	379	15.42%
10/6/90	1679	297	100	0	0	2076	2456	380	15.47%
10/7/90	1888	330	100	0	0	2318	2619	301	11.49%
10/12/90	3420	297	100	0	0	3817	4276	459	10.73%
10/13/90	3147	297	100	0	0	3544	4043	499	12.34%
10/14/90	2826	495	100	0	0	3421	3867	446	11.53%
10/15/90	2812	297	100	0	0	3209	3599	390	10.84%
10/16/90	1927	396	100	0	0	2423	2827	404	14.29%
10/17/90	2532	396	100	0	0	3028	3449	421	12.21%
10/18/90	2864	396	100	0	0	3360	3757	397	10.57%
10/19/90	2133	396	100	0	0	2629	3028	399	13.18%
10/20/90	2249	396	100	0	0	2745	3253	508	15.62%
10/21/90	1569	462	100	0	0	2131	2457	326	13.27%
10/22/90	1797	297	100	0	0	2194	2546	352	13.83%
10/23/90	1708	231	100	0	0	2039	2441	402	16.47%
10/24/90	1925	396	100	0	0	2421	2713	292	10.76%
10/25/90	1660	297	100	0	0	2057	2422	365	15.07%
10/26/90	2861	363	100	0	0	3324	3678	354	9.62%
10/27/90	2519	330	100	0	0	2949	3326	377	11.33%
10/28/90	3151	429	100	0	0	3680	4013	333	8.30%
10/29/90	3775	264	100	0	0	4139	4479	340	7.59%
10/30/90	4216	363	100	0	0	4679	5319	640	12.03%
10/31/90	4050	297	100	0	0	4447	5051	604	11.96%
11/1/90	1669	330	100	0	0	2099	2500	401	16.04%
11/2/90	1399	396	100	0	0	1895	2287	392	17.14%
11/3/90	1484	396	100	0	0	1980	2333	353	15.13%
11/8/90	3371	99	100	0	0	3570	4174	604	14.47%
11/9/90	2708	264	100	0	0	3072	3589	517	14.41%
11/10/90	2384	297	100	0	0	2781	3249	468	14.40%
11/11/90	1862	363	100	0	0	2325	2728	403	14.77%
11/12/90	1728	363	100	0	0	2191	2584	393	15.21%
11/13/90	1631	264	100	0	0	1995	2383	388	16.28%
11/14/90	1425	330	100	0	0	1855	2320	465	20.04%
11/15/90	1847	363	100	0	0	2310	2685	375	13.97%
11/16/90	1578	297	100	0	0	1975	2355	380	16.14%
11/17/90	1518	363	100	0	0	1981	2409	428	17.77%
11/18/90	1403	297	100	0	0	1800	2277	477	20.95%
11/19/90	1535	264	100	0	0	1899	2305	406	17.61%
11/20/90	1322	297	100	0	0	1719	2205	486	22.04%
11/21/90	2812	264	100	0	0	3176	3538	362	10.23%
11/22/90	1965	297	100	0	0	2362	2813	451	16.03%
11/23/90	1877	165	100	0	0	2142	2542	400	15.74%
11/24/90	1511	396	100	0	0	2007	2434	427	17.54%

TABLE 6.2 (continued)

Mean daily discharges for Romeoville AVM and Lockport. October - December, 1990

	Mean daily discharge, in cubic feet per second								
Date	Turbine	Leakage	Lenkage	Sluice gate	Control works	Lockport total	Romeoville AVM	Absolute Difference	Percent Difference
11/25/90	1199	330	100	0	0	1629	2082	453	21.76%
11/26/90	1737	264	100	0	0	2101	2518	417	16.56%
12/1/90	3953	297	100	0	0	4350	5160	810	15.70%
12/2/90	3802	429	100	0	0	4331	4992	661	13.24%
12/4/90	3680	330	100	0	0	4110	4701	591	12.57%
12/5/90	3096	330	100	0	0	3526	4288	762	17.77%
12/6/90	2724	396	100	0	0	3220	3775	555	14.70%
12/7/90	2200	363	100	0	0	2663	3007	344	11.44%
12/8/90	2293	198	100	0	0	2591	2983	392	13.14%
12/9/90	2195	264	100	0	0	2559	2878	319	11.08%
12/10/90	1890	264	100	0	0	2254	2552	298	11.68%
12/11/90	2110	264	100	0	0	2474	2791	317	11.36%
12/12/90	2044	396	100	0	0	2540	2825	285	10.09%
12/13/90	1699	396	100	0	0	2195	2472	277	11.21%
12/14/90	1701	363	100	0	0	2164	2461	297	12.07%
12/15/90	2150	330	100	0	0	2580	2876	296	10.29%
12/16/90	1900	363	100	0	0	2363	2701	338	12.51%
12/17/90	2051	231	100	0	0	2382	2750	368	13.38%
12/18/90	1964	330	100	0	0	2394	2732	338	12.37%
12/19/90	2153	231	100	0	0	2484	2815	331	11.76%
12/20/90	2087	165	100	0	0	2352	2618	266	10.16%
12/21/90	2698	231	100	0	0	3029	3354	325	9.69%
12/22/90	2110	264	100	0	0	2474	2798	324	11.58%
12/23/90	2208	132	100	0	0	2440	2718	278	10.23%
12/24/90	1965	363	100	0	0	2428	2580	152	5.89%
12/25/90	1448	264	100	0	0	1812	2157	345	15.99%
12/26/90	1635	330	100	0	0	2065	2344	279	11.90%
12/27/90	1733	264	100	0	0	2097	2453	356	14.51%
12/28/90	1826	264	100	0	0	2190	2569	379	14.75%
12/31/90	3662	330	100	0	0	4092	4461	369	8.27%

TABLE 6.3

Mean daily discharges for Romeoville AVM and Lockport. July- September, 1991

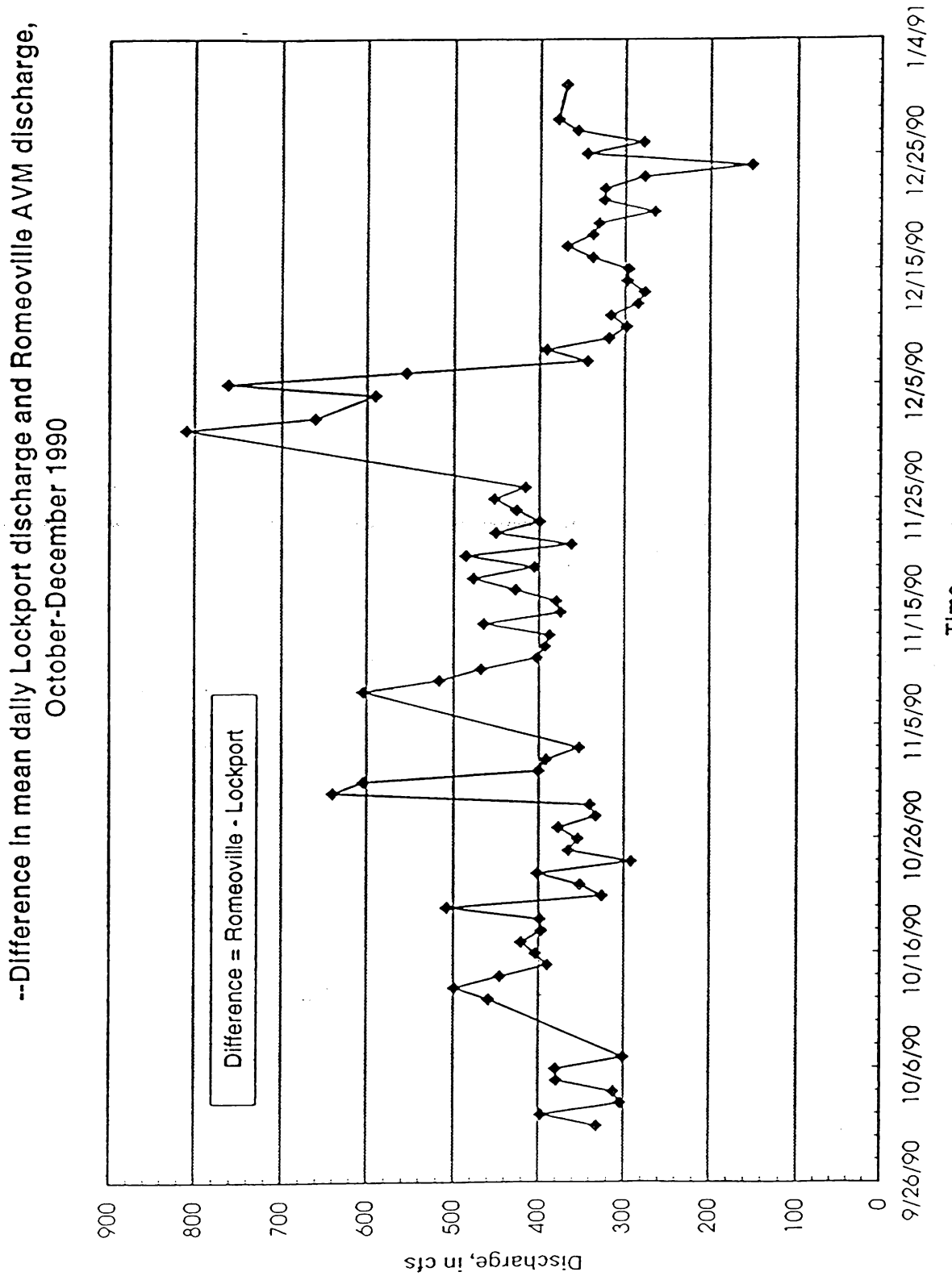
	Mean daily discharge, in cubic feet per second								
Date	Turbine	Lockage	Leakage	Sluice gate	Control works	Lockport total	Romeoville AVM	Absolute Difference	Percent Difference
7/1/91	2949	297	100	0	0	3346	3760	414	11.01%
7/2/91	3310	330	100	0	0	3740	4055	315	7.77%
7/3/91	3488	264	100	0	0	3852	4219	367	8.70%
7/4/91	3002	429	100	0	0	3531	3960	429	10.83%
7/5/91	3031	429	100	0	0	3560	3965	405	10.21%
7/6/91	3166	363	100	0	0	3629	4001	372	9.30%
7/8/91	3011	165	100	0	0	3276	3650	374	10.25%
7/9/91	3490	297	100	0	0	3887	4303	416	9.67%
7/10/91	2940	363	100	0	0	3403	3718	315	8.47%
7/11/91	3236	297	100	0	0	3633	3948	315	7.98%
7/12/91	2968	363	100	0	0	3431	3787	356	9.40%
7/13/91	3243	429	100	0	0	3772	4115	343	8.34%
7/14/91	2935	495	100	0	0	3530	3877	347	8.95%
7/15/91	2896	297	100	0	0	3293	3760	467	12.42%
7/16/91	3295	231	100	0	0	3626	4002	376	9.40%
7/17/91	3023	297	100	0	0	3420	3783	363	9.60%
7/18/91	3481	330	100	0	0	3911	4244	333	7.85%
7/19/91	3261	297	100	0	0	3658	4021	363	9.03%
7/20/91	3644	330	100	0	0	4074	4527	453	10.01%
7/21/91	2999	429	100	0	0	3528	3878	350	9.03%
7/22/91	3225	297	100	0	0	3622	3995	373	9.34%
7/23/91	3225	297	100	0	0	3622	3887	265	6.82%
7/24/91	3291	297	100	0	0	3688	4066	378	9.30%
7/25/91	3185	330	100	0	0	3615	3931	316	8.04%
7/26/91	3282	330	100	0	0	3712	4032	320	7.94%
7/27/91	3169	330	100	0	0	3599	3924	325	8.28%
7/28/91	3316	495	100	0	0	3911	4212	301	7.15%
7/29/91	2767	363	100	0	0	3230	3580	350	9.78%
7/30/91	3079	330	100	0	0	3509	3956	447	11.30%
7/31/91	2969	363	100	0	0	3432	3728	296	7.94%
8/1/91	3341	231	100	0	0	3672	3985	313	7.85%
8/2/91	3112	396	100	0	0	3608	3895	287	7.37%
8/3/91	3393	297	100	0	0	3790	4175	385	9.22%
8/4/91	3393	297	100	0	0	3790	4192	402	9.59%
8/10/91	3599	495	100	0	0	4194	4614	420	9.10%
8/11/91	3360	396	100	0	0	3856	4162	306	7.35%
8/12/91	3281	396	100	0	0	3777	4150	373	8.99%
8/13/91	3001	330	100	0	0	3431	3834	403	10.51%
8/14/91	3081	396	100	0	0	3577	3941	364	9.24%
8/15/91	3201	297	100	0	0	3598	3945	347	8.80%
8/16/91	3382	429	100	0	0	3911	4178	267	6.39%
8/17/91	2877	330	100	0	0	3307	3646	339	9.30%
8/18/91	3155	330	100	0	0	3585	3952	367	9.29%
8/19/91	3768	363	100	0	0	4231	4736	505	10.66%
8/20/91	4188	330	100	0	0	4618	5090	472	9.27%
8/21/91	3125	231	100	0	0	3456	3883	427	11.00%
8/22/91	3284	132	100	0	0	3516	3883	367	9.45%

TABLE 6.3 (continued)

Mean daily discharges for Romeoville AVM and Lockport, July- September, 1991

Date	Mean daily discharge, in cubic feet per second								
	Turbine	Lockage	Leakage	Sluice gate	Control works	Lockport total	Romeoville AVM	Absolute Difference	Percent Difference
8/24/91	3225	363	100	0	0	3688	3956	268	6.77%
8/25/91	3358	297	100	0	0	3755	4091	336	8.21%
8/26/91	3048	297	100	0	0	3445	3828	383	10.01%
8/27/91	2954	231	100	0	0	3285	3595	310	8.62%
8/28/91	3312	264	100	0	0	3676	4042	366	9.05%
8/29/91	3097	330	100	0	0	3527	3920	393	10.03%
8/30/91	3205	462	100	0	0	3767	4212	445	10.57%
8/31/91	3120	495	100	0	0	3715	4145	430	10.37%
9/1/91	3456	429	100	0	0	3985	4429	444	10.02%
9/2/91	2462	363	100	0	0	2925	3680	755	20.52%
9/4/91	3079	297	100	0	0	3476	3876	400	10.32%
9/5/91	3328	330	100	0	0	3758	4148	390	9.40%
9/6/91	3131	330	100	0	0	3561	3929	368	9.37%
9/7/91	2969	462	100	0	0	3531	3985	454	11.39%
9/8/91	3119	495	100	0	0	3714	4051	337	8.32%
9/10/91	3025	330	100	0	0	3455	3940	485	12.31%
9/11/91	3346	363	100	0	0	3809	4210	401	9.52%
9/14/91	3504	396	100	0	0	4000	4510	510	11.31%
9/15/91	3882	363	100	0	0	4345	4875	530	10.87%
9/16/91	2956	264	100	0	0	3320	3764	444	11.80%
9/20/91	1619	330	100	0	0	2049	2428	379	15.61%
9/24/91	1365	396	100	0	0	1861	2216	355	16.02%
9/25/91	1310	396	100	0	0	1806	2146	340	15.84%
9/26/91	1552	429	100	0	0	2081	2283	202	8.85%
9/27/91	1398	297	100	0	0	1795	2009	214	10.65%
9/28/91	1644	297	100	0	0	2041	2281	240	10.52%
9/29/91	1583	396	100	0	0	2079	2328	249	10.70%
9/30/91	1504	231	100	0	0	1835	2087	252	12.07%

FIGURE 6.3



oct-90

FIGURE 6.4

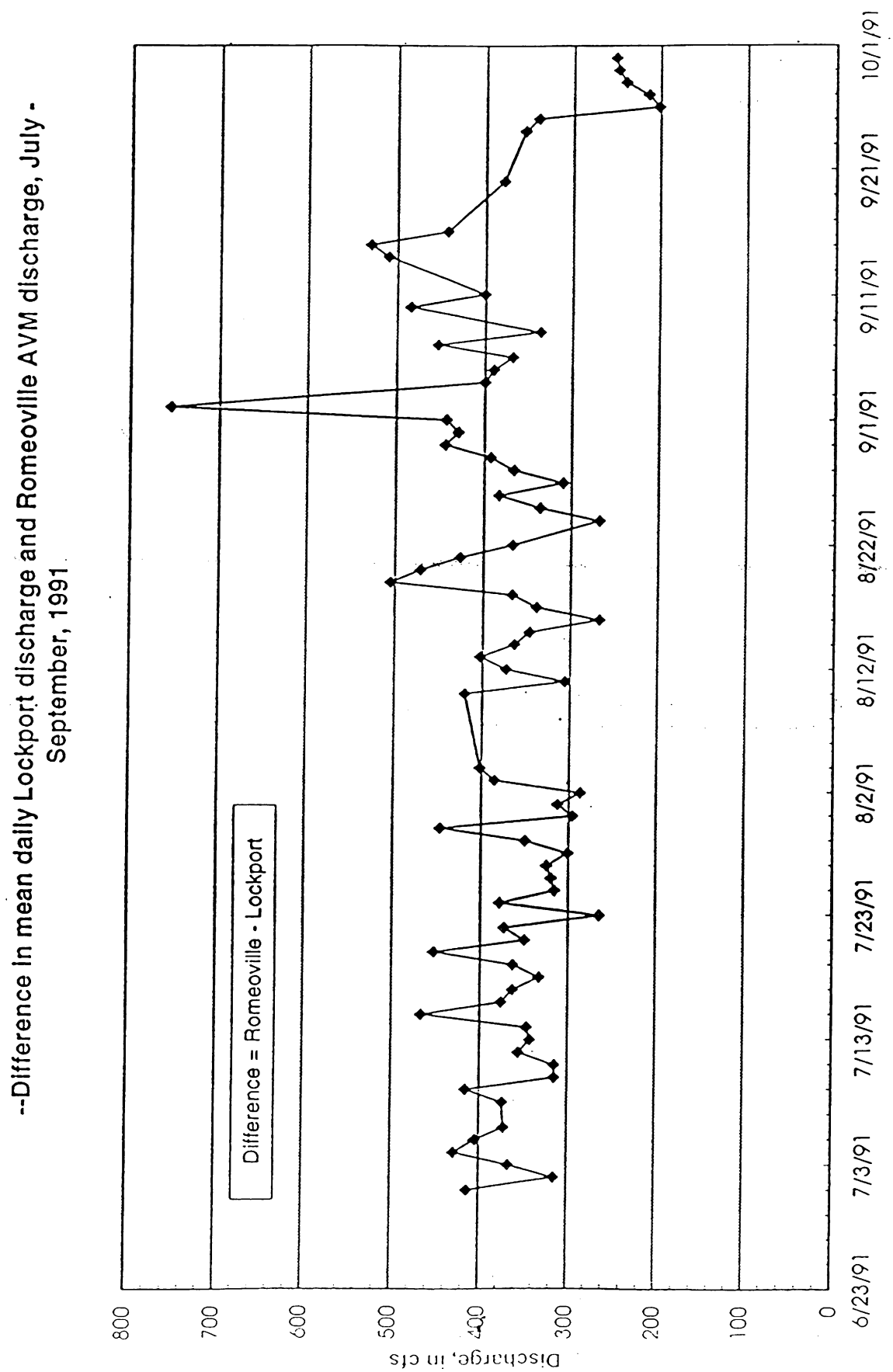

--Difference in mean daily Lockport discharge and Romeoville AVM discharge, July - September, 1991
Difference = Romeoville - Lockport
Discharge, in cfs
800
700
600
500
400
300
200
100
0
6/23/91
7/3/91
7/13/91
7/23/91
8/2/91
8/12/91
8/22/91
9/1/91
9/11/91
9/21/91
10/1/91
Time

TABLE 6.4

Selected Mean Daily Lockport and Romeoville AVM Discharges
August and October 1992

Year	Mean Daily Discharge, in cfs			Difference in cubic feet	Lockages		Lockport Adj for Lockage	Percent Difference
	Lockport	Romeoville	Difference		Compute	Reported		
24-Aug-92	3,010	3465	455	39,312,000	15.7	133	3416	-1.40%
25-Aug-92	3,273	3606	333	28,771,200	11.5	14	3650	1.23
26-Aug-92	3,949	4022	73	6,307,200	2.5	10	4239	5.40
04-Oct-92	3,118	3439	321	27,734,400	11.1	18	3641	5.86
05-Oct-92	2,628	2988	360	31,104,000	12.4	16	3092	3.50
06-Oct-92	2,595	2943	348	30,067,200	12.0	14	3001	1.98

Lock Width	110 feet
Lock Length	600 feet
Normal Head	38 feet
Volume per Lockage	2,508,000 cubic feet
Total lockage time	20 minutes

7.0 RECOMMENDATIONS

This Committee is gratified by the improvement achieved in the accounting procedure, particularly in the quality of the AVM records. The U.S. Army Corps of Engineers, U.S. Geological Survey, and the State of Illinois should be commended for assigning to this project highly qualified personnel and supplying state-of-the-art instrumentation and technical support. The primary reason for the diversion exceeding the flow limits of the Supreme Court decree is the improved accuracy of the accounting procedures. A major part of this improved accuracy can be attributed to the AVM system at Romeoville. This Committee is in general agreement with the findings and recommendations made by the Second Technical Committee. In most instances, actions have been taken to comply with the recommendations and significant progress has been made.

Some of the recommendations made by the Second Technical Committee are still current and may be repeated here to emphasize their importance.

7.1 FIRST AND SECOND COMMITTEE RECOMMENDATIONS

The Committee is in agreement with the findings and recommendations of the Second Committee (Espey, et al, 1987).

7.2 MASTER PLAN

7.2.1 The draft of the Master Plan for Lake Michigan Diversion Accounting Program should be finalized.

7.2.2 The Master Plan should include an "Operational Procedures Manual" documenting technical procedures and methods used in the Lake Michigan diversion computations.

7.3 QUALITY ASSURANCE PLAN

The draft Quality Assurance Plan (draft - October 1988) should be updated and finalized based on the present status (1994) of Lake Michigan diversion computational procedures and measurements. Basic elements of the plan are as follows:

- Develop documentation for measurements;
- Develop documentation for methods of data collection;
- Develop "Standard Operation Procedure" for calibration and verification of measurement components;
- Measurement site field evaluation schedule;
- Establish methods of verifying the accuracy of data used in diversion computation; and
- Maintain permanent records of measurements, verification tests, and other sources of diversion data.

7.4 AVM SYSTEM RECOMMENDATIONS

7.4.1 Update the AVM Quality Assurance Plan to include:

7.4.1.1 Uniform guidelines for the operation of AVM installations and development of discharge ratings.

7.4.1.2 Policy and technical guidance on Broadband Acoustic Doppler Current Profilers, including discharge-measurement procedures.

7.4.2 A technical review of the Romeoville AVM discharge ratings and flow records should be conducted annually.

7.4.3 The AVM discharge ratings and flow records at Lockport should also be reviewed annually using the technical guidelines described in the Quality Assurance Plan.

7.4.4 The mean bed elevation of the canal at the AVM measuring reach should be surveyed periodically to detect any changes in the mean bottom elevation and to determine if a correction or shift to the area rating is needed.

7.4.5 An examination of the range of discharge measurements indicates that about 80 percent of the measurements were made at gage heights between 24.7 and 25.7 feet. If at all possible, it would be very useful in the development of discharge ratings to obtain more discharge measurements at the 21 to 24 foot range.

7.4.6 The ACDP (Broadband) system should be used to calibrate and verify the AVM Romeoville system operations. The ACDP can be a valuable tool for measurement during low flow and/or unsteady flow conditions.

7.5 LAKEFRONT MEASUREMENTS

Investigate the feasibility of developing ratings between the leakage flow through the gates at the lakefront and the water surface elevation of the lake.

7.6 LOCKPORT/ROMEOVILLE REGRESSION EQUATIONS

7.6.1 Use the AVM turbine (August 1992) and Romeoville AVM data to revise the turbine rating.

7.6.2 Based on results of recommendation 7.6.1, review and revise, if appropriate, USGS regression equations.

7.6.3 Based on the 7.6.2, recalculate the Lake Michigan diversion for the 1981, 1982, 1983, and 1984 Water Years.

7.7 ANNUAL DIVERSION REPORT

Annual Lake Michigan diversion results should be published in a more timely fashion.

7.8 DES PLAINES PUMPING STATION

Field investigation of flow characteristics of the Des Plaines pumping station, including bypass flow, be conducted to improve the accuracy of inflow and infiltration characteristics used in the hydrologic simulation.

7.9 GRAND CALUMET RIVER

Based on the measured flows of the current Grand Calumet River gage, if the regression equation that estimates Grand Calumet River flow results in a significant error in the deductible water supply from Indiana (contained in column 5), the impact on the historical diversion record should be reviewed.

REFERENCES

REFERENCES

Burke, Christopher B. (Engineering Ltd.), 1990, Infiltration and Inflow Study and Diversion Accounting Model Modifications, Contract Report for U.S. Army Corps of Engineers, Chicago District, CBBEL Project Number 88-70.

Cunge, J. A., Holly, F. M., and Verway, A., 1980, Practical Aspects of Computation River Hydraulics, Pitman Advanced Publishing Program, Boston, MA

Espey, W. H., Barnes, H. H. Jr., and Vigander, S., 1981, Lake Michigan Diversion, Findings, of the Technical Committee for Review of Diversion Flow Measurements and Accounting Procedures: U.S. Army Corps of Engineers, Chicago District.

Espey, W. H., Barnes, H. H. Jr., and Westfall, D.E., 1987, Lake Michigan Diversion, Findings, of the Technical Committee for Review of Diversion Flow Measurements and Accounting Procedures: U.S. Army Corps of Engineers, Chicago District.

Gordon, R. L., 1989, Acoustic Measurement of River Discharge: Journal of Hydraulic Engineering, American Society of Civil Engineers, v. 115, p. 925-936.

Gordon, R. L., Bornhoft, J., 1991, BroadBand ADCP Discharge Demonstration Tests in the Rivers Rhine, Moselle, Waterweg, and Schelde. RD Instruments Report.

Hitt, K. L., Miller, J. B., 1985, Great Lakes Set Record High Water Levels, National Water Summary 1985, Hydrologic Events and Surface-Water Resources, U.S. Geological Survey, Water Resources Paper 2300, pp 35-40.

Laenen, A., 1992 Defining the Relationship Between Mean Velocity and the Line Velocity of an Acoustic Meter, U.S. Geological Survey, preliminary draft report.

Melching, C. H., Oberg, K. A., 1993, Comparison, Analysis, and Estimation of Discharge Data from Two Acoustic Velocity Meters on the Chicago Sanitary and Ship Canal at Romeoville, Illinois: U.S. Geological Survey Water Resources Investigations Report 93-4048.

Rantz, S. E., et al, 1982, Measurement and Computation of Streamflow: Volume 1. Measurement of Stage and Discharge: U.S. Geological Survey WSP 2175.

Simpson, R. M., Oltman, R. N., 1990 An Acoustic Doppler Discharge System, Hydraulic Engineering Proceedings, 1990 National Conference, HH/Division, ASCE, pp 903-908.

APPENDIX 1

COLUMN 4

GROUNDWATER PUMPAGE DISCHARGED INTO THE CANAL

+ LMW GW REACHING CANAL (NOT INCLUDING INDIANA)

+ DPW GW REACHING CANAL (NOT INCLUDING O'HARE TRANSFER)

+ O'HARE FLOW TRANSFER (GW PORTION OF SANITARY COMPONENT)

    - SANITARY % BASED ON UDPPS
    - GW % OF WATER SUPPLY COMPUTED ON INDIVIDUAL COMMUNITY BASIS

+ MAINSTREAM TARP GW INFILTRATION

+ CALUMET TARP GW INFILTRATION

- SUBTRACT 10 % OF SANITARY PORTION OF THE DPW CSO's NOT REACHING CANAL

    - 10% GW ASSUMED (MAY REVISE TO MATCH O'HARE GW %)
    - COMPUTED BY SUMMING SCA's THAT OVERFLOW TO WATERWAYS NOT TRIBUTARY TO CANAL.

COLUMN 5

WATER SUPPLY PUMPAGE FROM INDIANA REACHING CANAL

COMPONENTS

1 - WATER SUPPLY FROM - DYER
- SCHERERVILLE
- ST. JOHN

DISCHARGE TO HART DITCH (LITTLE CALUMET)

2 - GRAND CALUMET WATER SUPPLY DEDUCTION

- LESSER VALUE OF:

A. GRAND CALUMET FLOW FROM REGRESSION EQNS.
B. SUM OF WATER SUPPLY PUMPAGE FOR
   - EAST CHICAGO
   - WHITING
   - HAMMOND (INCLUDES PUMPAGE TO MUNSTER, HIGHLAND, AND GRIFFITH)

GRAND CALUMET FLOW FROM REGRESSION EQUATIONS

IF LM STAGE $\leq$ 1.0 CCD GC FLOW = 23 CFS

IF LM STAGE > 1.0 CCD

$$GC\ FLOW = 29.9\ (LMS)^{5/3} + 0.13\ (HDR)^{2/3} - 9.6$$

WHERE LMS = LAKE MICHIGAN STAGE @ CALUMET HARBOR
HDR = HART DITCH RUNOFF AS INDICATOR OF GC RUNOFF

COLUMN 6

RUNOFF FROM DES PLAINES WATERSHED REACHING THE CANAL

COMPONENTS		MODEL
NORTHSIDE WRP	- DPW INFLOW - DPW INFILTRATION	SCALP
WSW WRP	- DPW INFLOW - DPW INFILTRATION	SCALP
CALUMET WRP	- DPW INFLOW - DPW INFILTRATION	SCALP
LEMONT WRP	- INFLOW - INFILTRATION	SCALP
DPW CSO's TO CANAL	- INFLOW - INFILTRATION	SCALP & TNET
O'HARE TRANSFER	- INFLOW - INFILTRATION	SCALP FOR UDPPS
WSW TARP PUMPAGE	- DPW INFLOW - DPW INFILTRATION	TNET
CALUMET TARP PUMPAGE	- DPW INFLOW - DPW INFILTRATION	TNET
SUMMIT CONDUIT RUNOFF (I/I)		HSPF
LDP UNGAGED WATERSHED RUNOFF		SIMULATION

COLUMN 7

LAKE MICHIGAN PUMPAGE BY FEDERAL FACILITIES DISCHARGING TO CANAL

COMPONENTS:

- HINES HOSPITAL
- FORT SHERIDAN
- GLENVIEW NAVAL AIR STATION
- U.S. ARMY CORPS EMERGENCY NAVIGATION MAKEUP

COLUMN 9

LAKE MICHIGAN PUMPAGE NOT DISCHARGED TO CANAL

COMPONENTS:

+ LINCOLNSHIRE
+ RIVERWOODS
+ LIBERTYVILLE
+ WAUKEGAN --- GURNEE, PARK CITY

+ N.W. SUBURBAN JOINT ACTION WATER AGENCY
  - ELK GROVE VILLAGE, HOFFMAN ESTATES, MOUNT PROSPECT, SCHAUMBURG, HANOVER PARK, ROLLING MEADOWS, AND STREAMWOOD

+ N.W. WATER COMMISSION
  - ARLINGTON HEIGHTS, BUFFALO GROVE, PALATINE, WHEELING.

+ LAKE CO. PUBLIC WATER DISTRICT
  - KNOLLWOOD - ROUNDOUT, VERNON HILLS, WILDWOOD - GAGES LAKE

+ LAKE CO. PUBLIC WORKS - BRADLEY ROAD
+ 38.2% DES PLAINES (38.2% TO O'HARE STP)
+ 76% NORTH CHICAGO (76% TO GURNEE STP) MINUS LAKE CO. PUBLIC WKS (FROM NORTH CHICAGO)
+ 90% DPW SANITARY PORTION CSO NOT TO CANAL (90% LM WATER 10% GW ASSUMED)
  - MAY REVISE TO MATCH O'HARE LM & GW %'s

- (SUBTRACT) 95.4% SANITARY PORTION O'HARE FLOW TRANSFER (95.4% LM WATER , 4.6% GW)

- MOST WATER SUPPLY DATA SUPPLIED BY IDOT DIVISION OF WATER RESOURCES ON LMO-3 FORMS

- COMPUTATIONS BASED ON LM WATER SUPPLY DISTRIBUTION NETWORK DIAGRAM FROM IDOT. (DYNAMIC IN NATURE)

COLUMN 11

LAKE MICHIGAN PUMPAGE ACCOUNTABLE TO ILLINOIS

COMPONENTS:

+ CHICAGO
+ EVANSTON
+ GLENCOE
+ HIGHLAND PARK
+ HIGHWOOD
+ KENILWORTH
+ LAKE FOREST
+ LANSING (PURCHASED FROM HAMMOND, IN)
+ CHICAGO HEIGHTS (PURCHASED FROM HAMMOND, IN)
+ NORTH CHICAGO
+ NORTHBROOK
+ WAUKEGAN
+ WILMETTE
+ WINNETKA
+ ACME STEEL
+ LAKE CO. PUBLIC WATER DISTRICT
+ LTV STEEL
+ REPUBLIC STEEL
+ U.S. STEEL SOUTH WORKS

(SUBTRACT) - GLENVIEW NAVAL AIR STATION (SUPPLIED BY WILMETTE)
(SUBTRACT) - HINES HOSPITAL (SUPPLIED BY CHICAGO)

WATER SUPPLY DATA FROM LMO-3 FORMS SUPPLIED BY IDOT. PUMPAGE RECORDS ON LMO-3 FORMS INCLUDE PUMPAGE TO SECONDARY USERS

COLUMN 12

RUNOFF FROM THE DIVERTED LAKE MICHIGAN WATERSHED

COMPONENTS:

- NORTH BRANCH CHICAGO RIVER @ NILES RUNOFF (FROM BUDGET 3)
- LITTLE CALUMET RIVER @ STATE LINE RUNOFF (FROM BUDGET 4)
- THORN CREEK @ THORNTON RUNOFF (FROM BUDGET 5)
- LITTLE CALUMET RIVER @ S. HOLLAND RUNOFF (FROM BUDGET 6)

• NORTH SIDE WRP	- LMW INFLOW	
	- LMW INFILTRATION	
• WSW WRP	- LMW INFLOW	SCALP
	- LMW INFILTRATION	STP
• CALUMET WRP	- LMW INFLOW	
	- LMW INFILTRATION	
• WSW TARP PUMPAGE	- LMW INFLOW	
	- LMW INFILTRATION	TNET
• CALUMET TARP PUMPAGE	- LMW INFLOW	
	- LMW INFILTRATION	
• INFLOW PORTION CSO's FROM UNGAGED SCA's		SCALP &
• INFILTRATION PORTION CSO's FROM UNGAGED SCA's		TNET

- UNGAGED CALUMET RUNOFF
  - DETERMINED BY MULTIPLYING UNIT RUNOFF FILES IMPRO, OLFRO, SUBRO (HSPF) BY THE APPROPRIATE DRAINAGE AREAS FOR IMPERVIOUS, GRASSLAND , AND FOREST LAND

- GRAND CALUMET RIVER RUNOFF
  - CRUDE ESTIMATION
  - SUBTRACT WATER SUPPLIES OF EAST CHICAGO, WHITING, AND HAMMOND FROM ESTIMATED GRAND CALUMET RIVER FLOW (REGRESSION EQUATIONS)

COLUMN 13

DIRECT DIVERSIONS
ACCOUNTABLE TO ILLINOIS

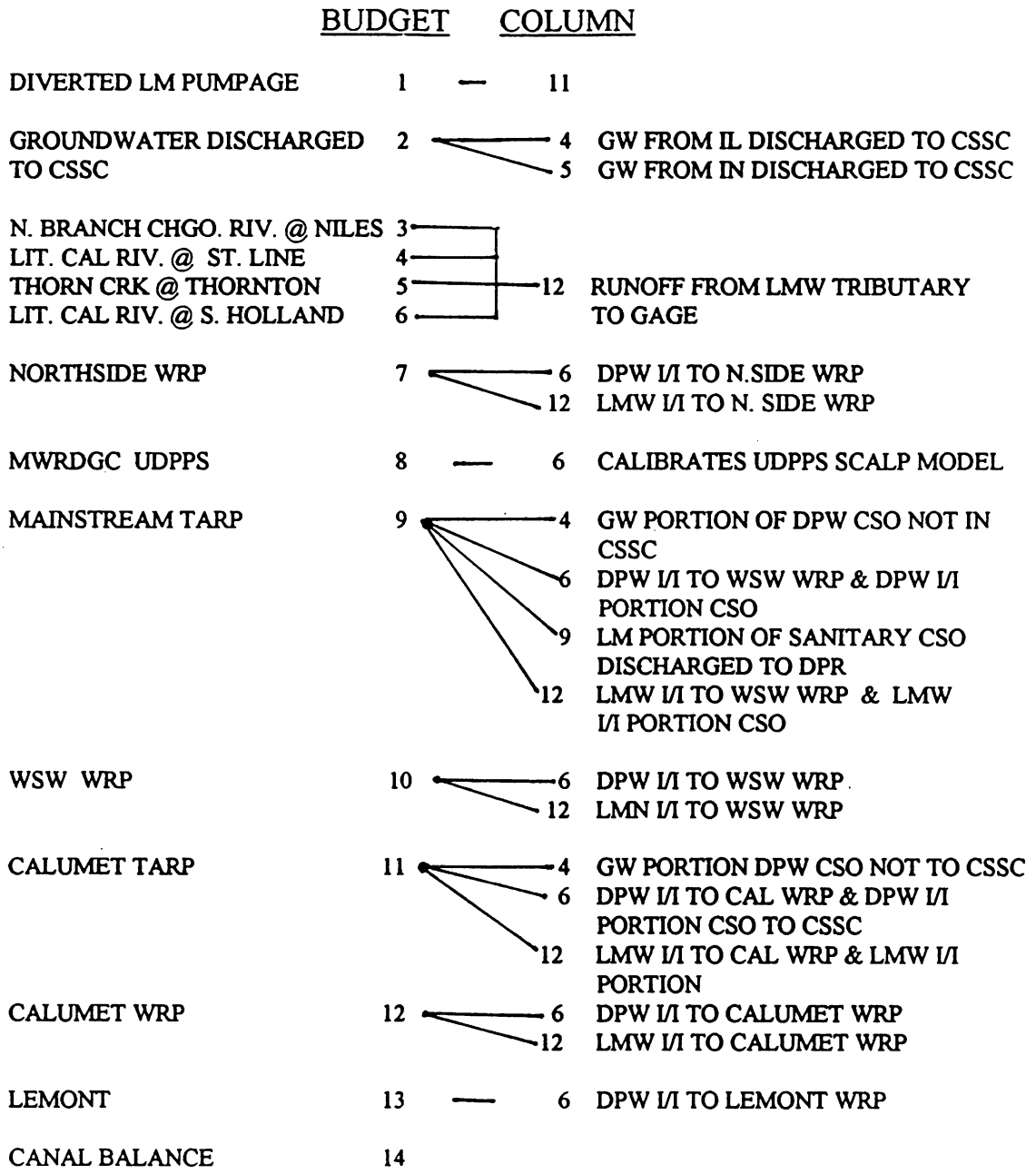
COMPONENTS:

+ CHICAGO RIVER CONTROLLING WORKS
+ O'BRIEN LOCK
+ WILMETTE CONTROLLING WORKS

LOCKAGE,
LEAKAGE,
NAVIGATION MAKEUP,
& DISCRETIONARY

- CRCW BACKFLOWS
- O'BRIEN BACKFLOWS
- WILMETTE BACKFLOWS

BUDGET & COLUMN INTERACTIONS



Lake Michigan Diversion Accounting Data Sources and Problems

Precipitation Network

- Original 13 gages, (4 NWS, 4 City of Chicago, and 5 MWRDGC) gages, through Water Year 1989
- MWRDGC gages are digitized from circular charts, Chicago City gages from tabulations, and four NWS gages from EarthInfo CD-ROM
- Data through Water Year 1989 adjusted by Illinois State Water Survey
- ISWS 25 gage network began WY 1990
- ISWS delivered data on disk as hourly data

Problems with precipitation data

- For 13 gage network include rain shadows at some locations, differing equipment and differing techniques

- MWRDGC circular charts could have digitizing errors

- For both networks include missing data and damaged gages

Meteorological Data

- Includes wind run, dew point, maximum and minimum temperatures, and cloud cover
- Temperatures from four NWS gages, and other meteorological data from O'Hare
- Solar radiation and evapotranspiration simulated by HSPFPREP
- Read from climatilogic reports as daily values

Water Supply Pumpage from Lake Michigan

- Lake Michigan withdrawals reported by Illinois Dept. of Transportation and Indiana Dept of Natural Resources

- Monthly Illinois report of daily pumping from lake (primary use) and monthly pumping to secondary users in form LMO-3, and annual distribution network

- Flows are based on pump ratings

Water Supply Pumpage from Groundwater

- Groundwater pumpage reported by Indiana Dept of Environmental Management, and Illinois State Water Survey and USGS Illinois office
- Annual and monthly totals are converted to average daily flows
- Flows are based on pump ratings and are averaged rather than daily flows

Waste Treatment Plant Pumpage

- Four plants used to check runoff and tunnel flows to treatment plant (Calumet, O'Hare, Northside, and Stickney)
- Eight plants used to check simulations of runoff at stream gages (IL - Nile, Deerfield, Plum Creek, University Park, Willowbrook, and Homewood; IN - Dyer and Schererville)
- Delivered on paper as pump time charts, or daily or monthly tabulations
- Based on pump ratings and digitized or averaged flows

Pump Station Data

- Sewer pumps at Calumet, Mainstream, O'Hare, and Upper Des Plaines
- Calumet, Mainstream, and O'Hare pump water from respective TARP tunnels
- Upper Des Plaines pumps water from upper Des Plaines River watershed
- Data is from weekly circular pump records with some missing data, multiple weeks on same chart
- Flow based on pump ratings and digitizing of charts
- Upper Des Plaines does not measure overflows

Streamflow Data from USGS

- Flows at:
  - Grand Calumet River at Hammond
  - Hart Ditch at Munster
  - Little Calumet River at Munster
  - Little Calumet River at South Holland
  - North Branch of Chicago River at Niles
  - Romeoville Acoustic Velocity Meter (AVM)
  - Thorn Creek at Thornton

- Flows are received direct from USGS on disk or read from Earthinfo CD-ROM

Possible Errors in USGS Data

- Possible errors in ratings, gage breakdowns, and AVM data replacement based on Lockport flows

- Hammond flow before WY91 simulated from Hart Ditch at Munster and Lake Michigan elevations

Streamflow Data from Metropolitan Water Reclamation District of Greater Chicago

- Canal flows (lockage, leakage, navigation makeup, direct diversion, backflows) at:
  O'Brien Lock and Dam
  Chicago River Controlling Works
  Lockport (turbine, sluice, and Controlling Works flows)
  Wilmette Controlling Works
- Flows are recieved on paper as daily values
- Leakage assumed to be constant and is not measured
- Flow calculations for navigation makeup, direct diversions, backflows, sluice and Lockport Controlling Works are of dubious accuracy
- Turbine flows now measured with AVMs since March 1992, of dubious accuracy before then
- Lockage flows derived by Corps of Engineers, Chicago District based on lock usage

Data from Industry

- Withdrawals from canal, Lake Michigan (by industry or other entity), groundwater, or storm runoff
- Discharges to canal or to sewers
- Data from:
  - Acme Steel at Chicago and Riverside
  - Argonne Laboratories
  - LTV Steel
  - Material Service
  - Republic Steel
  - Rhone-Doulenc Basic Chemical
  - Texaco Oil
  - Uno-ven Corporation
  - Underwriter's Laboratory
- Variations in data collection techniques and in data quality

APPENDIX 2

Description of the Diversion Accounting Computational Budgets

Budget No.	Title	Description
1	Diverted Lake Michigan Pumpage	This budget sums Lake Michigan water diverted by the State of Illinois in the form of municipal and industrial water supply. The results of this budget are used in Column 11.
2	Groundwater Discharged to the CSSC	This budget sums groundwater pumpages that are discharged to the CSSC. The results of this budget are used in Column 4.
3	North Branch Chicago River at Niles, IL	This budget performs a simple separation of stream flow into sanitary and runoff portions. The results this budget are used in Budget 13 and Column 12.
4	Little Calumet River at the IL-IN State Line	This budget performs a simple separation of stream flow into sanitary and runoff portions. The results this budget are used in Budget 13 and Column 12.
5	Thorn Creek at Thornton, IL	This budget performs a simple separation of stream flow into sanitary and runoff portions. The results this budget are used in Budget 13 and Column 12.
6	Little Calumet River at South Holland, IL	This budget performs a simple separation of stream flow into sanitary and runoff portions. The results this budget are used in Budget 13 and Column 12.
7	MWRDGC Northside Water Reclamation	This budget performs hydrologic and hydraulic simulation of the service basin tributary to the MWRDGC Northside Water Reclamation Facility. The simulations estimates the runoff from portions of the Lake Michigan and Des Plaines River watersheds within the Northside service basin that is diverted to the CSSC in the form of inflow-infiltration. The budget provides an internal verification of the accounting procedures. The results of this budget are used on Budget 13 and Columns 6 and 12.

Table 2 (cont)

Description of the Diversion Accounting Computational Budgets

Budget No.	Title	Description
8	MWRDGC Upper Des Plaines Pumping Station	This budget performs hydrologic and hydraulic simulation of the MWRDGC Upper Des Plaines Pumping Station. This budget provides a calibration point to verify models of the Des Plaines River watershed
9	MWRDGC Mainstream TARP Pumping Station	This budget performs hydrologic and hydraulic simulation of the MWRDGC Mainstream TARP Pumping Station. The results of this simulation are used in Budgets 10 and 13 and Columns 6 and 12. The budget also provides internal verification of the accounting procedures.
10	MWRDGC Stickney Water Reclamation Facility	This budget performs hydrologic and hydraulic simulation of the service basin tributary to the MWRDGC Stickney Water Reclamation Facility. The simulations estimates the runoff from portions of the Lake Michigan and Des Plaines River watersheds within the Stickney service basin that is diverted to the CSSC in the form of inflow-infiltration. The budget provides an internal verification of the accounting procedures. The results of this budget are used in Budget 13 and Columns 6 and 12.
11	MWRDGC Calumet Water Reclamation Facility	This budget performs hydrologic and hydraulic simulation of the service basin tributary to the MWRDGC Calumet Water Reclamation Facility. The simulations estimates the runoff from portions of the Lake Michigan and Des Plaines River watersheds within the Calumet service basin that is diverted to the CSSC in the form of inflow-infiltration. The budget provides an internal verification of the accounting procedures. The results of this budget are used in Budget 13 and Columns 6 and 12.

Description of the Diversion Accounting Computational Budgets

Budget No.	Title	Description
12	MWRDGC Lemont Water Reclamation Facility	This budget performs hydrologic and hydraulic simulation of the service basin tributary to the MWRDGC Lemont Water Reclamation Facility. The simulations estimates the runoff from portions of the Des Plaines River watershed within the Lemont service basin that is diverted to the CSSC in the form of inflow-infiltration. The budget provides an internal verification of the accounting procedures. The results of this budget are used in Budget 13 and Column 6.
13	Chicago Canal System	This budget performs a water balance of the Chicago Canal System which includes the CSSC and adjoining channels. This budget provides a verification point for the accounting procedures.

APPENDIX 3

Enclosure 1

ROMEOVILLE AVM STATION DESCRIPTION REV. 05/19/93

Quadrangle - Romeoville, 7.5' series

STATION NUMBER 05536995

WRITTEN BY: D.A. Stedfast, 08/14/91
CHECKED BY: D.P. Morgan, 08/14/91
REVISED BY: D.P. Morgan, 05/19/93

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES DIVISION

Description of Gaging Station on Chicago Sanitary & Ship Canal at Romeoville, IL

Location.--Lat 41 38' 26", long 88 03' 38", in SE1/4SW1/4 sec. 35, T.37 N., R.10 E., Will County, Hydrologic Unit 07120004, on the left bank 40 feet upstream from Romeoville Road (135th Street) bridge over the canal, at Romeoville, 5.2 miles upstream from Lockport Lock and Dam, and at river mile 6.2.

To reach gage.-- Exit from I80 at Richards Street in Joliet, IL. Drive north on Richards Street approximately 0.9 miles to Jefferson Street (just past RR Viaduct). Proceed right on Jefferson Street which turns north and becomes Collins Street and later route 171 for 5.5 miles to New Avenue (0.9 miles north of route 171 and route 7 intersection). Turn left onto New Avenue and proceed 2.8 miles to 135th Street. At 135th Street turn left and drive 0.6 miles to the Romeoville Road Bridge over the canal. Turn right between the bridge and the railroad tracks, and proceed 150 ft north to gage.

Establishment and history.--April 1974 to September 19, 1977, water-temperature records were collected at a site on the right downstream side of the bridge. Water quality samples have been collected at the site from 1974-77 and from 1987 to the current year. The first acoustical velocity meter and the wire-weight gage were installed during the week of March 18-23, 1984, by A. W. Noehre, G. G. Fisk, J. K. LaTour, and personnel from Sarasota Automation, W. Buck and A. Rouse. A manometer and digital water-stage recorder were installed on April 3, 1984, by G. G. Fisk. The Sarasota AVM was replaced by a Ferranti O.R.E. Inc. AVM during the week of November 7-11, 1988. The O.R.E. AVM was installed by M. P. DeVries, D. P. Morgan, J. J. Duncker, S. M. Robinson, and a representative from O.R.E.. At the time of the O.R.E. AVM installation the downstream set of three transducers (upper, middle, and lower paths) was relocated 20 feet downstream from the original location. On November 2, 1990 the left upstream set of three transducers (upper, middle, and lower paths) was relocated approximately 20 feet downstream to a natural crevis in the canal wall. A staff gage was installed on November 2, 1990, by D. P. Morgan and L. J. Mansue. A PVC stilling well with a Handar SDI-12 shaft encoder were installed during the week of July 6-9, 1992, by D. P. Morgan, L. J. Mansue and L. C. Schideman.

Drainage area.--739 sq mi.

Gage.-- A four path Ferranti O.R.E. 7410 Acoustical velocity meter, Decwriter printer, Campbell Scientific CR-10 datalogger with a temperature thermocouple, Handar SDI-12 shaft encoder operated over a float stilling well in canal, a stage potentiometer operated by a manometer, and a telemetry system within an 8 ft x 8 ft x 8 ft concrete block shelter. AC power is available to run the equipment, heater, air conditioner, telephone modem and security system.

Eight velocity transducers are installed at four locations along the canal walls with two locations on each bank approximately 170 ft apart and 162 ft across from the other bank. The velocity transducers are installed in or attached to 3 or 6 inch plastic pipes which were fabricated to be adjusted from the top of the canal wall. The transducers attached to the 6 inch pipe are secured by being directly inserted into holes in the 6 inch pipe. The transducers attached to the 3 inch pipes are secured by being epoxied into a 3 inch to 2 inch reducer which is then inserted into a 2 inch hole in the 3 inch plastic pipe. This assembly is then secured with lag bolts through each side of the pipe. These pipes are located within chases cut into the limestone canal walls or within natural recessed openings. They are secured to the canal walls by a pair of steel strap mounting brackets located above the water surface to hold the pipes in a fixed position. One set of three transducers (upper, middle, and lower paths) is located along the east (left) bank approximately 130 feet upstream of the gage. The other set of three transducers is located along the west (right) bank near the upstream side of the bridge. The remaining pair of transducers make up the cross-path. The upstream cross-path transducer is located approximately 170 feet upstream of the bridge on the west (right) bank and the downstream transducer is located on the east (left) bank near the upstream side of the bridge. Each pair (path) of transducers are at approximately the same elevation, across the canal from one another at approximately a 45 degree angle to the canal sides, and at different vertical locations within the cross-section. Each "pair" forms a velocity path. One stage transducer is attached to the east (left) canal wall near the gage house and is pointed at the water surface. All transducers are linked to the acoustical velocity meter by individual electrical wires. The wires from the west (right) bank are attached to the canal wall, submerged along the canal bottom, attached to the east (left) wall within the chase, and buried under ground to the gage house.

A PVC stilling well is mounted next to stage transducer on the same mounting bracket that is attached to the east (left) canal wall. A float operated SDI-12 shaft encoder is linked to CR10 datalogger by buried cable.

The orifice for the manometer is attached to the east (left) canal wall located near the stage transducer and is pointed at the canal bottom.

An outside staff gage is bolted to the east (left) canal wall, in the same natural crevice as the uplooker transducer and the manometer orifice.

A security system is installed with a direct telephone line to the Romeoville Police Station. Anyone entering the gage house has 30 SECONDS TO TURN OFF THE SYSTEM by using the key found on the top of the security control box. The security control box is located at the left side of the door as you enter. Upon leaving the gage house to activate the system, shut the front door, turn the system on, remove the key and return to top of the security control box, turn out the overhead light, open the door, exit, and close the door. YOU HAVE 45 SECONDS TO COMPLETE THE ABOVE PROCEDURE.

Elevations of prominent features, in feet above gage datum, are as follows:

Velocity path no. 1 (top) transducers	16.51
Velocity path no. 2 (middle) transducers	12.26
Velocity path no. 3 (cross) transducers	13.16
Velocity path no. 4 (bottom) transducers	8.28
Stage transducer	15.75
Orifice	18.00

AVM transducer path lengths and angles are as follows:

Length of velocity paths 1,2, and 4	236. feet
Length of velocity path 3	235. feet
Angle of velocity paths 1,2, and 4	44.5 degrees
Angle of velocity path 3	44.0 degrees

SPECIAL NOTE: Repair work at this gage sometimes requires a dive. If a dive is required then TWO DIVERS AND A LINETENDER (SPOTTER) MUST BE ON HAND! ALL DIVES will be conducted using the "buddy system" as per safety regulations. BOTH DIVERS WILL BE DRESSED IN DRY SUITS.

Gage datum.--551.89 ft NGVD of 1929, determined from USAE BM 296.1 No. 113. The bench mark is the top of a copper bolt leaded vertically into the top of the bridge seat on the pier at the east end of the swing-bridge over Chicago Sanitary and Ship Canal at Romeoville. The bolt is near the south end of coping and 0.95 ft west of east end. Elevation 595.064 ft NGVD of 1929.

Reference and bench marks.--BM-1.--Copper bolt on top of and near downstream end of left bridge pier. Elevation = 43.17 ft.

Regulation and diversion.--Flow is diverted from Lake Michigan and is regulated by the Lockport Lock and Dam. The amount of diversion (3,200 cfs) is set by the December 1, 1980 amendment to the Supreme Court of the United States Decree of June 12, 1967.

Accuracy.-- The discharge records should be excellent.

Cooperation.-- U.S. Army Corps of Engineers, Chicago District

Observer.--None

Twenty-four verticals should be taken at the following predetermined stations:

6	32	60	88	116	148
10	39	67	95	123	158
18	46	74	102	130	163
25	53	81	109	138	168

note: Stationing is in feet from the right edge of water at the canal wall (station 6ft is located at REW and station 168ft at LEW).

The mean velocity for each section should be determined using the ".2, .6, and .8" velocity method in each vertical to describe the flow more accurately for calibration of the acoustical velocity meter. The equation for the mean section velocity is as follows:

$$v(avg) = [((v(.2) + v(.8)) / 2) + v(.6)] / 2$$

The AVM discharge during the discharge measurement shall be weighted based on data recorded during the discharge measurement. Intermediate watch times (beginning and ending times) at each measurement station shall be recorded on the measurement note sheet. One minute AVM stage, velocity, and discharge readings shall recorded on the Decwriter. These recorded values shall then be used to compute the weighted AVM discharge using the following formula:

$$\text{Weighted AVM Q} = \text{sum}[\ Q_{meas} \times Q_{avm}\ ] / Q_{meas(total)}$$

where: Q_{meas} = the discharge measured in each sub-area (section).

Q_{avm} = the average AVM discharge recorded during the time that the section was measured.

$Q_{meas(total)}$ = the total discharge measured.

Discharge measurements can also be made using an acoustical doppler meter from one boat pulled across canal attached to tagline as mention above. This will become the preferred way to make measurements because one measurement can be made in 4 minutes, therefore a series of measurements can be made in the same time a single conventional boat measurement can be made.

Floods.-- None, due to regulation.

Point of zero flow.--Can occur at any stage.

Winter flow.--Free of ice formation.

BM-2.--Bronze gaging station tablet in gage house concrete pad near landward side of gage door. Elevation = 35.44 ft.

RP-3.--Head of lag-bolt in power pole located 15 feet upstream of left end of bridge. Elevation = 37.65 ft.

RP-4.--Carragebolt in canal wall near top of staff board. Elevation = 29.34 ft.

Channel and control.--The canal at this location was constructed by using dynamite to cut an opening in the limestone outcrop of near vertical walls 34 ft deep and a top width of 162 ft. The channel bottom is almost horizontal with some rounding at the face of each bank. The control for low and medium flows is the Greater Chicago Metropolitan Sanitary District Dam and the U.S. Army Corps of Engineers Lock at Lockport, 5.2 mi downstream. High flow is controlled by the Lock and Dam and the Greater Chicago Metropolitan Sanitary District Controlling Works, 3.0 mi downstream, which diverts water to the Des Plaines River. The water surface elevation in the canal is maintained at about a 25 ft stage at the gage. When heavy rains are forecasted the water surface is drawn down about 4-5 ft by opening the controlling works to temporarily lower flood waters and to increase the discharge from the Chicago area. During these periods the stage at the gage decreases and the velocity and discharge increase.

Discharge measurements.--Measurements are made from boats using a tagline stretched across the channel at a point halfway between the upstream and downstream transducers. This measurement point is marked on each canal wall with red paint and two bolt anchors. The tagline is attached to "J" bolts which are secured into these anchors. Then the tagline can be pulled up tight using a winch. Measurement time should be minimized by having two boat crews measuring at the same time, starting from opposite sides of the canal. Full 40 second velocity sampling times are required to maintain the high degree of accuracy required for this site. Discharge measurements should also be made during periods of minimal changes in discharge (ie. minimal barge/boat traffic and changes in the gate openings at the dam 3 miles downstream). Possible variations in the flow should be checked by telephoning the Greater Chicago Metropolitan Sanitary District's (GCMSD) Supervising Civil Engineer, Bill Eyre, of the General Division for Maintenance and Operation (312-751-5102).

APPENDIX 4

05536995 CHICAGO SANITARY/SHIP CANAL AT ROMEOVILLE, IL
1989 WATER YEAR

STATION ANALYSIS
URBANA FIELD HEADQUARTERS

EQUIPMENT.--An acoustic velocity meter (AVM), is used to collect stage and velocity record and to compute discharge. Velocity data are collected with 4 pairs of ultrasonic transducers; stage data is collected with a vertically-mounted ultrasonic transducer.

Other equipment housed in a 8' x 8' concrete block shelter includes: a digital water-stage recorder operated by a manometer which is used to check AVM stage readings; A CR-10 datalogger, which is used to record AVM data; a printer, which also records AVM data; a telephone connected to the AVM modem system on which the gage may be called for readings; a Data Collection Platform (DCP); an air conditioner and electric heater. The equipment is powered by AC current and is secured by an eight foot high cyclone and barbed-wire fence and an alarm system which is connected to the Romeoville Police Department. The outside gage is a wire-weight gage which is mounted to the upstream guard rail.

AVM RECORD.--Two different AVM's were in operation during the water year. The old Sarasota AVM was in operation from Oct 1 to Nov 3. A new ORE AVM was installed during the period from Nov 3 to Nov 17 and was officially in operation from Nov 17 to Sept 30. Both AVM's furnished satisfactory record throughout the water year, except as follows: Nov 3 to Nov 17, the new AVM was being installed; Jan. 31 to Feb. 3, the AVM was down due to failure of the power surge protector; June 12 to June 14, the AVM was down for repairs to uplooker transducer; August 22 to August 23, the AVM had shut itself down possibly due to a current surge from a nearby lightning strike(no apparent damage to equipment); and September 26 to September 30, the AVM was down due to the destruction of the submarine cables by barge traffic. Estimated daily discharges for these periods were provided by the Army Corps of Engineers. The estimates were determined from the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) records for the Lockport Dam which were adjusted based on a regression equation developed by the Corps.

Velocity paths 1 and 3 were not working from Oct 1 (previous WY) to Nov 3. The correction factor of 0.988 applied during the 1988 WY was also applied to the daily mean discharges computed using paths 2 and 4 for this year. This correction factor was based on an analysis made during the 1988 WY which investigated the accuracy of computing daily discharge using various combinations of the velocity paths.

The uplooking stage transducer started giving bad data on June 1, failed on June 12, and was replaced on June 14. After the new stage transducer was installed the gage height datum was incorrectly set in the AVM and was not corrected until after Sept. 30. Therefore, manometer stage data collected during the period June 1 to Sept. 30 was used for computing discharge rather than the AVM stage data.

Various path lengths, path angles, and all path elevations stored in the O.R.E. AVM were incorrect during the period Nov. 17 to Sept. 30. Therefore, the AVM recorded discharges and velocities during this period were incorrect. In addition, the discharge equations in

05536995 CHICAGO SANITARY/SHIP CANAL AT ROMEOVILLE, IL
1990 WATER YEAR

Correction factors for the bad velocity readings was developed using the AVM velocity equation, V=(L*dt)/(2*T^2*cos(theta)). Therefore, Vcorrected= Vold(Lcorrect/Lold), which by substitution becomes Vcorrect=Vold(0.961).

b) The path angles for the bottom, middle, and top paths were incorrectly set at 41 degrees from Oct. 1 to Mar. 21. The correct path angle was 42 degrees + or - 5 minutes. Utilizing the AVM velocity equation resulted in the correction factor, Vcorrect=Vold(1.016). The path angle for the cross path was also set incorrectly at 44.5 degrees during the same period. The correct path angle for this path was 44 degrees + or - 5 minutes. Which, utilizing the AVM velocity equation resulted in the correction factor, Vcorrect= Vold(0.992).

c) All velocity path elevations were incorrectly set in the AVM for the period Oct. 1 to Apr. 5. These errors resulted in incorrect discharge values being computed by the AVM. However, since the AVM computed discharges are not used for the official record these errors only resulted in relatively small "real time" discharge data errors. The correct path elevations were measured on March 19, 1990, and can be found listed in the Station Description dated August 1, 1990. The official discharge equations and a description of the methodology can be found in the Station Quality Assurance Plan dated August 1, 1990.

d) The path velocities were corrected and the discharge was correctly computed using a computer program on the Prime Mini Computer. All correctly computed daily discharge records were reviewed and down loaded into the ADAPS data base.

GAGE-HEIGHT RECORD: The gage-height datum for the uplooking stage transducer that was installed June 14, 1989 was incorrectly set in the AVM and was not corrected until 1400 hours March 21, 1990. The uplooking stage transducer did not provide satisfactory gage-height record during the period Oct. 1 to Mar. 21 due to incorrect datum setting; May 8, transducer assembly slipped down and was at an angle; May 9, assembly dangling by one strap; May 24, removed assembly for repairs and reinstalled on May 25; June 12, found assembly at an angle; and following the end of the water year on Oct. 15, 1990 found assembly gone. The manometer provided satisfactory stage record during the year. Therefore, manometer stage record collected during the period Oct. 1 to Sept. 30 was used to compute discharge in lieu of the AVM uplooking stage record.

DATUM CORRECTIONS.-- Corrections to (wire-weight) gage

	DATE	FOUND	LEFT
Levels	06-03-87	+.003'	+.003'

No datum corrections were warranted during the water year.

VELOCITY CORRECTIONS.--Application of velocity correction factors:

From October 01, 1989 at 0015 hours to March 19, 1990 at 1745 hours:

Vcorrect VEL-U(1) = Vold VEL-U(1) * 1.016 * 0.961
Vcorrect VEL-M(2) = Vold VEL-M(2) * 1.016 * 0.961
Vcorrect VEL-X(3) = Vold VEL-X(3) * 0.992

05536995 CHICAGO SANITARY/SHIP CANAL AT ROMEOVILLE, IL
1989 WATER YEAR

during the 1989 water year. These measurements range in discharge from 2,072 to 14,900 ft3/s. The daily discharge range experienced during the water year was from 1,790 ft3/s to 12,800 ft3/s. With the exception of measurements no. 43, 44, and 46 all measurements confirmed the AVM discharge within 4.3 percent of the measured discharge. These measurements were believed to have been made with polymer cups and were not used in the analysis.

All measurements were made with the .2, .6, and .8 method to obtain point velocities in the verticals. Mean velocity for each vertical was computed in the following manner:

$$v(avg) = [((v(.2) + v(.8)) / 2) + v(.6)] / 2$$

AVM discharges were discharge measurement weighed to adjust for change in discharge during the measurement.

DISCHARGE.-- Sarasota AVM discharges were used from October 1 to Nov 2. Discharge was computed using the attached equations (note: the equation format and detailed description of the method can be found in the Station Quality Assurance Plan dated 8/1/90) for the period, Nov 17 to Sept. 30. The periods of no records or less than 80 AVM readings per day are Nov. 3-17, Jan. 31 - Feb. 3, June 12-14, Aug. 22-23, and Sept. 26-30. Estimated daily discharges for these periods were determined from the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) records for the Lockport Controlling Works which were adjusted based on a regression equation developed by the Corps of Engineers.

On April 5, 1990, a meeting was held at the AVM site to discuss the 1989 WY discharge records with representatives from the Corps of Engineers (COE), Chicago District, and IDOT Divisiont of Water Resources (DWR). Attendees included David Stedfast (USGS), David Morgan (USGS), Tom Price (COE), Tim Lormand (COE), and Dan Injerd (DWR). The status of the current AVM equipment, proposed improvements, and quality of this years record were discussed. All parties agreed that the O.R.E. equipment was both doing well and was a definite improvement over the Sarasota AVM. The proposed improvements included moving the upstream transducers on the gage side downstream 20 feet to a natural crack in the canal wall and locating the manometer orifice in a stilling well. Both DWR and COE representatives agreed that the 1989 daily discharge records were acceptable and appeared very good after comparison to daily discharges computed by the COE using MWRDGC Lockport records.

REMARKS.--AVM discharge records are good except for periods of no record, which are fair.

Written by David A. Stedfast
Checked by David P. Morgan
Date August 3, 1990

05536995 CHICAGO SANITARY/SHIP CANAL AT ROMEOVILLE, IL
1990 WATER YEAR

The records for water year 1990 were re-computed in November 1992 by Kevin Oberg and Steve Melching, Illinois District, USGS. The records were re-computed to correct for an error in the average depth for the reach measured by the AVM. See report "Discharge and Regression Analyses for Acoustic Velocity Meter Data for Chicago Sanitary and Ship Canal at Romeoville, Illinois", by C.S. Melching and K.A. Oberg for more details. In addition, Melching and Oberg developed a new set of regression equations for estimating missing record, using data provided by MWRDGC for the Lockport Powerhouse and Controlling Works. These new equations, also described in the above report, were used to estimate days of missing record for the 1990 water year.

Revised by: Kevin Oberg
Date: May 27, 1993

05536995 CHICAGO SANITARY/SHIP CANAL AT ROMEOVILLE, IL
1990 WATER YEAR

STATION ANALYSIS
URBANA FIELD HEADQUARTERS

EQUIPMENT.--A Ferranti O.R.E. Inc. Accusonic Flowmeter Model 7410, hereafter referred to as an acoustic velocity meter (AVM), is used to collect stage and velocity record and to compute discharge. Velocity data are collected with 4 pairs of ultrasonic transducers; stage data is collected with a vertically mounted ultrasonic transducer.

Other equipment housed in a 8' x 8' concrete block shelter includes: a manometer which is used to check AVM stage readings and serves as a back-up system for collecting stage; A CR1O datalogger, which is used to record AVM and manometer data; a printer, which also records AVM data; a telephone connected to the CR10 modem system on which the gage may be called for readings; a Data Collection Platform (DCP); an air conditioner and electric heater. The equipment is powered by AC current and is secured by an eight foot high cyclone and barbed-wire fence and an alarm system which is connected to the Romeoville Police Department.

The outside gage is a wire-weight gage mounted to the upstream guard rail of the Romeoville Road (135th Street) bridge and was used until September 18, 1990, at which time the bridge was condemned and removed from service because of an unsafe structural condition. An outside staff gage was installed on November 2, 1990, and is now the base outside gage.

AVM RECORD.--The AVM furnished satisfactory record throughout the water year, except as follows: Oct. 1 to 6, the AVM was down due to the destruction of the submarine cables by barge traffic; Oct. 10 to 13, transducer assembly knocked out of alignment by a barge; Nov. 15 to 21, bad velocity transducers plus the transducers were dropped from their mounting into the canal; Mar. 16 to 19, transducer assembly knocked out of alignment by a barge; Apr. 29 to May 1, a short in the junction box caused the AVM to shut down and stop taking velocity measurements; May 5 to 7, no gage-height record due to equipment malfunction; May 9 to 30, the AVM was down due to the destruction of the submarine cables by barge traffic; and June 14, AVM transceiver damaged by lightning.

VELOCITY RECORD: Values for the path lengths, path angles, and path elevations were incorrectly stored in the AVM during most of the 1989 water year. These errors continued into the 1990 water year until March 1990. Therefore, the AVM recorded velocites and discharges during this period were incorrect. The equations in the AVM are not the official equations for computing discharge record, however, they do provide useful provisional "real-time" information to the cooperator.

Discussion of the correction factors as applied to the path length, path, angle, and path elevations:

a) The path lengths for the bottom and middle path were incorrectly set at 255.9 ft. and 255.6 ft. respectively during the period Oct. 1 to Mar. 19. The top path length was incorrectly set to 255.9 ft. during the period Oct. 1 to Mar. 19. The correct path length for all three of these paths was 246 ft. as measured on March 19, 1990.

Correction factors for the bad velocity readings was developed using the AVM velocity equation, V=(L*dt)/(2*T^2*cos(theta)). Therefore, Vcorrected= Vold(Lcorrect/Lold), which by substitution becomes Vcorrect=Vold(0.961).

b) The path angles for the bottom, middle, and top paths were incorrectly set at 41 degrees from Oct. 1 to Mar. 21. The correct path angle was 42 degrees + or - 5 minutes. Utilizing the AVM velocity equation resulted in the correction factor, Vcorrect=Vold(1.016). The path angle for the cross path was also set incorrectly at 44.5 degrees during the same period. The correct path angle for this path was 44 degrees + or - 5 minutes. Which, utilizing the AVM velocity equation resulted in the correction factor, Vcorrect= Vold(0.992).

c) All velocity path elevations were incorrectly set in the AVM for the period Oct. 1 to Apr. 5. These errors resulted in incorrect discharge values being computed by the AVM. However, since the AVM computed discharges are not used for the official record these errors only resulted in relatively small "real time" discharge data errors. The correct path elevations were measured on March 19, 1990, and can be found listed in the Station Description dated August 1, 1990. The official discharge equations and a description of the methodology can be found in the Station Quality Assurance Plan dated August 1, 1990.

d) The path velocities were corrected and the discharge was correctly computed using a computer program on the Prime Mini Computer. All correctly computed daily discharge records were reviewed and down loaded into the ADAPS data base.

GAGE-HEIGHT RECORD: The gage-height datum for the uplooking stage transducer that was installed June 14, 1989 was incorrectly set in the AVM and was not corrected until 1400 hours March 21, 1990. The uplooking stage transducer did not provide satisfactory gage-height record during the period Oct. 1 to Mar. 21 due to incorrect datum setting; May 8, transducer assembly slipped down and was at an angle; May 9, assembly dangling by one strap; May 24, removed assembly for repairs and reinstalled on May 25; June 12, found assembly at an angle; and following the end of the water year on Oct. 15, 1990 found assembly gone. The manometer provided satisfactory stage record during the year. Therefore, manometer stage record collected during the period Oct. 1 to Sept. 30 was used to compute discharge in lieu of the AVM uplooking stage record.

DATUM CORRECTIONS.-- Corrections to (wire-weight) gage

	DATE	FOUND	LEFT
Levels	06-03-87	+.003'	+.003'

No datum corrections were warranted during the water year.

VELOCITY CORRECTIONS.--Application of velocity correction factors:

From October 01, 1989 at 0015 hours to March 19, 1990 at 1745 hours:

Vcorrect VEL-U(1) = Vold VEL-U(1) * 1.016 * 0.961
Vcorrect VEL-M(2) = Vold VEL-M(2) * 1.016 * 0.961
Vcorrect VEL-X(3) = Vold VEL-X(3) * 0.992

05536995 CHICAGO SANITARY/SHIP CANAL AT ROMEOVILLE, IL
1990 WATER YEAR

Vcorrect VEL-L(4) = Vold VEL-L(4) * 1.016 * 0.961

From March 19, 1990 at 1800 hours to March 21, 1990 at 1800 hours:

Vcorrect VEL-U(1) = Vold VEL-U(1) * 1.016
Vcorrect VEL-M(2) = Vold VEL-M(2) * 1.016
Vcorrect VEL-X(3) = Vold VEL-X(3) * 0.992
Vcorrect VEL-L(4) = Vold VEL-L(4) * 1.016

From March 21, 1990 at 1815 hours to September 30, 1990: No correction factors warranted.

GAGE-HEIGHT CORRECTIONS.--Manometer gage-heights were corrected to the outside gage due to manometer / potentiometer drift as shown on attached ADAPS printout of gage-height corrections, based on gage inspections during the year.

RATING.--Eleven discharge measurements, Nos. 52-62, are available to verify the AVM discharges during the 1990 water year. These measurements range from 1,870 to 14,400 ft3/s. The daily range experienced during the year was from 1,762 to an estimated 17,870 ft3/s. With the exceptions of measurements Nos. 54 and 55, all measurements confirmed the AVM discharge within 5.2 percent of the measured discharge. An explanation for measurements Nos. 54 and 55 not confirming the AVM discharge has not been determined. Both measurements were made back-to-back on January 11, 1990.

All measurements were made with the .2, .6, and .8 method to obtain point velocities in the verticals. Mean velocity for each vertical was computed in the following manner:

$$v(avg) = [((v(.2) + v(.8)) / 2) + v(.6)] / 2$$

The AVM discharges (computed using the attached official set of discharge equations) during the discharge measurement are weighted to adjust for change in discharge during the measurement. The procedure for weighted is provided in the Quality Assurance Plan (REV. 08/01/90).

DISCHARGE.--Discharge was computed using AVM velocity and manometer gage height records and the attached equations except for periods of no records or when less than 80 AVM readings were available in any given day. The periods of no records or less than 80 AVM readings per day are Oct. 1 to Oct. 6, Oct. 10 to 13, Nov. 15 to Nov. 21, Mar. 16 to 19, Apr. 29 to May 1, May 4 to 30, and June 14. Estimated daily discharges for these periods were provided by the Army Corps of Engineers. The estimates were determined from the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) records for the Lockport Dam which were adjusted based on a regression equation developed by the Corps of Engineers.

REMARKS.--AVM discharge records are good except for periods of no record, which are fair.

Written by G.W. Curtis, February 27, 1991
Checked by D.P. Morgan, March 1, 1991

05536995 CHICAGO SANITARY/SHIP CANAL AT ROMEOVILLE, IL
1990 WATER YEAR

The records for water year 1990 were re-computed in November 1992 by Kevin Oberg and Steve Melching, Illinois District, USGS. The records were re-computed to correct for an error in the average depth for the reach measured by the AVM. See report "Discharge and Regression Analyses for Acoustic Velocity Meter Data for Chicago Sanitary and Ship Canal at Romeoville, Illinois", by C.S. Melching and K.A. Oberg for more details. In addition, Melching and Oberg developed a new set of regression equations for estimating missing record, using data provided by MWRDGC for the Lockport Powerhouse and Controlling Works. These new equations, also described in the above report, were used to estimate days of missing record for the 1990 water year.

Revised by: Kevin Oberg
Date: May 27, 1993

05536995 CHICAGO SANITARY AND SHIP CANAL AT ROMEOVILLE, IL
1991 WATER YEAR

STATION ANALYSIS
ILLINOIS DISTRICT, U.S.G.S.

EQUIPMENT.--A Ferranti O.R.E. Inc. Accusonic Flowmeter Model 7410, hereafter referred to as an acoustic velocity meter (AVM), is used to collect gage-height and velocity record and to compute discharge. Velocity data are collected with 4 pairs of ultrasonic transducers; gage-height data is collected with a vertically mounted ultrasonic transducer.

Other equipment housed in a 8' x 8' concrete block shelter includes: a manometer which was used as the primary gage-height record and to check AVM gage-height readings; A CR10 datalogger, which is used to record AVM and manometer data; a printer, which also records AVM data; a telephone connected to the CR10 modem system on which the gage may be called for readings; a Data Collection Platform (DCP); an air conditioner and electric heater. The equipment is powered by AC current and is secured by an eight foot high cyclone and barbed-wire fence and an alarm system which is connected to the Romeoville Police Department.

The outside gage is a staff gage mounted to the canal wall in a natural niche where the uplooking gage-height assembly for the AVM and manometer orifice is located. The staff gage was installed on November 2, 1990. Prior to September 18, 1990 the outside gage was a wire-weight gage mounted to the upstream guard rail of the Romeoville Road (135th Street) bridge and was used until the bridge was condemned and removed from service because of an unsafe structural condition.

AVM RECORD.--The AVM furnished satisfactory record throughout the water year, except as follows: March 27 AVM was down due to loss of AC power caused by high winds; April 20 to 24 AVM was down due to the measurement section of AVM locking-up; and May 31 to June 3 AVM was down due to the measurement section of AVM locking up. For the period of no CR10 datalogger record and at other times when the CR10 datalogger failed to record AVM record, the Decwriter AVM printouts were used to fill in the gaps.

VELOCITY RECORD: The parameters for path lengths, path angles, and path elevations were correct in the AVM for the entire year. This information is required in order to compute velocity and discharge correctly. The equations in the AVM are not the official equations for computing discharge record, however, they do provide useful provisional "real-time" information to the cooperator.

Due to prior history of barges hitting or scraping the canal wall and either knocking the upstream main three transducer assembly out of alignment or destroying transducers / assembly, the entire assembly was moved downstream approximately 20 feet into a natural niche on November 2, 1990. This changed the path length, path angle of the upper, middle and lower paths. This move also allowed for the lowering of the lower transducer to be in vertical elevation alignment of the downstream lower transducer. The

05536995 CHICAGO SANITARY AND SHIP CANAL AT ROMEOVILLE, IL
1991 WATER YEAR

path length, path angle and path elevation were determined by station levels ran on November 1, 1990.

Path length, path angle and path elevation are as follows:

Path	Length	Angle	Cosag	Elevation
upper	236	44.5	0.713	16.51
middle	236	44.5	0.713	12.26
x-path	235	44.0	0.719	12.78
lower	236	44.5	0.713	8.28

On several occasions during the year barge traffic hit and destroyed the upstream x-path transducer assembly and once destroyed the downstream x-path transducer assembly which caused several replacements of these assemblies. On May 15 and 16, 1991, both the upstream and downstream mounting brackets for the x-path were changed in hope of making the assemblies and transducers more secure from barge traffic. Also at this time the downstream x-path was replaced with a new assembly. This new assembly changed the x-path elevation from 12.78 feet to13.16 feet. Path length and path angle remained the same.

On June 5, 1991 levels were ran into each of the upper mounting brackets of all transducer assemblies to establish reference point elevations. This was done in order to make it easier to maintain transducer elevation in case of transducer / assembly replacement. However, on June 26, 1991 the reference point for the downstream main three assembly was transferred from the mounting bracket to the top of eye bolt on metal cap of permanent pipe on which the bracket is mounted to.

Using "PLOTWAT" within "ADAPS" all working path velocities were compared graphically to one another and any bad velocity values were deleted from the original Prime datalogger files. These files were used to input the velocity record into ADAPS. Once all the bad velocities were deleted from the Prime datalogger files a computer program on the Prime Mini Computer was used to compute an average AVM velocity using the following relationships:

Mean AVM Vel. = (Vel-U + Vel-M + Vel-X + Vel-L)/4, if all four paths are working, or

If all four paths are not working then the mean AVM velocity is computed by first correcting each working path velocity by the appropriate (Mean AVM Vel)/(path velocity) ratio and then taking their arithmetic mean. The correction ratios are listed below:

(Mean AVM Vel)/(Vel-U) = 0.964
(Mean AVM Vel)/(Vel-M) = 0.989
(Mean AVM Vel)/(Vel-X) = 1.000
(Mean AVM Vel)/(Vel-L) = 1.053

05536995 CHICAGO SANITARY AND SHIP CANAL AT ROMEOVILLE, IL
1991 WATER YEAR

These relationships were determined by David A. Stedfast, who used a Minitab Statistical Analysis program on a Sun Workstation Computer. The results of his analysis can be found in a folder marked "Velocity Rating" in the Romeoville file cabinet.

GAGE-HEIGHT RECORD: The uplooking gage-height transducer did not provide satisfactory gage-height record during the following periods: October 1-18 due to improper (temporarily) mounting and uplooking assembly gone; March 27 due to loss of AC power caused by high winds; April 12-20 due to uplooking malfunction; April 20-24 AVM was down due to the measurement section of AVM locking-up; and May 31 to June 3 AVM was down due to the measurement section of AVM locking-up. There were also other periods of time including the April 12-20 where the uplooking gage-height would for no reason calculate gage-height a foot or more off in either direction for short periods of time and then come back on to the correct gage-height on its own. Because of these random occurrences of improper gage-heights the valid manometer gage-height record collected during the water year was used to compute discharge in lieu of the AVM uplooking gage-height record.

MANOMETER GAGE-HEIGHT RECORD: The manometer furnished satisfactory record throughout the water year, except as follows: December 25 to February 7 manometer malfunction and NON-U.S.G.S. removal of potentiometer sprocket; June 25 and 26 due to drained battery to CR10 datalogger - no data collected. During periods of no manometer gage-height record the AVM uplooking gage-height was used. The AVM uplooking gage-height was inputted into the same DD as the manometer gage-height and is stored without any tag following the value when you do a unit-value retrieval.

DATUM CORRECTIONS.--

	DATE	FOUND	LEFT	Remarks
Levels	11-01-90	-----	-.001'	Established staff gage
Levels	06-05-91	-----	-----	Established RPs for Transducer assemblies

No datum corrections were warranted during the water year.

VELOCITY CORRECTIONS.--

No correction factors were warranted during the water year.

GAGE-HEIGHT CORRECTIONS.--Manometer gage-heights were corrected to the outside gage due to manometer / potentiometer drift as shown on attached ADAPS printout of gage-height corrections, based on gage inspections during the year.

The project chief of water year 1990 ended the year with a +0.10 ft gage-height correction. Looking at the measurement sheets and inspections for water years 1990 and

1991 there was not enough information to base this correction, so no correction was used to start water year 1991.

RATING.--Seven discharge measurements, Nos. 63-69, are available to verify the AVM discharges during the 1991 water year. These measurements range from 1,918 to 16,941 ft3/s. The daily range experienced during the year was from 2,009 to 17,530 ft3/s. With the exceptions of Measurements Nos. 63, 64, and 68, all measurements confirmed the weighted AVM discharge within 4.61 percent of the measured discharge.

The AVM discharges computed during the discharge measurement are weighted to adjust for change in discharge during the measurement. The procedure for weighted is provided in the Quality Assurance Plan (REV. 11/13/92). The printout results of weighted discharge for each measurement is attached.

An explanation for Measurement No. 63 for being +5.74% difference is because it was abbreviated measurement of only 13 verticals from a single boat using only .2 and .8 method. Also the AVM discharge per section went from -17.9% to +21.4% difference to the discharge measured in 40 minutes.

An explanation for Measurement No. 64 for being +9.62% difference is because it was noted on the measurement that the field people had to change current meter about a third of the way through the measurement because of problems. Comparing the velocities for the sections prior to the meter change are consistently lower than the AVM. The velocities for the sections after the meter change compare favorably with the AVM. Although the measurement took 1 3/4 hours to make, the AVM discharges were relatively steady except during the last two sections where the discharge increased by 43.9% compared to the measured or 31.3% to the total weighted AVM discharge.

An explanation for Measurement No. 68 for being +5.1% difference is because although the AVM discharge per section only changed by 459 cfs during the measurement, this change at the range of discharge measured amounts to a 19.4% difference. This fluctuation in AVM discharge exceeded the 5% difference to the measured for 57% of the measurement and was up and down throughout the measurement. Actual flow condition during the measurement for the purpose of making a discharge measurement were unsteady.

Discharge Measurement No. 67 was made approximately 4.3 miles upstream of gage off of the downstream side of Lemont Road Bridge due to high velocities and low stages. The AVM discharges used to compute a weighted discharge were time adjusted for travel time of the flood wave using the following equation.

$$t = 1 / (1.3 * v)$$

where t = time of travel of flood wave between QM section and control (gage) in

05536995 CHICAGO SANITARY AND SHIP CANAL AT ROMEOVILLE, IL
1991 WATER YEAR

seconds

l = length of reach between measuring section and control (gage) in feet
v = mean velocity of QM in feet/sec

All measurements except No. 63 were made with the .2, .6, and .8 method to obtain point velocities in the verticals. Mean velocity for each vertical was computed in the following manner:

$$v(avg) = [((v(.2) + v(.8)) / 2) + v(.6)] / 2$$

DISCHARGE.--Discharge was computed within "ADAPS" using a velocity-velocity rating (No. 2) and a stage-area rating (No. 1) except for periods of no records or when less than 80 AVM readings were available in any given day. The periods of no records or less than 80 AVM readings per day are March 27, April 20-24 and May 31 to June 3. Estimated daily discharges for these periods were determined from the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) records for the Lockport Dam which were adjusted based on a regression equation developed by the Illinois District, U.S.G.S. See report "Discharge and Regression Analyses for Acoustic Velocity Meter Data for Chicago Sanitary and Ship Canal at Romeoville, Illinois", by C.S. Melching and K.A. Oberg.

VELOCITY-VELOCITY RATING.--Mean Channel Vel. = 0.92 X (Mean AVM Vel.)

STAGE-AREA RATING.--Channel Flow Area = (GH X 162) + 251.0

REMARKS.--AVM discharge records are good except for periods of no record, which are fair.

Written by: D.P. Morgan, June 29, 1992
Revised by: D.P. Morgan, May 19, 1993
Checked by: J.J. Duncker, May 27, 1993

05536995 CHICAGO SANITARY AND SHIP CANAL AT ROMEOVILLE, IL
1992 WATER YEAR

STATION ANALYSIS
ILLINOIS DISTRICT, U.S.G.S.

EQUIPMENT.--A Ferranti O.R.E. Inc. Accusonic Flowmeter Model 7410, hereafter referred to as an acoustic velocity meter (AVM), is used to collect gageheight and velocity record and to compute discharge. Velocity data are collected with 4 pairs of ultrasonic transducers; gage-height data is collected with a vertically mounted ultrasonic transducer.

Other equipment housed in a 8' x 8' concrete block shelter includes: a manometer which was used as the primary gage-height record and to check AVM gage-height readings; A CR10 datalogger, which is used to record AVM and manometer data; a printer, which also records AVM data; a telephone connected to the CR10 modem system on which the gage may be called for readings; a Data Collection Platform (DCP); an air conditioner and electric heater. The equipment is powered by AC current and is secured by an eight foot high cyclone and barbed-wire fence and an alarm system which is connected to the Romeoville Police Department.

The outside gage is a staff gage mounted to the canal wall in a natural niche where the uplooking gage-height assembly for the AVM and manometer orifice is located.

AVM RECORD.--The AVM furnished satisfactory record throughout the water year, except as follows: April 17 to 21 due to a barge hitting uplooking assembly and pulling out submarine cables for west (right) transducers; July 2 to 6 due to the AVM being hit by lightning. For the periods of no CR10 datalogger record and at other times when the CR10 datalogger failed to record AVM record, the Decwriter AVM printouts were used to fill in the gaps.

VELOCITY RECORD: The parameters for path lengths, path angles, and path elevations were correct in the AVM for the entire year. This information is required in order to compute velocity and discharge correctly. The equations in the AVM are not the official equations for computing discharge record, however, they do provide useful provisional "real-time" information to the cooperator.

Path length, path angle and path elevation are as follows:

Path	Length	Angle	Cosag	Elevation
upper	236	44.5	0.713	16.51
middle	236	44.5	0.713	12.26
x-path	235	44.0	0.719	13.16
lower	236	44.5	0.713	8.28

On April 24 a barge hit and destroyed the upstream main three (upper, middle and lower) transducer assembly. The assembly was replaced on April 29. On May 22 upper, middle and lower paths were shut off in order to pull up the downstream main three transducer assembly to replace the lower transducer. Also during the period of August 7 to 25 the upper, middle and lower paths would cut out from sunset to sunrise, but would

work fine during daylight hours. During these periods of no upper, middle and lower velocities the x-path was working satisfactory to compute an average AVM velocity.

Using "PLOTWAT" within "ADAPS" all working path velocities were compared graphically to one another and any bad velocity values were deleted from the original Prime datalogger files. These files were used to input the velocity record into ADAPS. Once all the bad velocities were deleted from the Prime datalogger files a computer program on the Prime Mini Computer was used to compute an average AVM velocity using the following relationships:

Mean AVM Vel. = (Vel-U + Vel-M + Vel-X + Vel-L)/4 if all four paths are working, or

If all four paths are not working then the mean AVM velocity is computed by first correcting each working path velocity by the appropriate (Mean AVM Vel)/(path velocity) ratio and then taking their arithmetic mean. The correction ratios are listed below:

(Mean AVM Vel)/(Vel-U) = 0.967
(Mean AVM Vel)/(Vel-M) = 0.990
(Mean AVM Vel)/(Vel-X) = 1.003
(Mean AVM Vel)/(Vel-L) = 1.049

These new relationships were determined by C.S. Melching and L.C. Schideman by using SAS programs on Data General Workstation Computer. The results of this analysis can be found in a folder marked "Velocity Rating" in the Romeoville file cabinet.

GAGE-HEIGHT RECORD: The uplooking gage-height transducer did not provide satisfactory gage-height record during the following periods: April 17 to May 14 due to a barge hitting the uplooking assembly and assembly gone; May 21 due to maintenance; June 17 and June 19 to July 8 due to uplooking transducer malfunction. There were also other periods of time when the uplooking gage-height would for no reason calculate gage-height a foot or more off in either direction for short periods of time and then come back on to the correct gage-height on its own. Because of these random occurrences of improper gage-heights the valid manometer gage-height record collected during the water year was used to compute discharge in lieu of the AVM uplooking gage-height record.

MANOMETER GAGE-HEIGHT RECORD: The manometer furnished satisfactory record throughout the water year, except as follows: December 23 to January 13 due to the potentiometer going bad; April 15, July 2 to 6 due to manometer malfunction. During periods of no manometer gage-height record the AVM uplooking gage-height was used. The AVM uplooking gage-height was inputted into the same DD as the manometer gage-height and is stored without any tag following the value when you do a unit-value

retrieval.

DATUM CORRECTIONS.--

	DATE	FOUND	LEFT	Remarks
Levels	11-01-90	-----	-.001'	Established staff gage
Levels	06-05-91	-----	------	Established RPs for Transducer assemblies

No datum corrections were warranted during the water year.

VELOCITY CORRECTIONS.--No correction factors were warranted during the water year.

GAGE-HEIGHT CORRECTIONS.--Manometer gage-heights were corrected to the outside gage due to manometer / potentiometer drift as shown on attached ADAPS printout of gage-height corrections, based on gage inspections during the year. Whenever possible an average gageheight readings (during QM's) were used to determine correction factors due to the stage elevation fluctuation in the canal.

RATING.--Four conventional discharge measurements, Nos. 70-74D, and one series of Acoustic Doppler Meter measurements (June 26) are available to verify the AVM discharges during the 1992 water year. These measurements range from 2,297 to 3,977 ft3/s. The daily range experienced during the year was from 2,077 to 10,650 ft3/s. With the exceptions of Measurements No. 71 all measurements confirmed the weighted AVM discharge within 4.5 percent of the measured discharge.

The AVM discharges computed during the discharge measurement are weighted to adjust for change in discharge during the measurement. The procedure for weighted is provided in the Quality Assurance Plan (REV. 05/19/93). The printout results of weighted discharge for each measurement is attached.

An explanation for Measurement No. 70 for being +2.78% difference and Measurement No. 71 for being -22.78% difference is as follows: During both measurements the AVM discharges indicate flow conditions were unsteady. During measurement No. 70 the fluctuation in AVM discharge during the last third went from 2656 cfs up to 4291 cfs and back down to 1940 cfs and that during the first two thirds the AVM discharge was approximately 15.7% less than measured. Because of this large change in AVM discharge a check measurement was made. The fluctuation in AVM discharge during measurement No. 71 was less but went from 1686 cfs up to 2454 cfs back down to 1790 cfs and was consistently less than measured. The measured discharge between the two measurements were within 4.2% of each other and verify each other. The last third of measurement No. 70 compensates the first two thirds and because of the unsteady flow condition at these low flow conditions would be hard to measure correctly and any

change in flow would be significant.

No analytical procedures have been established yet for acoustical doppler meter (ADCP) measurements but provisional analysis shows that the velocities and discharges measured by the ADCP compare favorable with the AVM.

All conventional measurements were made with the .2, .6, and .8 method to obtain point velocities in the verticals. Mean velocity for each vertical was computed in the following manner:

$$v(avg) = [((v(.2) + v(.8)) / 2) + v(.6)] / 2$$

DISCHARGE.--Discharges were computed within "ADAPS" using a velocity-velocity rating (No. 2) and a stage-area rating (No. 1) except for periods of no record or when less than 80 AVM readings were available in any given day. The periods of no record or less than 80 AVM readings per day are April 17 to 21 and July 2 to 6. Estimated daily discharges for these periods were determined from the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) records for the Lockport Dam which were adjusted based on a regression equation developed by the Illinois District, U.S.G.S. See report "Discharge and Regression Analyses for Acoustic Velocity Meter Data for Chicago Sanitary and Ship Canal at Romeoville, Illinois", by C.S. Melching and K.A. Oberg.

VELOCITY-VELOCITY RATING.-- Mean Channel Vel. = 0.92 X (Mean AVM Vel.)

STAGE-AREA RATING.-- Channel Flow Area = (GH X 162) + 251.0

REMARKS.--AVM discharge records are good except for periods of no record, which are fair.

Written by: D.P. Morgan, May 20, 1993
Checked by: J.J. Duncker, May 27, 1993

APPENDIX 5

Stability of the Measuring Section

There is some evidence that from time to time varying amounts of debris are accumulated and transported away from the vicinity of the measuring section. The purpose of this analysis is to find out the extent to which the stability of the mean bottom elevation of the channel is affected by these events.

The data for this analysis was compiled from discharge measurements made at the downstream side of the Romeoville Road bridge see table 1a. The area listed in the table is described by equation 1a

$$A = C * (Gh + X) \qquad (1a)$$

Where:

A is the cross-sectional area in square feet

Gh gage height in feet

X depth correction factor (departure from the zero of the gage at time of measurement)

C is equal to 162 (width of the channel)

Equation 1a can be used to compute X when A is known

$$X = A/162 - Gh \qquad (2a)$$

For example the depth correction factor at the time of the first measurement made on March 22, 1984 was X=4190/162-25.5 = 0.36 feet.

Values of X for the rest of the discharge measurements in the data set were computed and listed in table 1a.

Table 1a. Depth Correction X

Meas. No.	Date	Width	Gage Height	Area	Measured Discharge	AVM Discharge	Observed Velocity	Index X
1	22-Mar-84	163	25.50	4190	6400	6310	1.53	0.36
2	12-Jun-84	164	25.20	4170	3710	3591	0.89	0.54
3	29-Jun-84	164	24.60	4088	3400	2436	0.83	0.63
4	16-Oct-84	164	22.20	3660	7670	8477	2.10	0.39
5	25-Feb-85	164	23.40	3830	5770	6117	1.51	0.24
6	04-Mar-85	164	22.20	3640	17700	17908	4.86	0.27
7	04-Apr-85	164	22.40	3680	17900	17842	4.86	0.32
8	29-Aug-85	164	24.90	4110	3460	2685	0.84	0.47
9	17-Oct-85	164	25.00	4090	2550	1486	0.62	0.25
10	19-Nov-85	164	23.20	3800	6660	6383	1.75	0.26
11	20-Nov-85	163	22.20	3650	8510	8230	2.33	0.33
12	20-Dec-85	164	25.10	4160	2920	2675	0.70	0.58
13	10-Jan-86	164	25.00	4280	1360	944	0.32	1.42
14	04-Apr-86	164	25.10	4210	1880	1392	0.45	0.89
15	01-Jul-86	164	24.30	3970	5100	4827	1.28	0.21
16	14-Jul-86	164	24.70	4150	3050	2678	0.73	0.92
17	07-Aug-86	164	24.70	4090	4720	4134	1.15	0.55
18	22-Sep-86	164	24.70	4150	4400	2541	1.06	0.92
19	25-Sep-86	164	24.80	4080	4460	4389	1.09	0.39
20	03-Oct-86	164	21.80	3660	15800	15725	4.32	0.79
21	12-Dec-86	164	25.00	4170	3430	3275	0.82	0.74
22	26-Jan-87	164	25.00	4190	2850	2903	0.68	0.86
23	16-Mar-87	164	25.10	4180	1950	2180	0.47	0.70
24	07-Apr-87	164	25.00	4170	2430	2383	0.58	0.74
25	15-May-87	164	25.10	4190	3120	3114	0.74	0.76
26	19-Jun-87	164	24.30	4080	7300	6838	1.79	0.89
27	24-Jul-87	164	24.70	4110	3970	3992	0.97	0.67
28	17-Aug-87	164	20.90	3450	14700	14325	4.26	0.40
29	09-Oct-87	164	24.90	4140	2670	2689	0.64	0.66
30	15-Oct-87	164	24.80	4160	2930	2837	0.70	0.88
31	15-Oct-87	164	24.90	4170	2460	2116	0.59	0.84
32	15-Oct-87	164	25.10	4160	2660	2271	0.64	0.58
33	19-Oct-87	164	25.20	4180	2320	1983	0.56	0.60
34	19-Oct-87	164	25.20	4170	2550	1983	0.61	0.54
35	03-Nov-87	164	25.10	4150	2020	1993	0.49	0.52
36	23-Nov-87	164	24.90	4150	2810	2847	0.68	0.72
37	30-Dec-87	164	25.00	4140	2840	3803	0.69	0.56
38	12-Feb-88	164	25.50	4220	2080	2181	0.49	0.55
39	10-Jun-88	164	25.00	4160	4110	4181	0.99	0.68
40	27-Jul-88	164	25.20	4150	3010	2942	0.73	0.42
41	15-Sep-88	164	24.80	4100	5230	5228	1.28	0.51

Table 1a. Depth Correction X (continued)

Meas No.	Date	Width	Gage Height	Area	Measured Discharge	AVM Discharge	Observed Velocity	Index X
42	18-Nov-88	164	25.10	4210	4510	3810	1.07	0.89
43	13-Jan-89	164	25.30	4210	2580	2677	0.61	0.69
44	03-Mar-89	164	25.50	4220	2250	2401	0.53	0.55
45	05-Apr-89	164	25.60	4230	2560	2634	0.61	0.51
46	03-May-89	164	25.40	4140	2070	2233	0.50	0.16
47	01-Jun-89	164	21.20	3530	14400	14323	4.08	0.59
48	01-Jun-89	164	21.10	3550	15100	14209	4.25	0.81
49	18-Jul-89	164	24.70	4140	5480	5330	1.32	0.86
50	04-Aug-89	164	25.10	4140	4040	4090	0.98	0.46
51	12-Sep-89	164	24.70	4200	6010	5932	1.43	1.23
52	21-Nov-89	163	25.60	4170	2690	2804	0.65	0.14
53	21-Nov-89	163	25.60	4190	2890	2705	0.69	0.26
54	11-Jan-90	164	25.20	4200	2070	2319	0.49	0.73
55	11-Jan-90	164	25.10	4140	1990	2241	0.48	0.46
56	06-Mar-90	164	25.60	4260	2270	2046	0.53	0.70
57	06-Mar-90	164	25.50	4250	2700	2929	0.64	0.73
58	24-Apr-90	164	25.70	4220	2730	2746	0.65	0.35
59	11-Jul-90	164	25.60	4160	5450	5467	1.31	0.08
60	11-Jul-90	164	25.40	4180	5310	5362	1.27	0.40
61	20-Jul-90	164	20.70	3400	14400	13942	4.24	0.29
62	22-Aug-90	164	25.30	4150	5250	5082	1.27	0.32

Figure 1a Variation of Depth correction

x with Time - March 84 to Aug 1990

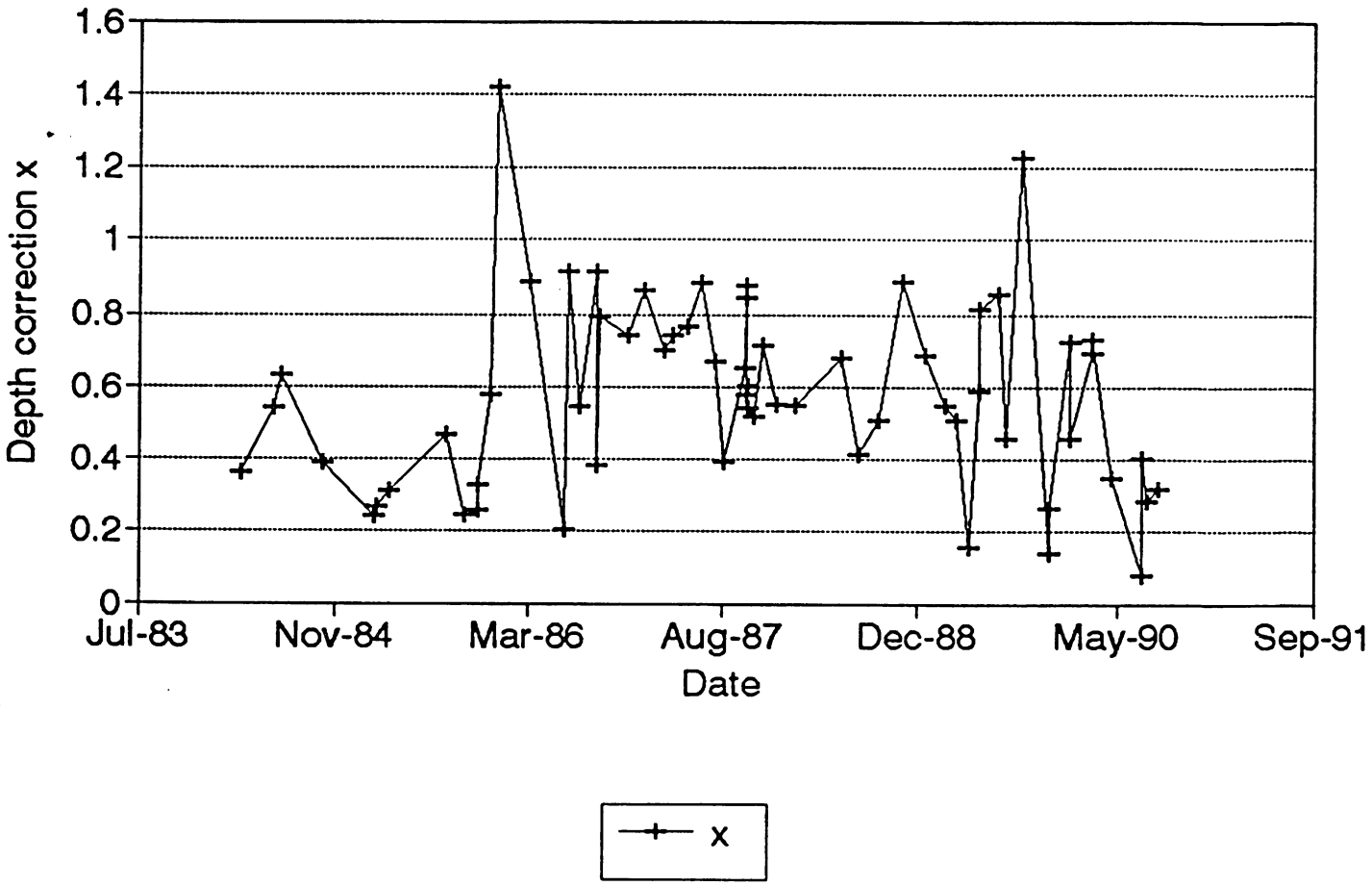


Figure 1a shows a plot of the X values vs. time for the period of record. Figure 1a is a graphic representation of the continuously changing mean bottom of the canal. It shows that during the period of record from March 1984 to August 1990 the mean bottom elevation ranged from about 0.2 to 1.4 feet. The mean for the period of record is about 0.60 feet below the zero of the gage. The results of this analysis indicate that it would be prudent to measure the cross section periodically to determine if a correction or shift to the rating is needed to account for changes in the mean bottom elevation.

APPENDIX 6

Appendix 6 is a USGS review (11/26/93) of a early draft chapter of the Third Technical Committee Report. Chapter 4, "AVM System at Romeoville", of the Committee's final report (August 1994) has been significantly revised, and, therefore, all references to pages, figures, and equations in the old draft chapter are inconsistent with the final report Chapter 4, pp 66-91.

United States Department of the Interior

GEOLOGICAL SURVEY
Water Resources Division
102 E. Main St., 4th Floor
Urbana, Illinois 61801

November 26, 1993

Oscar Lara
118 South Clinton
Suite 210
Iowa City, Iowa 52240

Dear Oscar:

I have completed my review of your findings concerning the stage-area rating for the acoustic velocity meter (AVM) on the Chicago Sanitary and Ship Canal at Romeoville, IL. Please accept my apologies for the delay in sending my comments. After discussing the issue in-house, we felt that it would be wise to have it reviewed prior to sending it to you. This is part of the reason for the delay.

Your suggestion of a real-time adjusted rating is excellent for most rivers in the Midwest. However, I do not feel that it is warranted at this location due to the low sediment load, the bedrock channel, the general lack of bed material, and a number of practical reasons. The documents and plots attached provide more details and evidence to support this. I do believe that the cross-sections should be measured at least once per year. If these measurements indicate that it should be done more frequently, then we will do this and apply shifts to the stage-area rating accordingly.

Also enclosed are some of the plots of ADCP data that I promised to send to you. I gave these to the other committee members and the Chicago Corps when we last met. I have also enclosed a diskette with the mean daily stages and velocities for Romeoville for the 1989 and 1990 water years. Please refer to the station analyses to know which stage was used during these two water years. In general the manometer stage was used throughout the whole period, but there is a substantial portion of the 1989 water year when the AVM stage was used.

Thanks for sharing your comments with me. They are valuable and have caused us to give greater consideration to this aspect of the data being collected at the AVM. If you have any questions, please do not hesitate to call. Please let me know if you would like me to come out to Iowa City when you meet with the rest of the review committee.

Sincerely,

Kevin

Kevin A. Oberg
Chief, Network Operations Section

Enclosure

Comments on Technical Committee Findings regarding the Romeoville AVM Stage-area Rating

The following comments are provided after reviewing pages 23-37 of the draft report of the Third Technical Review Committee for Lake Michigan Diversion Accounting. This portion of the report concerns the stage-area rating for the acoustic velocity meter (AVM) on the Chicago Sanitary and Ship Canal (the Canal) at Romeoville, Illinois.

In general, the use of a time-variant stage-area rating technique, as defined by equation 13 on page 27, is a sound practice for most streams in the Midwest. If the physical nature of the canal was not so well-known and the chance of rapid changes in the channel cross-section so small, the time-variant stage-area rating would unquestionably be used at this site. The following subsections discuss (1) the physical characteristics of the Canal, (2) the reasons for the changes in depth correction factors determined from individual discharge measurements, (3) the possibility of a rating bias and the consequences, (4) the cross-section measurements made at Romeoville, and (5) the conclusions that can be made from the above.

Physical characteristics

A substantial portion of the Chicago Sanitary and Ship Canal was cut out of bedrock, including the reach up- and downstream of the AVM. The bedrock cut is more than 40 feet deep near the gage. The cut is rectangular as can be seen from the cross-section plots (figures 1-3). Sediment concentrations in the canal are small compared to nearby rivers and streams. During water years 1987 to 1990, the maximum sediment concentration measured in the canal was 133 milligrams per liter (mg/l), compared to 282 mg/l at Des Plaines River at Riverside, IL, 550 mg/l at Illinois River at Marseilles, and 843 mg/l at Du Page River at Shorewood. Most sediment concentrations measured during this period were less than 50 mg/l. During a recent study of the quality of bed sediments in the Upper Illinois River watershed, bed material samples were obtained from eight sites along the canal. At most of these locations, it was very difficult to get a sample of sufficient size for analysis, and in some cases no sample could be obtained.

Prior to 1989, the AVM transducers had to be serviced using a diver. Several USGS employees, including Jim Duncker, made dives at the AVM. Although visibility is extremely poor, Duncker was able to describe the bottom of the canal based on "feel". The bottom is covered with some loose gravel and pebbles, approximately the size of a quarter or smaller. Duncker believes that the layer of material covering the bedrock is shallow because there is little or no displacement when standing on the bottom. Near the sides, especially at the downstream transducers, there is some material that has accumulated because of slumping of the side walls.

Changing Depth Correction Factors

In the draft report, equation 14 (p. 27) is used to compute a depth correction factor using the known width of the canal (162 feet), the area from a discharge measurement, and the gage height of the measurement. The statement is made that the depth correction factor "defines the bed

elevation at the time of the discharge measurement". This statement would be correct, assuming that stage is constant throughout the measurement. Further, it is important to stress that the bed elevation obtained represents only a single location in the reach, the velocity of which is measured by the AVM. Thus, the bed elevation that is computed using equation 14 is not directly comparable to the mean bed elevation used by the USGS in the slope-area rating that is computed for the reach defined by the upstream and downstream AVM transducer assemblies. Therefore, it is not appropriate to compare a stage-area rating based on discharge measurements at a single, non-constant, location in the reach and a stage-area rating computed for the reach using measured cross-sections at the upstream and downstream AVM transducers and a point halfway between the transducers.

There are also errors in the areas computed for each of the discharge measurements that need to be considered. These errors include equipment error (problem with sounding reel, and others) and human error. While neither of these are expected to be large, they will have an effect on the resulting value computed for the depth correction factor. Examination of depth correction factors for discharge measurements made on the same day (table 1) show that the values for the depth correction factor can vary as much as 0.38 feet from one measurement to the next.

Table 1.-- Discharge measurements made on the same day and the difference between depth correction factors computed for each pair of measurements.

Discharge measurement number	Difference in depth-correction factor (feet)
6-7	0.05
31-32	.30
33-34	.06
47-48	.12
52-53	.12
54-55	.27
56-57	.03
59-60	.38
70-71	.10

With this as a background, an alternative explanation for the apparent changes in the depth-correction factor with time is presented. Figure 10 of the draft report illustrates these changes well. Measurements 1-62 were all made from the upstream side of the Romeoville Road bridge. This approximately coincides with the location of the downstream cross-section. Measurement 67 was made from the Lemont Road bridge approximately 4 miles upstream. Measurements 63-66 and 68 were made using a boat, halfway between the downstream transducer assembly and the midpoint between the upstream and downstream transducer assemblies. Measurements 69-72 were all made halfway between the upstream and downstream transducer assemblies. As the measuring point moved further upstream, the mean depth increased.

This is supported by the discharge measurement results as well as the two sets of cross-section measurements made in June, 1991 and October, 1993. Both of these measurements show that the middle cross-section is the deepest of the three measured. This explains why the depth correction factor appears to increase with time. That is, the depth-correction factor changes with measurement location and measurement location changes with time. Figure 4 (which is a plot of the same data shown in Figure 10 of the committee's report) illustrates this. The mean depth correction factor for measurements 1 to 62 is 0.58 (table 2). This is not much different than the mean depth for the downstream cross-section (measured in June, 1991) of 0.41 feet. The mean depth-correction factor for measurements 69 to 72 is 2.10, which compares well to the mean depth for the middle cross-section (measured in June, 1991) of 2.09 feet.

Table 2.-- Mean depth correction factors for the corresponding set of discharge measurements.

Measurement numbers	1-62	63-72	63-68	69-72
Mean depth correction factor (feet)	0.58	2.09	1.41	2.10

Based on the discharge measurement data alone, it is evident that attributing the apparent change in the depth-correction factor over time to scour and fill is inappropriate. Rather most of the changes can be explained when the measurement location is taken into account. Furthermore, it is highly unlikely that significant scour and fill is taking place in the Canal, especially when one considers the physical conditions found in the Canal. Finally, it is also important to emphasize corrections probably should not be made to the AVM record based solely on data for one cross-section. Cross-sectional area from current-meter measurements can be used to indicate whether or not large changes in the area have occurred. However, they are only an indication, and should not be used to develop a stage-area rating for this station.

Fixed Rating Bias

The committee report states that use of a single, fixed rating for the entire period of record introduces a positive bias in the computation of flow areas. The possibility of such a bias is discussed in this section.

The single rating (Area=162*(Stage+1.55)) is used to compute the flow area for all of the AVM discharge records. It is assumed that little or no change in this relation occurs; this, however, is not very likely. Therefore, three questions may be asked. First, how much change is significant? Second, if a significant change is observed, what method should be used to phase in the changed stage-area rating? Finally, can one actually be sure that a positive bias results from use of the current rating, by comparing the rating to current-meter measurements?

It is possible that the use of a fixed rating could introduce a significant bias in the area obtained from the stage-area rating, if significant changes in the channel cross section were taking place over time. Although such changes seem rather doubtful at present, it is prudent to (1) determine whether the channel is changing significantly over time and (2) consider how to correct for any observed changes in the channel cross-section. At present, there is no method for phasing in a change in the stage-area rating over time (time shift). This is something that should undoubtedly

be pursued. In order to address (1) above, cross-section measurements should be made at three cross-sections (minimum) once per year. In addition, a method for extracting depth data from acoustic Doppler current profiler (ADCP) data collected at Romeoville should be developed. These data could be used to determine when any large changes in the measuring section have taken place.

Because discharge measurements are made at only one location (cross-section), it is doubtful that results (such as the average depth or depth correction factor) can be extrapolated to the other cross-sections defined by the two main transducer assemblies. The depth of the channel bottom varies significantly from the downstream to the upstream transducer assemblies (tables 3 and 4).

If a positive bias did exist, it will not cause a bias in discharges computed for the entire period, as stated in the committee report. Discharges for the 1991 and 1992 water years were computed using the index-velocity method. In this method, a bias in the stage-area rating will not affect the discharges computed.

Cross-section measurements

Two sets of cross-section measurements have been made in the Canal at Romeoville, one set in June, 1991 and a second in October, 1993. Cross-sections were measured using a boat and sounding weight attached to a tagline stretched across the Canal at the upstream transducer assembly, the downstream transducer assembly, and halfway in between. While soundings were made, the base gage (staff gage) was read at regular intervals. Both sets of measurements were made in the same locations. Results of these cross-section measurements are shown in tables 3 and 4 and figures 1-3.

Based on these measurements, the mean bed elevation has decreased by 0.17 feet. This reflects a change of less than one percent of the depth at normal stage (25.50 feet). It is doubtful that this change is significant and no doubt reflects primarily the error in making such a cross-section measurement. No change in the stage-area rating should be made unless similar results can be obtained from a repeat measurement. The upstream cross-section changed the most, with the mean bottom elevation decreasing by 0.35 feet (tables 3 and 4). However, the October, 1993 measurements show that the shape of the cross-section has changed little in the two years between cross-section measurements (figs. 1 to 3).

Conclusions

Upon consideration of the physical characteristics of the Canal, the changes in measurement location with time, and the lack of a large change in the channel cross-section since 1991, the following conclusions are made.

1. There is little physical evidence to support the assumption that the bed elevation at Romeoville is changing in time because of scour and fill.

2. It is not appropriate to directly compare a stage-area rating developed from discharge measurements taken at one, non-constant location to a stage-area rating based on the mean bed elevation between the upstream and downstream transducer assemblies. The discharge measurements can, however, be used to approximately verify the stage-area rating developed using the measured channel cross-sections.

3. The depth-correction factor computed using equation 14 in the draft committee report verifies the stage-area rating developed using the mean bed elevation between transducer assemblies, when the changes in measurement location are accounted for.

4. The cross-sectional area computed for the discharge measurement often does not account for changes in stage throughout the measurement.

5. Channel cross-sections measured in October, 1993 show that little change in the three cross-sections has occurred. This is significant in that the flow in the Canal was much higher than normal for an extended period in the spring of 1992.

6. Channel cross-section measurements should be made each year in the Canal at Romeoville. This should be incorporated into the QA plan for the gaging station.

7. A method for phasing in significant changes in the stage-area rating should be developed and used as appropriate.

8. Discharges for the 1991 and 1992 water years were computed using the index-velocity method. In this method, a bias in the stage-area rating will not affect the discharges computed.

Figure 1.--Cross-section at upstream transducer assembly, Romeoville, IL

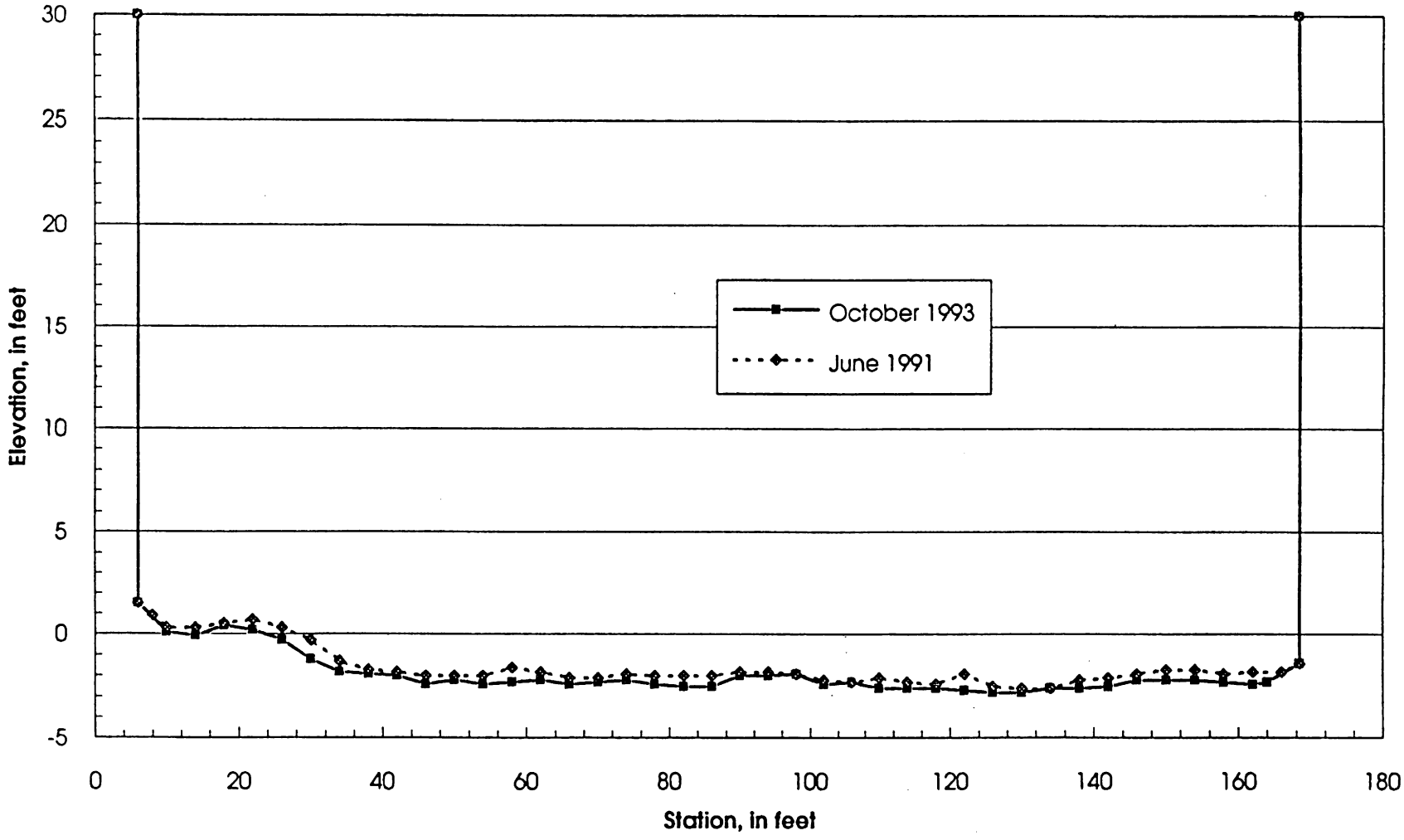
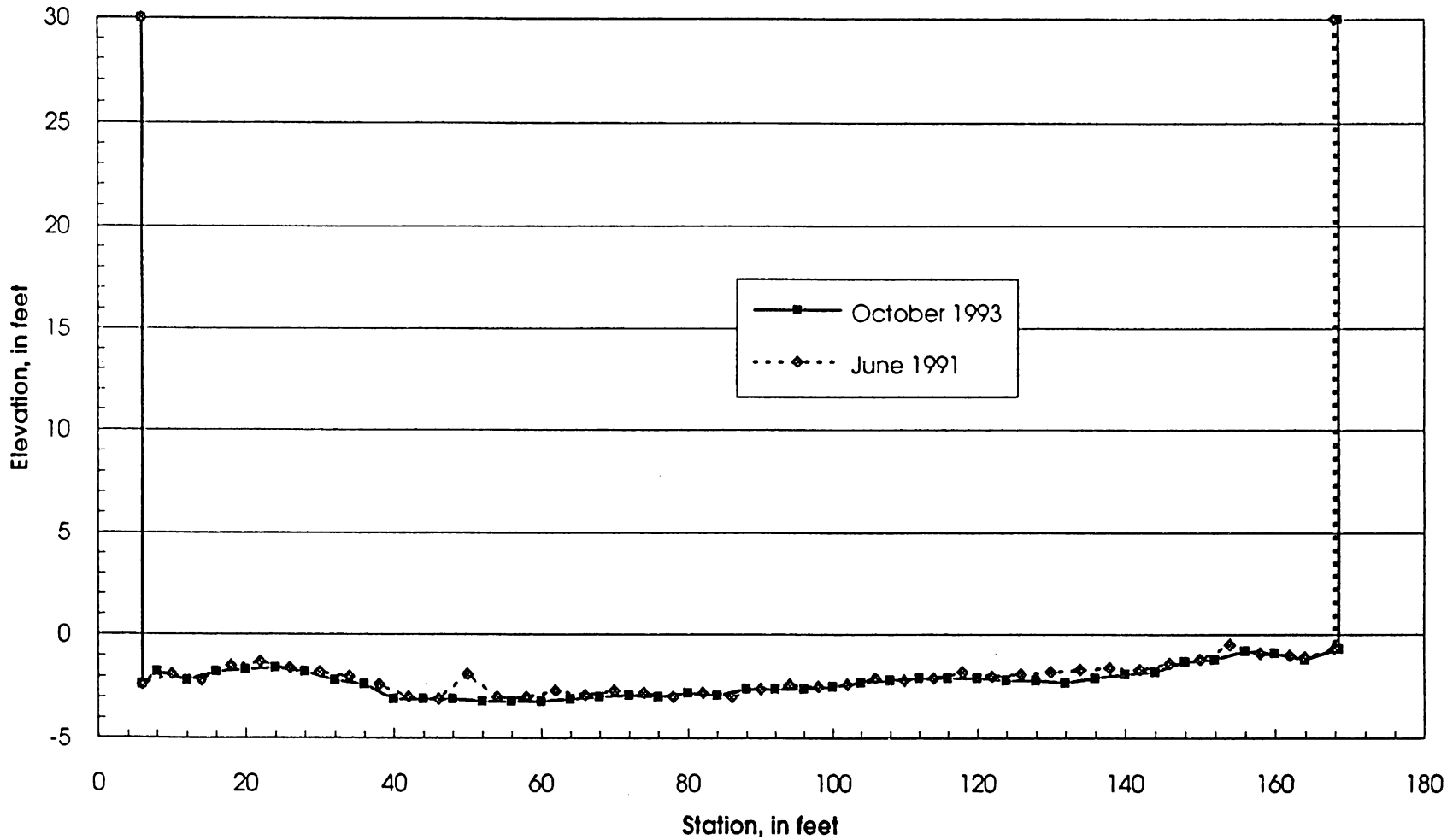


Figure 2.--Cross-section midway between upstream and downstream transducer assemblies, Romeoville, IL



F e 1

Figure 3.--Cross-section at downstream transducer assembly, Romeoville, IL

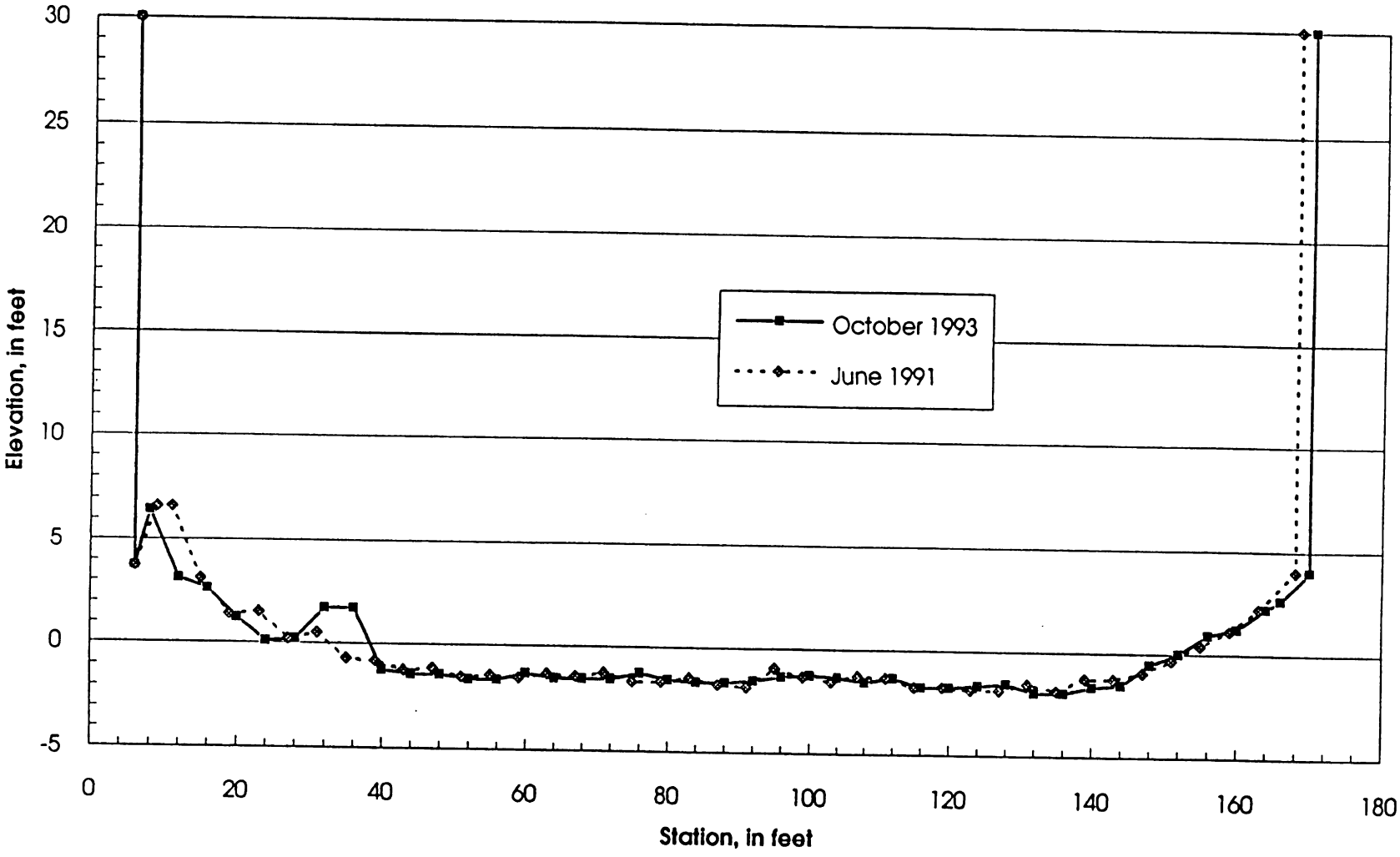


Figure 4.--Depth Correction Factor Computed for Discharge Measurements made at AVM at Romeoville, Illinois.

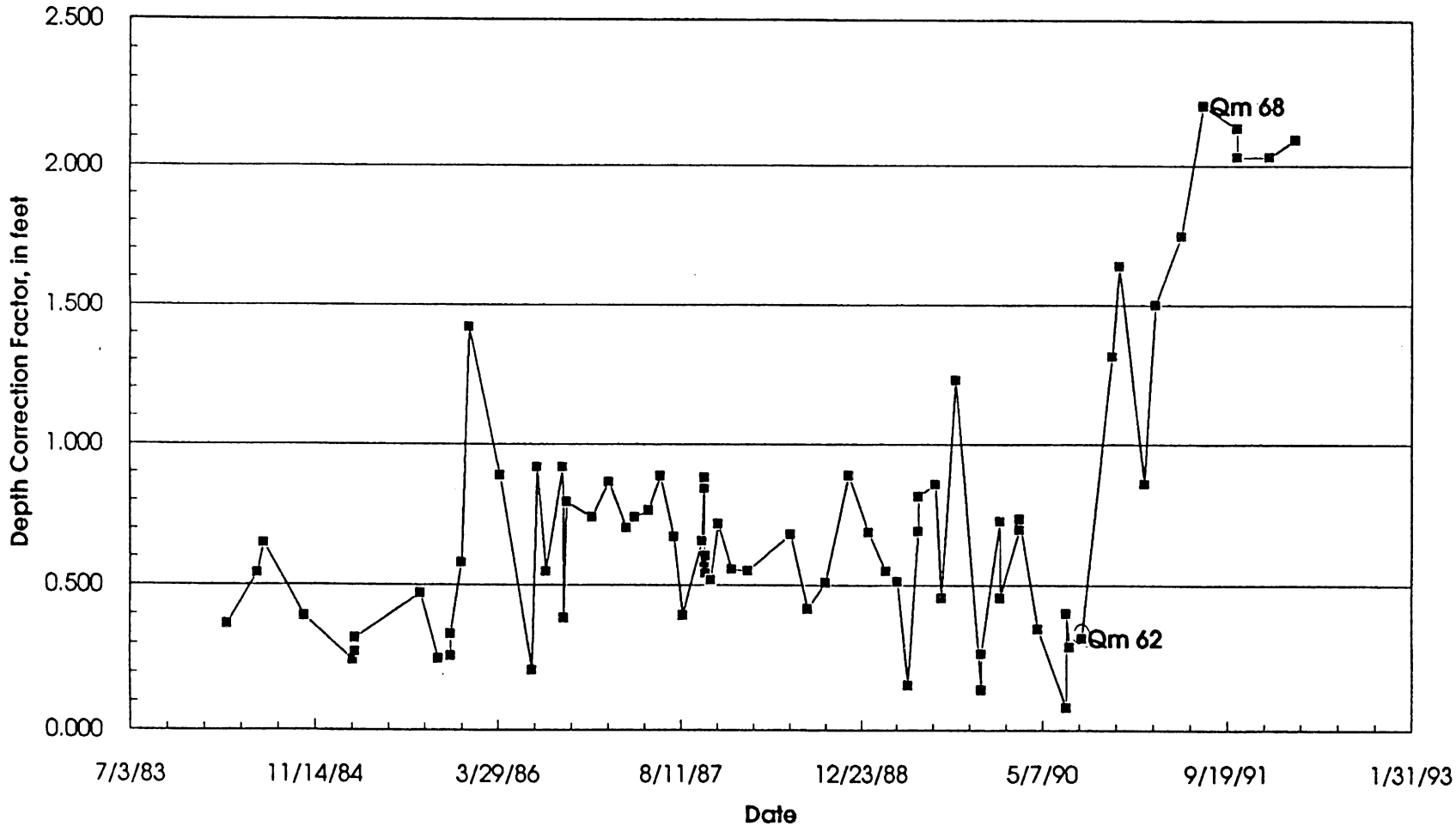


Table 3.–Calculated bottom-elevation data for the Chicago Sanitary and Ship Canal elevations measured at Romeoville, Ill., June 4 and 5, 1991

Upstream				Middle				Downstream			
		Average				Average				Average	
Station	Elevation	Depth	Area	Station	Elevation	Depth	Area	Station	Elevation	Depth	Area
6	30			6	30			6	30		
6	1.5			6	-2.4			6	3.7		
8	0.9	1.2	2.4	10	-1.9	-2.15	-8.6	9	6.6	5.15	15.45
10	0.3	0.6	1.2	14	-2.2	-2.05	-8.2	11	6.6	6.6	13.2
14	0.3	0.3	1.2	18	-1.5	-1.85	-7.4	15	3.1	4.85	19.4
18	0.5	0.4	1.6	22	-1.3	-1.4	-5.6	19	1.4	2.25	9
22	0.7	0.6	2.4	26	-1.6	-1.45	-5.8	23	1.5	1.45	5.8
26	0.3	0.5	2	30	-1.8	-1.7	-6.8	27	0.2	0.85	3.4
30	-0.3	0	0	34	-2	-1.9	-7.6	31	0.5	0.35	1.4
34	-1.3	-0.8	-3.2	38	-2.4	-2.2	-8.8	35	-0.7	-0.1	-0.4
38	-1.7	-1.5	-6	42	-3	-2.7	-10.8	39	-0.8	-0.75	-3
42	-1.8	-1.75	-7	46	-3.1	-3.05	-12.2	43	-1.2	-1	-4
46	-2	-1.9	-7.6	50	-1.9	-2.5	-10	47	-1.1	-1.15	-4.6
50	-2	-2	-8	54	-3	-2.45	-9.8	51	-1.5	-1.3	-5.2
54	-2	-2	-8	58	-3	-3	-12	55	-1.4	-1.45	-5.8
58	-1.6	-1.8	-7.2	62	-2.7	-2.85	-11.4	59	-1.5	-1.45	-5.8
62	-1.8	-1.7	-6.8	66	-2.9	-2.8	-11.2	63	-1.3	-1.4	-5.6
66	-2.1	-1.95	-7.8	70	-2.7	-2.8	-11.2	67	-1.4	-1.35	-5.4
70	-2.1	-2.1	-8.4	74	-2.8	-2.75	-11	71	-1.2	-1.3	-5.2
74	-1.9	-2	-8	78	-3	-2.9	-11.6	75	-1.6	-1.4	-5.6
78	-2	-1.95	-7.8	82	-2.8	-2.9	-11.6	79	-1.6	-1.6	-6.4
82	-2	-2	-8	86	-3	-2.9	-11.6	83	-1.4	-1.5	-6
86	-2	-2	-8	90	-2.6	-2.8	-11.2	87	-1.7	-1.55	-6.2
90	-1.8	-1.9	-7.6	94	-2.4	-2.5	-10	91	-1.8	-1.75	-7
94	-1.8	-1.8	-7.2	98	-2.5	-2.45	-9.8	95	-0.9	-1.35	-5.4
98	-1.9	-1.85	-7.4	102	-2.4	-2.45	-9.8	99	-1.3	-1.1	-4.4
102	-2.2	-2.05	-8.2	106	-2.1	-2.25	-9	103	-1.5	-1.4	-5.6
106	-2.3	-2.25	-9	110	-2.2	-2.15	-8.6	107	-1.2	-1.35	-5.4
110	-2.1	-2.2	-8.8	114	-2.1	-2.15	-8.6	111	-1.3	-1.25	-5
114	-2.3	-2.2	-8.8	118	-1.8	-1.95	-7.8	115	-1.7	-1.5	-6
118	-2.4	-2.35	-9.4	122	-2	-1.9	-7.6	119	-1.7	-1.7	-6.8
122	-1.9	-2.15	-8.6	126	-1.9	-1.95	-7.8	123	-1.8	-1.75	-7
126	-2.5	-2.2	-8.8	130	-1.8	-1.85	-7.4	127	-1.8	-1.8	-7.2
130	-2.6	-2.55	-10.2	134	-1.7	-1.75	-7	131	-1.5	-1.65	-6.6
134	-2.6	-2.6	-10.4	138	-1.6	-1.65	-6.6	135	-1.8	-1.65	-6.6
138	-2.2	-2.4	-9.6	142	-1.7	-1.65	-6.6	139	-1.2	-1.5	-6
142	-2.1	-2.15	-8.6	146	-1.4	-1.55	-6.2	143	-1.2	-1.2	-4.8
146	-1.9	-2	-8	150	-1.2	-1.3	-5.2	147	-0.9	-1.05	-4.2
150	-1.7	-1.8	-7.2	154	-0.5	-0.85	-3.4	151	-0.3	-0.6	-2.4
154	-1.7	-1.7	-6.8	158	-0.9	-0.7	-2.8	155	0.4	0.05	0.2
158	-1.9	-1.8	-7.2	162	-1	-0.95	-3.8	159	1.1	0.75	3
162	-1.8	-1.85	-7.4	164	-1.1	-1.05	-2.1	163	2.2	1.65	6.6
166	-1.8	-1.8	-7.2	168	-0.7	-0.9	-3.6	168	4	3.1	15.5
168.5	-1.4	-1.6	-4	168	30			168	30		
168.5	30										
Mean Depth			Total	Mean Depth			Total	Mean Depth			Total
-1.609			-261.4	-2.087			-338.1	-0.411			-66.65

Average Depth for the three cross-sections -1.549

Table 4.–Calculated bottom-elevation data for the Chicago Sanitary and Ship Canal elevations measured at Romeoville, Ill., October, 12, 1993

Upstream				Middle				Downstream			
		Average				Average				Average	
Station	Elevation	Depth	Area	Station	Elevation	Depth	Area	Station	Elevation	Depth	Area
6	30			6	30			6	30		
6	1.5			6	-2.4			6	3.7		
10	0.1	0.8	3.2	8	-1.8	-2.1	-4.2	8	6.4	5.05	10.1
14	-0.1	0	0	12	-2.2	-2	-8	12	3.1	4.75	19
18	0.4	0.15	0.6	16	-1.8	-2	-8	16	2.6	2.85	11.4
22	0.2	0.3	1.2	20	-1.7	-1.75	-7	20	1.2	1.9	7.6
26	-0.3	-0.05	-0.2	24	-1.6	-1.65	-6.6	24	0.1	0.65	2.6
30	-1.2	-0.75	-3	28	-1.8	-1.7	-6.8	28	0.2	0.15	0.6
34	-1.8	-1.5	-6	32	-2.2	-2	-8	32	1.7	0.95	3.8
38	-1.9	-1.85	-7.4	36	-2.4	-2.3	-9.2	36	1.7	1.7	6.8
42	-2	-1.95	-7.8	40	-3.1	-2.75	-11	40	-1.2	0.25	1
46	-2.4	-2.2	-8.8	44	-3.1	-3.1	-12.4	44	-1.4	-1.3	-5.2
50	-2.2	-2.3	-9.2	48	-3.1	-3.1	-12.4	48	-1.4	-1.4	-5.6
54	-2.4	-2.3	-9.2	52	-3.2	-3.15	-12.6	52	-1.6	-1.5	-6
58	-2.3	-2.35	-9.4	56	-3.2	-3.2	-12.8	56	-1.6	-1.6	-6.4
62	-2.2	-2.25	-9	60	-3.2	-3.2	-12.8	60	-1.3	-1.45	-5.8
66	-2.4	-2.3	-9.2	64	-3.1	-3.15	-12.6	64	-1.5	-1.4	-5.6
70	-2.3	-2.35	-9.4	68	-3	-3.05	-12.2	68	-1.5	-1.5	-6
74	-2.2	-2.25	-9	72	-2.9	-2.95	-11.8	72	-1.5	-1.5	-6
78	-2.4	-2.3	-9.2	76	-3	-2.95	-11.8	76	-1.2	-1.35	-5.4
82	-2.5	-2.45	-9.8	80	-2.8	-2.9	-11.6	80	-1.5	-1.35	-5.4
86	-2.5	-2.5	-10	84	-2.9	-2.85	-11.4	84	-1.6	-1.55	-6.2
90	-2	-2.25	-9	88	-2.6	-2.75	-11	88	-1.6	-1.6	-6.4
94	-2	-2	-8	92	-2.6	-2.6	-10.4	92	-1.5	-1.55	-6.2
98	-1.9	-1.95	-7.8	96	-2.6	-2.6	-10.4	96	-1.3	-1.4	-5.6
102	-2.4	-2.15	-8.6	100	-2.5	-2.55	-10.2	100	-1.2	-1.25	-5
106	-2.3	-2.35	-9.4	104	-2.3	-2.4	-9.6	104	-1.3	-1.25	-5
110	-2.6	-2.45	-9.8	108	-2.2	-2.25	-9	108	-1.5	-1.4	-5.6
114	-2.6	-2.6	-10.4	112	-2.1	-2.15	-8.6	112	-1.3	-1.4	-5.6
118	-2.6	-2.6	-10.4	116	-2.1	-2.1	-8.4	116	-1.7	-1.5	-6
122	-2.7	-2.65	-10.6	120	-2.1	-2.1	-8.4	120	-1.7	-1.7	-6.8
126	-2.8	-2.75	-11	124	-2.2	-2.15	-8.6	124	-1.6	-1.65	-6.6
130	-2.8	-2.8	-11.2	128	-2.2	-2.2	-8.8	128	-1.5	-1.55	-6.2
134	-2.6	-2.7	-10.8	132	-2.3	-2.25	-9	132	-1.9	-1.7	-6.8
138	-2.6	-2.6	-10.4	136	-2.1	-2.2	-8.8	136	-1.9	-1.9	-7.6
142	-2.5	-2.55	-10.2	140	-1.9	-2	-8	140	-1.6	-1.75	-7
146	-2.2	-2.35	-9.4	144	-1.8	-1.85	-7.4	144	-1.5	-1.55	-6.2
150	-2.2	-2.2	-8.8	148	-1.3	-1.55	-6.2	148	-0.5	-1	-4
154	-2.2	-2.2	-8.8	152	-1.2	-1.25	-5	152	0	-0.25	-1
158	-2.3	-2.25	-9	156	-0.8	-1	-4	156	0.9	0.45	1.8
162	-2.4	-2.35	-9.4	160	-0.9	-0.85	-3.4	160	1.2	1.05	4.2
164	-2.3	-2.35	-4.7	164	-1.2	-1.05	-4.2	164	2.2	1.7	6.8
168.5	-1.4	-1.85	-8.325	168.5	-0.7	-0.95	-4.275	166	2.6	2.4	4.8
168.5	30			168.5	30			170	4	3.3	13.2
								170	30		
Mean Depth			Total	Mean Depth			Total	Mean Depth			Total
-1.955			-317.6	-2.258			-366.9	-0.422			-67.5

Average Depth for the three cross-sections -1.723

APPENDIX 7

Quality Assurance Plan for the Acoustic Velocity Meters in the Chicago Sanitary and Ship Canal at Romeoville, Illinois.

PURPOSE AND SCOPE

The purpose of this quality assurance plan is to provide procedures for the standardization and documentation of streamflow record for the Acoustic Velocity Meter (AVM) on the Chicago Sanitary and Ship Canal at Romeoville, Illinois, also referred to as Romeoville. Procedures will be presented for the collection, analysis, computation, and review of AVM records for Romeoville.

This plan is to be used as a guide to aid personnel responsible for AVM site inspections, discharge measurements, data collection and analysis of records at the AVM at Romeoville. The plan describes procedures to be followed and steps to be taken to insure the quality of the record for this site.

DOCUMENTATION

A log book will be maintained in the gage house to document all activities related to the operation and maintenance of the AVM. All problems, changes to equipment or procedures, observations, and conversations with colleagues and representatives from other agencies and manufacturers which relate to the operation of the AVM will be noted.

GAGE INSPECTIONS

During visits to the AVM, the following tasks should be performed and recorded in the log book.

1. All stage-gage readings shall be read and recorded - (wire-weight, manometer, float and AVM depth gage) and readings shall be read before, after, and at 2 minute intervals during each discharge measurement.
2. On a quarterly basis, 15-second AVM velocity and stage readings shall be recorded on the Decwriter for a period of five minutes. The results of this test including the number of successful readings for each path shall be recorded in the log book. This test will insure that consistent and reasonable velocities and stage readings are being recorded by the AVM.
3. All parameters (data stored in the AVM memory and related to path lengths, transducers, elevations, channel widths), shall be read and checked against a master list, which is kept in the gagehouse on a quarterly basis. The results of this check shall be recorded in the logbook. Under NO circumstances should these parameters be changed. These parameters relate to the specific physical characteristics of the site.
4. The 15-minute data, which is recorded on the Decwriter, should be scanned to insure readings are reasonable and path velocities are not anomalous.

DISCHARGE MEASUREMENTS

1. The project chief shall be in complete charge of all aspects of the discharge measurements. The main responsibility of the project chief during the measurement shall be to read and record the outside gage, manometer, float and the AVM gage heights every two minutes. There will be a pre-measurement meeting so that all personnel involved will know what to do and when to do it.

2. Discharge measurements will be made at the rate of 6 per year and should cover the range in discharge experienced at the site during the year.

3. Discharge measurements should be made during periods of minimal changes in discharge (i.e. minimal barge/boat traffic and changes in the gate openings at the dam 3 miles downstream). Possible variations in the flow should be checked by telephoning the Metropolitain Water Reclamation District of Greater Chicago (MWRDGC) Supervising Civil Engineer, Bill Eyre, of the General Division for Operation and Maintenance (312-751-5102).

4. Measurement time shall be minimized by having two boat crews measuring at the same time, starting from opposite ends of the section. Using a digitizer, full 40 second velocity reading times shall be used. Also, each boat crew shall have a working headset and appropriate current meter rating table as a backup to digitizer failure. The minimum sounding weight shall be 50 lbs and a 75 lbs sounding weight shall be in the boat in case it is needed. In case of high flow, measurements shall be made from the Lemont Road bridge (next bridge upstream) and shall use 100 lbs sounding weights.

5. Twenty-four verticals at predesignated stationing, with readings at .2, .6 and .8 will be made for each measurement. Starting and ending times shall be recorded on the measurement sheet for each vertical. Each vertical shall be complete and uninterrupted. In case of equipment failure or boat traffic that vertical will be restarted. The boat tagline shall be secured the same way each time (6 ft station at REW) so that the twenty-four verticals will be taken at the same locations each time discharge measurements are made. The twenty-four verticals will be taken at the following predetermined stations:

6	32	60	88	116	148
10	39	67	95	123	158
18	46	74	102	130	163
25	53	81	109	138	168

note: Stationing is in feet from the right edge of water at the canal wall (station 6 ft is located at REW and station 168 ft at LEW).

The equation for the mean velocity for each section is as follows:

$$v(avg) = [((v(.2) + v(.8)) / 2) + v(.6)] / 2$$

6. Appropriate information on the measurement face sheet for each crew shall be filled out (Susp., method coef., Hor. angle coef., Susp. coef., meter no., type of meter, meter ___ ft. above bottom of weight., spin test info, measurement rating, cross section info, flow, comp. by _____, and any additional remarks).

7. The AVM discharge during the discharge measurement shall be weighted based on data recorded during the discharge measurement. Intermediate watch times (ending time) at each measurement station shall be recorded on the measurement note sheet. One minute AVM stage, velocity, and discharge readings shall be recorded on the Decwriter. These recorded values shall then be used to compute the weighted AVM discharge using the following formula:

$$\text{Weighted AVM Q} = \text{sum}[\ Q_{meas} \times Q_{avm}]/Q_{meas(total)}$$

Where: Q_{meas} = the discharge measured in each sub-area(section).
Q_{avm} = the average AVM discharge recorded during the time that the section was measured.
$Q_{meas(total)}$ = the total discharge measured.

8. ALL measurements shall be checked against the AVM weighted discharge and if there is a discrepancy of more than five percent a complete check measurement will be made.

9. Discharge measurements can also be made using an acoustical doppler meter from one boat pulled across canal attached to tagline as mention above. This will become the preferred way to make measurements because one measurement can be made in 4 minutes, therefore a series of measurements can be made in the same time a single conventional boat measurement can be made. The tagline may not be used if the project chief deems it to be unsafe or if accurate measurements may be made without the aid of the tagline. The tagline will only be absolutely necessary during measurements of very low flows.

No field personnel shall leave the measuring stie until the project chief verifies that a check measurement is not required.

ROUTINE MAINTENANCE

1. Routine station levels shall be run every two years.
2. Levels shall be run to the transducers every two years or if changes to the transducer locations are made. Level notes shall include all relevant information including tape downs from the reference mark on the transducer mounting bracket to any reference mark on the PVC pipe and/or to each transducer. If any relevant measurements were not made at the time of the level run then the location where this information can be found shall be referenced in the level notes.
3. Transducer path lengths and angles shall be measured and remeasured if physical changes to the transducer locations are made.
4. Voltage on memory circuit boards (Ram and R. T. clock) shall be checked annually.
5. The AVM shall be serviced by the manufacturer annually as part of a service contract.
6. Telemetry (telephone) shall be operated and maintained to facilitate reception of daily real-time data and to minimize AVM problems.
7. The Sutron DCP will be operated and maintained by the Rock Island District (RID) Corps of Engineers. Any identified problems with the DCP or DCP data will be relayed to RID (Bill McCutcheon).

DATA ANALYSIS

1. The AVM data being recorded by the CR10 shall be entered on the computer and checked for anomalies.
2. Velocity and depth unit values shall be graphically plotted to identify possible anomalous data.
3. Discharge measurements shall be furnished to the cooperator (CHI - COE) on a monthly basis.
4. Official discharge records shall be computed using a velocity-velocity rating and a stage-area rating in ADAPS. The stage-area and velocity-velocity ratings are given by the following equations:

$$\text{Channel Flow Area} = (\text{GH} \times 162) + 251.0$$

$$\text{Mean Channel Vel.} = 0.92 \times (\text{Mean AVM Vel.})$$

where:

GH = gage height, in feet

Mean AVM Vel. = the mean AVM velocity if all four paths are working or
= (Vel-U + Vel-M + Vel-X + Vel-L)/4

note: If all four paths are not working then the mean AVM velocity is computed by first correcting each working path velocity by the appropriate (Mean AVM Vel)/(path velocity) ratio and then taking their arithmetic mean. The correction ratios are listed below:

(Mean AVM Vel)/(Vel-U) = 0.967
(Mean AVM Vel)/(Vel-M) = 0.990
(Mean AVM Vel)/(Vel-X) = 1.003
(Mean AVM Vel)/(Vel-L) = 1.049

5. On at least an annual basis, the relation between measured discharge and the AVM weighted discharge shall be examined to ensure that there is no consistent bias.
6. An extensive and detailed station analysis shall be written every water year.

MISSING AVM RECORD

Missing record shall be estimated by using regression equations based on MWRDGC-reported flows at Lockport. The regression equations used to estimate missing record are documented in the report "Discharge and Regression Analyses for Acoustic Velocity Meter Data for Chicago Sanitary and Ship Canal at Romeoville, Illinois", by C.S. Melching and K.A. Oberg.

RECORD REVIEW

1. The AVM discharge records shall be reviewed annually by an independent district reviewer.

2. A comparison shall be made annually between the AVM computed daily discharges and those estimated using the regression with Lockport records.

3. Because this discharge record is being computed to meet specific legal requirements, the agencies involved shall meet annually to review and agree upon the data. A technical committee composed of representatives from USGS, CHI-COE, and IDOT-DWR shall review and document this record annually.

Written by: D.A. Stedfast, August 15, 1991
Checked by: D.P. Morgan, August 15, 1991
Revised by: D.P. Morgan, May 19, 1993

APPENDIX 8

Lake Michigan Diversion Accounting Third Technical Committee Workshop Number 1 Tentative Agenda

US Army Corps of Engineers, Chicago District
Suite 600, 111 North Canal Street
Chicago IL

Monday, 22 February 1993

1 to 4 pm NCC and Committee meeting in Engineering Conference Room
- Introductions to District Engineer, Chief of Engineering Division, and others
- Review of Scope of Work
- Goals
- Discussion

Tuesday, 23 February 1993

Field trip to AVM, WSW treatment plant, McCook Quarry, Mainstream Pump Station, and other sites

Wednesday, 24 February 1993

Presentations by other interested parties in Main Conference Room

8:30 am Welcome by DE, Introductions
9:00 am Overview of Diversion Accounting
9:30 am Illinois State Water Survey
10:00 am Break
10:15 am Metropolitan Water Reclamation District of Greater Chicago
11:30 am Illinois Department of Transportation
noon Lunch Break
1:00 pm United States Geologic Survey
3:00 pm Discussions

Thursday, 25 February 1993

Morning field trip to Des Plaines pump station, O'Hare TARP, and Chicago Lock

Afternoon presentation on diversion accounting procedures in Engineering Conference Room

1:00 pm Data sources and problems
1:45 pm Modeling procedures
3:30 pm Discussions

Friday, 26 February 1993

Meeting of Committee and NCC for review and summary with scheduling of topics for Workshop Number 2 in Engineering Conference Room

Lake Michigan Diversion Accounting
Third Technical Committee
Second Workshop
Tentative Schedule

Monday, 3 May 1993

1:00 pm	Discussion of Agenda Engineering Division Conference Room
1:30	Lakefront Leakage Measurements, Kevin Oberg USGS Engineering Division Conference Room
4:00 pm	Adjourn for day

Tuesday, 4 May 1993

9:00 am	Leave Chicago District offices for Chicago Lock
9:15 am	Arrive Chicago Lock
	Demonstration of Acoustic Doppler Current Profiler (ADCP) flow meter and dye study at Chicago Lock
	Leave Chicago Lock for Chicago District offices when Committee satisfied.
	Adjourn for day

Wednesday, 5 May 1993

8:00 am	Discussion of future lakefront leakage measurements (if not covered on Tuesday) Planning Division Conference Room
10:00 am	Status of Second Technical Committee recommendations Planning Division Conference Room
11:30 am	Lunch
12:30 pm	Review of Diversion Accounting Process Engineering Division Conference Room
4:00 pm	Adjourn for day

Thursday, 6 May 1993

8:00 am	Review of Diversion Accounting Process Planning Division Conference Room
11:30 am	Lunch
12:30 pm	Review of Diversion Accounting Process Engineering Division Conference Room
3:30 pm	Scheduling of Third Workshop Engineering Division Conference Room
4:00 pm	Adjourn until next workshop

Lake Michigan Diversion Accounting
Third Technical Committee
Schedule for Third Workshop

Monday, 19 July 1993
1:00 PM Discussion of week's events.
1:30 PM Discuss events since last meeting.
2:00 PM Discuss technical issues raised by the committee.
3:30 PM Adjourn.

Tuesday, 20 July 1993
8:30 AM Trip to Wilmette.
Visit a new precipitation gage site.
11:30 AM Lunch on the road.
12:30 PM Trip to O'Brien Lock.
3:30 PM Adjourn.

Wednesday, 21 July 1993
8:00 AM Discuss technical issues (as listed by Committee).
11:30 AM Lunch
12:30 PM Continue technical discussion, if needed.
Discuss issues (QA and Master Plan).
3:30 PM Adjourn.

Thursday, 22 July 1993
8:00 AM USGS discussion of Chicago Lock flow measurements.
Discussion of Des Plaines flow measurements.
Discussion of AVM index velocity & rating.
11:45 AM Lunch
12:30 PM Summary and wrap-up.
1:00 PM Technical Committee (members only) meeting

Schedule for Meeting Rooms

Monday, 19 July 1993		
	Engineering Conf. Room	10:30 am to 4:30 pm
Tuesday, 20 July 1993		
	Engineering Conf. Room	7:30 am to 9 am
	Rental van.	8 am to 5 pm
Wednesday, 21 July 1993		
	Main Conf. Room	7:30 am to 12:30 pm
	Engineering Conf. Room	12:30 pm to 4:30 pm
Thursday, 22 July 1993		
	Main Conf. Room	7:30 am to 11:30
	Engineering Conf. Room	Noon to 4:30 pm

6636 2417 1
09•07•99 MAB
ICI INFORMATION CONSERVATION