TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ACKNOWLEDGEMENTS

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EXECUTIVE SUMMARY

The Fourth Technical Committee was appointed by the USACE to conduct an assessment and evaluation of the accounting procedures and methodology used in the determination of diversion from Lake Michigan and to 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 appointed every five years, and a report evaluating the accounting and operational procedures is to be presented to the USACE and to other interested parties.

The Fourth Technical Committee was appointed by the USACE in July of 1998 and convened in September of 1998. The Committee was appointed to conduct a comprehensive review of the current diversion accounting procedures. The review included the following: 1.) current accounting results; 2.) current diversion related measurement techniques at the Romeoville acoustic velocity meter (AVM) site, the Lockport control structures, precipitation gages, and other pertinent structures; 3.) procedures used to calculate and verify flows that are not directly measured; and 4.) status of recommendations from previous committees. In addition, the technical adequacy of the lakefront AVMs was reviewed as well as the overall measurement techniques employed at the various water supply treatment plants and pumping stations. Determination of the adequacy of existing accounting procedures were made in accordance with the stipulations of the 1967 Supreme Court decree with the 1980 modifications. The Committee recognized that, because the State of Illinois has exceeded Lake Michigan diversion limits as stipulated by the 1967 Supreme Court and modified in 1980, efforts have been initiated since December of 1995 to mediate a resolution. The draft Great Lakes Mediation Memorandum of Understanding, (July 29, 1996) sets forth a transition period of Lake Michigan diversion accounting and a shift to lakefront diversion measurement and accounting. The Committee recognized the critical importance of various technical and accounting issues in shifting to the lakefront accounting for Lake Michigan diversion; and therefore has reviewed various lakefront measurement and accounting issues as a integral part of our review and evaluation in an attempt to resolve some of these issues in terms of recommendations and possibly assist in the resolution of these issues in the mediation process.

In general, the Fourth Technical Committee has found, based on our review, that the Lake Michigan Diversion Accounting is in compliance with the 1980 Modified Decree, with respect to the "best current engineering practices and scientific knowledge." The committee also acknowledges that the USACE has made significant progress in implementing many of the Third Technical Committee's recommendations. This Committee is pleased by the improvements in the accounting procedure, particularly in the quality of the AVM records. 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 Third Technical Committee. In most instances, actions have been taken to comply with the recommendations and significant progress has been made since the Third Committee.

The Committee identified some documentation of the procedures for diversion accounting that needs to be updated and finalized, as was recommended by previous committees. The quality-assurance plan (draft 1988) needs to be updated to reflect current procedures and finalized. This needs to include documentation, review, and verification of measurements, other information used in accounting, data provided by other agencies, simulation results, budgets and balances among components of diversion accounting, and final reports. The Lake Michigan Diversion Accounting Draft Manual of Procedures (1998a) needs to be updated to reflect current procedures and finalized. This should include use of template files with fields to be updated set to the missing data flag and description of the manual and automated quality-assurance checks from the quality-assurance plan.

Annual Report and Diversion Accounting Report Status

The 1993 Annual Report (USACE, 1994a) contained the water-year[1] 1990 (WY 1990) accounting year and was publicly released in September 1994. Modifications were made to the hydrology runoff model, the hydraulic sewer-routing models and the 25-gage precipitation networking incorporated into WY90 diversion accounting. The 1994 Annual Report (USACE, 1995) containing WY 1991 and WY 1992 accounting years was publicly released in October 1995. Modifications were made to the hydrologic models and the hydraulic sewer model in order to utilize the data storage system (DSS) database (USACE, 1994b). The 1995 Annual Report (USACE, 1997) containing WY 1993 and WY 1994 accounting years was publicly released in March 1997. Modifications were made to: (1) Des Plaines Tunnel and Reservoir Plan (TARP) system modeling, (2) deductible water supply from the Grand Calumet River, and (3) modeling of runoff from the ungaged Calumet watershed and these were incorporated into the WY 1993 and WY 1994 diversion accounting. The 1996 Annual Report (USACE, 1998b) containing the WY 1995 accounting year was publicly released in October 1998. During 1996 the USACE supported the Great Lakes Mediation Committee, and performed additional investigations of runoff and consumptive use.

Lake Michigan Diversion Accounting Modeling

The modeling procedures are essentially unchanged since 1995. There still are shortcomings calibrating the hydrologic model and the Special Contributing Area Loading Program (SCALP) model is still an empirical procedure. The quality of the flow data at the Des Plaines Pump Station, which could be an important calibration point, has not been improved.

One major change in the system since 1993, which has not been addressed is the gate operation in the model. The Des Plaines tunnel has been added to the TARP System over the past four years. The Des Plaines tunnel adds a significant volume of runoff from the Des Plaines Basin that is deducted from the Lockport flow. The water entering the Des Plaines tunnel is controlled by dropshaft gates. The gate operation procedure determines the volume of water entering the tunnel. The current Tunnel Network (TNET) model has a hypothetical operating procedure that was developed prior to construction. This operating procedure identifies index dropshafts for each gate. A single operating procedure is defined for all gates identified with a single dropshaft. Thus, varying the operating procedure for a dropshaft affects the operation of all gates identified with that dropshaft. The operating procedure of the prototype needs to be added into the model and the operating procedure needs to be verified against flow records.

The Committee identified some inconsistencies in model results and budgets from model results that should be reviewed further. The average annual deduction for Indiana water-supply pumpage has increased 34.7 percent coincident with a change in the calculation procedures for this value. Budgets 9, 10, 11, and 13 of the water-year accounting reports showed significant long-term biases for the six-year period of analysis. All of these issues should be reviewed further to determine their effect on historical and current diversion accounting.

Lake Michigan Diversion Accounting Measurements

The flow measurements used for diversion accounting are essentially unchanged since 1993. The primary measurement point is the AVM system on the Chicago Sanitary and Ship Canal at Romeoville, Illinois. For water years 1990-1995, between 92 and 97 percent of the total accountable diversion flowed past the

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[1] Water-year is defined as the 12-month period from October 1 through September 30 The water year is designated by the calendar year in which it ends and which includes nine of the twelve months. Thus the water year ending September 30, 1990 is called the "1990 water year."

Romeoville gage. Since July 1993, discharge measurements to develop the AVM rating were done using a broadband Acoustic Doppler Current Profiler (ADCP).

Review of the records for the Romeoville AVM indicated a possible discrepancy between the surveyed channel width and the width measured as part of ADCP measurements. The USGS should continue to investigate whether this has affected the measured discharges and the AVM rating equations. If this has affected the AVM ratings, the effect on historical diversion accounting should be reviewed.

Apart from the possible width error, records from the AVM at Romeoville are excellent. There were 110 days during the water years 1990 through 1995 where record was estimated from backup equations. Overall, the estimated long-term error in the AVM record is ± 93 ft^3/sec compared to an average annual flow of 3,567 ft^3/sec (1990-1996), which represents an error of ± 2.6 percent.

Runoff Study

In support of negotiations for changing the 1967 Supreme Court Decree, the Chicago District conducted a model study for estimating runoff from the Lake Michigan watershed for 44 years (1951 through 1994) (USACE, 1996a). The model was based on the hydrologic models used in accounting procedures. This report was reviewed and approved by the USACE, Hydrologic Engineering Center (HEC).

Consumptive Use

The USACE (USACE, 1996a) studied and modeled the water supply for metropolitan Chicago. The results of the study give a range of consumptive use estimates for water-supply pumpage. The issue of consumptive use arises because the potential of change to lakefront accounting and because the 1980 Modified Supreme Court Decree definition of diversion includes: "domestic pumpage from the lake by the state and its municipalities, political subdivision, agencies and instrumentalities, the sewage and sewage effluent derived from which reaches the Illinois waterway." Initial investigations by the USACE for WY 1991 and WY 1992 were based on influent records at three (3) treatment plants: West-Southwest (Stickney), Northside, and Calumet. The results of this investigation were reported in a draft report (USACE, 1996a). This report was reviewed and approved by the USACE (HEC). Consumptive-use values varied significantly for the period of analysis. In general, the USACE concluded that it was impossible to select either a consumptive use value from this analysis, or a "potential range". However, if extreme values were discounted, a potential range of 8-12 percent could be derived. The Committee believes that there is some inconsistency and confusion with regards to the definition of "consumptive use". Literature (USACE, 1996a) suggests that consumptive use in metered systems ranges from 10-16 percent of the total water entering the supply line system. Consumptive use represents the total loss between domestic pumpage and the resulting effluent from the domestic treatment plants. Therefore losses in the water-treatment plant, water-distribution system, consumer facilities (domestic, manufacturing, etc.), and wastewater-collection and treatment system should be included. The State of Illinois presented information to the Fourth Technical Committee with regards to unaccounted-for-flow (UFF). This information specified a goal for UFF not to exceed 8 percent and listed the permittees, as reported in the annual LMO-2[2] form, that have been consistently above the 8 percent standard for the last 9 years, or that have recently violated the 8 percent standard. This list of permittees with UFF exceeding 8 percent includes the City of Chicago, the largest water supplier in the diverted Lake Michigan watershed. This information suggests losses greater than 8 percent in the water distribution system. In conclusion, the Committee believe that the total losses (consumptive use) in the water/wastewater system could be significantly higher than the 8-12 percent range, as suggested by the USACE (1996a) draft report.

Specifically, consumptive use is going to be very difficult to isolate and determine based on the complexity of the water, wastewater and drainage systems and the associated difficulty and expense in

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[2] State of Illinois – Direct Diversion Flow Report Form for each month.

obtaining field measurements. A possible alternative is to use hydrologic modeling to quantify the base terms of diversion components in terms of diverted watershed runoff and consumptive use.

Lakefront Measurements

The consideration of lakefront measurements of diversion has been recommended by previous Technical Committees and is also recommended by the Fourth Technical Committee. Lakefront measurements could be more direct, simpler, and less complicated. With respect to present activities regarding lakefront measurements, the Technical Committee has the following comments:

1. Since domestic pumpage accounts for about 80 percent of the Lake Michigan Diversion under the lakefront accounting system, continued quality-assurance/quality-control (QA/QC) review of domestic pumpage systems is essential,
2. Support the implementation of acoustic water measurement instrumentation at Jardine Water Treatment Plant and Southside Plant,
3. Calibration of a backup system with respect to Chicago River controlling works sluice gates in terms of present conditions and future construction. This would include improvements in the measurement of river and lake stage and sluice gate opening (actual opening and time of gate-opening changes),
4. Calibration of a backup system with respect to O'Brien Lock and Dam gates in terms of present conditions and future construction. This would include improvements in the measurement of sluice gate opening (actual opening and time of gate-opening changes),
5. Continued calibration of the Columbus Avenue AVM site with respect to ADCP measurements for various conditions and consider "uplooking" velocity profiling system. Calibration needs to identify ways to reduce noise in AVM velocities and protocols to identify and screen erroneous AVM velocities,
6. Continued calibration of O'Brien Lock and Dam AVM site with respect to ADCP measurements for various conditions, and
7. Establishment and calibration of AVM at Wilmette Site and development of a back-up flow measurement system for missing record.

Great Lakes Mediation Memorandum of Understanding, July 1996

The July 1996 Great Lakes Mediation Memorandum of Understanding (MOU) prescribes a three-water-year transition period during which a dual reporting system will be operated. The purposes of the transition period are to assess the technical feasibility of moving the diversion measurement system to the lakefront and give additional time for AVM calibration and opportunity to complete the QA/QC program. The MOU describes this transition period as "beginning after the installation and initial calibration of the AVMs at the lakefront (WY 1997)."

Records from the lakefront AVMs show that the magnitude of the 'noise' in the velocity records will often exceed the mean velocity of the flow. Much of this noise is an artifact of the measurement conditions at these sites (very low velocities, bi-directional flow, thermal gradients, etc.). The noise affects development of index-velocity ratings for these sites, as well as the accuracy of the computed record. The USGS is continuing to refine the instrumentation for the lakefront AVMs to: (a) reduce the noise; (b) develop methods to better distinguish between water velocities and noise; (c) improve the index-velocity ratings; and (d) develop backup equations for these sites. At the present time (June, 1999) the USGS has yet to finalize and publish discharge record for the sites, because of the on-going work to reduce the noise. These refinements by the USGS to the Lakefront AVM instrumentation have the potential for improvement in the accuracy of the Lakefront AVM measurements. The Fourth Technical

Committee review was limited to the current available data and therefore does not represent potential improvement in measurement accuracy for the Lakefront sites.

In view of the on-going efforts to improve the record from these sites, the Technical Committee is concerned regarding the data reliability during the initial phase of the transition period. The USGS is using state-of-the-art technology to measure the velocities and develop the ratings at these sites. The Technical Committee feels that the accuracy of the record currently available for these sites does not reflect the potential of the current technology to measure flows at these sites. The Technical Committee does believe that the on-going refinements to the instrumentation at these sites and the continuing measurements with different instruments (e.g., upward-looking velocity profilers) may provide guidance for processing data already collected. The Technical Committee recommends that the USGS use data from on-going measurements with different instruments to attempt to develop methods to screen or filter the data already collected. These methods should define the accuracy of the records thus developed, as well as the accuracy that is achieved with the refined instrumentation. This will allow the transition period to begin at some time between October, 1996 and when the instrumentation refinements are accepted as operational, while also providing the data needed to assess the technical acceptability of lakefront accounting.

CONVERSION FACTORS

For readers who prefer to use metric (International System) units rather than inch-pound units, the conversion factors for the terms used in this report are listed below:

Multiply inch-pound units	By	To obtain metric units
foot (ft)	0.3048	meter
inches (in)	2.54	centimeter
miles (mi)	1.609	kilometer
square feet (ft^2)	0.0929	square meters
square miles (mi^2)	2.589	square kilometers
foot per second (ft/s)	0.3048	meter per second
cubic foot per second (ft^3/sec)	0.02832	cubic meter per second
millions gallons per day (MGD)	0.4382	cubic meter per second

1. INTRODUCTION

1.1. History of Lake Michigan Diversion

During the late 1800's, Chicago experienced serious water pollution problems. In 1854 and 1885, major storms caused massive amounts of untreated sewage and waste to be carried far out into Lake Michigan. This contaminated water found its way into the City of Chicago's water intakes and caused an outbreak of two waterborne diseases (typhoid and cholera). In the 1885 epidemic, 90,000 people were killed.

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. The CSSC was completed in 1900 by the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC). Prior to 1980 the MWRDGC was known as Metropolitan Sanitary District of Greater Chicago (MSDGC). Comments of the First Technical Committee Report (1981) indicate they were still MSDGC in 1982.

In 1901 the MSDGC was authorized by the Secretary of War to divert 4,167 ft^3/sec. In 1908 and again in 1913, the United States brought actions to enjoin the MSDGC from diverting more than the 4,167 ft^3/sec 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 same year, a permit was issued to divert 8,500 ft^3/sec 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 findings. Subsequently, the Court instructed the Special Master to determine the steps necessary for Illinois and the MSDGC to reduce diversion. Consequently, a 1930 Decree reduced the allowable diversion (in addition to domestic pumpage) in three steps: 6,500 ft^3/sec, after July 1, 1930; 5,000 ft^3/sec after December 30, 1935; and 1,500 ft^3/sec after December 31, 1938.

In 1967, a U.S. Supreme Court Decree limited the diversion of Lake Michigan water by the State of Illinois and its municipalities, including sewage and sewage effluent derived from domestic pumpage, to a five-year average of 3,200 ft^3/sec, effective March 1, 1970. This decree gave full responsibility to the State of Illinois for diversion measurements and computations. The USACE was to have a role of "general supervision and direction". The 1967 Decree limited the diversion, including domestic pumpage, to an average of 3,200 ft^3/sec over a five-year running accounting period. The first five year accounting period began March 1, 1970 and ended to February 28, 1975. During this period, the average diversion was 3,183 ft^3/sec The next accounting period began March 1, 1975 and ended February 29, 1980. During this period, the average diversion was 3,044 ft^3/sec.

The U.S. Supreme Court amended its 1967 Decree on December 1, 1980. The amendment changes, in part, provisions of the 1967 Decree that prevented the State of Illinois from effectively utilizing and managing the 3,200 ft^3/sec of Lake Michigan water, which had been allocated previously by the U.S. Supreme 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 annual accounting year shall not exceed 3,680 ft^3/sec, except that in any two (2) annual accounting periods within a forty (40) year period, the annual average diversion may not exceed 3,840 ft^3/sec, 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 ft^3/sec-years.

In addition, the modified U.S. Supreme Court Decree for the Lake Michigan Diversion at Chicago, Illinois, adopted by the Court on December 1, 1980, stipulates that the USACE convene a three-member Technical Committee at least once every five years to review and report on the methods of flow measurement and procedures for diversion accounting. The Committee review is to include: 1.) 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 2.) recommendations for any appropriate changes to those procedures.

1.2. Components of Diversion

The average annual value for each of the primary components of the Lake Michigan Diversion are averaged for accounting years 1990-95 are presented in Table 1.2-a and Figure 1.2-a. Presented in Figure 1.2-b is the average value for each component of Lake Michigan Diversion accounting of diversion for 1990-95. The components of Lake Michigan Diversion accounting are:

- water supply taken from Lake Michigan intake cribs and discharged into the river and canal system (in the greater Chicago area) as treated sewage
- storm runoff from the diverted watershed area of Lake Michigan, draining to the river and canal system in the greater Chicago area
- water from Lake Michigan entering directly into the river and canal system in the greater Chicago area. This component consists of the following three parts:
- water required for lockage at the Chicago River Controlling Works and the Thomas J. O'Brien Lock
- leakages occurring at the Chicago River Controlling Works, O'Brien Lock and Dam and Wilmette Pumping Station
- water taken in for the navigational make-up and discretionary purposes at the Chicago River Controlling Works, O'Brien Dam and Wilmette Pumping Station

Figure 1.2-a: Amount of total flow average, 1990-1995

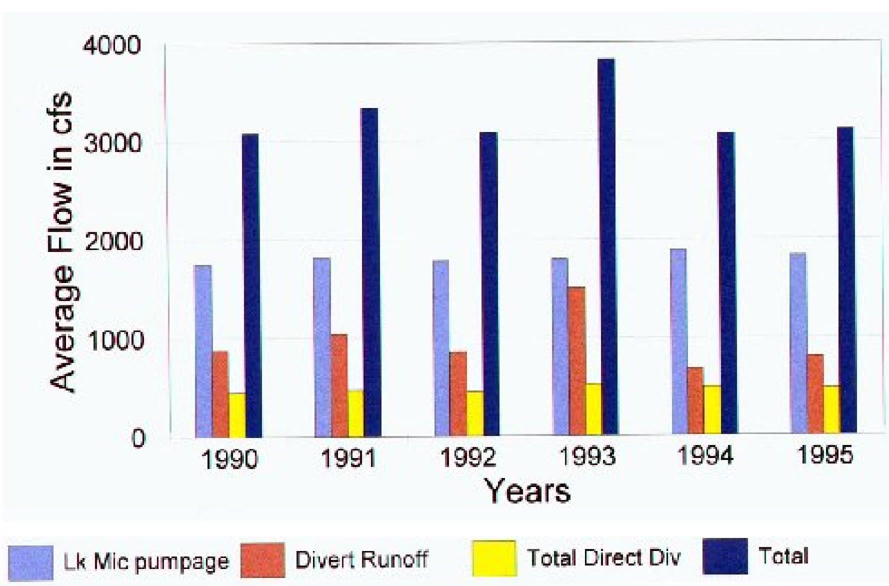

Figure 1.2-b: Percentage of total flow average, 1990-1995

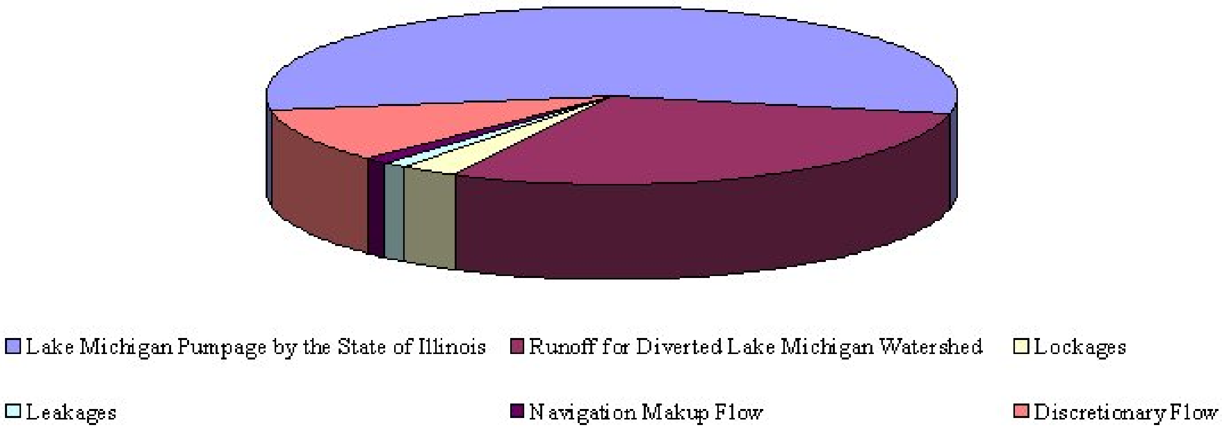

Table 1.2-a: Primary components of Lake Michigan diversion (1990 – 1995)

Description	1990		1991	
	Average Flow (ft^3/sec)	Percentage of Total Flow	Average Flow (ft^3/sec)	Percentage of Total Flow
Lake Michigan Pumpage by the State of Illinois	1,754.9	57.0%	1,819.0	54.6%
Runoff for Diverted Lake Michigan Watershed	872.9	28.4%	1,041.4	31.2%
Total Direct Diversions	450.9	14.6%	472.3	14.2%
Total	**3,078.7**		**3,332.7**	
Breakdown of Direct Diversions				
Lockages	71.8	2.3%	88.7	2.7%
Leakages	28.3	0.9%	31.1	0.9%
Navigation Makeup Flow	46.1	1.5%	37.4	1.1%
Discretionary Flow	304.7	9.9%	315.1	9.5%

Description	1992		1993	
	Average Flow (ft^3/sec)	Percentage of Total Flow	Average Flow (ft^3/sec)	Percentage of Total Flow
Lake Michigan Pumpage by the State of Illinois	1,785.1	57.9%	1,798.6	47.0%
Runoff for Diverted Lake Michigan Watershed	848.4	27.5%	1,504.7	39.4%
Total Direct Diversions	451.7	14.7%	519.0	13.5%
Total	**3,085.2**		**3,822.7**	
Breakdown of Direct Diversions				
Lockages	82.7	2.7%	91.6	2.4%
Leakages	32.6	1.1%	38.3	1.0%
Navigation Makeup Flow	43.4	1.4%	58.6	1.5%
Discretionary Flow	293.0	9.5%	330.5	8.6%

Description	1994		1995	
	Average Flow (ft^3/sec)	Percentage of Total Flow	Average Flow (ft^3/sec)	Percentage of Total Flow
Lake Michigan Pumpage by the State of Illinois	1,886.8	61.6%	1,827.8	58.9%
Runoff for Diverted Lake Michigan Watershed	681.1	22.2%	797.6	25.7%
Total Direct Diversions	497.3	16.2%	480.0	15.4%
Total	**3,065.2**		**3,106.2**	
Breakdown of Direct Diversions				
Lockages	117.9	3.8%	96.9	3.1%
Leakages	37.2	1.2%	35.0	1.1%
Navigation Makeup Flow	33.9	1.1%	28.3	0.9%
Discretionary Flow	308.3	10.1%	319.8	10.3%

Description	*Average*	
	Average Flow (ft^3/sec)	*Percentage of Total Flow*
Lake Michigan Pumpage by the State of Illinois	*1,812.0*	*56.2%*
Runoff for Diverted Lake Michigan Watershed	*957.7*	*29.1%*
Total Direct Diversions	*478.5*	*14.8%*
Total	***3,248.2***	
Breakdown of Direct Diversions		
Lockages	*91.6*	*2.8%*
Leakages	*33.8*	*1.0%*
Navigation Makeup Flow	*41.3*	*1.3%*
Discretionary Flow	*311.9*	*9.7%*

1.3. Review of Technical Committee's Findings

1.3.1. First Technical Committee

The first three-member Technical Committee convened in June 1981, and issued their final report, dated October 1981. The committee's report presented a discussion of the history of diversion, the various components of diversion, and the various flow measurements and computations used to determine Lake Michigan diversion as defined by the 1980 Modified Supreme Court Decree. The First Committee found virtually every aspect of the program to account for diversion from Lake Michigan to be in need of improvement. The diversion, measurement and accounting process "lacked credibility." The Lockport flow components, the cornerstone for diversion accounting, at that time, was determined to be deficient "in practically every aspect." The First Committee report was reviewed to establish a base of reference for the evaluation of diversion activities since 1981. The following is a brief summary of recommendations made by the First Committee:

- Preparation of a Master Plan for diversion accounting
- Establishment of a Quality-Assurance program including an <u>Operational Procedure Manual</u>
- Consideration of alternatives to measurements at Lockport facilities
- Modifications and improvements to Flow Measurement Practice for Facilities
- The Committee also recommended modifications to flow measurements practices for Lockport lock leakage.

1.3.2. Second Technical Committee

The Second Technical Committee was convened in July 1986 and reviewed accounting for water-years 1981 through 1983. The following is a brief summary of the major conclusions and recommendations of the Second Committee.

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 that were still lacking,
3. The diversion accounting certification report should provide the reader a narrative description of the facts which support the certification evaluation,
4. 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,
5. Columns 7 and 9 of the *<u>Diversion Accounting Procedures</u>* representing the so-called sewer induced groundwater inflow should be withdrawn from the diversion accounting format,
6. 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,
7. Examine the constancy of the relation between water-supply pumpage and sewage-treatment-plans inflows and its applications for the purpose of estimating infiltration and inflow deduction for the Des Plaines watershed,

8. Reconsider the alternatives (modeling, etc.) for estimating the annual runoff from the Lake Michigan watershed,
9. The effort by the USGS to establish guidelines to promote improvement in the quality of the AVM records should be continued,
10. The current regressions of the daily discharges for the AVM against MSDGC's records for flow at Lockport, used for the AVM back-up, should be reconsidered, specifically giving attention to the actual Lockport operating configurations,
11. A technical review of the AVM flow records should be conducted annually by the participating agencies,
12. The flow records for the AVM and flows at Lockport reported by MSDGC should be reviewed and compared for consistency on an annual basis,
13. The mean bed elevation for the canal in the reach delimited by the AVM transducer location should be determined, as well as along the transducer paths,
14. The Lockport facilities of MSDGC and USACE should be used for the back-up to the AVM system at Romeoville,
15. Execute a set of field measurements designed to verify the ratings developed by the USACE Waterways Experiment Station (WES) for both the Lockport Powerhouse sluice gates and the Lockport controlling works,
16. Infiltration and inflow of groundwater into the TARP tunnels should be treated as a deduction to the flows measured at Lockport, and
17. The runoff to the TARP system for the Lower Des Plaines combined sewer system should be determined and included as a deduction.

1.3.3. Third Technical Committee

The Third Technical Committee was convened in February of 1993 and reviewed water years 1984 through 1989. This Third Technical Committee was gratified by the improvement achieved in the accounting procedures, particularly in the quality of the AVM records. The primary reason for the diversion exceeding the flow limits of the 1980 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. 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 Third Technical Committee are still current and may be repeated here to emphasize their importance.

1. The draft of the Master Plan for Lake Michigan Diversion Accounting Program (Master Plan) should be finalized,
2. The Master Plan should include an "Operational Procedures Manual" documenting technical procedures and methods used in the Lake Michigan diversion computations,
3. The draft – Plan (draft – October 1988) should be updated and finalized based on the present status of Lake Michigan diversion computational procedures and measurements,
4. Update the AVM Quality-Assurance Plan,
5. A technical review of the Romeoville AVM discharge ratings and flow records should be conducted annually,

6. The mean bed elevation of the canal at the AVM measuring reach should be surveyed periodically,
7. 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,
8. The ADCP (Broadband) system should be used to calibrate and verify the AVM Romeoville system operations. The ADCP can be a valuable tool for measurement during low flow and/or unsteady flow conditions,
9. Investigate the feasibility of developing ratings between the leakage flow through the gates at the lakefront and the water surface elevation of the lake,
10. Annual Lake Michigan diversion results should be published in a more timely fashion, and
11. 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.

2. LAKE MICHIGAN DIVERSION ACCOUNTING – WATER YEAR 1990-1995

Presented in Table 2-a is a summary of chronological events regarding the Technical Committee's activities and Lake Michigan Diversion events for the period 1980-1998.

Table 2-a: Chronological summary of Technical Committee and Lake Michigan diversion events

FIRST TECHNICAL COMMITTEE Convened June 1981, Final Report – October 1981 (Espey and others, 1981) Reviewed Status of Diversion Computation as Stipulated by the 1980, Modified Decree of the U.S. Supreme Court		
SECOND TECHNICAL COMMITTEE Convened July 1986, Final Report – November 1987 (Espey and others, 1987) Reviewed Status of Diversion Computation as Stipulated by the 1980, Modified Decree of the U.S. Supreme Court		
Annual Report	**Water Year Diversion Results**	**Remarks**
1981, 1982 Annual Report 11/83 – Released	1981/1982	1. Lockport Measurement Site – First Committee Report (October 1981) 2. Harza report proposed new diversion accounting program (Harza Engineering, 1981) 3. WY 81-82 Diversion certified despite Technical Committee (1981) concerns regarding Lockport rating (Northeastern Illinois Planning Commission)
1983, 1984, 1985 Annual Report 2/86 – Released	1983	1. New Accounting System (NIPC) 2. Use hydrologic computer models. WES Report (Hart and McGee, 1985) Powerhouse and Controlling Works sluice gage – new rating resulted in a reduced diversion (180 ft^3/sec) for 1988 WY 3. AVM installation (March 18-23, 1984), AVM data suggest Lockport Turbine/low flows consistently low 4. 1983 diversion certified despite concerns on Lockport rating (Technical Committee, 1981) findings 5. Second Committee convenes (July 1986)
1986 Annual Report 3/87 – Released	No diversion results	1. TARP – Began new accounting system, development of a computerized water budget, HEC analysis of Hydrologic Simulation Procedures 2. Second Committee Report (Espey and others, November 1987)
THIRD TECHNICAL COMMITTEE Convened February 1983, Final Report – July 1994 (Espey and others, 1994) Reviewed Status of Diversion Computation as Stipulated by the 1980, Modified Decree of the U.S. Supreme Court		
Annual Report	**Water Year Diversion Results**	**Remarks**
1987 Annual Report 9/88 – Released	No diversion results	The Water Resource Development Act of 1986 gave USACE responsibility for the computation of diversion flow (effective October 1987)
1988 Annual Report 3/89 – Released	No diversion results	1. Continuing problems with AVM – new system; new AVM system to be installed 2. Diversion Accounting certification suspended in WY 1988 pending revision of modeling parameters 3. Second Technical Committee Final Report (November 1987)
1989 Annual Report 11/93 – Released	1984-1987	1. November 1988 – ORE, Inc. AVM installed 2. First Annual Report that USACE assumes responsibility for the compilation of diversion 3. Diversion Account report developed by NIPC, reviewed and updated by USACE 4. USACE updated model parameters and revised 1984-1985 flows based on AVM records
1990-92 Annual Report 1/94 - Released	1986-1987 1988 - 1989	1. New regression equations (USGS) (WY 1986, 1987, 1988, and 1989) 2. Modeling update – TARP 3. Lakefront measurements 4. New 25-gage rain gage network – installed (October 1990) 5. Grand Calumet River West Branch gage established 6. Diversion results indicated State of Illinois exceeded allowable diversion – 1988 7. 1986 problem with AVM 8. 1987 AVM – little missing record 9. 1988-89 Solar Radiation Correction

FOURTH TECHNICAL COMMITTEE Convened September 1998, Draft Report (January 2000) Reviewed Status of Diversion Computation as Stipulated by the 1980, Modified Decree of the U.S. Supreme Court		
Annual Report	**Water Year Diversion Results**	**Remarks**
1993 Annual Report 9/94 – Released	1990	1. Modification to the hydrologic runoff models and hydraulic sewer routing models to incorporate the 25-gage precipitation network into the WY 90 diversion accounting. This includes revision to map delineation for combined sewer specifications contributing areas, delineation of precipitation assigned area for the 25-gage network, revision and update of land-use/land-cover delineations. 2. Third Technical Committee – convened February 1993 3. Third Technical Committee final report (Espey and others, 1994)
1994 Annual Report 10/95 – Released	1991 1992	During WY 1994 and continuing into WY 1995 the hydrologic runoff and hydraulic sewer models were modified in order to utilize the DSS database as the sole database in all diversion accounting computations. The modified models were used for WY 91 and WY 1992 accounting
1995 Annual Report 3/97 – Released	1993 1994	1. Beginning in June 1993 the southern and middle portions of the Des Plaines TARP system became operational. These tunnels were added to the modeling of the TARP system of WY 1993. 2. The estimate of the Grand Calumet river portion of the water supply pumpage from Indiana that reaches the Chicago Sanitary and Ship Channel (CSSC) was revised to better account for the unique hydraulics of the river. 3. Prior to WY 1993 there existed a double accounting of a portion of the runoff from the ungaged Calumet watershed. The flow that was double accounted was the infiltration into the separate sanitary sewers within the ungaged Calumet watershed. This revision only impacts Column 12, the diverted runoff from the Lake Michigan watershed, which is used as a component verification of the overall diversion contained in Column 10.
1996 Annual Report 10/98 – Released	1995	The USACE supported the Great Lakes Mediation Committee with respect to various special studies: 1) runoff and 2) consumptive use

2.1. 1993 Annual Report (Water Year 1990)

The 1993 Annual Report contained the WY 1990 diversion accounting results (USACE, 1994a) and was released September 1994. The hydrologic runoff models and hydraulic sewer routing models were revised in 1993 to incorporate the new 25-gage precipitation network. In addition, improvements and updates were made for land use, land-cover delineation, and hydrologic-runoff-models input parameters. The USGS developed new regression equations to calculate flows for the Romeoville AVM when the AVM was not functioning. The USGS has reviewed the AVM flows for WY 1986-1992. The revised regression equations and the updated flows were included in the accounting reports for WY 1986-1990.

In 1993, an extensive series of measurements were taken from April to October 1993 that utilized an ADCP and dye testing to measure leakage and sluice gate flows at the Chicago River Controlling Works (CRCW).

2.2. 1994 Annual Report (Water Year 1991 and 1992)

The 1994 Annual Report contain the WY 1991-1992 diversion accounting results and was release in October of 1995 (USACE, 1995). Modifications were made to the hydrologic runoff model and hydraulic sewer-routing models in order to incorporate the conversion to the using DSS database. Modifications to the models was first reflected in the WY 1991-1992 accounting, eliminating much of the data manipulation between two different databases. The Third Technical Committee was convened during February 1993.

The annual averages in table 4 of the WY 1991 and 1992 reports were calculated from the monthly averages of the daily values. In general this had little effect, but did result in errors up to 0.6 percent in the annual-average values shown. This appears to have been corrected beginning with the water year 1993 report. The Technical Committee noted apparent errors in table 7 of the WY 1991 and 1992 reports regarding measured simulated to recorded flow (S/R) ratios.

2.3. 1995 Annual Report (Water Year 1993 and 1994)

There were several changes made to the Lake Michigan Accounting procedures during this reporting period. Beginning in June of 1993 the southern and middle portion of the Des Plaines TARP system became operational. As a result, these tunnels for the Des Plaines portion of the TARP system were added to the modeling of the TARP system for WY 1993. In addition, the deductible water supply of the Grand Calumet River was revised. This was based on better accounting for the unique hydraulic characteristics of the river. In addition, modifications were made to the models of the runoff from the ungaged area of the Calumet watershed. Prior to WY 1993, portions of the runoff from the ungaged Calumet watershed were double accounted. The flow that was double accounted was the infiltration into the separate sanitary sewers in the ungaged Calumet watershed. This revision has no direct impact on Column 12, the diverted runoff from the Lake Michigan watershed, which is used for a component verification of the overall diversion contained in Column 10. This revision has no direct impact on the computations of Lake Michigan diversion.

2.4. 1996 Annual Report (Water Year 1995)

The activities for WY 1996 centered on completing WY 1995 accounting reports. Additionally, the USACE supported the Great Lakes Mediation Committee. In response to a dispute over the alleged violation by the State of Illinois of the diversion limits set forth in the 1967 and 1980 Supreme Court Consent Decree in *Wisconsin v. Illinois*, 388 U.S. 426 (1967), as modified, 449 U.S. 48 (1980) (“Decree”), voluntary negotiations were carried out among the State of Illinois, the other seven Great Lakes states, the MWRDGC and the United States during mediation (The Great Lakes Mediation) that began in December 1995. Representatives from Canada and the Province of Ontario observed the negotiations and participated in the discussions. The negotiators involved in the Great Lakes Mediation agreed to principles set forth in the Memorandum of Understanding (“MOU”), dated July 29, 1996. The final acceptance of these terms was ratified by principals not present at the mediation. In support of the mediation process the USACE provided technical support, including long-term runoff and consumptive use studies. These studies provide the technical basis of an agreement between the states to potential move the accounting process to the lakefront. The following is a brief discussion of the technical support provided reflected in two special studies.

2.4.1. Runoff Study

In support of negotiations for changing the Decree, USACE conducted a model study to estimate runoff from the Lake Michigan watershed for 44 years (1951 through 1994). This analysis is referred to as the “period-of-record analysis”. The model was based on the hydrologic models used in the accounting procedures (USACE, 1996a). The total runoff from the Lake Michigan watershed was computed by summing the following elements:

1. The total inflow and infiltration components of interceptor and inflows for all 137 Special Contributing Areas (SCAs) within the Lake Michigan watershed and within the three MWRDGC water reclamation plant (WRP) service areas,
2. The total runoff, sewered and unsewered, from the 84 square mile “ungaged” Calumet watershed,
3. Runoff from streamflow separation techniques applied at four streamflow gages (North Branch Chicago River at Niles, Illinois; Little Calumet River at Munster,

Indiana; Thorn Creek at Thornton, Illinois; Little Calumet River at South Holland, Illinois),

4. Runoff from streamflow separation and a simulation analysis for the Grand Calumet River, and

5. The baseflow entering the canals and watershed channels between gages and the downstream end of the diverted watershed.

The first two elements were computed using the simulation models. The models were calibrated against influent pumping records for the three MWRDGC WRPs. Statistical analyses of the simulated recorded flows at the three MWRDGC WRPs were done for each water year. The Corps reported that these showed a "good correlation, both with respect to the correlation coefficient and the simulated to recorded ratios". Table 2.4-a lists the correlation coefficients and the ratios of simulated to recorded flows. The correlation coefficients in Table 2.4-a are presented as r^2, rather than as r, because r^2 has a physical meaning as the percent of the variance in the recorded data that is explained by the simulated flows. Prior to WY 1990, the total simulated flows were somewhat less (average of 0.4 ft^3/sec for all three WRPs, WY 1983 through 1989) than the total recorded inflows at the three WRPs. The revised models used for WY 1990 and thereafter show total simulated flows that are slightly higher (average of 27.9 ft^3/sec for all three WRPs, WY 1990-1992) than the total recorded inflows at the water reclamation plants. The diverted Lake Michigan watershed was found, during WY 1990 modeling revisions, to contain significantly more impervious area than was modeled prior to WY 1990.

Table 2.4-a – Summary of simulation statistics for the three MWRDGC WRPs for water years 1986 through 1995.

[Flow, annual mean recorded discharge; ft^3/sec, cubic feet per second; r^2, correlation coefficient; S/R, ratio of mean simulated to mean recorded flows]

Accounting year	Northside WRP			Stickney WRP			Calumet WRP		
	r^2	S/R	Flow (ft^3/sec)	r^2	S/R	Flow (ft^3/sec)	r^2	S/R	Flow (ft^3/sec)
1986	0.61	0.95	451	0.62	1.08	1,140	0.22	0.84	382
1987	0.48	0.95	443	0.58	0.99	1,200	0.18	0.86	361
1988	0.38	0.97	411	0.49	0.93	1,240	0.27	0.80	360
1989	0.64	0.97	422	0.55	1.03	1,150	0.52	0.99	421
1990	0.78	0.94	437	0.74	1.07	1,050	0.79	1.00	386
1991	0.59	0.94	440	0.69	1.04	1,120	0.71	1.00	387
1992	0.67	0.95	433	0.58	1.09	1,050	0.76	1.05	369
1993	0.81	0.95	478	0.71	1.07	1,170	0.72	1.06	399
1994	0.61	0.97	403	0.59	1.04	1,050	0.64	1.02	362
1995	0.61	0.95	416	0.64	0.98	1,140	0.81	0.99	378

The runoff from areas of the Lake Michigan watershed not included in the above simulations were based primarily on stream-gage records. These areas include the northern and southeastern extents of the watershed. In these areas, a streamflow-separation technique was used, in which estimated sanitary discharges upstream from the stream gage were subtracted from the flow at the gage.

The runoff was estimated by simulation for approximately 361 square miles, while using streamflow separation techniques for approximately 312 square miles. Some areas overlap in that they fall within both the simulated area and the stream gaged area. These areas are separately sewered where the sanitary sewers convey flow to the water reclamation plants while the storm sewers discharge into streams to be measured by the gages. Overlapping areas were generally classified as gaged areas.

The USACE and the USGS undertook an analysis of the groundwater discharge to the canals and watercourses, downstream of any gages, within the diverted watershed. The streams include portions of: the Chicago River, the North Branch of the Chicago River, the South Fork of the South Branch of the Chicago River, the CSSC, the Calumet River, the Grand Calumet River and the Cal-Sag Channel. This analysis determined the total annual baseflow to be approximately 4.0 ft^3/sec.

The annual runoff from the diverted watershed was computed by summing the simulated flows (for the SCAs and the ungaged Calumet watershed), the gaged flows (from the North Branch of the Chicago River, the Little Calumet River and the Grand Calumet Rivers) plus the baseflow. The results of the WY 1951-1994 continuous period simulation of the diverted Lake Michigan diversion accounting runoff was 785.2 ft^3/sec.

To gain a better understanding of the long term runoff values a series of comparisons and analyses was undertaken by the USACE (*Lakefront Account Technical Analysis*, 1996). The analysis consisted of:

- Results of the USACE's period-of-record analysis were compared with the analysis of long term average runoff values computed by the NIPC.
- The period-of-record runoff values were compared with those computed for the diversion accounting reports. A series of trend analyses compared the increases over time in station rainfalls, modeled rainfalls, and modeled runoffs. Generally, the results showed that both simulated and recorded rainfall and runoff were increasing over time, but at a diminishing rate.
- An analysis of the sensitivity of the computed runoff to the set of rainfall gages utilized was performed. The average runoff computed for the 5-year period, WY 1990-1994, was 866.2 ft^3/sec for the three gages used in the period-of-record study. Using only the Midway gage (the procedure NIPC used) the average runoff for the five-year period was 887.4 ft^3/sec, or a 2.4 percent increase. Using 21 of the 25 gages currently employed in the Lake Michigan diversion accounting resulted in increasing the average annual runoff for the five-year period to 916.0 ft^3/sec, or a 5.6 percent increase.
- A sensitivity analysis of the effects of imperviousness was undertaken, and the average runoff computed for the 5-year period, WY 1990-1994, using the impervious and pervious breakdowns applied in the period-of-record study was 866.2 ft^3/sec. Increasing the impervious areas by 10 percent resulted in the average runoff for the five-year period to 885.9 ft^3/sec, or a 2.3 percent increase. Decreasing the impervious areas by 10 percent resulted in the average runoff for the five-year period to 846.5 ft^3/sec, or a 2.3 percent decrease.
- A comparison with the historic record at Lockport was made using a rather involved analysis procedure. The results are not clearly defined in the Lakefront Accounting Technical Analysis (USACE, 1986a). This comparison showed that regression against the Lockport flows only explained 22 percent of the variance in the runoffs.

In addition, water balances were prepared for selected portions of the system. The water balances showed runoff was 47 percent of the rainfall over the basin. In contrast, the period-of-record analysis showed runoff as 44.7 percent of the rainfall, a difference of 40 ft^3/sec, or a 5.1 percent.

The USACE's sensitivity analysis results are summarized. Each item has a qualitative sense of concern, as well as the impact that item may have on the diversion period of record results.

Table 2.4-b: Summary of sensitivity analyses for period-of-record analysis.

Issue	Concern	Impact
USACE's analysis versus NIPC's	Moderate	Low
Period of record versus accounting flows	Moderate	Low
Trend analyses of rainfall and runoff values	Moderate	Low
Sensitivity analysis of the rainfall gages	High	High
Sensitivity analysis of imperviousness	High	High
Comparison with the record at Lockport	Moderate	Low
Mass balance of rainfall and runoff	Low	Low

The high concern and high impact issues -- the sensitivity of the simulations results of the rain gages and the imperviousness -- generate the largest uncertainty in the runoff values. Clearly, it would be desirable to use more rainfall gages for computing runoff for the period of record; however the three that were used are the only long-term gages available in the basin. A further review of the rain-gage sensitivity analysis could serve to diminish the concerns. The uncertainty in the correct values of imperviousness, with respect to the ungaged Calumet area and the overflows.

2.4.2. Consumptive Use Study

The USACE (1996a) studied and modeled the water supply for metropolitan Chicago. The results of the study gave a range of consumptive use estimates for water-supply pumpage.

The issue of consumptive use arises because of changing to lakefront accounting (domestic pumpage) and because the 1980 Modified Supreme Court Decree definition of diversion includes: domestic pumpage from the lake "the sewage and sewage effluent derived from which reaches the Illinois waterway." Initial investigations by the USACE for WY 1991 and WY 1992 were based on influent records at three WRP: West-Southwest (Stickney), Northside, and Calumet. The results of this investigation were reported in a draft report (USACE 1996a). Consumptive-use values varied significantly for the period of analysis. In general, the USACE concluded that it was impossible to select either a consumptive-use value from this analysis, or a "potential range". However, if extreme values were discounted, a potential range of 8-12 percent could be derived. The Committee believes that there is some inconsistency and confusion with regards to the definition of "consumptive use". Literature (USACE, 1996a) suggests that consumptive use in metered systems ranges from 10-16 percent of the total water entering the supply line system. Consumptive use represents the total loss between domestic pumpage (distribution) and the resulting effluent from the domestic treatment plants. Therefore losses in the water-treatment plant, water-distribution system, consumer facilities (domestic, manufacturing, etc.), and wastewater-collection and treatment system should be included. The State of Illinois presented information to the Fourth Technical Committee with regards to UFF. This information specified a goal for UFF not to exceed 8 percent and listed the permittees, as reported in the annual LMO-2 form, that have been consistently above the 8 percent standard for the last 9 years, or that have recently violated the 8 percent standard. This list of permittees with UFF exceeding 8 percent includes the City of Chicago, the largest water supplier in the diverted Lake Michigan watershed. This information suggests losses greater than 8 percent in the water distribution system. In conclusion, the Committee believes that the total losses (consumptive use) in the water/wastewater system could be significantly higher than the 8-12 percent range, as suggested by the USACE draft report (1996a).

Specifically, consumptive use is going to be very difficult to isolate and determine based on the complexity of the system and the difficulty and expense in obtaining field measurements. A possible alternative is reflected in the following analysis which reflected the base terms of lakefront diversion and the corresponding water budget terms.

Diversion accounting flow (**DA**) can be defined as:

$$DA = WS - CU + R + DD \tag{2.4-a}$$

Where **WS** is water-supply pumpage, **CU** is consumptive use, **R** is runoff from the diverted watershed, and **DD** is direct diversion from Lake Michigan.

CU can be defined as:

$$CU = WS - (P_{WS} + O_{WS}) \tag{2.4-b}$$

Where $\mathbf{P_{WS}}$ is water supply discharged from the WRPs (effluent) and O_{WS} is water supply that overflows the sanitary or combined sewers to the canal system.

The following flow balance can be defined as:

$$E + O = (P_{WS} + P_{RG}) + (O_{WS} + O_{RG}) \tag{2.4-c}$$

E is effluent (WRP flows), **O** is total overflows to the canal system, P_{RG} is WRP effluent that are runoff and groundwater inflows, and O_{RG} is overflows that are runoff and groundwater inflows.

$$E = P_{WS} + P_{RG} \tag{2.4-d}$$

$$O = O_{WS} + O_{RG} \tag{2.4-e}$$

Assuming complete mixing; the ratio of water-supply pumpage to runoff and groundwater should be the same in overflow and WRP flows; or:

$$\frac{P_{WS}}{O_{WS}} = \frac{P_{RG}}{O_{RG}} \tag{2.4-f}$$

Rearranging equation 2.4-c gives:

$$E + O = (P_{WS} + O_{WS}) + (P_{RG} + O_{RG}) \tag{2.4-g}$$

Combining with 2.4-b gives:

$$E + O - WS = (P_{RG} + O_{RG}) - CU \tag{2.4-h}$$

Which can be rearranged to:

$$CU = (WS - E) + (P_{RG} + O_{RG}) - O \tag{2.4-i}$$

E and **WS** are measured at either end of the water system, and P_{RG}, O_{RG} and O can be determined in the simulation in "lump" parameter form. For model calibration based on measured flows at the treatment plant, the water supply pumpage and effluent pumpage are known and the difference between these should be runoff and infiltration captured by the sewers less overflows to the canal system prior to the treatment plant and less consumptive use. In fact, even if the simulated breakdown between runoff captured by the sewer and overland flow is wrong, the overall balance will be correct. This is because an error in the amount of runoff directly to streams will result in a compensating error in the consumptive use. However, it is critical that the overall runoff simulation be correct. Neither runoff not captured by sewers nor overflows are part of the calibration at treatment plants. Therefore, the model calibration also needs to include a calibration of the water budget at downstream gages, which will reflect these two terms. Unfortunately, any assumptions about consumptive use will affect this term. The hydrologic model includes a component for sanitary flow, which is based on a per capita loading factor times a population equivalent for each SCA. Therefore, implicit in these calculations is a consumptive use value

"**CU**" in the underlying assumptions utilized in the model. The Committee therefore concludes that the utilization of the hydrologic model to determine the summation of "**CU"** will require the calibration of the hydrologic model with respect to not only WRP pumpage records, but sanitary sewer overflow in a isolated defined sanitary sewer system in order to measure these components.

2.5. Water Supply Pumpage from Indiana

The accounting for water supply pumpage from Indiana reaching the CSSC was changed in the Water Year 1992 and Water Year 1993 reports. For the water year 1990 and 1991 reports, the flow in the Grand Calumet River at the Indiana-Illinois state line (Hohman Avenue) was computed from a regression equation developed by Kieffer and Associates in 1978. This was assumed to be comprised entirely of water-supply pumpage until the calculated annual-mean flow exceeded the annual-mean discharge from the upstream WRPs. If the estimated annual-mean flow at Hohman Avenue exceeded the annual-mean discharge from the upstream treatment plants, the entire annual-mean discharge from the upstream treatment plants was assumed to flow past Hohman Avenue to the CSSC.

The USGS began operating a streamflow-gaging station on the Grand Calumet River at Hohman Avenue at Hammond Indiana on October 2, 1991. The measured annual-mean flow from this gage replaced the estimated flows from the regression equation in the water year 1992 accounting report. A comparison of flows from the old regression equation with the measured flows from January 1, 1995 through September 30, 1998 indicated that the old regression equation underestimated flows at this site by an average of 6.9 ft^3/sec. Assuming that the average error is random over time, this could increase the deductions for the water year 1990 and 1991 accounting reports by up to 6.9 ft^3/sec[1] representing a .2 percent in the annual diversion. For the water year 1992 accounting report, the measured flows were compared to the WRP discharges. Since the measured annual-mean flow at Hohman Avenue (24.9 ft^3/sec) exceeded the treatment plant discharges (24.5 ft^3/sec) all the water-supply pumpage was assumed to flow to the CSSC.

Beginning with the water year 1993 accounting report, the portion of discharges from the Hammond, East Chicago, and Whiting WRPs that flowed past Hohman Avenue were pro-rated based on the Lake Michigan water elevation. According to the USACE (1996a, Appendix A, pp. 12-13) an unsteady-state hydraulic model was developed to investigate flow in the Grand Calumet River. Based on results from this model, the following equations were developed to allocate the portion of sanitary flows (water-supply pumpage) flowing west to Hohman Avenue rather than east to Lake Michigan (Table 2.5-a).

Table 2.5-a: Equations used to estimate water supply pumpage from Indiana reaching the CSSC from the *Lakefront Accounting Technical Analysis* (USACE, 1996a).

Lake Michigan level (ft, CCD)	Water-supply estimation equation	
$GH_{LkMi} <= 1.0$	$Q_{WS} = 0.446 * Q_{HW}$	(2.5-a)
$1.0 < GH_{LkMi} <= 1.4$	$Q_{WS} = 0.446 * Q_{HW} + \frac{(GH_{LkMi} - 1.00)}{0.4} * 0.554 * Q_{HW}$	(2.5-b)
$1.4 < GH_{LkM} <= 1.8$	$Q_{WS} = Q_{HW} + \frac{(GH_{LkMi} - 1.40)}{0.4} * Q_{EC}$	(2.5-c)
$1.8 < GH_{LkMi}$	$Q_{WS} = Q_{HW} + Q_{EC}$	(2.5-d)

Modified from USACE, 1996a, Appendix A, pp. 12-13

[1] The total effluent discharge and water-supply pumpage were not listed, only that they exceeded the estimated flow at Hohman Avenue, and thus the entire flow at Hohman Avenue was deducted.

Where GH_{LkMi} is the Lake Michigan water elevation at Calumet Harbor, in feet above the Chicago City datum (CCD); $\boldsymbol{Q_{WS}}$ is the water-supply pumpage past Hohman Avenue; $\boldsymbol{Q_{HW}}$ is the combined water supply pumpage from Hammond and Whiting plants; and $\boldsymbol{Q_{EC}}$ is the water supply from the East Chicago.

These equations have been modified slightly from the original presentation. In the USACE (*Lakefront Accounting Technical Analysis* – 1996a, Appendix A, pp. 12-13), the minimum flow from the Hammond WRP of 35 ft^3/sec is used to estimate the water supply from Hammond and Whiting and the minimum flow from the East Chicago WRP of 25 ft^3/sec is used to estimate water supply from East Chicago. If these values are substituted into Equations 2.5-a through 2.5-d they will simplify to those shown in by the USACE (1996a).

Although the analysis described by the USACE (1996a) is the source of the equations used in the water year 1993 through 1995 accounting reports, different equations were used in these reports than in the *Lakefront Accounting Technical Analysis*. The equations used in the accounting reports are (Table 2.5-b):

Table 2.5-b: Equations used to estimate water supply pumpage from Indiana reaching the CSSC used in the water year 1993-1995 accounting reports (USACE, 1994, 1995, 1997, and 1998).

Lake Michigan level (ft, CCD)	Water-supply estimation equation	
$GH_{LkMi} < 0.3$	$Q_{ws} = 0.45*Q_{HW}$	(2.5-e)
$0.3 <= GH_{LkMi} < 1.5$	$Q_{ws} = Q_{HW}\left(0.22GH_{LkMi}^3 - 0.15GH_{LkMi}^2 + 0.06GH_{LkMi} + 0.45\right)$	(2.5-f)
$1.5 <= GH_{LkMi} < 1.8$	$Q_{ws} = Q_{HW} + \frac{\left(GH_{LkMi} - 1.5\right)}{0.3} \times Q_{EC}$	(2.5-g)
$1.8 <= GH_{LkMi}$	$Q_{ws} = Q_{HW} + Q_{EC}$	(2.5-h)

For Lake Michigan levels near 1.0 ft CCD, the equations used in the accounting reports estimate about 4.0 ft^3/sec more flow than the equations used in the Lakefront Accounting Technical Analysis. For Lake Michigan levels near 1.5 ft CCD, the equations used in the accounting reports estimate about 6.9 ft^3/sec less flow than the equations used in the Lakefront Accounting Technical Analysis. This corresponds to errors of –30 percent to +17 percent of the typical annual water-supply pumpage at Hohman Avenue. Figure 2.5-a shows the differences between the two sets of equations for the range of Lake Michigan levels. The potential long-term effect of the differences between these equations was evaluated by evaluating the differences for Lake Michigan levels from January 1996 through August 1998. The average difference in estimated water-supply pumpage past Hohman Avenue was 1.6 ft^3/sec less flow was estimated by Equations 2.5-e through 2.5-h than by Equations 2.5-a through 2.5-d.

The Technical Committee recommends that the basis for the equations used in the *Lakefront Accounting Technical Analysis* (USACE, 1996a) and the water year accounting reports, as well as the basis for using different equations for Romeoville and lakefront accounting, be defined clearly.

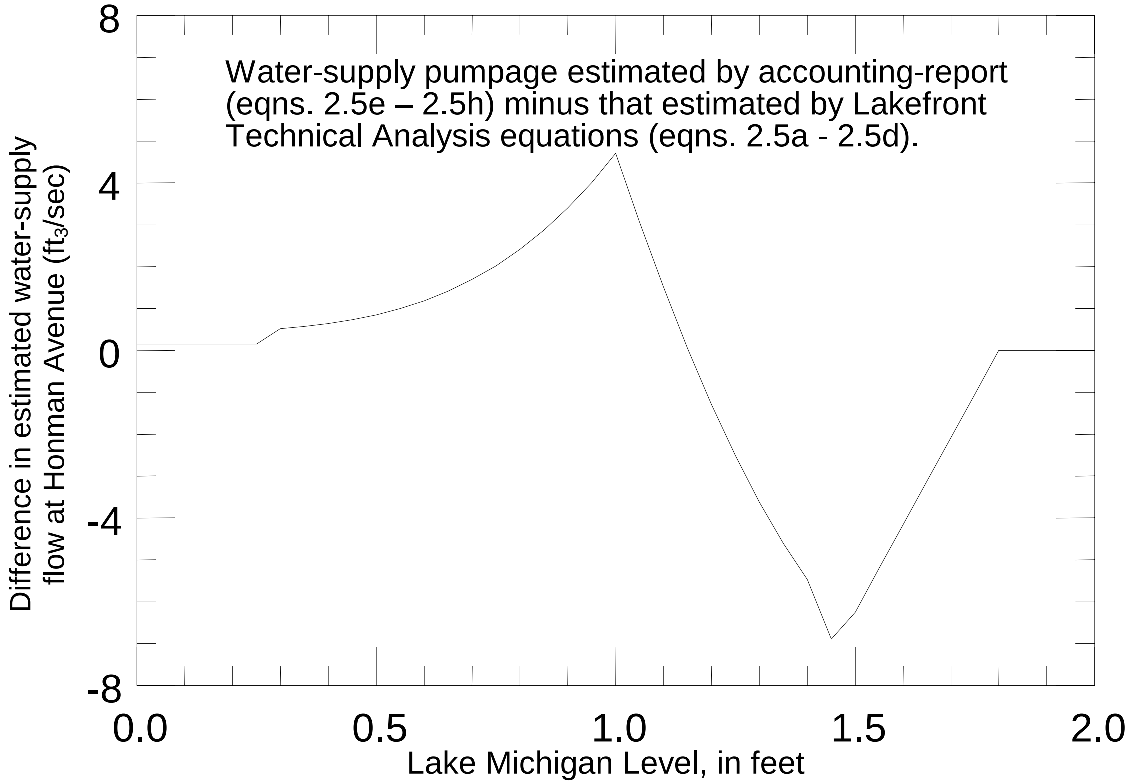

Figure 2.5-a: Differences in water-supply pumpage estimated by equations for diversion accounting and for *Lakefront Accounting Technical Analysis*

A double-mass curve analysis was used to look for changes in the relation of deductible Indiana water-supply pumpage to other components of the diversion accounting. Double-mass curves were used to compare the Indiana pumpage to: (a) the Romeoville gage record; (b) the groundwater pumpage discharged to the canal (Figure 2.5-b); (c) pumpage from Lake Michigan accountable to Illinois (Figure 2.5-c); and (d) the total diversion accountable to Illinois. All of these showed a similar break around September, 1992. The double-mass curve shown in Figure 2.5-b compares the Indiana water-supply pumpage to the groundwater pumpage discharged to the canal. Since both axes are water-supply pumpage, any relations or breaks are less likely to be masked by any external influences (i.e., unusually wet or dry years). This assumes that nothing affected Indiana's water-supply pumpage differently than Illinois' groundwater pumpage. Many Illinois communities have been switching from groundwater to Lake Michigan as the source of their water. This could potentially affect the double-mass curve in Figure 2.5-b. Figure 2.5-c is a double mass curve comparing the Indiana water-supply pumpage to the pumpage from Lake Michigan accountable to Illinois. If the break in Figure 2.5-b was from communities switching from groundwater to Lake Michigan as the source of their water, Figure 2.5-c should have a break at the same time but in the opposite direction (Lake Michigan pumpage should increase relative to Indiana's pumpage). Since the break for both curves indicates Indiana's pumpage is increasing relative to Illinois water-supply pumpage, this break is likely to represent a change in the Indiana pumpage values.

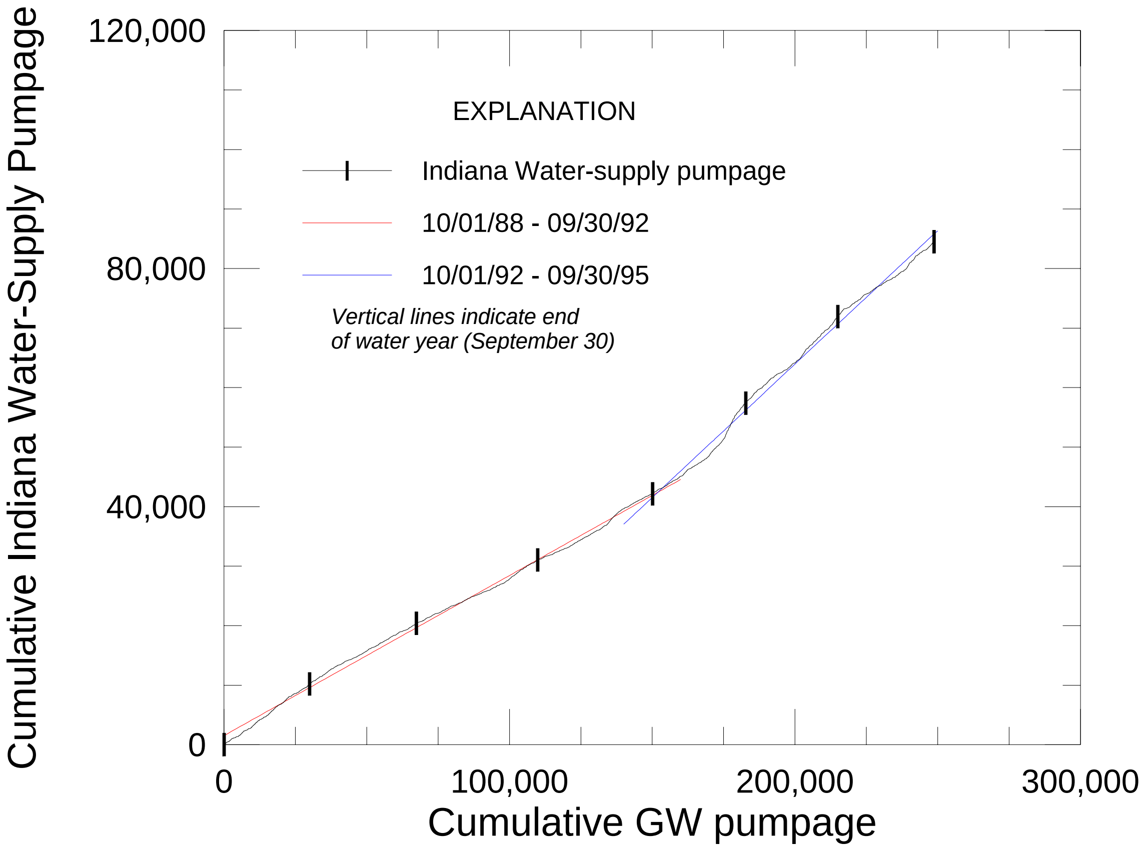

Figure 2.5-b: Double-mass-curve comparison of Indiana water-supply pumpage and groundwater pumpage discharged to the canal, water years 1989 through 1995.

Beginning in water year 1993 (October 1, 1992), the accounting procedure for Indiana pumpage discharged to the CSSC was changed as described above. The average annual Indiana pumpage discharged to the CSSC from water years 1989 through 1992 was 28.8 ft^3/sec. The average annual Indiana pumpage discharged to the CSSC from water years 1993 through 1995 was 38.7 ft^3/sec, an increase of 34.7 percent. Although other factors, such as the level of Lake Michigan, may have changed Indiana's pumpage discharged to the CSSC at this time, they are not identified in the accounting reports. If the change in procedure is the cause of the change in Indiana pumpage discharged to the CSSC, and the new procedure is accepted as more accurate, the accounting for water years prior to 1993 should be reviewed.

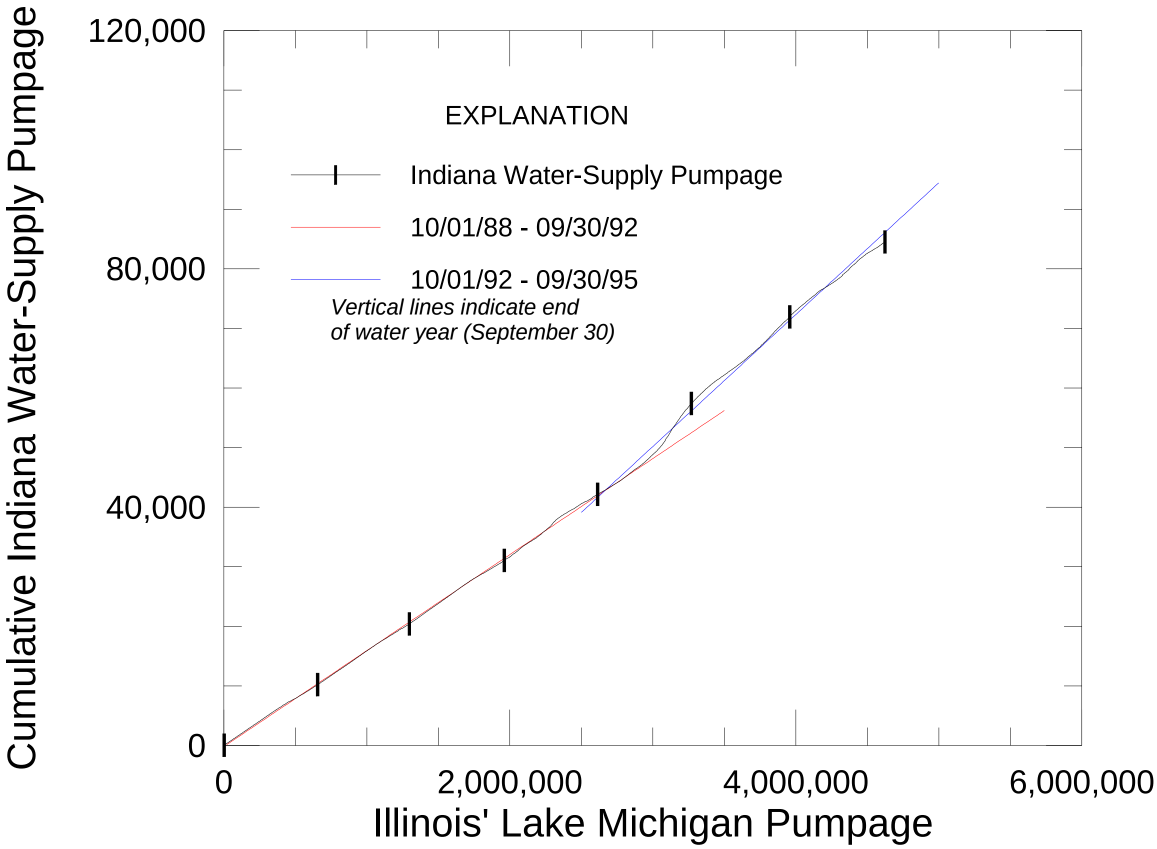

Figure 2.5-c: Double-mass-curve comparison of Indiana water-supply pumpage and Lake Michigan pumpage accountable to Illinois, water years 1989 through 1995.

2.6. Status of Lake Michigan Diversion

Lake Michigan diversion certified (USACE) flow is summarized in Table 2.6-a. Since implementation of the modified Supreme Court Decree of December 1, 1980 and before this report, the Corps of Engineers has certified diversion flows for WY 1981 through WY 1994. Table 2.6-b shows the accounting year, the certified flows, the running average flows, and the cumulative deviation from the allowable diversion of 3,200 ft^3/sec (Decree, 1980).

The running average diversion for the period WY 1981 through WY 1995 is 3,439 ft^3/sec, 239 ft^3/sec greater than the 3,200 ft^3/sec 40-year average diversion specified by the Decree. The annual average diversion exceeded 3,680 ft^3/sec three times, once more than the maximum number of times allowed in the Decree (1980). The absolute annual maximum of 3,840 ft^3/sec also was exceeded during the WY 1993 accounting period. The cumulative deviation, the sum of difference between the annual average flows and 3,200 ft^3/sec, is –3,586 ft^3/sec-years. The negative cumulative deviation indicates a cumulative flow deficit. The Decree (1980) specifies a maximum allowable deficit of 2,000 ft^3/sec-years over the first 39 years of the 40 year averaging period. Presented in Figure 2.6-a is this history of diversion since 1981 compared to the Decree (1980) limits.

Table 2.6-a: Status of the State of Illinois Diversion (1980 Modified U.S. Supreme Court Decree)

Accounting Year	Certified Flow (ft^3/sec)	Running Average (ft^3/sec)	Cumulative Deviation (ft^3/sec)
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	-2,001
1989	3,378	3,443	-2,189
1990	3,531	3,452	-2,520
1991	3,555	3,461	-2,875
1992	3,409	3,457	-3,084
1993	3,841	3,487	-3,725
1994	3,064	3,456	-3,589
1995	3,197	3,439	-3,586

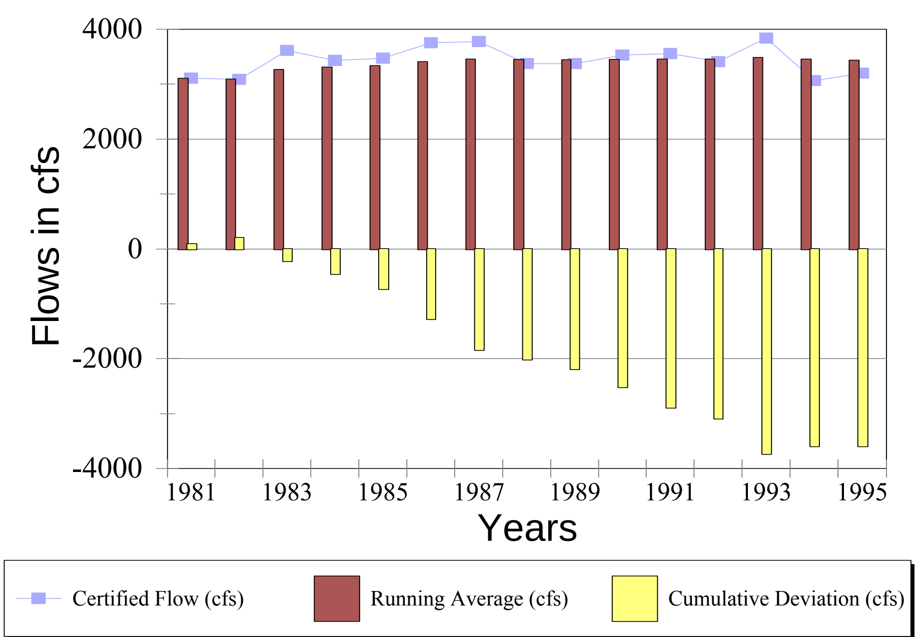

Figure 2.6-a: Illinois Lake Michigan diversion (1980 Modified Decree)

3. REVIEW OF CURRENT ACCOUNTING SYSTEM

3.1. Accounting Report

A sample diversion accounting table for 1995 is shown in Table 3.1-a. A summary of the column entries is shown in Table 3.1-b. Columns 1 through 3 are the total flow entering the CSSC. Column 4 through Column 7 are the deductions from the CSSC flows. The total deduction is in Column 8. Column 9 is the pumpage discharged, such that it is not measured at Romeoville, and represents an addition to the CSSC record. Column 10 is the Lake Michigan diversion accountable to Illinois and is equal to the CSSC flow (Column 3) minus the deductions (Column 8) plus the additions (Column 9). 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. Columns 11 through 13 are not used in the diversion calculation but are included to verify the diversion calculation and to estimate the three diversion components. The sum of the Columns 11 through 13 should theoretically equal the flow in Column 10, but errors in the simulation of runoff (a portion of which is determined by streamgage measurements) and the measurement of leakage and flow past the sluice gates may cause this number to be different.

Table 3.1-a - Diversion accounting table for 1995

Lake Michigan Diversion Accounting WY 1995	Romeoville AVM gage Record	Diversion 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
Oct-94	2712.5	4.2	2716.7	68.5	40.6	109.0	1.1	219.2	234.8	2732.3	1755.5	648.5	639.8
Nov-94	3277.8	1.6	3279.4	122.0	33.3	220.3	1.0	376.6	222.4	3125.2	1656.8	1082.7	125.6
Dec-94	2808.2	2.6	2810.8	88.6	33.3	183.3	0.9	306.1	215.7	2720.4	1637.2	813.9	83.5
Jan-95	3540.0	1.6	3541.6	105.6	32.2	319.3	1.0	458.1	224.5	3308	1659.6	1487.5	78.2
Feb-95	2284.9	3.1	2288	71.5	24.7	124.4	1.0	221.6	223.7	2290.1	1660.6	312.1	88.8
Mar-95	2621.4	1.2	2622.6	90.5	25.2	165.0	0.9	281.6	218.9	2559.9	1649.4	651.7	81.5
Apr-95	3919.4	1.3	3920.7	111.0	28.2	328.1	0.8	468.1	220.3	3672.9	1641.1	1715.0	129.8
May-95	3441.5	1.2	3442.7	106.9	37.3	242.0	0.9	387.1	243.1	3298.7	1711.6	1044.8	229.8
Jun-95	3288.4	3.7	3292.1	89.4	49.3	139.4	1.3	279.4	325.3	3338	2137.0	472.3	702.8
Jul-95	3752.1	2.4	3754.5	87.6	44.3	68.1	1.4	201.4	337.1	3890.2	2281.0	414.1	1207.1
Aug-95	3941.8	1.8	3943.6	100.2	31.2	79.6	1.3	212.3	349.6	4080.9	2161.9	805.8	1136.5
Sep-95	3175.1	3.2	3178.3	65.3	33.4	24.6	1.2	124.5	243.9	3297.7	1970.5	92.8	1228.5
Average	3234.8	2.3	3237.1	92.3	34.5	167.8	1.1	295.6	255.2	3196.7	1827.8	797.6	480.1

3.1.1. Description of Columns in Diversion Accounting Table

Table 3.1-b - 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

The following is a brief description of each column:

Column 1: CSSC (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 USGS using the AVM gage at this station location. Records were based on the Sarasota AVM from June 12, 1984 to November 3, 1988. A new AVM manufactured by ORE became operational on November 17, 1988.

Column 2: Diversion from the CSSC above the Gage

Column 2 is municipal or industrial diversions from the CSSC upstream of the Romeoville gage. Presently, only Argonne National Laboratories and Citgo Corporation divert water from the CSSC upstream of the Romeoville gage.

Column 3: Total Flow through the CSSC

Column 3 is the sum of columns 1 and 2 and represents the total flow entering the canal system.

Column 4: Groundwater Discharge to the CSSC and Adjoining Canals

Column 4 is the groundwater pumped by communities, industrial users, and other private users as reported by the Illinois State Water Survey (ISWS). Column 4 includes groundwater seepage into the TARP that is discharged to the canal. 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, Des Plaines, 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 that 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.

The groundwater constituent of combined sewer overflow is determined entirely through simulation. Groundwater pumpage whose effluent is discharged to the canal is a deduction.

The value of Column 4 for Groundwater Pumpage Discharged to the Canal is based on water-supply pumpage records and does not consider consumptive use. The Supreme Court Decree specifies sewage effluent for the accounting, which would require subtracting the consumptive use from the groundwater-pumpage records. However, consumptive use varies widely among water suppliers and among geographic regions of the greater Chicago metropolitan area and the range of potential consumptive use values is not well defined. Therefore, the present best engineering practice does not provide clear guidance as to the value that should be subtracted from the pumpage records to account for consumptive use.

Column 5: Water-Supply Pumpage From Indiana Reaching the CSSC

Column 5 is the water supply pumpage by the State of Indiana which reaches the canal as effluent. This water is not charged to Illinois' allotment. It is a deduction from the flow measurement at Romeoville.

Column 6: Runoff from the Des Plaines River watershed (DPW) Reaching the CSSC

Column 6 consists of the following components, which are determined by simulation:

1. Infiltration and inflow from the DPW discharged to the WRPs,
2. Infiltration and inflow from the DPW reaching the CSSC through combined sewer overflows,
3. Direct runoff, including runoff from storm sewers, that discharges to watercourses from the Lower Des Plaines watershed, and the Summit conduit area, and
4. The runoff portion of the O'Hare flow transfer.

Column 7: Lake Michigan Pumpage by Federal Facilities which Discharge to CSSC

Column 7 represents Lake Michigan diversion by federal facilities not chargeable to the State of Illinois allocation. Federal facilities represented by the column are as follows:

- Hines VA Hospital
- Fort Sheridan
- Glenview Naval Air Station
- USACE emergency navigation makeup water

Column 8: Total Deduction from CSSC at Romeoville

Column 8 is the sum of the columns 4, 5, 6, and 7 and represents the total deductions from Romeoville records.

Column 9: Lake Michigan pumpage not discharged to the CSSC

Column 9 is water supply pumpage from Lake Michigan that is not discharged to the CSSC. The water supply pumpage not discharged to the CSSC has two components:

1. Water supply used by communities whose sewage effluent is not discharged into the CSSC, and
2. The sanitary portion of combined sewer overflows that are not discharged to the CSSC, from Lake Michigan water supply, originating from communities whose sewage effluent is tributary to CSSC.

The value in Column 9 for Lake Michigan Pumpage not discharged to the Canal is based on water-supply pumpage records and does not consider consumptive use. The Supreme Court Decree specifies sewage effluent for the accounting, which would require subtracting the consumptive use from the water-supply-pumpage records. However, consumptive use varies widely among water suppliers and among geographic regions of the greater Chicago metropolitan area and the range of potential consumptive use values is not well defined. Therefore, the present best engineering practice does not provide clear guidance as to the value that should be subtracted from the pumpage records to account for consumptive use.

Column 10: Total diversion

Column 10 is the total Lake Michigan diversion that is accountable to the State of Illinois. Column 10 is equal to column 3 minus column 8 and plus column 9.

Column 11: Lake Michigan pumpage

Column 11 is the total Lake Michigan pumpage for which Illinois is accountable. The Lake Michigan pumpage is from water pumpage records of primary diverters of Lake Michigan water. They are measured at water-treatment plants or pumping stations.

Column 12: Simulated runoff from diverted Lake Michigan watershed

Column 12 is 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 programs Hydrologic Simulation Program – Fortran (HSPF), SCALP, and TNET models, and from streamflow-separation techniques.

Column 13: Total direct diversion from Lake Michigan

Column 13 represents the total direct diversion of Lake Michigan water into the diverted rivers systems through the controlling structure at Wilmette, the CRCW, and the O'Brien Lock. The values are reported by MWRDGC on their LMO-6 reports.

3.1.2. Description of Computational Budgets

Thirteen computational budgets compile input for the diversion calculation and estimate flows that cannot be measured. A summary of these budgets is presented in Table 3.1-c, Budgets 1 and 2 are summation of critical water-supply pumpage data. Budgets 3 through 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). Budgets 7 through 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.

Table 3.1-c - Description of the diversion accounting computational budgets

Budget Number	Title	Description
1	Diverted Lake Michigan Pumpage	This budget sums the Lake Michigan water diverted by the State of Illinois in the form of Industrial and Municipal 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 of 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 of 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 of 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 of this budget are used in Budget 14 and Column 12.
7	MWRDGC Northside Water Reclamation Plant	This budget performs hydrologic and hydraulic simulation of the service basin tributary to the MWRDGC Northside Reclamation Facility. The simulations estimates the runoff from portions of the Lake Michigan and Des Plaines River watershed 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 in Budget 14 and Columns 6 and 12.
8	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 simulation 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 simulations 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.
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 Column 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.

3.2. Hydrologic Modeling

The flow record at the AVM at Romeoville represents the majority (over 90 percent) of the volume of water diverted from Lake Michigan. But in addition to the CSSC 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, either as interceptor flows to water reclamation plants, as overflows to rivers tributary to the canal or to TARP, or as direct runoff from the Lower Des Plaines watershed, Summit conduit, or ungaged Calumet watershed.
- Groundwater pumpage that was treated and discharged from sewage treatment plants into the canal system.
- Water-supply pumpage from the State of Indiana.
- Water-supply pumpage from Federal facilities that was discharged into the canal.

The hydrologic model computes the runoff from the 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 function of the hydrologic and hydraulic modeling is to estimate the volume of runoff 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 function of the hydrologic and hydraulic modeling is to compute the components of budgets 7 through 14. From these budgets, the runoff from the diverted Lake Michigan watershed can be calculated. In addition, the budgets provide a verification of simulated flows and indicate problems 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 HSPF model simulates the hydrology and computes the runoff in inches for a majority of the basin. The runoff is applied to the SCALP program, which converts the runoff into sewer discharge and routes the flow through the sewer network. Flow greater than the capacity of the interceptor sewers overflows into dropshafts that convey water into the TARP tunnels. When the TARP tunnels are filled, the water overflows into the canal system or into the Des Plaines River. The one-dimensional unsteady flow model, TNET, routes the flow through the TARP tunnels and simulates the pumpage of wastewater into MWRDGS's Stickney and Calumet WRPs. The outfalls from the sewage treatment plants flow into the CSSC. There is no routing through the CSSC.

3.2.1.1. Precipitation Network

Rainfall is measured by a new (1990) network of 25 precipitation gages that is maintained by the ISWS, (Vogel, 1988; Pepler, 1991a). The gages were installed on a rectangular grid with a spacing of from 5 to 7 miles between the gages. Figure 3.2-a shows the location of the rainfall gages. The gages are all of a single type, a universal weighing bucket. The gages were located such that they were urban areas and were as free as possible in from obstructions. A quality-assurance program has been developed to estimate missing values and check for the consistency of the data. Average rainfall over a subarea is calculated using Thiesen polygons.

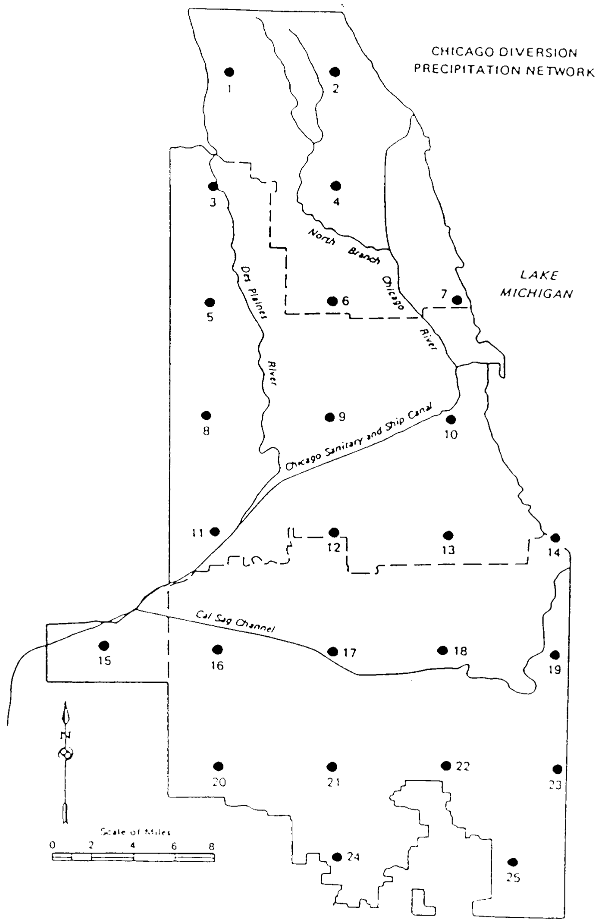

Figure 3.2–a - The 25-gage precipitation gage network.

3.2.1.2. HSPF Model

HSPF (Johanson and Others, 1984) is a continuous hydrologic model, which simulates 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 elements are divided into impervious, grassland, and forest areas. 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 the equations are determined by calibration to observed data. The program, which has its roots in the Stanford watershed model (Crawford and Linsley, 1966), 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:

3.2.1.2.1. Interception Storage

Precipitation is first lost to interception – retention on leaves, branches, and stems of vegetation. The HSPF model simulates 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.

3.2.1.2.2. Impervious Area Runoff

Precipitation onto an impervious area, such as a road or a parking lot, does not infiltrate and the entire precipitation is assumed to be surface runoff.

3.2.1.2.3. Infiltration

A portion of the precipitation that falls onto the pervious ground infiltrates into the soil. In the HSPF model, 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.

3.2.1.2.4. 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.

3.2.1.2.5. 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.

3.2.1.2.6. 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 evapotranspiration, and percolation to the deep groundwater storage.

3.2.1.2.7. 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.

3.2.1.2.8. 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.

3.2.1.2.9. 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 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.

3.2.1.2.10. Output

The output from the HSPF program is the surface and subsurface runoff in inches for the impervious, grassland, and forest portions of each subarea. In the runoff modeling, the HSPF program is only used to simulate the interaction between precipitation and the soil moisture reservoir.

3.2.1.3. SCALP

The SCALP program converts the unit surface and subsurface runoff from the HSPF model for each subarea into flow and routes the flow through separate and combined sewers. In addition, the SCALP program adds a sanitary component to the flow. Separate sewer systems have independent sanitary and storm sewers. For separate sewer systems, the SCALP program only simulates flow through the sanitary sewers, ignoring the storm sewers. In addition to the sanitary flow, the sanitary sewers also carry stormwater flow from infiltration before and after the storm event. The combined sewers carry both stormwater and sanitary flow. The water is conveyed through the sewer network using linear routing to the WRP. When the capacity of an interceptor sewer is exceeded, the water first overflows into a TARP dropshaft up to the capacity of the TARP tunnels, and then overflows into the CSSC.

The SCALP model reads the three HSPF unit runoff files: subsurface runoff (SUBRO), impervious runoff (IMPRO), and overland pervious-surface runoff (OLFERO). 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, respectively. 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.

Groundwater infiltration seeps into the sewer system through joints and fissures. The sewers also receive inflow from unregulated connections to the sewer system. Two examples of these 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 surface runoff be assigned as infiltration and inflow. Assigning 100% of the subsurface runoff to infiltration on the surface seems unusual. However, the model covers an urban area where the natural drainage ways, ditches and creeks, have been replaced with sewers; which receive the subsurface runoff.

The sanitary inflow is estimated by multiplying the population equivalent of a subarea by the per capita loading. The sanitary flow is the dry period base flow of the sewers. The per capita loadings for the three WRPs are shown in Table 3.2-a.

Table 3.2-a - Per capita loading for each WRP area

Water Reclamation Plant	Minimum Per Capita Loading in ft^3/sec	Maximum Per Capita Loading in ft^3/sec	Average Per Capita Loading in ft^3/sec
Northside	2.468×10^{-4}	2.550×10^{-4}	2.509×10^{-4}
West-Southwest (Stickney)	1.928×10^{-4}	4.488×10^{-4}	2.679×10^{-4}
Calumet	1.228×10^{-4}	9.299×10^{-4}	3.670×10^{-4}

The sanitary runoff has been calibrated using the per capita loading rather than population equivalents. Once again this is unusual but not incorrect.

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,

$$Q = \frac{1}{K} S \tag{3.2-a}$$

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 flow, 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 HEC DSS data base.

3.2.1.4. TNET

TARP consists of a network of deep tunnels underlying the City of Chicago. The tunnels collect sanitary and stormwater runoff that overflows from the interceptor sewers up to the storage capacity of the tunnels. The Mainstream tunnel system extends from Wilmette to the Hodgkins pump station near the West-Southwest (Stickney) WRP. As of the 1995 diversion-accounting report (USACE, 1998), only the middle (below Dempster Avenue) and south legs of the Des Plaines tunnel were operational. The northern leg was completed in early 1999. The Des Plaines tunnel is not connected directly to the Mainstream tunnel, but the tunnels are both evacuated by the Hodgkins pumping station. The Chicago pumping station evacuates the Calumet TARP System to the Calumet WRP. Figure 3.2-b shows the tunnel system as of 1994, based on the 1995 diversion accounting report.

Gates on the drop shafts control the flow into the tunnel systems. An operator starts closing the gates when the storage inside the tunnel exceeds 40 percent of the total capacity. The gates are completely closed before the tunnels are completely filled to prevent pressure surges and to provide storage for ungated sewers directly connected to the tunnels. When the gates are closed, wastewater overflows into the canal system.

After the storm event, the storage of the tunnel is pumped to the West Southwest (Stickney) WRP and Calumet WRP and then discharged to the canal system.

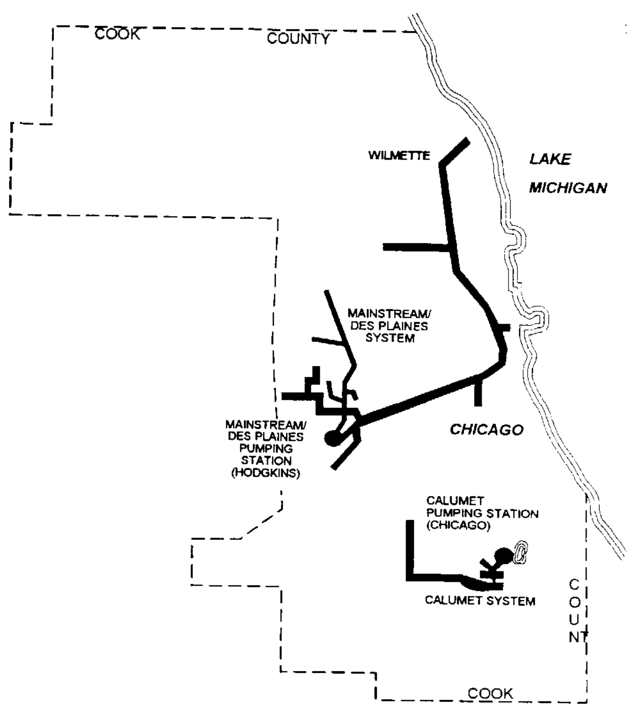

Figure 3.2–b - The Chicago TARP tunnels as of 1996.

To increase storage, reservoirs are being designed for the Mainstream and Calumet tunnel systems.

The TNET model (Barkau, 1991) simulates the tunnel systems. 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 Preissman slot (Cunge and others, 1986), 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 is the same as the celerity of a pressure wave. The width of the slot is generally about 0.001 ft in width, which produces a wave celerity of about 4,900 ft/sec.

The overflows, which were computed by SCALP, are input to the drop shafts. The TNET program sets the drop shaft gates according to elevation of water in the tunnel. When the gates are closed, the overflow is dumped to the canal and written to DSS. The operating plan in the TNET program for the dropshaft gate closures, roughly approximates the current operating plan used by MWRDGC. However, the operator may deviate from this policy and the volume of water input into the tunnels by TNET and the volume of water in the actual operation may be different.

3.2.2. Simulation Results

The overall goal of the hydrologic modeling is to reproduce the annual runoff from the basin. Therefore, the S/R ratio in budgets 7, 8, 9, 10, 11, 12, 13 and 14 provide an overall picture of the quality of the hydrologic modeling. Table 3.2-b summarizes the hydrologic modeling from 1990 through 1995.

The model reasonably reproduces the annual runoff of the system. However the model is not perfect. Figures 3.2-c to 3.2-e compare simulated and observed flow for 1995 at the Northside – 5 percent error, Stickney – 2 percent error, and Lemont – 32 percent error WRPs. Errors can be seen in the reproduction of both high and low flow, but overall, the errors balance producing annual runoff that ranges from 2 to 32 percent less than the measured flows. The performance of the model can be improved by significant effort, calibrating the model by event and by adjusting sanitary inflow by month, but the annual runoff would not be changed.

The simulation of the TARP pumping station and the Calumet pumping station using TNET is not as accurate. Both models show clear biases; the simulation of the TARP pumping station, with an average S/R ratio of 1.22, consistently overestimates the pump station flow and the simulation of the Calumet pumping station, with an average S/R ratio of .73, consistently underestimates flow.

Table 3.2-b - Simulated to recorded ratios for budgets 7 through 14 from 1990 to 1995.

Budget	Description	S/R Ratio for 1990	S/R Ratio for 1991	S/R Ratio for 1992	S/R Ratio for 1993	S/R Ratio for 1994	S/R Ratio for 1995	Average S/R
7	Northside WRP	0.94	0.94	0.95	0.95	0.97	0.95	0.95
8	Upper Des Plaines Pumping Station	1.08	1.04	1.00	0.92	0.86	NA	0.98
9	Mainstream TARP Pumping Station	1.05	1.35	1.51	1.06	1.23	1.14	1.22
10	Stickney WRP	1.07	10.4	1.09	1.07	1.04	0.98	1.05
11	Calumet TARP Pumping Station	0.73	0.81	0.89	0.61	0.75	0.58	0.73
12	Calumet WRP	1.00	1.00	1.05	1.06	1.02	0.99	1.02
13	Lemont WRP	0.86	0.76	0.79	0.88	0.82	0.68	0.80
14	Chicago Canal System Balance	1.15	0.90	0.90	0.99	0.98	1.00	0.98

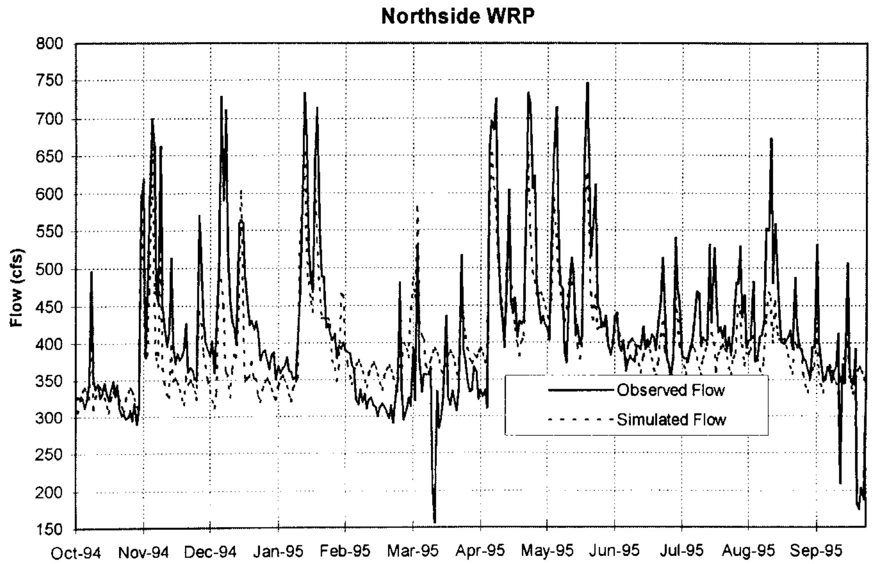

Figure 3.2–c - Comparison between simulated and observed flow at Northside WRP for 1995.

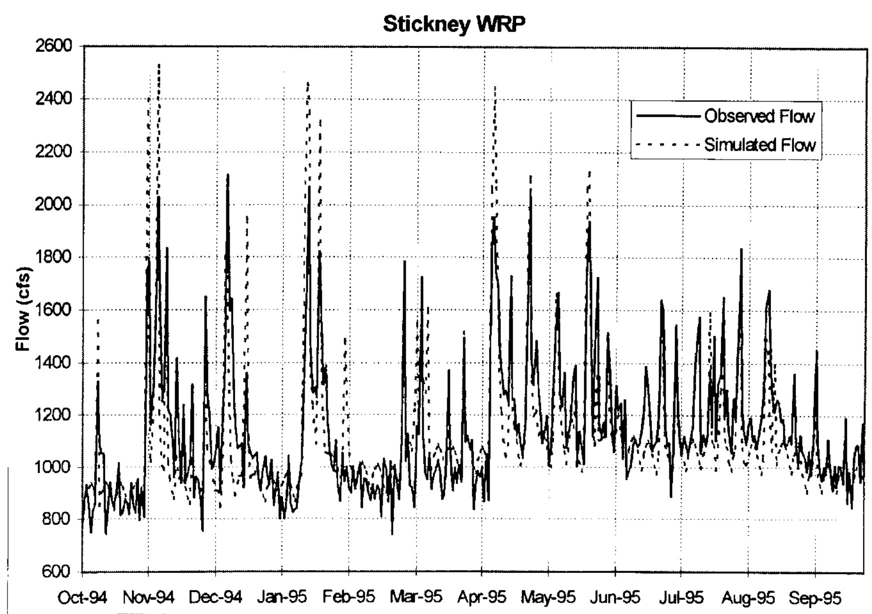

Figure 3.2–d - Comparison between simulated and observed flow at Stickney WRP for 1995.

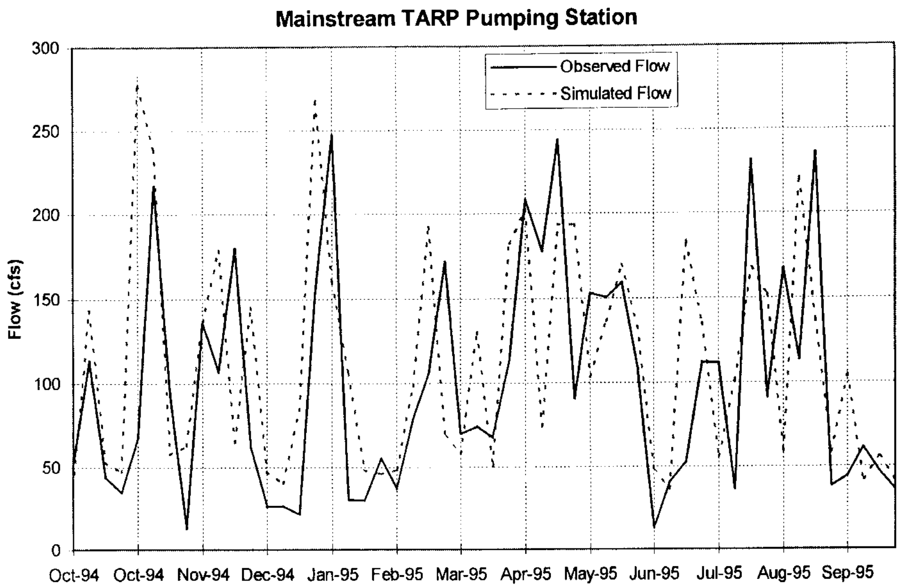

Figure 3.2–e - Comparison between simulated and observed flow at the Lemont WRP for 1995.

Figure 3.2-f compares simulated and observed flow at the Hodgkins TARP pumping station for 1995. For the TARP system, the groundwater flow of 55 ft^3/sec is about 50 percent of the total inflow into the tunnels. The Technical Committee recommends that the USACE investigate reducing the groundwater inflow and increasing the inflow to the tunnel by postponing the closure of the drop shaft gates. The latter can be accomplished by increasing the elevations at the index stations when the drop shaft gates close. Calibrating by storm event may also improve the simulation of the individual events. This procedure would divide the time series into a series of events. The model parameters are assumed constant over the event and the parameters are optimized to provide the best simulation of the event. This is a technique that is used on period-of-record simulation of large rivers, where hydrologic parameters are not constant. The technique does provide a better answer, but at increased work. For the diversion-accounting problem, where an annual flow is all that is required, the procedure will not improve the annual number, but it will improve the overall simulation.

Another problem is the starting tunnel storage. The TNET program pumps down the tunnel when the water level exceeds a starting elevation; thus the tunnel may be above minimum storage at the start of an event. At the forecast of significant runoff, MWDRGC pumps down the tunnel, hence, the tunnel is always at minimum storage at the start of an event. Adding a forecast capability to the TNET program would pump the tunnel down prior to an event.

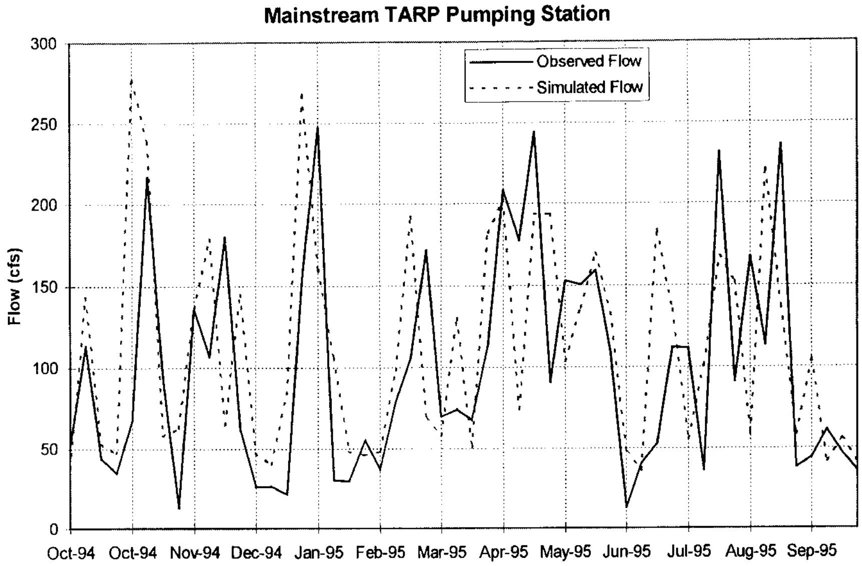

Figure 3.2–f - Comparison between simulated and observed flow at the Hodgkins (mainstream) TARP pumping station for 1995.

The Calumet model is a more complex problem. Figure 3.2-g compares simulated and observed flow at the TARP pumping station for 1995. The pump-station algorithm of the TNET program was developed for the TARP pump station and not the Calumet pumping station; therefore, the timing of the pumping may be improper. The pump-station algorithm will not effect the overall volume of the pumping. The USACE has contracted to upgrade the pump-station algorithm. There is a deficiency of volume entering the tunnels, both during base flow and during storm events. The inflow can be increased by postponing the closure of the drop shaft gates and by increasing the groundwater flow into the tunnels. Secondly, the sewer connections to the tunnel and should be checked to ensure that all inflow enters the tunnel. The Calumet pumping station has no impact on the diversion accounting flow since the overflows from the drop shaft gates are returned to the canal system.

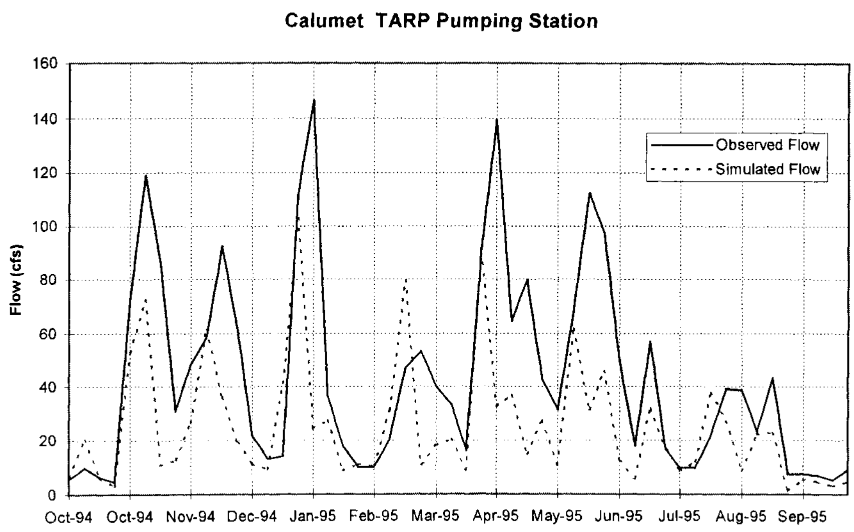

Figure 3.2–g - Comparison between simulated and observed flow at the Calumet pumping station for 1995.

The per capita load has been used as a calibration factor of the sanitary flow (Calumet – Table 3.2-a). The range of per capita loading from 1.228×10^{-4} to 9.229×10^{-4} ft^3/sec is unusual. The Technical

Committee recommends that the population equivalent be used as a calibration factor and a more realistic per capita loading be used for each basin.

The Des Plaines pumping station is still a problem. The Des Plaines pumping station is the calibration point for the runoff model of the Des Plaines watershed. Runoff from the Des Plaines watershed is a deduction. In the report of the Third Committee, the Committee recommended improving the measurement of discharge at the pumping station. The improved measurements have not been made because of the cost. Since the Third Committee, the amount of missing data has increased to a point where a comparison could not be made for 1995. At present the station is of no value as a calibration point.

3.2.3. Hydrologic Balance

A double-mass curve analysis was used to look for changes in the relations among components of the diversion accounting that are determined by modeling of the watershed and drainage system. Several changes have been made in the modeling procedures and assumptions since the review by the Third Technical Committee in 1993 (Espey and others, 1994). Among these changes are:

- Implementation of the 25-station raingage network (Peppler, 1991a);
- Revision of the land-cover assignments and associated model parameters for the 214 Special Contributing Areas (SCA's) (Rust Environment and Infrastructure, 1993);
- Improvements on HSPF model parameters (Corps of Engineers, 1995, Appendix B, page 8); and
- Addition of the southern and middle portions of the Des Plaines TARP system to the model (Corps of Engineers, 1997, p. 11).

Double-mass curves were used to compare computed runoff components with the average precipitation for water years 1990 through 1995. The double-mass curve comparing the runoff from the Des Plaines River watershed that reaches the CSSC (Column 6 of diversion-accounting table) with the precipitation (Figure 3.2-h) shows a break in the slope near the end of water-year 1993. Linear regression was used to fit straight lines to the data prior to and after the southern and middle portions of the Des Plaines TARP system were placed into service (June 6, 1993). The break in the slope of the curve indicates that, on the average, the modeled runoff from the Des Plaines watershed from a unit rainfall has decreased three percent since June, 1993. As double-mass curves provide a more qualitative, rather than quantitative analysis, this break may be the result of one or several of the changes in the accounting system around this period.

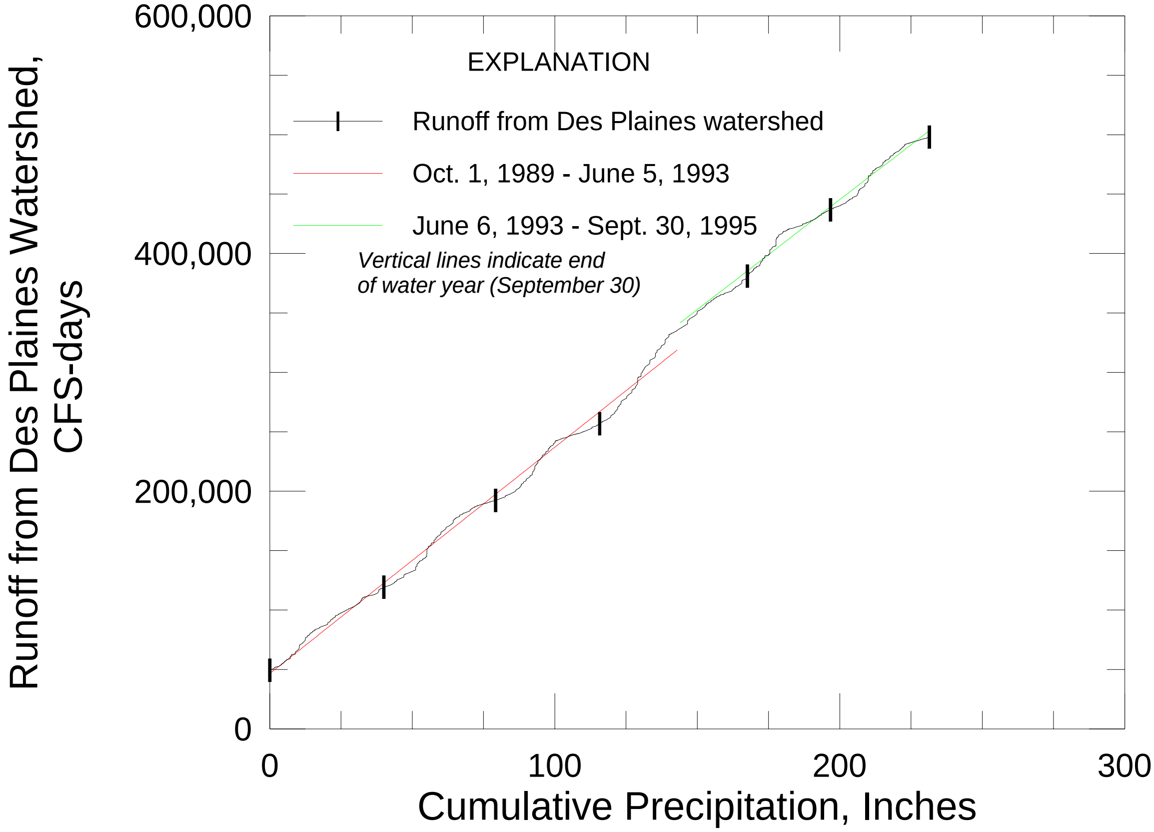

Figure 3.2–h - Double-mass-curve comparison of runoff from the Des Plaines River watershed reaching the canal and precipitation, water years 1990 through 1995.

A double-mass curve comparing the runoff from the diverted Lake Michigan watershed (Column 12 of diversion-accounting table) with the precipitation (Figure 3.2-i) for the same period showed a similar break in slope. Linear regression was used to fit straight lines to the data prior to and after June 6, 1993. The slopes of the curves before and after this date showed that the runoff from a unit rainfall decreased approximately eight percent after June 6, 1993. This indicates that the revisions to the diversion-accounting methods around this period adjusted the modeled relation between rainfall and runoff, and that the break in the runoff from the Des Plaines watershed reflected this overall change in the rainfall-runoff relation, as well as the increased runoff from bringing the Des Plaines TARP system on-line.

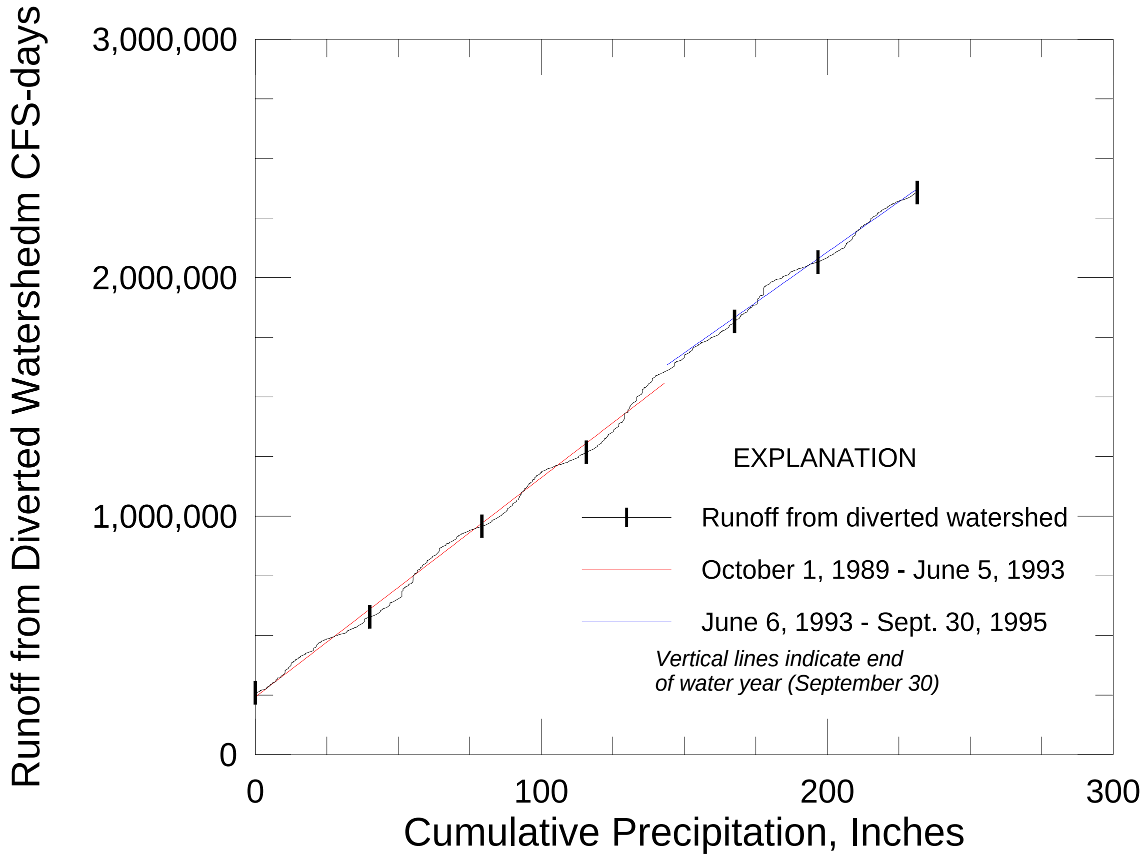

Figure 3.2–i - Double-mass-curve comparison of runoff from the diverted Lake Michigan watershed and precipitation, water years 1990 through 1995.

3.2.4. Accounting Report Budgets

Fourteen water budgets are presented in the annual accounting reports. These budgets are used: (1) to sum the diverted water supply; (2) as part of the calculation of runoff from the diverted Lake Michigan watershed; (3) to compare flows estimated from the model with measured flows; (4) to compute column inputs used in the diversion computations and summary tables; and (5) to examine the balance of total inflows and outflows. The budgets comparing flows estimated from the model with measured flows provide an indication of the errors associated with using the model to estimate various components of the accounting calculations.

The models are an approximation to the physical processes occurring in the basin. Flows estimated from the models will be less accurate than measured flows, since a perfect calibration to the measurements would be limited to the accuracy of the measurements. The errors between the modeled and measured flows should be normally distributed with a mean of zero for the model to provide a good estimate of the flows. If the distribution of errors is non-normal or is not centered around zero, use of modeled flows will introduce a bias into the calculations made using model results. While data to thoroughly examine the distribution of errors between the modeled and measured flows were not provided to the Technical Committee, results from the annual accounting reports (USACE, 1994, Appendix B; 1995a, Appendix B; 1995, Appendix C; 1997, Appendix A; 1997 Appendix B; 1998, Appendix A) provide sufficient data to identify biases in some of the budgets. Table 3.2-c summarizes the ratio of flows estimated from the model to measured flows (S/R ratio) for water years 1990 through 1995 for seven budgets that compare flows estimated from the model with measured flows and for the canal system balance. Correlation coefficients (r^2) reported for budgets 7, 10, 12, and 14 indicated that these four budgets consistently explained more than 64 percent of the variance between the modeled and measured flows. The mean (1.000) and standard deviation (0.063) of ratios from these four budgets was assumed to represent the normal distribution of S/R ratios for model results that provide a reasonable estimation of the measured flows. The frequency of ratios: (1) more than one standard deviation smaller than one (S/R less than 0.94); (2) within 1 standard deviation of one (0.94 <= S/R <= 1.06); and (3) more than one standard deviation greater than one (S/R > 1.06) were quantified for each budget. The Chi-squared (X^2) test was used to determine whether the observed ratios differed from the normal distribution (at the 90-percent significance level). Results from the X^2 test indicate that budgets 9, 10, 11, and 13 do not follow the expected normal distribution. Budgets 9 and 10 indicate that modeled flows consistently overestimate the measured flows, while budgets 11 and 13 indicate that modeled flows consistently underestimate the measured flows.

Biases in the modeled flows for Budgets 9 and 10 may affect the computed diversion. Results from modeling of the Mainstream and Des Plaines TARP pump station (Budget 9) are used to compute part of the Des Plaines River watershed runoff deduction (Column 6) and part of the groundwater seepage into the TARP system, which is part of the Column 4 deduction. The observed bias of simulated flows exceeding measured flows implies that the Des Plaines River watershed runoff deduction and/or the groundwater seepage into the TARP system will be overestimated by this model. Accounting reports for water years 1990 through 1992 stated that, 'base flows appear to be overestimated in the simulation. This is probably due to overestimation of groundwater infiltration into the TARP tunnels.' (USACE, 1994, Appendix B, p. 31; 1995, Appendix B, p. 31; and 1995, Appendix C, p. 31). The probable overestimation of groundwater inflows into the TARP tunnels was not mentioned in the accounting report for water years 1993 through 1995, but the simulated flows remain 6 to 23 percent greater than the measured flows.

Table 3.2-c - Ratio of flows estimated from the model to measured flows (S/R ratio) for budgets 7 through 14 for water years 1990 through 1995.

[Bold font indicates budgets where X^2 test indicates significant difference from the normal distribution]

	Budget							
	7	**8**	**9**	**10**	**11**	**12**	**13**	**14**
Year	Northside WRP	Upper Des Plaines pump station	Mainstream TARP pump station	Stickney WRP	Calumet TARP pump station	Calumet WRP	Lemont WRP	Chicago canal system balance
1990	0.94	1.08	1.05	1.07	0.73	1.00	0.86	1.15
1991[1]	0.94	1.04	1.35	1.04	0.81	1.00	0.76	0.9
1992[1]	0.95	1.00	1.51	1.09	0.89	1.05	0.79	0.88
1993	0.95	0.92	1.06	1.07	0.61	1.06	0.88	0.99
1994	0.97	0.86	1.23	1.04	0.75	1.02	0.82	0.98
1995	0.95	--	1.14	0.98	0.58	0.99	0.68	1.00
Average	0.950	0.980	1.223	1.048	0.728	1.020	0.798	0.9833
Standard Deviation	0.011	0.089	0.180	0.039	0.118	0.029	0.073	0.096
S/R < 0.94	0	2	0	0	6	0	6	2
0.94<= S/R <= 1.06	6	2	2	3	0	6	0	3
S/R > 1.06	0	1	4	3	0	0	0	1
	Chi-Square test for Budgets							
Test statistic	2.789	2.475	**11.784**	**5.652**	**31.818**	2.789	**31.818**	1.450
x^2	4.605	4.605	**4.605**	**4.605**	**4.605**	4.605	**4.605**	4.605

[1] Values for all water years are based on table 7 in published water-year accounting reports. Values for water years 1991 and 1992 do not agree with ratios calculated from mean simulated and recorded flows from same table.

Accounting reports for water years 1990 through 1992 also stated that, ‘bypass flows are discharged to TARP, when available, via drop shaft 11 (DSN 11)’ (USACE, 1994, Appendix B, p. 31; 1995, Appendix B, p. 31; and 1995, Appendix C, p. 31). The water year 1990 accounting report indicates that this would ‘account for the simulation of a pumpage volume that is less than the recorded pumpage volume’ (USACE, 1994, Appendix B, p. 31). It is reasonable that bypass flows to the TARP tunnels that were not included in the model would, as this report stated, result in simulated flows that are less than the recorded flows; however, the simulated flows are consistently **greater** than the recorded flows. If these bypass flows are ‘a frequent occurrence,’ this would indicate that the bias in the simulated flows is greater than the S/R ratio indicates, as there is a known (but not quantified) inflow to the TARP system that is not included in the model. This same explanation for the large S/R ratios is given in the accounting reports for water years 1991 and 1992 without the statement that this would ‘account for the simulation of a pumpage volume that is less than the recorded pumpage volume’ (USACE, 1995, Appendix B, p. 31; and 1995, Appendix C, p. 31). The un-quantified bypass flows into the TARP tunnels was not mentioned in the accounting report for water years 1993 through 1995, but the simulated flows remain 6 to 23 percent greater than the measured flows. It is not stated whether these flows are now included in the model or not.

The biases in budgets 11 (Calumet TARP pump station) and 13 (Lemont water reclamation facility) may affect the groundwater infiltration (column 4) component of the current diversion accounting. Flows from two of the major SCA’s flow into the Calumet TARP system without restriction, essentially using TARP as a sanitary interceptor sewer. Several attempts have been made to improve the simulation of these flows into the Calumet TARP system, and thereby improve the calibration of this component of the accounting models. These are described in detail in the water year 1989 accounting report (USACE, 1992, Appendix E). The current simulation assumes that only interceptor overflow from two separately-sewered SCAs (CA19-R1 and CA1W) and from one combined-sewer SCA (CA14) are routed through TARP. The simulated overflows (0.29 ft^3/sec) were subtracted from the average TARP flows for eight dry-weather periods in water year 1989 (7.32 ft^3/sec). The difference (7.03 ft^3/sec) is used as the groundwater infiltration rate for the TARP tunnels. The Technical Committee recommends that the analysis of groundwater infiltration into the Calumet TARP tunnels needs to be reviewed using data from more than one year. The modeling procedures should then be revised to reflect any changes to the groundwater infiltration. In light of the consistent underestimation of flows by 11 to 42 percent by these models, the basic procedure used to estimate the flows routed through the TARP tunnels needs to be addressed for these models to be used to help determine runoff from the diverted Lake Michigan watershed.

3.3. Romeoville AVM System

The USGS streamgage on the CSSC at Romeoville, Illinois is the primary measurement point for the Lake Michigan diversion accounting. For water years 1990-1995, between 92 and 97 percent of the total accountable diversion flowed past the Romeoville gage.

3.3.1. Description

The gauge at the CSSC at Romeoville, Illinois, is immediately north of the Romeoville Road bridge on the east side of the canal, approximately 5 miles north of Lockport Lock and Dam. This station is an Accusonic O.R.E. AVM with four acoustic paths. Three paths are installed pointing southeast at an angle of 44.5 degrees to the flow, path lengths of 236 ft, and at elevations of approximately 18.1, 13.8, and 9.8 ft above the canal bed. The fourth path points southeast at an angle of 44 degrees to the flow, with a path length of 235 ft and at an elevation of 14.7 ft above the canal bed. The station was installed in March, 1984. The original AVM was replaced with the current O.R.E. unit in November, 1989.

Until July 1993, discharge measurements for rating analysis at this site were done using Price AA current meters. Beginning in July 1993, discharge measurements were done using a broadband ADCP. Most discharge measurements were done at a defined cross-section located half-way between the upstream and downstream AVM transducers.

3.3.2. Measurement errors

The measurement cross-section was surveyed in June, 1991 and October, 1993. The 1993 survey showed the cross-section area to be 41.5 ft^2 (0.9 percent) greater than the 1991 survey at the normal stage of 25.50 ft. The cross-section areas determined from the published stage-area rating for this site were 87 ft^2 (2.0 percent) lower than measured during the 1991 survey and 128.5 ft^2 (2.9 percent) lower than measured during the 1993 survey for the normal stage of 25.50 ft.

The error between the stage-area rating and the 1991 survey are constant with depth, and essentially are the equivalent of a datum offset of 0.54 ft. The error between the stage-area rating and the 1993 survey changes slightly with depth, ranging from 126 ft^2 (3.5 percent) at the lowest stages during rating measurements to 128.5 ft^2 (2.9 percent) at the highest stages during rating measurements.

These errors in the stage-area rating will result in a bias in the index-velocity rating developed from discharge measurements at this site. The measured discharge is divided by the rated area for the stage during the measurement to determine the mean velocity in the cross-section. Because the rated area is consistently too small, the calculated mean velocities will be consistently too high. When these are used to develop the index-velocity rating, the mean velocities calculated from the rating will also be too high. An error analysis of discharge measurements 52 through 77 made from November 1986 through November 1993 indicates errors in the calculated mean velocities ranging from 0.009 ft/s to 0.103 ft/s (2.0 to 2.4 percent, with an average of 2.0 percent). These will translate into a bias in the index-velocity rating of 2.0 percent.

The discharge record for this station is calculated by: (a) calculating the cross-section area of the flow from the stage and the stage-area rating; (b) calculating the mean velocity in the cross-section from the index-velocity rating; and (c) multiplying the area times the mean velocity to calculate the discharge. Since the same stage-area rating used in the development of the index-velocity rating is used to calculate the discharge, the error in the index-velocity rating (+2.0 percent) will tend to cancel the error in the stage-area rating (-2.0 percent), resulting in an average bias in the calculated discharges of 0.04 percent, or about 1 ft^3/s.

The errors in the cross-sectional area measurements are less than expected measurement errors. The USGS reports slumping of the canal walls that can significantly affect the measured depth and area with a small longitudinal change in the measurement section. The USGS has decided to continue to use the original stage-area rating because the changes in area and their effect on the calculated discharges are not significant. As part of their quality-assurance plan for this station, the USGS continues to make periodic measurements of the cross-sectional area of the measurement section.

3.3.3. ADCP Measurement Errors

Until July 1993, discharge measurements at this site were done using Price AA current meters. Beginning in July 1993, discharge measurements were done using a broadband ADCP. The accuracy of discharge measured with Price AA current meters has been well-documented (Carter and Anderson, 1963; Dickenson, 1967; Herschy, 1970, 1975, 1978; Pelletier, 1988; and Smoot and Carter, 1968; Wahl, 1977), and will not be explored further in this review. Use of an ADCP to measure discharge could potentially improve the accuracy of the discharge measurements for rating analysis. The measurement method used by the ADCP addresses and improves several of the sources of error in discharge measurements. The ADCP integrates water velocities over a series of uniform-depth ‘bins’ rather than measuring at discrete points in the vertical, and typically measures more ‘bins’ than the number of points measured in each vertical with a Price AA current meter (or other point-velocity meters). ADCP measurements often will include more verticals (‘ensembles’) than would be measured with a point-velocity meter, although there are tradeoffs between the number of ‘ensembles’ measured, the averaging time for each ‘ensemble’, and the overall measurement time. The ADCP is not affected by oblique flow as it calculates the flow normal to the instrument path for each ensemble. In general, the effects of vertical motion (boat pitch and roll and vertical velocity components) do not affect the accuracy of the ADCP. In addition, an ADCP measurement requires significantly less time than a Price AA measurement (3-5 minutes with an ADCP, compared to 50-75 minutes with a Price AA meter). This allows more accurate comparison between ADCP and AVM measurements during unsteady-flow conditions.

3.3.3.1. Random ADCP Measurement Errors

There are several sources of error in ADCP measurements that may not be smaller than those for Price AA measurements. These can generally be categorized into uncertainties in the current meter, uncertainties in the discharge computations, uncertainties in the estimation of unmeasured near-shore discharges, and uncertainties from operator error. These can be further subdivided into random uncertainties and systematic bias. Simpson and Oltman (1992, Appendix B) provide a detailed error analysis for discharge measured with a narrow-band ADCP. Although a broadband ADCP was used for the measurements at the Romeoville AVM, parts of Simpson and Oltman's error analysis are applicable to evaluate the rating measurements at Romeoville.

Random uncertainties in the current meter are uncertainties in the measurements of the water velocity, the boat velocity, the water depth, and the pitch and roll of the instrument. The manufacturer lists the precision of a 'single-ping' determination of water velocity for different instruments and configurations. For a 1,200 kilohertz (kHz) broadband ADCP with a 0.5-meter bin size, the 'single-ping' standard deviation is ±4 cm/sec (±0.13 ft/sec). The precision of a water-velocity determined from averaging multiple 'pings' is the 'single-ping' precision divided by the square-root of the number of 'pings' averaged. For a configuration averaging twelve 'pings' per 'ensemble,' the standard deviation of the measured water velocity will be 1.15 cm/sec (0.038 ft/sec).

The manufacturer lists the precision of a 'single-ping' determination of bottom velocity[2] as a function of the frequency, the boat velocity, and the depth. For a 1,200 kHz broadband ADCP in 8.2 meters of water (27 ft) and a boat speed of 17 cm/sec (0.6 ft/sec), the 'single-ping' standard deviation is ±0.94 cm/sec (±0.03 ft/sec). The precision of a boat-velocity determined from averaging multiple 'pings' is the 'single-ping' precision divided by the square-root of the number of 'pings' averaged. For a configuration averaging six 'pings' per 'ensemble,' the standard deviation of the measured boat velocity will be 0.19 cm/sec (0.006 ft/sec). This was checked by comparing the standard deviation of measured distance for 59 measurements on a measurement section, and the using this to estimate the standard deviation of the measured boat velocities. The measured standard deviation of boat velocities was within 0.03 cm/sec of the theoretical standard deviation.

The manufacturer lists the precision of the depth measurement as 1 percent of the measured depth ±120/frequency (kHz). For the 1,200 kHz ADCP in 8.2 meters of water, the precision of the depth measurement is 8.2 cm ±10 cm; using the maximum error gives a precision of 18.2 cm (0.60 ft).

The manufacturer lists the accuracy of the pitch and roll sensors as ±1 degree. A first-order error analysis (Ang and Tung, 1975, p. 199) was used to estimate the effect of an error of ±1 degree on the measured mean water velocities for the conditions of the Romeoville site. The error would vary with the orientation of the ADCP relative to the mean water velocity. For mean water velocities at Romeoville of 1.1 ft/sec, the single-ping error in the measured water velocity would range from 0.1 to 5.1 percent (0.001 to 0.056 ft/sec). For a configuration averaging twelve 'pings' per 'ensemble,' the error of the measured water velocity should be 1.5 percent or smaller.

Simpson and Oltman (1992) list the random uncertainties in the discharge calculation as uncertainty due to the inability of point sampling to adequately define the velocity distribution and area in a subsection.

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[2] The boat velocity is equal in magnitude and opposite in sign to the bottom velocity.

The errors in defining the velocity distribution are broken down into (a) the short-term instrument precision (discussed above); (b) the effects of short-term turbulence on the determination of a depth-averaged water velocity; and (c) errors caused by difference between the true and estimated water velocities in unmeasured portions of the section. Simpson and Oltman (1992) discussed these errors independently, but lumped them into a 'reasonable and conservative' error of 5.55 percent. For the mean water velocities of 1.1 ft/sec at Romeoville, the short-term instrument precision of 0.038 ft/sec would be a random error of 3.4 percent. Using Simpson and Oltman's value of 5.55 percent for the error in defining the velocity distribution, would give the error due to the inability of point sampling to adequately define the velocity distribution as 4.3 percent $\left(0.0555 = \sqrt{(0.043)^2 + (0.034)^2}\right)$.

The errors due to the inability of point sampling to adequately define the area in a subsection depends on the cross-section geometry. For a rectangular channel such as Romeoville, the cross-section can be adequately defined with only a few depth measurements; therefore this term was considered to be negligible.

The ADCP cannot measure an area near to the edges of the channel because of 'side-lobe' echoes from the channel walls. The discharge in these unmeasured areas ('near-shore discharges') is estimated based on the measured depth, velocity, and distance from the shore for the last measured 'ensemble'. The default estimation for discharge in this unmeasured area is based on an equation presented by Simpson and Oltman (1992, p. 9). This estimation was developed by applying a velocity interpolation equation described by Fulford and Sauer (1986) to a triangular section. This equation was modified by the USGS for vertical walls while measuring in lock chambers (Oberg and Schmidt, 1994). Assuming this same equation was used for Romeoville, the near-shore discharges were calculated as:

$$Q_e = 0.91 V_m L d_m \tag{3.3-a}$$

where Q_e is the near-shore discharge; V_m is the mean measured velocity from the last ensemble, or the mean velocity from all valid bins over the specified averaging interval (space or time averaging) nearest the shore; L is the distance from the last 'ensemble' to the shore; and d_m is the last measured depth.

A first-order error analysis was done of Equation 3.3-a using 0.038 ft/sec[3] as the variance of the velocities, 0.6 ft as the variance of the depth, and the variance in the measured edge distances from 118 observations on a defined measurement section (1.1 ft). The calculated variance in the near-shore discharges was 34.7 ft^3/sec or 11.9 percent of the edge discharge.

For the Romeoville measurements many of the effects of operator error should be negligible, because the cross-section, starting and ending points, method and speed of crossing the channel, and instrument configuration are predefined in the quality-assurance plan. Operator error may still be introduced, however in how the ADCP is positioned, in measuring the transducer depth, in measuring the near-shore distances, and in starting and finishing the measurement. Many of these errors have already been estimated as part of the variance in other components of the error analysis. Therefore, no separate term will be included for operator error.

Based on the above calculations for 59 ADCP measurements at Romeoville, the random error in the ADCP measurements is 41.8 ft^3/sec or 0.9 percent of the total discharge.

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[3] Based on a single ping

3.3.3.2. Systematic Errors in ADCP Measurements

Systematic errors (bias) are significant because, unlike random errors, systematic errors cannot be reduced by data averaging. Systematic errors have a direction and can add to or subtract from the overall systematic uncertainty. Systematic instrument errors are more likely to affect ratings developed from ADCP measurements than those developed from conventional current-meter measurements. Two primary factors increase the potential for bias from ADCP measurements compared to conventional measurements: (1) typically a single ADCP is used for all measurements; and (2) there are not well-defined, documented calibration procedures for ADCPs. However, it should be noted that while most measurements were made with a single ADCP, the USGS has used at least four different ADCPs in developing the rating for Romeoville.

Standard practice for discharge measurements will result in eliminating bias from the current-metering instrument. It is very unlikely that the same current meter will not be used for all measurements at a site. In developing procedures for streamgauging, the USGS has taken care to eliminate systematic error to the extent possible from the standard procedures. Rantz (1982b, p.346) states:

> *In making a check measurement, the possibility of systematic error is eliminated by changing the measurement conditions as much as possible. The* ***meter and stopwatch are changed*** *[emphasis added], or the stopwatch is checked against the movement of the second hand of a standard watch. If the measurements are made from a bridge, boat, or cableway, the measurement verticals are changed by measuring at verticals between those originally used; if wading measurements are being made, a new measurement section is sought or the measurement verticals in the original measurement are changed.*

This practice of changing meters, measurement locations, watches, etc., will have the effect of randomizing the bias, and thus allowing the error to be reduced by averaging.

In contrast, a single ADCP is often used for all measurements at a site, because these instruments are not as readily available as conventional current meters. Because of this, any systematic error in the instrument will <u>not</u> be randomized, and thus will result in a systematic error in the rating developed from the measurements. In addition, because a fixed measurement section in used for this site, any systematic error from the cross section also is <u>not</u> randomized. As was indicated earlier, the USGS has used at least four different ADCPs in developing the rating for Romeoville.

Unlike point-velocity meters, which can be calibrated relatively easily in a tow-tank, there are currently not accepted procedures to calibrate ADCPs. The USGS has "no official position at present on the accuracy of ADCPs (K. Oberg, oral commun., February 2, 1999). Several studies have been done to investigate methods to calibrate ADCPs and/or components of ADCP measurement and processing. The National Ocean Service has done studies in tow-tanks and lakes (Appell and others, 1988). The USGS has done some tests in tow tanks (K. Oberg, oral commun., February 2, 1999). The USGS evaluated ADCP-measured discharges with river discharges from 'conventional' methods for twelve sites (Morlock, 1996). This study was done because "although there have been several laboratory studies and some field experiments, quantitative information on the performance of ADCPs under field conditions is relatively rare but essential to proper assessment of the potential uses and limitations of these instruments." The following is excerpted from the USGS's quality-assurance plan for broadband ADCPs (Lipscomb, 1995, p. 7) and indicates that the standard for ADCP calibration is ±5 percent.

> *Each ADCP should be checked annually by making a discharge measurement at a site where the ADCP-measured discharge can be compared with a known discharge derived from some other source. An example of such a site would be one where a stable stage-discharge relation with no significant shifting has been established over a period of several years. The site ideally would be chosen to minimize the amount of unmeasured sections near the banks or in shallows and should no be near any large steel structures, such as bridges, that might affect the ADCP's compass. The discharge obtained using*

the ADCP must be within 5 percent of the known discharge. If these measurement fail to agree with the known discharge, the ADCP must be returned to the manufacturer for further evaluation and calibration if necessary. These check measurements must be fully documented and a summary log of the results kept on file in the District or Field Office and noted in the applicable station analysis.

Results from the USGS's evaluation (Morlock, 1996, p. 27) indicate that 26 of the 31 measurements differed by less than 5 percent from the discharges determined by conventional methods and all 31 ADCP measurements were within 8 percent of the discharges determined by conventional methods. Morlock (1996, p. 1) also noted that standard deviations of the ADCP measurements were generally higher than measurement errors predicted by error-propagation analysis of ADCP instrument performance. Morlock (1996, p. 1) indicated that "substantial portions of measurement error may be attributable to sources unrelated to ADCP electronics or signal processing and are functions of the field environment."

Simpson and Oltman (1992, pp. 26-30) list several sources of systematic errors in ADCP-measured discharges. The *Systematic Uncertainty due to Uncompensated Vessel-Attitude changes* will not be a factor, as the broadband ADCPs used at Romeoville have built-in pitch and roll compensation. The *Systematic Uncertainty due to Improper Profiler Beam Geometry* also is assumed to be negligibly small, as the manufacturer has implemented beam-angle-calibration procedures since the ADCP referred to in Simpson and Oltman (1992) was manufactured.

The *Systematic Uncertainty due to Use of the 1/6-Power Curve-Fitting Method for Estimating Unmeasured f-Values* describe systematic errors that can result when the 1/6-power curve used by the ADCP to estimate unmeasured discharges at the top and bottom of the cross section consistently does not accurately depict the actual river vertical velocity profile. Simpson and Oltman (1992, p. 27) show that for only two 'bins' measured in the vertical, the systematic error is the same magnitude as that of a conventional current meter measurement with velocities measured and $^{2}/_{10}$ and $^{8}/_{10}$ of the total depth. Analysis of data sets collected by Simpson and Oltman (1992) indicate errors in the depth-weighted cross-product (*f*-value) that ranged from -0.08 percent to +0.90 percent, with a mean of 0.45 percent. If all other factors remain constant, this will cause an error of +0.45 percent in the measured discharges. The factors described as leading to *Systematic Uncertainty due to operator-Caused Errors* are essentially eliminated with the established procedures and cross-section for the Romeoville measurement. Since these procedures are always followed at Romeoville, however, any error in these procedures will result in a bias rather than a systematic error. A value of ±0.25 percent of the total discharge was arbitrarily chosen (this is half the value arbitrarily chosen by Simpson and Oltman 1992, p. 31).

Based on the above results, the total systematic uncertainty for a typical ADCP measurement at Romeoville is:

$$Q'_{s\,\max} = (1.0045)(1.0025) - 1 = +0.7 \text{ percent, and} \tag{3.3-b}$$

$$Q'_{s\,\min} = (1.0045)(0.9975) - 1 = +0.2 \text{ percent.} \tag{3.3-c}$$

3.3.3.3. Errors in Measuring Cross-Section Width

The surveyed width of the measurement section at Romeoville is 162 ft (USGS, written commun., "Romeoville AVM Quality Assurance Plan, Rev. 04/06/98"). The total width used in 59 ADCP discharge measurements made at this site from July 22, 1993 to October 24, 1995, ranged from 148.0 ft (-8.6 percent) to 168.6 ft (+4.1 percent), with a mean of 158.6 ft (-2.1 percent). It in not clear whether the differences in the measured width are errors in estimating the edge distances or errors in the ADCP's measurement of distance traveled. This apparent error also could be the result of compass errors in the ADCP. Such compass errors can affect the distance reported by the ADCP but will not affect the measured discharge. This is because the compass error will affect the water and boat velocities identically and because the ADCP calculates the discharge by determining the velocity normal to the boat path for each ‘ensemble’ and summing for all ensembles. In contrast, the distance is the straight-line distance from the starting to the ending point of the measurement. Because of the nearly rectangular cross-section at Romeoville, the effect of this error was estimated as translating directly to a bias of the same amount (-2.1 percent) in the measured discharge. It is recommended that the USGS investigate further to determine: (a) whether this error persists in more recent data; (b) whether this is an error in distance measurement or the result of a compass errors; and (c) the effect of this on the discharge.

3.3.3.4. Total Uncertainty in Typical Discharge Measurement

The total uncertainty in the measured discharge was calculated by applying the results of the preceding discussions to each individual discharge measurement. For 59 ADCP discharge measurements made at this site from July 22, 1993 to October 24, 1995, the theoretical errors, including the error from the measurement of cross-section width, ranged from –566 ft^3/sec to -352 ft^3/sec, with a mean of –78.5 ft^3/sec (-1.7 percent). The calculations were repeated without adding the effect of the error from the measurement of cross-section width; this showed theoretical errors ranging from –40 ft^3/sec to +137 ft^3/sec, with a mean of 19.9 ft^3/sec (+0.4 percent of the mean of the measured discharges, 4,530 ft^3/sec).

USGS reports describing the quality-assurance for ADCP measurements indicate that the standard deviation of ADCP measurements is typically greater than that predicted by error-propagation analysis (Lipscomb, 1995; Morlock, 1996). The above analysis would indicate that theoretically, accuracies on the order of 0.5 percent are possible. The ADCP calibration standard used by the USGS of ±5 percent (Lipscomb, 1995) defines a more appropriate upper limit to the potential error in the ADCP measurements. The way the USGS standard is implemented would not flag an instrument for recalibration until the error from a rated discharge exceeded 5 percent, regardless of whether the error was random or systematic. Without results from ADCP test measurements, it cannot be determined whether error in the instrument is random or systematic. If the error in an instrument is random, it will be reduced by the square root of the number of measurements used in the analysis and will appear as part of the analysis of rating-bias errors (see section 3.3.4). If the error is systematic, it is only reduced by using more than one meter; for a single meter it will not be included in the analysis of rating bias but need to be explicitly added to the bias error. The USGS indicates that four ADCPs have been used at this site for the record of ADCP measurements provided. The USGS indicates that at least one different ADCP was used from February, 1997 through November, 1998. Since one ADCP was used through water year 1995, the 5-percent error could potentially be a bias. This was examined further by comparing the index-velocity rating based on 23 Price AA measurements from November 1989 through October, 1993 with the index-velocity rating developed based on 59 ADCP measurements from July 1993 through October, 1995. This analysis indicated a decrease of 5 percent in the mean velocities (and thus discharges) predicted by the rating developed from ADCP measurements compared with the rating developed from Price AA measurements (Figure 3.3-a).

Given the different measurement characteristics of Price AA current meters and ADCPs, it would be reasonable to expect a greater standard error from Price AA measurements than ADCP measurements. Price AA measurements take significantly longer than ADCP measurements and are thus affected more by unsteady flows. It is likely that several Price AA meters were used to make these 23 measurements. Both of these factors will increase the random error of the Price AA measurements. However, the 5-percent difference between the ADCP and the AA ratings may indicate a bias error. In addition, the bias is more likely to be in the ADCP data, given the tendency of errors from long-term series of Price AA measurements to be random. The committee recommends that further study be done to identify whether the differences between the Price AA measurements and the ADCP measurements represent a bias error in the record for the Romeoville AVM.

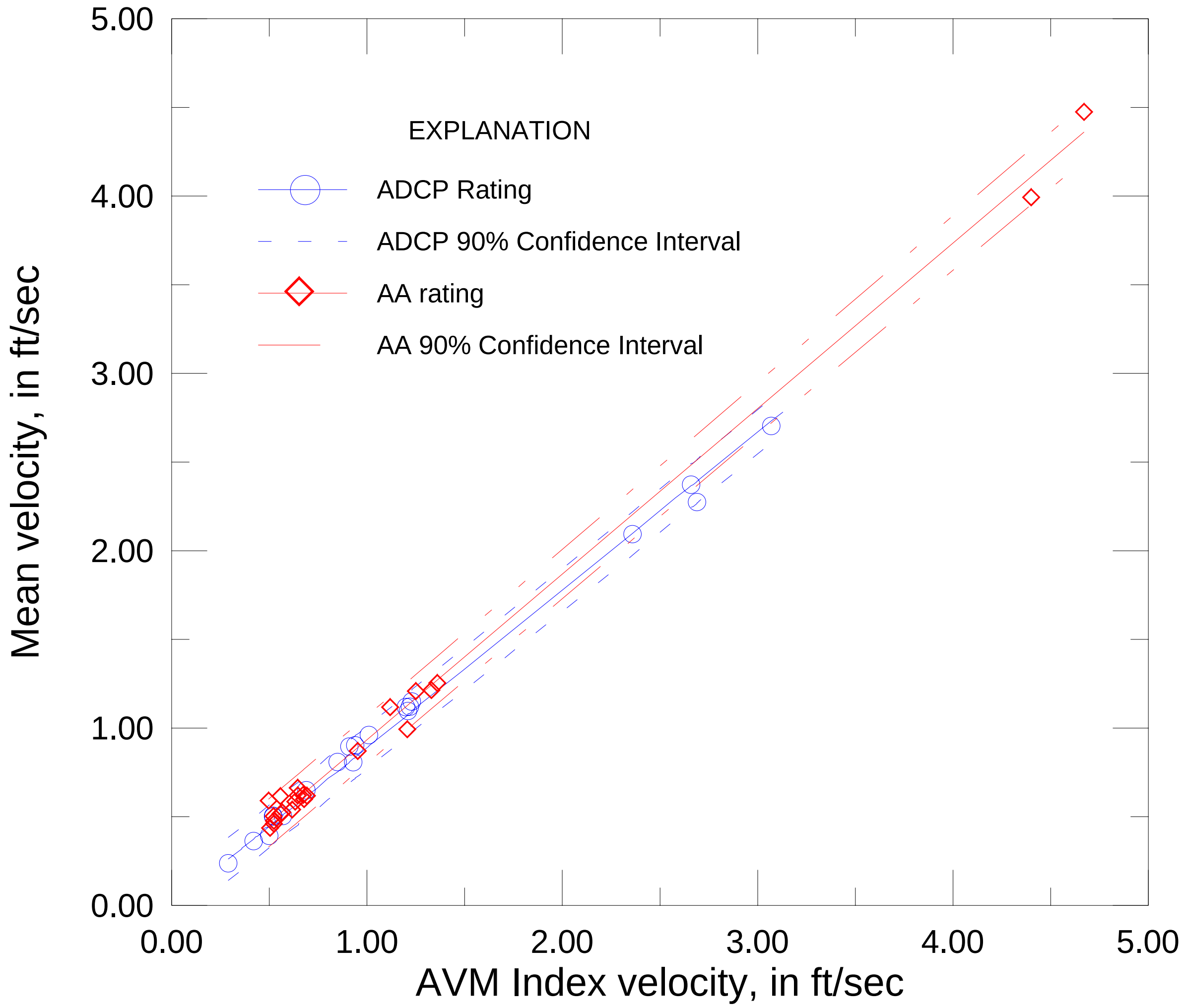


Figure 3.3–a - Graph showing ratings and 90-percent confidence intervals for index-velocity ratings developed from Price AA measurements and from ADCP measurements at Romeoville between November 1989 and October 1995.

3.3.4. Backup System

The backup system is a means to estimate flow at Romeoville for periods when the AVM is inoperative or is not functioning properly. The backup system for the Romeoville AVM is a set of regression equations which relate the flow at Romeoville to the MWRDGC's discharge estimates at the Lockport powerhouse, lock, and controlling works. These estimation equations are described in detail by Melching and Oberg (1993). These equations estimate the daily-mean flow at Romeoville based on MWRDGC's reported flows at Lockport. These equations were developed based on 1,640 days of concurrent discharges from Romeoville and at the Lockport powerhouse, lock, and controlling works. Three different equations were developed based on the flow conditions at Lockport. For periods when the flow at Lockport was comprised entirely of turbine flows, lockages, and leakage, the estimated daily-mean discharge at Romeoville is given by:

$$Q_{Romeoville} = 75.48 + 1.127 Q_{TLL} \tag{3.3-d}$$

where $Q_{Romeoville}$ is the daily-mean discharge at Romeoville; and Q_{TLL} is MWRDGC's estimated discharge from turbines, lockage, and leakage.

The standard error of this equation is 155 ft^3/sec, or 4.69 percent of the mean flow for the conditions where this equation applies.

For periods when the flow at Lockport is comprised of turbine flows, lockages, leakage, and sluice-gate flows, with the sluice-gate flows less than 5,000 ft^3/sec, the estimated daily-mean discharge at Romeoville is given by:

$$Q_{Romeoville} = 219.7 + 1.127Q_{TLL} + 0.6842Q_{SG} \tag{3.3-e}$$

where Q_{SG} is MWRDGC's estimated discharge from the sluice gates; and all other terms are as defined above.

The standard error of this equations is 296 ft^3/sec, or 6.25 percent of the mean flow for the conditions where this equation applies.

For periods when the flow at Lockport is comprised of turbine flows, lockages, leakage, and sluice-gate flows, with the sluice-gate flows greater than 5,000 ft^3/sec or when the controlling works are releasing flow, the estimated daily-mean discharge at Romeoville is given by:

$$Q_{Romeoville} = 1{,}086 + 1.127Q_{TLL} + 0.4361Q_{SG} + 0.3228Q_{CW} \tag{3.3-f}$$

where Q_{CW} is MWRDGC's estimated discharge from the controlling works; and all other terms are as defined above.

The standard error of this equations is 1,245 ft^3/sec, or 13.1 percent of the mean flow for the conditions where this equation applies.

According to Melching and Oberg (1993, p. 39) these Equations (3.3-d through 3.3-f) are only valid for estimating missing discharge until August 20, 1992. Beginning on August 21, 1992, AVMs installed in the intakes of the Lockport turbines replaced the flow estimates from the turbine efficiency (rating) curves as the official MWRDGC reported flows at Lockport. The regression analysis used to develop Equations 3.3-d though 3.3-f should be repeated to develop new regression equations for periods when the reported flows at Lockport are based on the turbine AVMs

The USGS has estimated flows based on Equations 3.3-d though 3.3-f since August 20, 1992. Table 3.3-a lists the estimated daily-mean discharges for Romeoville for water years 1990 – 1995. The MWRDGC used the turbine AVMs as the official MWRDGC reported flows at Lockport from August 21, 1992 through July 31, 1994. From August 1, 1994 through November 30, 1996, discharges from the turbine-efficiency curves were used as the official MWRDGC reported flows at Lockport. From December 1, 1996 through August 1998, MWRDGC used the turbine AVMs as the official MWRDGC reported flows at Lockport, except for days when the turbine AVMs were not operational (Ram Kaduir, MWRDGC, written commun. to USGS, February 5, 1999). Table 3.3-b lists the days from December 1, 1996 through August, 1998 when the turbine AVMs were not used to report flow. The Technical Committee recommends that the USGS be informed whenever changes are made to the methods used to determine the reported flows at Lockport.

Table 3.3-a - Daily-mean discharges at Romeoville estimated from MWRDGC-reported discharges at Lockport for water years 1990 through 1995.

[USGS-Estimated discharge is from annual data reports; Turbine flow basis describes whether MWRDGC-reported flows are from turbine rating or turbine AVMs; shaded cells indicate days for which estimation equations are not valid; blank cells indicate no flow for this component; dashes indicate no data; standard error for periods with no MWRDGC data available (standard error is italicized) was estimated based on probability of applicable equation; Std. Error, the standard error in the estimated discharge for the given water year that results from uncertainty in the backup equations.]

		MWRDGC reported				
Date (yyyy.mm.dd)	USGS-Estimated Discharge (ft^3/sec)	Turbine, leakage, and lockage flow (ft^3/sec)	Turbine flow basis	Sluice - gate flow (ft^3/sec)	Controlling works Flow (ft^3/sec)	Standard Error (ft^3/sec)
WATER YEAR 1990						
1989.10.01	2,976	2,574	Rating			155
1989.10.02	2,600	2,240	Rating			155
1989.10.03	3,003	2,598	Rating			155
1989.10.04	2,725	2,351	Rating			155
1989.10.05	3,007	2,601	Rating			155
1989.10.06	3,161	2,738	Rating			155
1989.10.10	2,712	2,339	Rating			155
1989.10.11	2,534	2,182	Rating			155
1989.10.12	2,456	2,112	Rating			155
1989.10.13	2,866	2,476	Rating			155
1989.11.15	7,064	3,868	Rating	3,632		296
1989.11.16	5,160	4,512	Rating			155
1989.11.17	3,400	2,950	Rating			155
1989.11.18	3,447	2,992	Rating			155
1989.11.19	3,002	2,597	Rating			155
1989.11.20	2,660	2,293	Rating			155
1989.11.21	2,350	2,018	Rating			155
1990.03.16	3,737	3,249	Rating			155
1990.03.17	3,607	3,134	Rating			155
1990.03.18	3,278	2,842	Rating			155
1990.03.19	3,213	2,784	Rating			155
1990.04.29	2,182	1,869	Rating			155
1990.04.30	2,611	2,250	Rating			155

1990.05.01	2,210	1,890	Rating			155
1990.05.04	8,410	4,176	Rating	5,689	427	1,245
1990.05.05	4,880	4,265	Rating			155
1990.05.06	3,920	3,411	Rating			155
1990.05.07	2,830	2,445	Rating			155
1990.05.08	3,260	2,827	Rating			155
1990.05.09	8,760	3,466	Rating	6,322	3,117	1,245
1990.05.10	17,900	3,978	Rating	20,217	10,795	1,245
1990.05.11	14,100	4,273	Rating	16,622	2,888	1,245
1990.05.12	10,800	4,397	Rating	10,563	432	1,245
1990.05.13	8,990	4,700	Rating	5,986		1,245
1990.05.14	8,540	4,629	Rating	5,127		1,245
1990.05.15	7,960	4,565	Rating	3,786		296
1990.05.16	6,470	3,688	Rating	3,056		296
1990.05.17	4,710	4,112	Rating			155
1990.05.18	3,530	3,063	Rating			155
1990.05.19	6,590	4,332	Rating	2,177		296
1990.05.20	4,240	3,691	Rating			155
1990.05.21	3,620	3,144	Rating			155
1990.05.22	3,250	2,820	Rating			155
1990.05.23	3,010	2,600	Rating			155
1990.05.24	2,940	2,543	Rating			155
1990.05.25	4,840	4,231	Rating			155
1990.05.26	4,620	4,030	Rating			155
1990.05.27	4,100	3,572	Rating			155
1990.05.28	2,860	2,474	Rating			155
1990.05.29	2,620	2,259	Rating			155
1990.05.30	2,700	2,324	Rating			155
1990.06.14	4,740	3,786	Rating	363		296
			WATER YEAR 1991			
1991.03.27	7,510	4,362	Rating	3,465		296
1991.04.20	4,500	3,928	Rating			155
1991.04.21	3,820	3,319	Rating			155
1991.04.22	1,810	1,542	Rating			155
1991.04.23	5,200	2,506	Rating	3,152		296

1991.04.24	4,040	3,514	Rating			155
1991.05.31	5,520	2,368	Rating	3,846		296
1991.06.01	4,480	2,630	Rating	1,888		296
1991.06.02	3,320	2,552	Rating	333		296
1991.06.03	3,360	2,450	Rating	554		296
WATER YEAR 1992						
1992.04.17	7,111	--	Rating	--	--	823
1992.04.18	7,340	--	Rating	--	--	823
1992.04.19	7,092	--	Rating	--	--	823
1992.04.20	7,050	--	Rating	--	--	823
1992.04.21	6,761	--	Rating	--	--	296
1992.07.02	6,037	--	Rating	--	--	296
1992.07.03	5,013	--	Rating	--	--	199
1992.07.04	4,244	--	Rating	--	--	199
1992.07.05	4,428	--	Rating	--	--	199
1992.07.06	4,079	--	Rating	--	--	199
WATER YEAR 1993						
1992.12.05	2,166	--	AVM	--	--	155
1992.12.06	2,008	--	AVM	--	--	155
1992.12.07	2,493	--	AVM	--	--	155
1992.12.13	2,272	--	AVM	--	--	155
1992.12.14	2,437	--	AVM	--	--	155
1992.12.15	3,359	--	AVM	--	--	155
1993.02.27	2,266	--	AVM	--	--	155
1993.02.28	1,864	--	AVM	--	--	155
1993.03.17	3,628	--	AVM	--	--	199
1993.03.18	3,273	--	AVM	--	--	155
1993.05.04	3,245	--	AVM	--	--	155
1993.05.05	3,901	--	AVM	--	--	199
1993.07.13	4,791	--	AVM	--	--	199
1993.07.14	4,224	--	AVM	--	--	199
1993.07.15	4,108	--	AVM	--	--	199
WATER YEAR 1994						
1993.11.13	3,611	--	AVM	--	--	199
1993.11.14	3,101	--	AVM	--	--	155
1993.11.15	2,385	--	AVM	--	--	155

1993.12.03	2,948	--	AVM	--	--	155
1994.07.04	3,830	--	AVM	--	--	199
1994.07.05	3,913	--	AVM	--	--	199
1994.08.11	5,083	--	Rating	--	--	199
			WATER YEAR 1995			
1995.01.03	2,187	--	Rating	--	--	155
1995.01.04	3,041	--	Rating	--	--	155
1995.01.05	1,973	--	Rating	--	--	155
1995.01.08	2,204	--	Rating	--	--	155
1995.01.09	2,332	--	Rating	--	--	155
1995.01.10	2,317	--	Rating	--	--	155
1995.02.07	2,493	--	Rating	--	--	155
1995.02.08	2,349	--	Rating	--	--	155
1995.02.09	1,928	--	Rating	--	--	155
1995.02.10	2,231	--	Rating	--	--	155
1995.02.11	2,342	--	Rating	--	--	155
1995.02.12	2,420	--	Rating	--	--	155
1995.02.13	2,142	--	Rating	--	--	155
1995.02.14	2,625	--	Rating	--	--	155
1995.06.07	3,895	--	Rating	--	--	199
1995.09.10	3,711	--	Rating	--	--	199
			WATER YEAR 1996			
1995.10.18	2,351	2,019	Rating			155
1996.01.19	3,777	3,284	Rating			155
1996.01.22	2,580	2,222	Rating			155
1996.01.24	2,660	2,293	Rating			155
1996.01.25	1,926	1,642	Rating			155
1996.01.26	2,710	2,338	Rating			155
1996.01.29	2,277	1,953	Rating			155
1996.02.05	2,274	1,951	Rating			155
1996.02.07	2,256	1,935	Rating			155
1996.02.08	2,814	2,430	Rating			155
1996.02.12	2,068	1,768	Rating			155
1996.02.13	2,014	1,720	Rating			155
1996.03.07	2,385	2,049	Rating			155

1996.06.17	6,835	7,106	Rating	2,439	714	1,245
1996.06.18	12,177	20,652	Rating	10,351	6,257	1,245
1996.06.19	5,013	4,381	Rating			155
1996.06.20	4,668	4,075	Rating			155
1996.06.21	4,603	4,017	Rating			155
1996.09.22	3,268	2,833	Rating			155
1996.09.23	3,014	2,607	Rating			155
1996.09.24	3,035	2,626	Rating			155
1996.09.25	3,001	2,596	Rating			155
1996.09.26	5,478	5,074	Rating	1,040		296
1996.09.27	7,510	8,194	Rating	3,043	881	1,245

Daily-mean discharges at Romeoville were estimated based on turbine AVMs rather than turbine efficiency (rating) curves for 15 days in water year 1993 and 6 days in water year 1994. The error in the estimated daily-mean discharges resulting from using turbine AVMS rather than turbine efficiency curves is expected to be small. The effect of this will be a small fraction of the annual-mean flow at Romeoville (if the error is 20 percent, the resulting error in the annual-mean flow at Romeoville will be 0.5 percent); however, when new back-up equations are developed for the turbine AVMS, the effect of this error should be quantified and documented.

Table 3.3-b - Days since December 1, 1996 where turbine AVMs were not used as the official MWRDGC reported flows at Lockport (MWRDGC, written commun., February 5, 1999).

Date of missing AVM record
February 27-28, 1997
December 30-31, 1997
February 5-10, 1998
May 3, 1998
June 9-12, 1998
June 16, 1998
July 1-6, 1998
July 24-29, 1998

3.3.5. Rating Bias

The discharge at the Romeoville AVM is calculated from observed water elevations and velocities using empirical stage-area and index-velocity ratings. The stage-area rating was calculated from surveyed channel cross sections. The index-velocity rating was developed from regression analysis of velocities measured by the AVM and determined from concurrent discharge measurements, as described in Section 3.3.2. The index-velocity rating is the ‘best-fit’ line through the observed data. Random error in the rating may result from: (a) the conceptual model represented by the rating equation not correctly representing the actual physical relation; (b) random errors in the stage, discharge, and AVM velocity measurements and in the stage-area rating; and (c) natural variability in the system. The random errors in the ADCP measurements would fall into (b) above. The random error in the empirical rating can be reduced by increasing the number of data points used in developing the rating. The rating itself, however, is a single estimate of the true relation between the AVM (index) velocity and the mean velocity in the channel. Any error in the rating is incorporated into every instantaneous discharge calculated from the rating, resulting in a bias in the calculated discharges (Gain, 1998; Sloat and Gain, 1995). The index-velocity rating for the Romeoville AVM is given by the following equation and is shown in Figure 3.3-b:

$$V_{mean} = 0.90(V_{Index}) \tag{3.3-g}$$

where V_{mean} is the mean velocity in the channel; and V_{Index} is the mean AVM velocity.

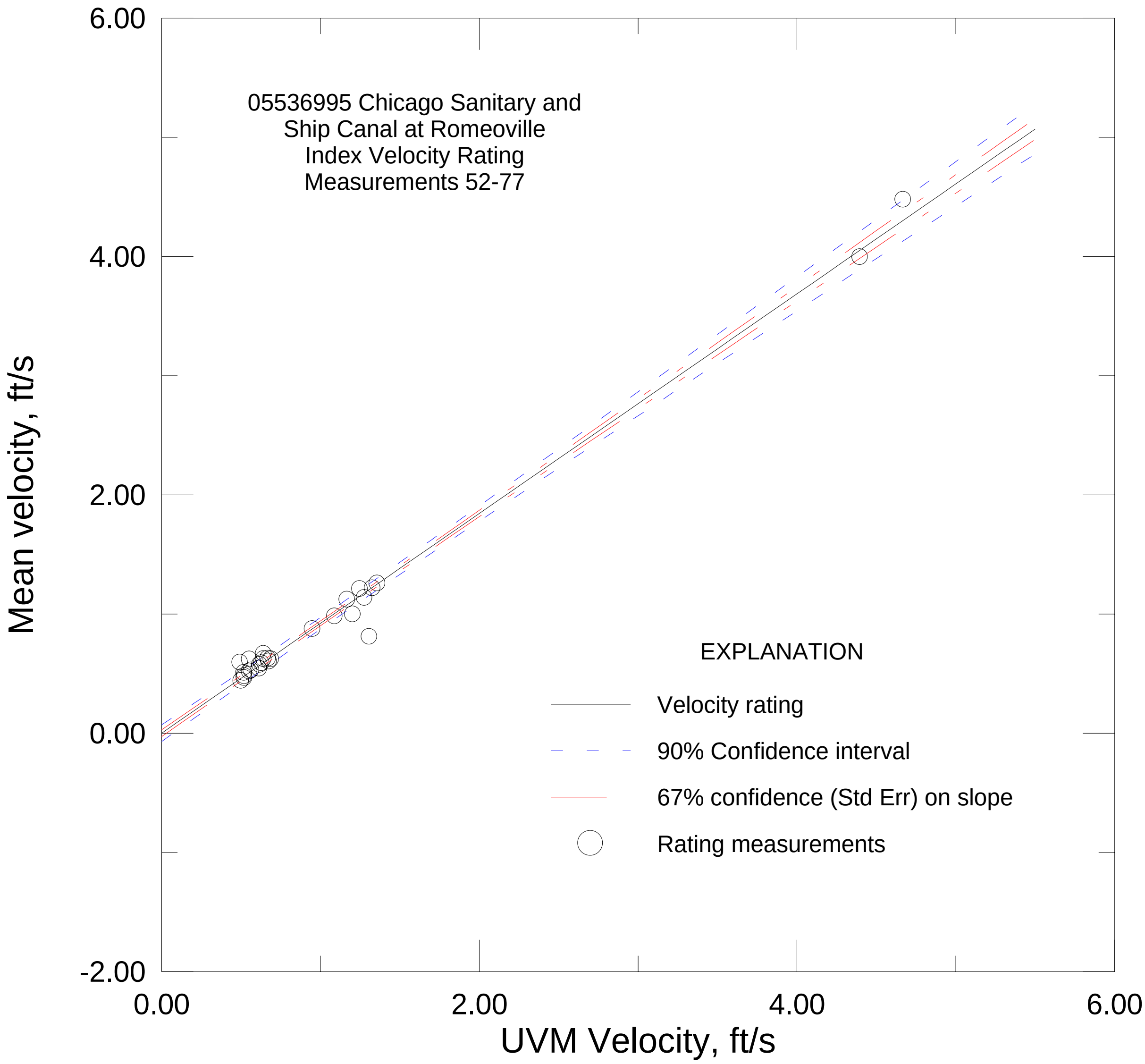


Figure 3.3-b - Index-velocity rating for Romeoville AVM.

The standard error of a value estimated from the regression equation is given by the following equation:

$$SE\left(\hat{V}_{Mean}\left(V_{Index}\right)\right) = s_{\varepsilon}\sqrt{1 + \frac{1}{n} + \frac{\left(V_{Index} - \overline{V}_{Index}\right)^2}{S_{Index}}} \tag{3.3-h}$$

where $SE\left(\hat{V}_{Mean}\left(V_{Index}\right)\right)$ is the standard error estimate of the mean velocity from the regression equation for the given AVM-Index velocity; $\widehat{V}_{mean}$ is the predicted mean velocity, s_{ε} is the standard error of the residuals from the regression equation; n is the number of observations (discharge measurements) in the data set used to develop the regression equation, V_{Index} is the observed AVM index velocity, $\overline{V}_{Index}$ is the mean AVM index velocity in the data set used to develop the regression equation, and S_{Index} is the standard deviation of the AVM observations in the data set used to develop the regression equation.

The prediction interval for the regression (rating) equation describes the range of values within which the true mean velocity is expected to fall a specified percentage of the time. The 90-percent prediction interval indicates that for a given AVM index velocity, the actual instantaneous mean (for the cross-section) velocity is expected to fall within this interval 90 percent of the time. The prediction interval is determined by the following equation: $\widehat{V}_{90\%} = \widehat{V}_{mean} \pm t_{\alpha/2} s_{\varepsilon} \sqrt{1 + \frac{1}{n} + \frac{\left(V_{Index} - \overline{V}_{Index}\right)^2}{S_{Index}}}$

where $\widehat{V}_{90\%}$ is the upper or lower limit of the prediction interval, $t_{\alpha/2}$ is the Student's t value for the 1-α (α=0.10 for this report) and n-2 degrees of freedom; and all other terms are described above.

It should be noted that $SE\left(\hat{V}_{Mean}\left(V_{Index}\right)\right)$, the standard error estimate of the mean velocity from the regression equation for the given AVM-Index velocity, is the same as the width of the 67.3-percent predication-interval.

Figure 3.3-a shows the standard error and 90-percent prediction interval for mean velocities for the rating for Romeoville. Based on equation 3.3h, the standard error for an instantaneous velocity from the rating equation is ±0.21 ft/sec for velocities between 0 and 2.3 ft/sec.

When daily-mean discharges are being considered, the confidence interval should be considered, rather than the prediction interval. The confidence interval describes the range of values within which the average of the predicted velocities is expected to fall a specified percentage of the time. The 90-percent confidence interval is determined by the following equation:

$$\overline{V}_{90\%} = \overline{V}_{mean} \pm t_{\alpha/2} s_{\varepsilon} \sqrt{\frac{1}{n} + \frac{\left(V_{Index} - \overline{V}_{Index}\right)^2}{S_{Index}}} \qquad (3.3\text{-j})$$

where $\overline{V}_{90\%}$ is the upper or lower limit of the confidence interval, $\overline{V}_{mean}$ is the mean of the predicted mean velocities, and all other terms are described above.

The standard error of a the average of the predicted mean velocities estimated from the regression equation is given by the following equation:

$$SE\left(\overline{V}_{Mean}\left(V_{Index}\right)\right) = s_{\varepsilon} \sqrt{\frac{1}{n} + \frac{\left(V_{Index} - \overline{V}_{Index}\right)^2}{S_{Index}}} \qquad (3.3\text{-k})$$

where $SE\left(\overline{V}_{Mean}\left(V_{Index}\right)\right)$ is the standard error estimate of the average of the predicted mean velocities from the regression equation for the given AVM-Index velocity; $\overline{V}_{mean}$ is the average of the predicted mean velocities, and all other terms are described above.

Since the data available for evaluating the bias in the AVM discharges are daily-mean values, the confidence interval and the standard error estimate of the average of the predicted mean velocities are the appropriate equations to use. Based on these data, the standard error of the average of the predicted mean velocities is ±0.04 ft/s for velocities ranging from 0.6 to 1.7 ft/sec.

The error in the daily-mean discharge for every day in water years 1990 through 1996 was estimated from the daily-mean discharge and equation 3.3-k as follows:

1. The daily-mean discharge was divided by the median cross-sectional area from all the observations used in developing the discharge rating (4,379 ft^2) to estimate a daily-mean water velocity;
2. The daily-mean index velocity was estimated from the inverse of the index-velocity rating, $V_{Index} = 0.90(V_{mean})$;
3. The standard error for the daily-mean water-velocity was estimated from equation 3.3-k; and
4. The standard error for the daily-mean discharge was estimated by multiplying the error in velocity times the median cross-sectional area.

This analysis will underestimate the magnitude of the error because (a) using daily-mean values will bias the calculations toward the mean of the observations, where the confidence interval is smallest; and (b) using the median area will again bias the velocities toward the mean of the observations. This analysis does give a more accurate estimate of the bias in the daily-mean discharges than earlier estimates done using the long-term mean AVM velocity and cross-sectional area. The results of this analysis show that the uncertainty in daily-mean discharges for water years 1990 through 1996 ranges from ±88 to ±296 ft^3/sec, with a median of ±93 ft^3/sec.

The uncertainties described above in the daily-mean discharges are a bias, because they represent the error in the rating as an estimate of the true mean velocity. This error is incorporated into every daily-mean discharge calculated from the rating.

3.3.6. Error Analysis

The error in the annual mean discharge at Romeoville is the result of all the errors discussed previously. The random errors in the ADCP discharge measurements are assumed to compose part of the random error in the rating analysis. The random error in the index-velocity rating will result in a bias in the calculated discharges, as the error is incorporated each time the rating is applied. Similarly, the random error in the three equations used to estimate missing record at Romeoville will result in a bias in the discharge, as the error is incorporated each time each equation is applied. The bias error from each of these equations could be either positive or negative, and thus errors from different equations may be offsetting. The following presents the worst case of all bias errors being in the same direction.

The error from the measured widths consistently underpredicting the known channel width is a negative bias. The discharges measured are expected to consistently be 2.1 percent smaller that the actual discharge in the channel. This will result in the ‘mean’ velocities used in development of the regression equation that are 2.1 percent too small, and thus the regression equation will have a –2.1 percent bias. This will eventually result in daily-mean discharges which are 2.1 percent too small.

The error in the annual-mean discharge was estimated from the published daily-mean discharges as follows:

1. The potential bias from the index-velocity rating was calculated as described in Section 3.3.3 for days when the AVM was operational;

2. The bias from the error in measured widths was calculated as –0.021 times the daily-mean discharge for days when the AVM was operational;
3. The potential bias from the equations for estimating missing record as shown in Table 3.3-a was used for days when the AVM was not operational;
4. The daily bias errors were summed for (a) index-velocity and missing-record bias being positive, and (b) index-velocity and missing-record bias being negative; and
5. The two sums were divided by the number of days of record to determine the range of potential bias in the annual-mean discharge.

The potential bias in annual-mean discharge at Romeoville (WY 1990 – 1996) are summarized in Table 3.3-c.

For the water years 1990 through 1996, the potential bias error in the mean-annual discharge is in the range of –191 to +61 ft^3/sec. If the possible error in the measured width is resolved, the potential bias error for these seven years would range from ±96 ft^3/sec for water year 1994 to ±126 ft^3/sec for water year 1990.

Table 3.3-c - Potential bias in annual-mean discharge at Romeoville, water years 1990-1996.

Water year	Annual-mean discharge	Without width error		Including potential error in measured width			
				Minimum bias		Maximum bias	
	(ft^3/sec)	(ft^3/sec)	(percent)	(ft^3/sec)	(percent)	(ft^3/sec)	(percent)
1990	3,749	±126	3.4%	-191	-5.1%	61	+1.6%
1991	3,791	±99	2.6%	-175	-4.6%	22	+0.6%
1992	3,860	±104	2.7%	-182	-4.7%	27	+0.7%
1993	4,074	±99	2.4%	-182	-4.5%	16	+0.4%
1994	3,095	±96	3.1%	-160	-5.2%	33	+1.1%
1995	3,235	±97	3.0%	-162	-5.0%	32	+1.0%
1996	3,162	±111	3.5%	-172	-5.4%	50	+1.6%

3.4. Other Gages

Although the Romeoville AVM is the primary gage for the Lake Michigan diversion accounting, records from five other gages operated by the USGS are included in the Lake Michigan diversion accounting. Records from four of these stations are used together with streamflow separation to estimate the runoff from portions of the diverted Lake Michigan Watershed. The fifth on the Grand Calumet River at Hohman Avenue at Hammond Indiana is used for estimating the water-supply pumpage from Indiana that flows past the Romeoville gage. Table 3.4-a lists these gages.

Table 3.4-a - Streamflow-gaging stations used for runoff analysis and calculation of Indiana water-supply pumpage.

USGS Station Number	Location	Drainage area (mi^2)	Period of record	Mean-annual discharge (ft^3/sec)
05536000	North Branch Chicago River at Niles, Illinois	100	1951-1997	100
05536195	Little Calumet River at Munster, Indiana	90	1958-1997	73
05536275	Thorn Creek at Thornton, Illinois	104	1948-1997	106
05536290	Little Calumet River at South Holland, Illinois	208	1948-1997	189
05536357	Grand Calumet River at Hohman Avenue at Hammond, Indiana	Indeterminate	1991-1997	46

Both the Little Calumet River at Munster, Indiana and Thorn Creek at Thornton, Illinois are upstream from the Little Calumet River at South Holland, Illinois, and account for 93 percent of the drainage area and 95 percent of the flow at the Little Calumet River at South Holland, Illinois.

3.4.1. Station 05536000, North Branch Chicago River at Niles, Illinois

Station 05536000 on the North Branch of the Chicago River at Niles, Illinois was used to estimate the runoff from 100 square miles of the diverted Lake Michigan watershed. The runoff from this gage was part of the value reported in Column 12 of the diversion accounting summary tables and also part of 'Budget 14—the Canal System Balance.' Although errors in the runoff calculated from this station did not directly affect the calculated diversion, these were considered in the evaluation of the validity of simulations which were part of the diversion calculation. The runoff flowing past this station also is part of the calculation of the runoff value for lakefront accounting.

The records from this station for water years 1990-1995 were rated by the USGS as 'good, except for estimated daily discharges, which are poor.' The estimated daily discharges include periods where the record was affected by ice. The ice-affected periods often were among the lowest flows at the station. Paragraph 31 on page 11 of Appendix A of the *Lakefront Accounting Technical Analysis* (USACE, 1996a) states that runoff was calculated by subtracting dry-weather treatment-plant flows from measured streamflow. Although paragraph 31 refers specifically to the station on the Grand Calumet River at Hohman Avenue, if a similar analysis was done for the North Branch of the Chicago River, care should be exercised that periods of estimated flow were excluded from the analysis.

This station had a rock riffle control at low flow, channel control at medium and high flows, and was controlled by a bridge opening at extremely high flows. Between seven and ten discharge measurements were made each year at this station. The same discharge rating was used for the entire six-year period. No datum corrections were necessary at this site for the six-year period. Significant shifts were regularly required at this station. Vegetation growth in the summer required a negative shift which ranged from –0.18 to –0.39 ft. High water resulted in positive shifts to low-and medium flows; these were always less than 0.10 ft.

In general the records for this site showed no problems. The station analysis for the 1995 water year should be re-visited. Although flows for 1995 were well inside the normal range at this station, this is the only year that showed no negative shifts to the rating. There may be other factors which affected the station in 1995; these should be documented in the station analysis.

3.4.2. Station 05536195, Little Calumet River at Munster, Indiana

Station 05536195 on the Little Calumet River at Munster, Indiana was used to estimate the runoff from 90 square miles of the diverted Lake Michigan watershed. The runoff from this gage was part of the value reported in Column 12 of the diversion accounting summary tables and also part of 'Budget 14—the Canal System Balance.' Although errors in the runoff calculated from this station did not directly affect the calculated diversion, these were considered in the evaluation of the validity of simulations which were part of the diversion calculation. The runoff flowing past this station also was part of the calculation of the runoff value for lakefront accounting.

No station analyses were reviewed for this station.

3.4.3. Station 05536275, Thorn Creek at Thornton, Illinois

Station 05536275 on Thorn Creek at Thornton, Illinois was used to estimate the runoff from 104 square miles of the diverted Lake Michigan watershed. The runoff from this gage was part of the value reported in Column 12 of the Diversion accounting summary tables and also part of 'Budget 14—the Canal System Balance.' Although errors in the runoff calculated from this station did not directly affect the calculated diversion, they were considered in the evaluation of the validity of simulations which were part of the diversion calculation. The runoff flowing past this station also was part of the calculation of the runoff value for lakefront accounting.

The records from this station for water years 1990-1995 were rated by the USGS as 'good, except for estimated daily discharges, which are poor.' The estimated daily discharges include periods where the record was affected by ice. The ice-affected periods often were among the lowest flows at the station. Paragraph 31 on page 11 of Appendix A of the *Lakefront Accounting Technical Analysis* (USACE, 1996a) states that runoff was calculated by subtracting dry-weather treatment-plant flows from measured streamflow. Although paragraph 31 refers specifically to the station on the Grand Calumet River at Hohman Avenue, if a similar analysis was done for the Thorn Creek at Thornton, care should be exercised that periods of estimated flow were excluded from the analysis.

Extreme low flow at this station was controlled by a gravel bar at the gage. Low and medium flows were controlled by the remains of a rock dam. The channel controlled high stages. Between seven and ten discharge measurements were made each year at this station. Discharge rating 25 was used starting October 1, 1990 and was used for the entire six-year period. No datum corrections were necessary at this site for the six-year period. Significant shifts were regularly required at this station. Vegetation growth and accumulation of leaves and debris on the low-water control required negative shifts which ranged to –0.29 ft. Fill and scour frequently affected the stage-discharge relation at this station and resulted in shifts ranging from –0.44 to +0.60 ft. A shift of –0.73 ft in water year 1991 affected high stages and was from debris in the floodplain.

Records for water year 1992 indicated a sluggish float that was not responsive to daily fluctuations in stage. This affected the record from October 1, 1991 through August 25, 1992.

3.4.4. Station 05536290, Little Calumet River at South Holland, Illinois

Station 05536290 on the Little Calumet River at South Holland, Illinois was used to estimate the runoff from 208 square miles of the diverted Lake Michigan watershed. The runoff from this gage was part of the value reported in Column 12 of the Diversion accounting summary tables and also part of 'Budget 14—the Canal System Balance.' Although errors in the runoff calculated from this station did not directly affect the calculated diversion, they were considered in the evaluation of the validity of simulations which were part of the diversion calculation. The runoff flowing past this station also was part of the calculation of the runoff value for lakefront accounting.

The records from this station for water years 1990-1995 were rated by the USGS as 'good, except for estimated daily discharges, which are poor.' The estimated daily discharges included periods where the record was affected by ice. The ice-affected periods often were among the lowest flows at the station.

Paragraph 31 on page 11 of Appendix A of the *Lakefront Accounting Technical Analysis* (USACE, 1996a) states that runoff was calculated by subtracting dry-weather treatment-plant flows from measured streamflow. Although paragraph 31 refers specifically to the station on the Grand Calumet River at Hohman Avenue, if a similar analysis was done for the Little Calumet River at South Holland, care should be exercised that periods of estimated flow were excluded from the analysis.

The channel controlled flow at this station at all but extremely high stages, which were controlled by bridge openings downstream. Between six and nine discharge measurements were made each year at this station. Discharge rating 29 was used until February 23, 1990. Discharge rating 30 was used from February 23, 1990 until some point during water year 1992 (station analysis for water year 1992 was not available). Discharge rating 31 was used through the end of water year 1996. No datum corrections were necessary at this site for the six-year period. Many corrections, ranging from –0.13 to +0.07 ft were applied to the gage height record. Observations indicate that the manometer consistently read approximately 0.04 too low at high stages. This was noted and corrected in water years 1995 and 1996 but not in prior years. Significant shifts were regularly required at this station. Shifts at this station were attributed to scour and fill of the channel, aquatic growth, and debris downstream from the gage. Shifts ranged from –0.13 to +0.39 ft. In water year 1994, a shift of –0.72 ft was observed on June 15. The cause of this shift was unknown. A drop in gage height of 0.73 ft on September 30, 1994 removed this shift.

3.4.5. Station 05536357, Grand Calumet River at Hohman Avenue at Hammond, Indiana

Station 05536357 on the Grand Calumet River at Hammond, Indiana was used to revise the regression equations that are used to estimate Indiana water-supply pumpage that flows to the CSSC. These equations are based on the flows from wastewater treatment plants and the Lake Michigan water level measured at Calumet Harbor. Records from this gauge are compared with the calculated Indiana water-supply pumpage. If the flow from this gauge exceeds the calculated water-supply pumpage, the total water-supply pumpage is deducted. Otherwise the entire flow at this gauge is considered as water supply pumpage and this value is used as the deduction. In addition, records from this station were used in the lakefront accounting runoff analysis.

Prior to water year 1992, flows into Illinois from the Grand Calumet River were estimated from a regression equation that related flow at Hohman Avenue to flow on Hart Ditch at Munster, Indiana and the water elevation of Lake Michigan Level. This equation was developed by Kiefer and Associates (1978) and presumably was reviewed by the First Technical Committee (Espey and others, 1981, p. 65). The First Technical Committee recommended that:

- Domestic pumpage factors to determine the portion of flow from East Chicago and Hammond sewage treatment plants passing Lockport should be re-considered, and
- A procedure for estimating the annual net flow across the hydraulic summits of the Grand and Little Calumet Rivers be developed.

The Second Technical Committee (Espey and other, 1987, p. 3-34) indicated that they 'reviewed the methodology used by NIPC and the State of Illinois to account for these deductions and concurs with the procedures used which are entirely consistent with the findings of the First Technical Committee.' The procedures used and reviewed are not described; it is assumed that these are the same procedures described in the Lake Michigan Diversion Accounting Water Year 1990 Report (USACE, 1994, Appendix B). The Third Technical Committee (Espey and others, 1994, p. 122) recommended that, 'if the regression equation that estimates Grand Calumet River flow results in a significant error in the deductible water supply from Indiana, the impact on the historical diversion record should be reviewed.'

For water years 1983 through 1992, flows on the Grand Calumet River at Hohman Avenue were estimated using the following equations:

if the Lake Michigan elevation is 1.00 ft Chicago City Datum (CCD) or less

$$Q_{GrandCal} = 23 \text{ ft}^3/\text{s} \quad (3.4\text{-a})$$

where $Q_{GrandCal}$ is the flow in the Grand Calumet River at Hohman Avenue, in ft^3/sec; and if the Lake Michigan elevations is greater than 1.00 ft CCD

$$Q_{GrandCal} = 29.9 * (\text{Lake Michigan elevation})^{(5/3)} + 0.13 * (\text{Hart Ditch flow})^{(2/3)} - 9.6 \quad (3.4\text{-b})$$

The regression analysis for flow in the Grand Calumet River at Hohman Avenue was revised based on data from the streamgage at this location. The revised regression equations were presented in the *Lakefront Accounting Technical Analysis* (USACE, 1996a, Appendix A, p.12). The data used for the regression analysis and the statistics describing the goodness-of-fit of the regression were not presented. The revised equations used to estimate flows on the Grand Calumet River at Hohman Avenue are:

if the Lake Michigan elevations is 1.00 ft Chicago City Datum (CCD) or less

$$Q_{GrandCal} = 17.9 * \text{Lake Michigan elevation} + 0.038 * \text{Hart Ditch flow} + 19.6 \quad (3.4\text{-c})$$

if the Lake Michigan elevations is greater than 1.00 ft CCD

$$Q_{GrandCal} = 65.4 * \text{Lake Michigan elevation} + 0.054 * \text{Hart Ditch flow} - 29. \quad (3.4\text{-d})$$

The records from this station for water years 1992 were rated by the USGS as 'good, except for estimated daily discharges, and those above 200 ft^3/sec, which are poor.' Records for water years 1993 through 1996 were rated as 'poor, because of the many rating shifts which appear to be caused by variable backwater and because the station is not comparable to other gaging stations.'

The control at this station was three corrugated metal culverts that run under Hohman Avenue just downstream from the gage. Between six and eight discharge measurements were made each year at this station. Discharge rating 1 was used for water year 1992. Rating 2 was used for water years 1993 and 1994. Rating 3 was used for water year 1995, and rating 4 was used for water year 1996. No datum corrections were necessary at this site for the six-year period. Many corrections, ranging from –0.09 to -0.02 ft were applied to the gage height record. These were applied to correct for differences between the recorded stage and the wire-weight gage; in all cases purging the orifice line corrected the discrepancies.

Significant shifts were regularly required at this station. Shifts at this station were attributed to growth of aquatic weed debris at the inlets to the culverts, and backwater. Shifts ranged from –1.74 to +0.22 ft. As stated earlier, because of the variable backwater and shifts, the records at this site are rated as 'poor.'

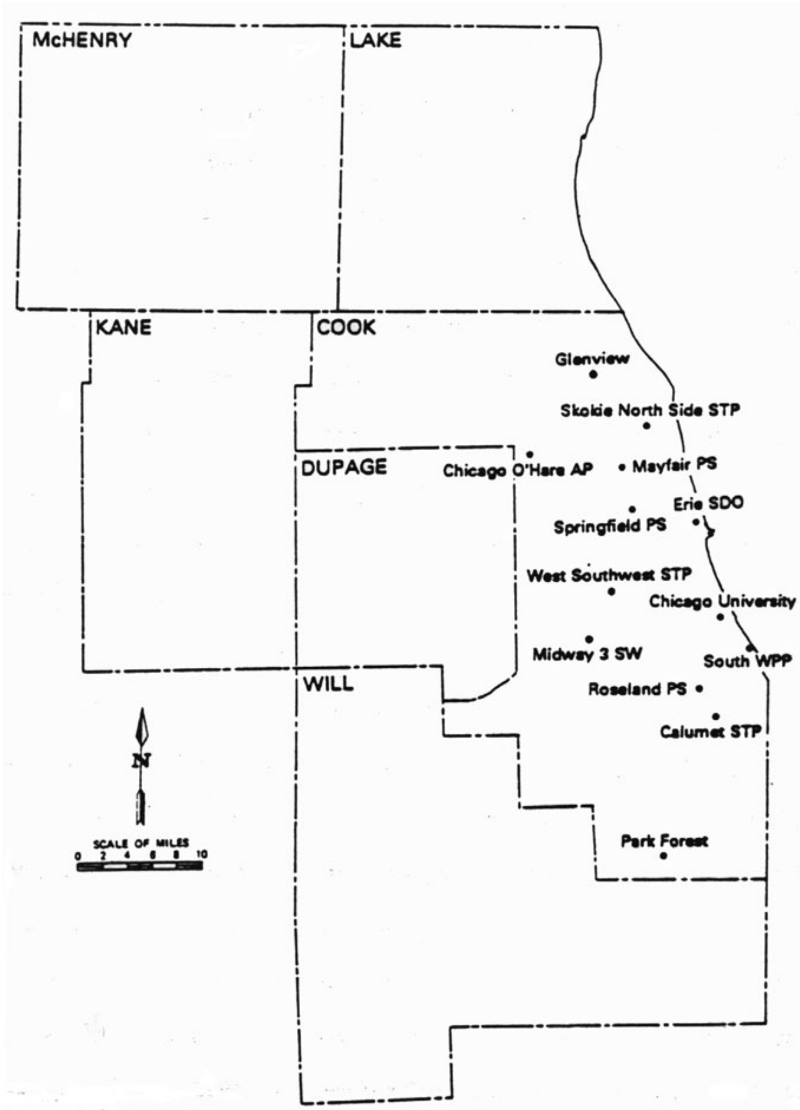
The regression equations used for water-year accounting prior to 1992, the regression analyses used in the Lakefront Accounting Technical Analysis, and the measured flows at the Hohman Avenue gage were compared using monthly Lake Michigan levels, and daily discharges from Hart Ditch at Munster, Indiana and the Grand Calumet River at Hohman Avenue for January 1 1996 through September 30, 1998. For this period, the old regression underestimated the flow by an average of 6.9 ft^3/s and the new regression overestimated the flow by an average of 10.7 ft^3/sec.

3.4.6. Twenty-five-Gage Precipitation Network

Precipitation information reflecting both the spatial and temporal distribution of precipitation over the diverted Lake Michigan watershed are essential for the runoff models to accurately determine the runoff from the diverted Lake Michigan watershed. Prior to water year 1990, precipitation data required for the modeling were from 13 raingages (Figure 3.4-a). Analysis of records from these gages indicated problems with the records from these sites including (1) irregular raingage distribution over the watershed; (2) low precipitation totals from a number of the stations; and different data collection, data reduction, and quality-assurance practices by different agencies operating different raingages (Vogel, 1988). In response to this, a network of 25 raingages was designed and installed by the ISWS in September through October, 1989 (Peppler, 1991a) Figure 3.4-b. The ISWS has been operating this raingage network since its inception. A single team of observers visits each site every 6-8 days to replace the charts and service the gages. Approximately once every four months the raingage service leader from the ISWS visits all the sites and performs maintenance and repairs for which the observers do not have adequate expertise. Detailed records of all inspections and service are maintained by the ISWS.

The chart from each raingage was manually inspected and edited before it was digitized into the computer. This allowed various potential errors with the raingage and recording mechanism to be identified from the charts. This also allowed potential observer errors (i.e. watch error at time change in April and October) to be identified and corrected. The software used to process the data identified whether the clock at a site is running fast or slow. The software defines hourly precipitation based on interpolation between digitized points from the chart. The array of hourly precipitation for an entire month was inspected for consistency. Missing values were filled in with interpolated information. Storm periods were identified and isohyetal patterns were drawn for each period. An ‘objective analysis program’ was used to determine new values for missing data and for values flagged as ‘questionable’ from the storm analysis. New values were compared with the values interpolated and any unrealistic values were adjusted. After all values were verified the hourly record for the month was complete.

Overall, the design and operation of this network should provide an accurate and complete record of precipitation over the watershed. The quality-assurance practices and the procedures for estimating missing values as documented by the Illinois State Water Survey (Westcott, 1998) are should ensure the quality of the record. Maintenance and modifications to the stations are well-documented.



EXPLANATION

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 Stations; and STP = Sewage Treatment Plant.

Figure 3.4-a - The thirteen raingage sites used prior to Water Year 1990.

Site Number	Latitude	Longitude
1	42°06'39"	87°52'06"
2	42°06'29"	87°45'06"
3	42°01'21"	87°52'39"
4	42°01'37"	87°45'20"
5	41°55'56"	87°52'41"
6	41°56'16"	87°45'39"
7	41°56'33"	87°38'43"
8	41°50'42"	87°52'53"
9	41°50'47"	87°45'28"
10	41°50'42"	87°38'27"
11	41°45'29"	87°52'20"
12	41°45'32"	87°46'06"
13	41°45'34"	87°38'07"
14	41°45'27"	87°32'38"
15	41°40'37"	87°59'52"
16	41°39'46"	87°52'13"
17	41°40'32"	87°45'00"
18	41°40'36"	87°39'07"
19	41°40'21"	87°32'22"
20	41°35'09"	87°52'35"
21	41°35'15"	87°44'52"
22	41°35'11"	87°38'00"
23	41°35'09"	87°32'14"
24	41°31'05"	87°44'00"
25	41°31'14"	87°34'25"

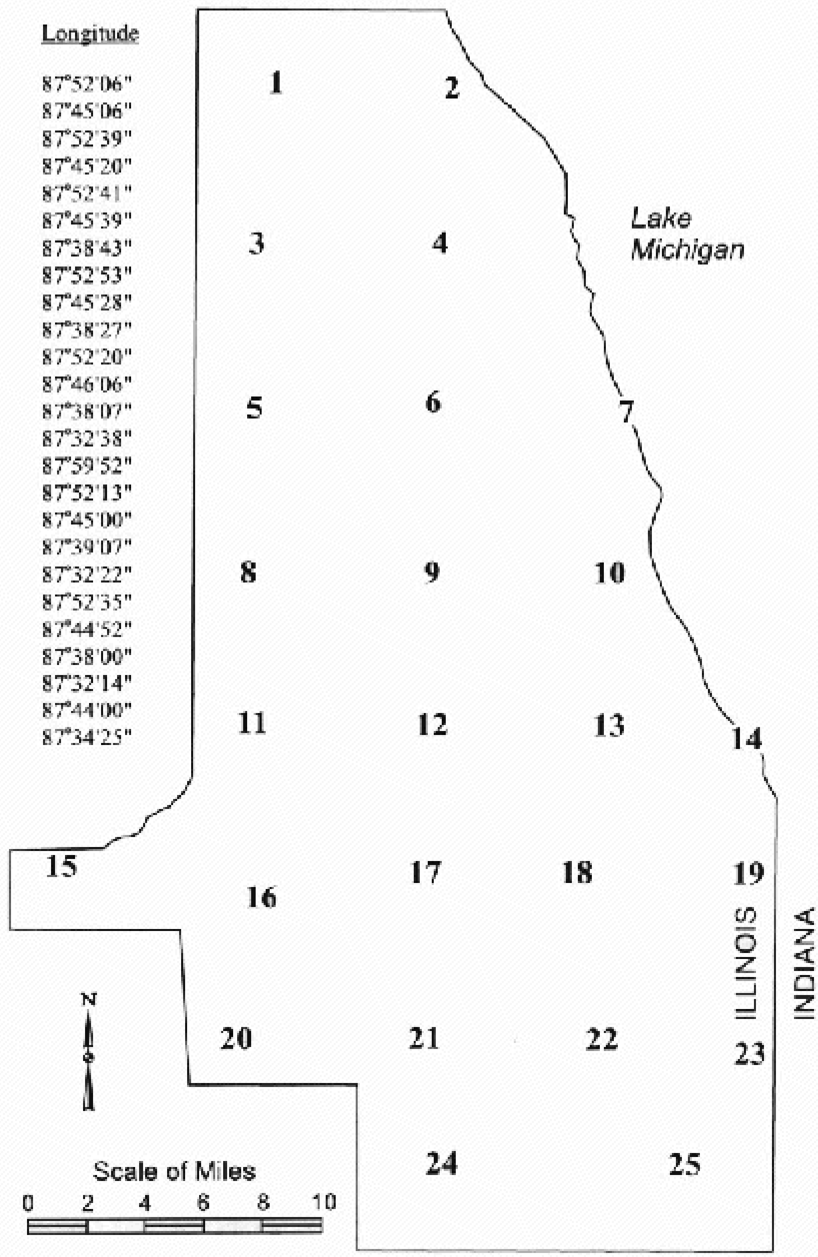


Figure 3.4-b - Configuration of the 25-site raingage network used during Water Years 1990-1993 (modified from Peppler, 1994).

It appears that the precipitation record has not been evaluated for changes in the 'catch' of the gages, especially after modifications to the gages. Double-mass curves are often used to compare records among gages. The ordinate is the cumulative sum of the record for the site being inspected; the abscissa is the cumulative sum of the record for the same period for a site or group of sites selected for comparison. Double-mass curves were done comparing each site to the average of all 25 sites. The double-mass curves were examined visually and the goodness-of-fit statistics for least-squares regression lines fit to the double-mass curves were examined to select the site that best represented the entire basin. Site 11 was selected for the base site. In addition to having the best fit to the average of all 25 sites, water-year reports on this network from the Illinois State Water Survey (Peppler, 1991b, 1993, 1994, 1995; Westcott, 1996, 1998) indicate that there were no significant modifications or repairs necessary at this site over the period of record.

Double-mass curves comparing each site to site 11 identified four sites where significant breaks were observed in the relation between the cumulative precipitation at the site and that from site 11. For all four of these sites the observed breaks were coincident with the gage being moved or replaced (Westcott, 1998, pp. 45-48). Linear least-squares regression was used to fit lines to the data before and after the break. The line fit to data preceding the break was extrapolated to estimate the cumulative precipitation from the site had the relation with site 11 remained constant. This was compared with the cumulative precipitation at the end of the period of record (September 30, 1998) to estimate the cumulative deviation of the record at the site. This was divided by the time between the break and September 30, 1998 to estimate the annual effect of this break in terms of inches of precipitation and percentage of mean-annual precipitation over the period of record. Table 3.4-b lists the stations and the possible error in the record at these stations. Figures 3.4-c through 3.4-f are the double-mass curves for these sites. No further attempts were made to determine: (a) whether the perceived change in the relation to site 11 was a change in the 'catch' of the gages; (b) whether the record preceding or following the change is 'correct'; and (c) what effect of the perceived change in annual precipitation would have on the calculated runoff from the diverted watershed.

Table 3.4-b - Results of double-mass curve comparisons of stations from the 25-gage precipitation network with Station 11, October 1, 1989 through September 30, 1998.

[Cumulative precipitation, period of record, estimated from regression with Site 11 was obtained by applying the regression between the site and site 11 prior to the break in the data with the measured cumulative precipitation at Site 11 for September 30,1998 (343.01 inches); the cumulative and average annual deficit are the amount that the gage underreported, negative values indicate that the gage overreported; mm/dd/yyyy, month/day/year]

Site	Time of break (mm/dd/yyyy)	Cumulative precipitation, period of record (inches)	Cumulative precipitation, period of record, estimated from regression with Site 11 (inches)	Cumulative deficit (inches)	Average annual deficit (inches)
6	07/13/1993	339.77	355.82	16.05	3.08 (8.1%)
14	10/31/1994	322.54	340.09	17.55	4.48 (12.5%)
15	11/22/1994	363.93	352.28	-11.65	-3.02 (-7.5%)
23	05/09/1996	344.86	355.73	10.87	4.54 (11.9%)

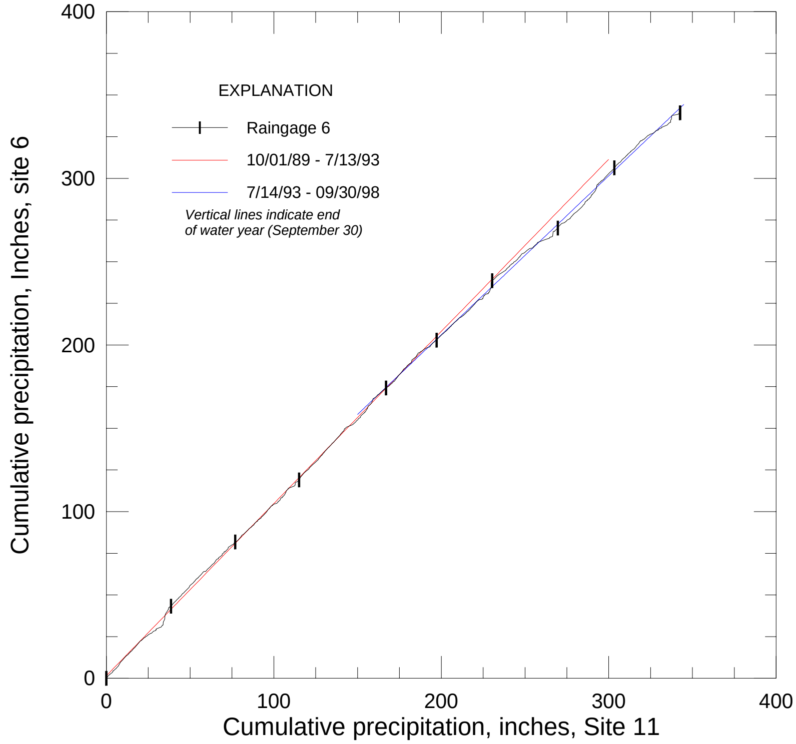


Figure 3.4-c - Double-mass curve comparing cumulative precipitation at sites 6 and 11, October 1, 1989 through September 30, 1998.

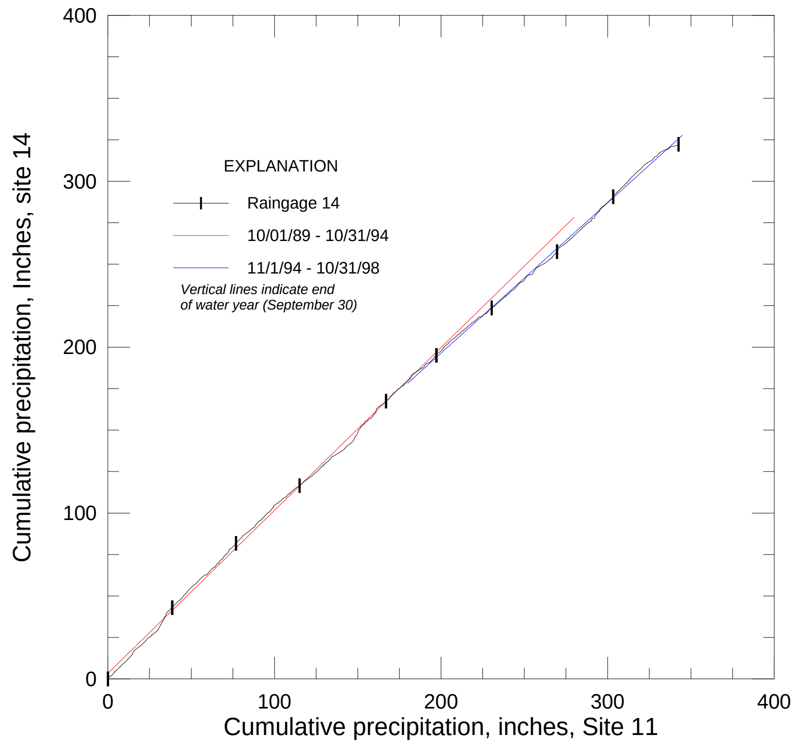


Figure 3.4-d - Double-mass curve comparing cumulative precipitation at sites 14 and 11, October 1, 1989 through September 30, 1998.

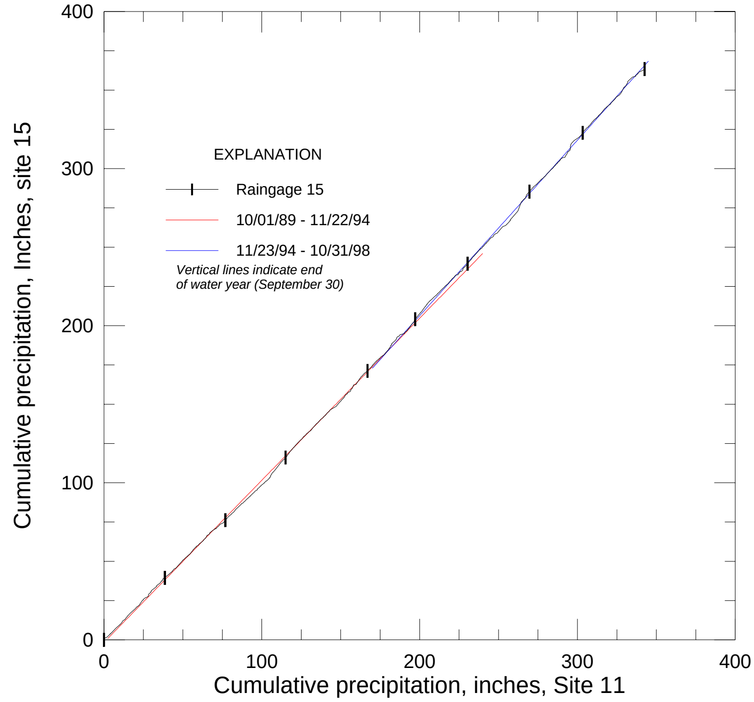


Figure 3.4-e - Double-mass curve comparing cumulative precipitation at sites 15 and 11, October 1, 1989 through September 30, 1998.

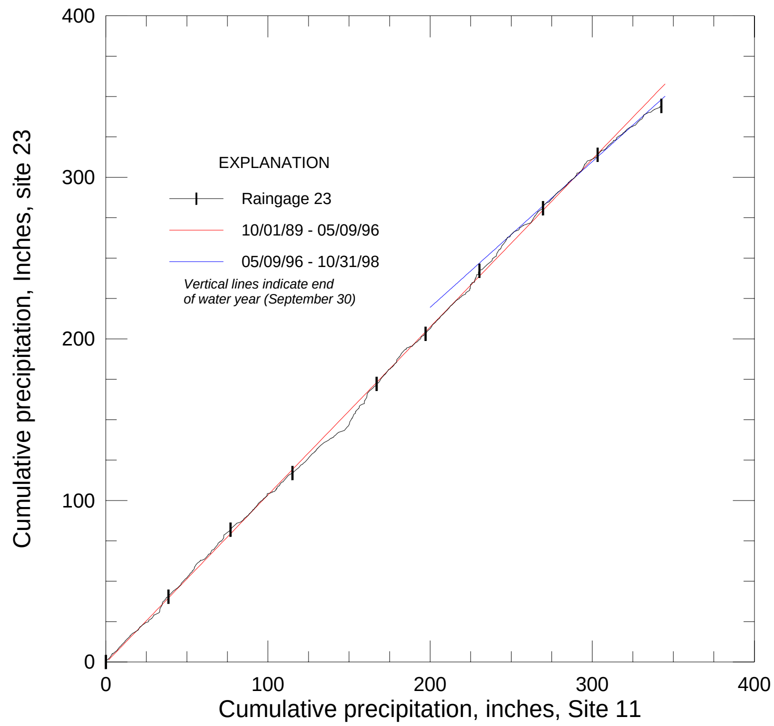


Figure 3.4-f - Double-mass curve comparing cumulative precipitation at sites 23 and 11, October 1, 1989 through September 30, 1998.

4. REVIEW OF LAKEFRONT ACCOUNTING

The consideration of Lakefront measurements of diversion has been recommended by previous Technical Committees and is also recommended by the Fourth Technical Committee. The July 1996 Great Lakes Mediation MOU prescribes a three-year transition period during which a dual-reporting system (lakefront and Romeoville accounting) will be operated. The purposes of the transition period are to assess the technical feasibility of moving the diversion measurements system to the lakefront and to give additional time for AVM calibration and opportunity to complete the QA/QC program. The MOU describes this period as “beginning after the initial calibration of the AVM’s at the lakefront (WY 1997).”

Lakefront accounting consists of four components: (1) the water-supply pumpage from Lake Michigan; (2) the direct diversions from Lake Michigan into the canal system; (3) the runoff from the diverted Lake Michigan watershed; and (4) the consumptive use of water supply pumped from Lake Michigan.

The water-supply pumpage is measured at the water treatment plants or pumping stations. The measurement accuracies for some of these facilities are reviewed in following sections. The direct diversions are measured by AVMs at the North Shore channel at Wilmette, at O’Brien locks and dams, and at the CRCW. The runoff from the diverted watershed and the consumptive use are fixed values that were agreed to during the mediation by all parties of the mediation.

The error in the calculated annual-mean discharges based on lakefront accounting is the errors from the water-supply pumpage measurements, the error from the measurement of lockage, leakage, and discretionary diversions, the error in the runoff, and the error in the consumptive use. Errors in the runoff and consumptive use, however, are irrelevant, as these are fixed by the mediation agreement. Therefore, the error in the calculated annual-mean discharges based on lakefront accounting is only based on the former two measurements. The following section gives more detail about the accuracy of the lockage, leakage, and discretionary diversion measurements and the water-supply pumpage measurements.

4.1. Measurement of Lockage, Leakage, and Discretionary Diversions

Flows resulting from lockages, leakage, and discretionary releases occur in the Calumet River, the Chicago River, and the North Shore Channel. These are measured by AVMs installed and operated by the USGS at the Calumet River at O’Brien Locks and Dam and the Chicago River at Columbus Drive. An AVM also has been installed (July 27, 1999) for the North Shore Channel at Wilmette. Acoustic velocity meters were selected for metering these flows because (1) the flows at these sites are not described by stage-discharge relations; (2) flow reversals occur at times at these sites; and (3) gauging these sites requires accurately measuring very low velocities in a large cross section. While AVMs are state-of-the-art flow-measuring instruments, flow conditions at these sites often are near the limitations of these instruments. The following sections describe the expected errors at these sites. Because these sites were still in a ‘troubleshooting’ mode of operation (Scott Gain, USGS, oral commun., 28 October, 1998), the data available for calibration and evaluation were somewhat limited at this time. The following sections are based on the ‘good’ data available at this time.

The AVM averaged water-velocity and stage and recorded them at 5-minute intervals. Discharge was calculated from the measured stage and AVM velocities by: (1) determining the cross-sectional area of the flow from the stage and a ‘stage-area rating’; (2) determining the mean velocity in the cross-section from the AVM velocity and a ‘index-velocity rating’; and (3) multiplying the mean velocity and area to determine the discharge for that 5-minute period. The ‘stage-area rating’ was determined from a survey of the channel cross section. The accuracy of this rating is determined by the accuracy of the survey equipment and techniques that were used. Because the same ‘stage-area rating’ was used in developing the ‘index-velocity rating’ and in determining discharges from the AVM velocities and because changes in stage are typical small at these sites, errors in the ‘stage-area rating’ will have a negligibly small effect on the calculated discharges (see discussion in section 3.3.2).

The ‘index-velocity rating’ was developed from measurements of the discharge in the channel. Measured discharges were divided by the cross-sectional area (obtained from the measured stage and the ‘stage-area rating’) to calculate the mean velocity in the cross section for each measurement. A discharge measurement typically took longer than the AVM averaging interval to complete; thus all the AVM measurements during the discharge measurement were averaged. Least-squares regression was used to determine a relation between the AVM velocity (the path or combination of paths used was determined as part of the rating analysis).

4.2. Lakefront Gages

4.2.1. Calumet River at O’Brien Locks and Dam (AVM system)

4.2.1.1. Description

The gauge at Calumet River at O’Brien Lock and Dam in Chicago, Illinois, is located on the downstream landwall of O’Brien locks, approximately $^{2}/_{10}$ of a mile southeast of the locks. This station is an AFFRA® AVM with two acoustic paths. The paths were installed at so that they angle upward as they cross the channel from the right bank to a pile about 100 ft from the left bank. The depths at the right guidewall are -6.0 ft CCD and the depths at the pile are at –4.0 ft CCD. The paths were oriented at approximately 52º and 62º to the flow, with one set of paths oriented upstream (into the flow), and one oriented downstream. The station was installed by the USGS in August and September 1996, and has been operating since October 1, 1996.

4.2.1.2. Measurement errors

Analysis of the time-series record of water-surface elevation and AVM velocities for the station at O’Brien Lock and Dam indicates that much of the record may not be suitable for analysis. Data are missing for large periods of the record. It also appears that sensors were adjusted or malfunctioning at times during the record. Figure 4.2-a shows time-series record of water-surface elevation and AVM velocities for this site. It appears that when the site was returned to operation on November 24, 1997, the distribution of measured path velocities was different than for the period prior to November 17, 1997. The stage sensor was not operating correctly from January 1, 1998 through February 24, 1998. It appears that the AVM began to malfunction on April 1, 1998 and this continued until the path velocities were out of range on May 23 (Path 1) and June 5 (Path 2). The AVM was returned to service on July 15, 1998 and appears to have functioned correctly from that time through December 31, 1998.

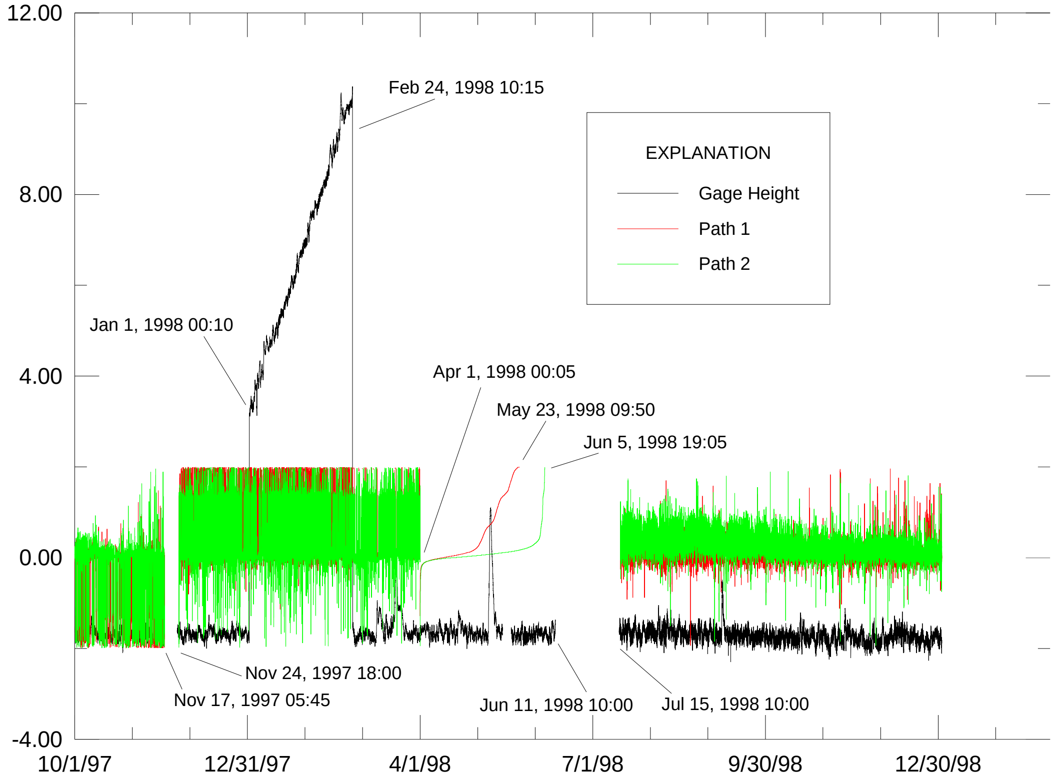


Figure 4.2-a - Record of water elevations and AVM velocities from the Calumet River at O'Brien Locks and Dam, October, 1997 through December, 1998.

It also appears that the AVM is set to 'clip' data exceeding ±2.00 ft/sec. Velocities of this magnitude are unreasonable for this site, so this is reasonable to screen the data; however, the missing values affect the filtering used to further analyze the data. Therefore, while the following paragraphs give an estimate of the noise in the data, this estimate is based on analysis of an incomplete record and therefore is only an approximation.

Analysis of the time-series record of AVM velocities for this site indicates a great deal of 'noise' in the measurements. Graphical analysis of the data indicated an apparent signal overlain with noise as large a ±1.5 ft/sec (Figure 4.2-b). The data were filtered to attempt to remove the noise from the underlying signal. Data for each day of ADCP measurements, as well as for the 12 hours preceding and following that day were selected for filtering and analysis. A linear trend was fit to the data, and then subtracted from the time-series record to make the data 'stationary.' A low-pass filter was then used to remove frequencies greater than 0.5 hour^{-1}. The linear trend was then added back to the filtered record to produce a final estimate of the long-term 'true' AVM velocities with the high-frequency noise removed. This analysis was intended to provide data to examine the potential accuracy of the discharges at this site. If a similar approach is used to develop an operational rating for the site, a more thorough analysis is needed to: (1) determine if a filter is appropriate; (2) to select an appropriate filter; and (3) to quantify the error introduced by the filter. Figure 4.2-b shows the AVM velocities for November 5, 1997, the filtered AVM velocities, and the mean velocities from ADCP measurements.

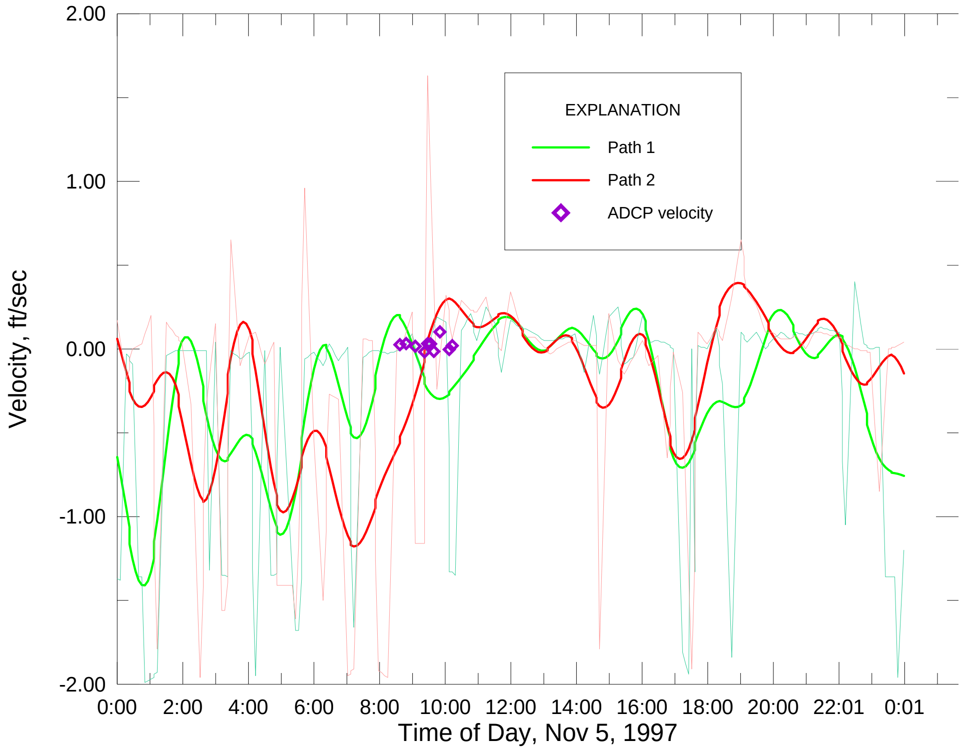


Figure 4.2-b - Graph showing measured and filtered AVM velocities and mean velocities from ADCP measurements at the O'Brien Locks and Dam AVM site, November 5, 1997.

The noise in the 5-minute AVM record at this site is significant. The filtering described above was done for every day with ADCP measurements. The root-mean-square (RMS) of the difference between the measured path velocities and the filtered path velocities for the entire data set was calculated as a measure of the noise in the data. The RMS differences for paths 1 and 2 were 0.337 and 0.322 ft/sec, respectively. This is compared to median velocities for these paths of 0.030 and 0.070 ft/sec, respectively, for the same data. The median discharge from three days that flow measurements were done was 500 ft^3/sec, which corresponds to a channel velocity of 0.10 ft/sec. Given the low velocities typical of this site, the magnitude of the noise in the AVM velocities often exceeded the mean velocity at this site. A method should be developed to filter the AVM record to improve the interpretation of these data.

Grouping replicate ADCP measurements as was done for the Columbus Avenue AVM (see Section 4.2.2) was not possible because of the limited data available to the Committee. Discharge measurements were done on November 5, 1997; November 25, 1997; June 11, 1998; June 16, 1998; and August 14, 1998. The AVM was not operating during the June, 1998 measurements so these could not be used in the rating analysis.

The channel geometry at this site includes an area about one-quarter of the channel width near the left (northeast) shore that is too shallow to measure with the ADCP. Discharge in this part of the channel was estimated based on the depth and velocity measured by the ADCP in the vertical nearest that edge of the channel. Analysis of 57 ADCP measurements indicated that this area had a median width of 98 ft out of a total channel width of 402 ft. The estimated flow in this unmeasured area ranged from –126 to +690 percent of the total flow in the channel, with a median absolute value of 7.5 percent of the flow. The equation used to estimate flow in this area assumes that the depth decreases linearly from the last measured depth to zero at the shore, and that the velocity decreases with the square root of the distance from the last vertical to the shore. Given the large portion of the channel width represented by the unmeasured area on the left bank, the Committee recommends that measurements be done by another means to verify and/or adjust the method used to calculate the discharge in this unmeasured area. These measurements should be done concurrently with ADCP measurements. They should be done over a range of flow conditions, as the curvature in the channel near this site and the sluice-gate location near the left side of the channel may result in different flow distribution between the unmeasured area and the main channel at different flows and stages.

No statistically significant relation between the mean velocity and the AVM path velocities was identified from these data. AVM data from November 24, 1997 through April 1, 1998 appear to be from a different distribution than those from other periods. Velocity data from this period appear to have the opposite sign than data from prior to November 17, 1997. Regression analysis of all the ADCP data and reversing the sign of the AVM velocities for the November 25, 1997 data resulted in the following equation, which explained 25 percent of the variance in the data and had a standard error of 0.07 ft/sec (corresponds to about ±340 ft^3/sec):

$$V_{mean} = 0.40\ V_{avg\ 1\&2} \tag{4.2-a}$$

where V_{mean} is the mean velocity in the channel, and $V_{avg\ 1\&2}$ is the average of the filtered velocities for paths 1 & 2

The Committee recommends that the record from this site should be examined to determine whether the sign of the velocities reported by the AVM was reversed on November 24, 1997, and if so, which is the correct sign for these velocities.

AVM data collected after July 15, 1998 have less noise than data collected earlier. Comparisons of measured and filtered AVM velocities for August 14, 1998 have RMS differences for paths 1 and 2 of 0.160 and 0.182 ft/sec, respectively. This is compared to median velocities for these paths of 0.154 and 0.423 ft/sec, respectively. A similar comparison was done for a low-flow period from September 22-28, 1998. Comparison of measured and filtered AVM velocities for this period have RMS differences for paths 1 and 2 of 0.111 and 0.152 ft/sec, respectively. This is compared to median velocities for these paths of 0.111 and 0.368 ft/sec, respectively. These results indicate a two- to three-fold reduction in the noise in the AVM velocities after July 15, 1998.

Discharges measured by the ADCP on August 14, 1998 ranged from 206 to 1,474 ft^3/sec. Regression analysis of the mean velocities from the ADCP measurements with the concurrent AVM velocities resulted in the following equation, which explained 17 percent of the variance and had a standard error of 0.06 ft/sec.:

$$V_{mean} = 0.35\ V_{avg\ 1\&2} \tag{4.2-b}$$

Equations 4.2-a and 4.2-b cannot be used to determine discharges for this site and no meaningful error analysis can be done for these equations. However, the similar slope of these equations indicates a consistent relation emerging from the data. The reduced noise in the AVM velocities since July 15, 1998 may indicate that the early operational problems with the AVM have been resolved with the modifications made in July, 1998. A continuing program of discharge measurements at this site should allow development of an operational rating and a detailed analysis of the errors in the discharges from the rating. The existing data are likely to define an upper limit to the error in the rating. Based on the standard error of Equation 4.2-b, and a median cross-sectional area of 4,876 ft^2, the error in the discharges computed for this site is expected to be less than ±300 ft^3/sec.

The Committee recommends that discharge measurements should be done to allow development and evaluation of a rating for this site. Based on an examination of the AVM data provided to the Committee, it appears that the period of good record for this site will begin on July 15, 1998, with much greater errors in the record preceding this date.

4.2.1.3. Backup system

The backup system is a means to estimate flow at O'Brien Locks and Dam for periods when the AVM is inoperative or is not functioning properly. The backup system for the AVM is a set of regression equations that relate the flow at the AVM to the head across the dam and the sluice-gate openings. The estimation equations described in this report are based on measurements of flow for four gate openings as well as two measurements of leakage. These equations were developed to present a possible format for the backup equations and to give a first approximation of the error to be expected from the backup equations. These equations are based on very limited data and are not intended for operational use. These equations also do not consider the flow from lockages. Much more data should be obtained to develop a set of backup equations for operational use.

The backup equations were developed by first developing an equation to estimate the leakage past the structure based on the headwater and tailwater elevations. Data provided to the Technical Committee included two sets of ADCP measurements for leakage conditions. After the equation to estimate leakage was developed, it was used to estimate the leakage for the sluice-gate measurements. Four sets of ADCP measurements were provided to the Committee for sluice-gate openings of 1, 3, 5, and 10 ft. Headwater and tailwater elevations for all the ADCP measurements were obtained from the Illinois' LMO-6 reports for the dates of these measurements. Table 4.2-a summarizes all the measurements used to develop these equations.

Table 4.2-a - Summary of sluice-gate and leakage measurements used to develop backup equations for AVM at Calumet River at O'Brien Locks and Dam.

Date (mm/dd/yyyy)	Gate opening (ft)	Headwater elev. (CCD)	Tailwater elev. (CCD)	Head (ft)	Discharge Mean measured (ft^3/sec)	Standard deviation (ft^3/sec)	Coefficient of Variation (percent)
11/5/1997	Leakage	1.70	-1.84	3.54	110	164	149%
6/11/1998	Leakage	1.52	-1.62	3.14	151	63.0	42%
6/17/1998	1	1.58	-1.85	3.43	185	66.9	36%
11/25/1998	5	0.73	-1.75	2.48	647	158.	24%
11/25/1998	10	0.73	-1.75	2.48	1,150	148	13%
8/14/1998	3				583	334	57%

Leakage was assumed to be through a vertical, rectangular orifice with a constant, but unknown, width extending the entire height of the structure. The theoretical equation for flow through a submerged orifice can be re-written for these assumptions as:

$$Q_L = C_L h_1 \sqrt{\Delta h} \tag{4.2-c}$$

where Q_L is the leakage discharge; C_L is the leakage coefficient; h_1 is the upstream depth of water above the gate sill; and Δh is the head difference across the structure.

Based on the two sets of measurements, the average leakage coefficient, C_L, is 4.92. The accuracy of this equation was estimated as ±124 ft^3/sec, based on the root mean square of the standard deviations of the measured discharges.

The flow through the sluice gates is expected to be described by the theoretical equation for flow through a submerged orifice. The basic form of the equation is:

$$Q = C_{gs} B h_3 \sqrt{2g\Delta h} \tag{4.2-d}$$

where Q is the discharge; C_{gs} is the gate coefficient; B is the gate width; h_3 is the downstream depth of water above the gate sill; g is the acceleration of gravity, 32.2 ft/sec^2; and Δh is the head across the structure.

The following empirical equation was developed to estimate the gate coefficient, C_{gs}, based on the ratio of the gate opening to the downstream depth of water.

$$C_{gs} = 1.1\left(\frac{h_3}{h_g}\right)^{-1.38} \qquad (4.2\text{-e})$$

where h_g is the gate opening, and all other terms are defined above.

This equation has a standard error of –32 percent to +48 percent. Based on the measurements available, the estimated error in the discharge from the backup equations ranged from –145 to +155 ft^3/sec for the 1-foot gate opening to –550 to +760 ft^3/sec for the 10-foot gate opening. More detailed analysis of the error is not warranted, given the paucity of data for backup-equation development. The Committee recommends that more measurements need to be done to develop backup equations for this site. The Committee recommends that detailed records of gate opening, headwater, and tailwater elevations, and lockages should be maintained to develop backup equations using the AVM record, once the AVM is operational. These records need to include which gates are open, how far each gate is open, the times that gates are opened and closed, and the times of lock-gate operations. Errors in the AVM record at low flows are likely to be large, relative to the magnitude of the flow. Therefore, the Committee recommends that ADCP measurements should be done to quantify the leakage and the sluice-gate flow for a variety of heads at smaller gate openings, rather than using the AVM record to develop the backup equations for low flows.

4.2.2. Chicago River at Columbus Avenue (AVM system)

4.2.2.1. Description

The gauge at Chicago River at Columbus Avenue in Chicago, Illinois, is located underneath the Columbus Avenue bridge, approximately $^4/_{10}$ of a mile west of the CRCW. This station is an AFFRA® AVM with four acoustic paths. The paths are installed at two depths; paths 1 and 2 are at -8.0 ft CCD and paths 3 and 4 are at –15.0 ft CCD. The USGS is adjusting path depths to achieve the most robust acoustic signals, which will change the rating and error analyses from this report. The paths are oriented at about a 60º angle to the river wall, with one set of paths oriented toward the northwest (paths 1 and 3), and one set oriented toward the southwest (paths 2 and 4). The station was installed in November 1996, and has been operating since December 2, 1996.

4.2.2.2. Measurement errors

Velocities from the four acoustic paths and the water stage were recorded at five-minute intervals. The discharge was calculated by determining the mean velocity in the channel and multiplying this by the cross-sectional area of the flow. The cross-sectional area was calculated from a stage-area rating based on a survey of the channel cross section near the gage. The relation between area and stage is:

Area = (208.33 x GH) + 5061 (4.2-f)

Where Area is the channel flow area, in ft^2; and GH is the gage height relative to the CCD, in ft.

The mean velocity was determined from the path velocities and an 'index-velocity rating' that relates the average of the path velocities to the mean velocity in the channel. The mean velocity in the channel was developed from the measured total discharge in the channel divided by the cross-sectional area of the flow. The cross-sectional area was determined from the gage height during the measurement and the stage-area rating (Equation 4.2-f). The discharge was measured using a broadband ADCP. Measurements were done at a defined cross section located just upstream from the Columbus Avenue bridge. Because of the low velocities in the channel, measurements were typically done using a tagline and an electric winch to maintain a constant rate across the channel. The USGS has made over 350 individual discharge measurements at this site. Many of these are not independent measurements but are rather replicate measurements of the same conditions. Most measurements had between 3 and 20 replicates, with a median of eight replicates.

The USGS (written commun., May, 1998) developed the following index-velocity rating based on the mean AVM velocity:

$$V_{mean} = 0.917 \times V_{AVM} \qquad (4.2\text{-g})$$ [1]

where V_{mean} is the mean velocity in the channel; and V_{AVM} is the mean velocity from the AVM paths.

Analysis of a longer period of data indicated that Path 3 provided the best agreement with measured channel velocities. Therefore a new regression was developed to give the index velocity based on Path 3 from the AVM (USGS, written commun., October, 1998)

$$V_{mean} = 0.911 \times V_{Path3} + 0.005 \qquad (4.2\text{-h})$$

where V_{Path3} is the velocity from AVM path 3.

The regression analyses used to develop these ratings (Equations 4.2-g and 4.2-h) appear to have used all of the ADCP measurements as independent data points. One of the assumptions of regression analysis is that data points are statistically independent[2]. The Durbin-Watson statistics for the above regression analyses were 0.35 and 0.87, respectively, indicating that the assumption of statistically independent data points was not met. Another assumption in regression analysis is that the variance of the residuals is constant. Figures 4.2-c and 4.2-d show the residuals for these regressions. Residuals for the rating using the average of all AVM paths (Equation 4.2-g; Figure 4.2-c) showed a marked decrease in the residuals over time. Residuals for the rating using only path 3 (Equation 4.2-h; Figure 4.2-d) showed a marked decrease in the variance with increasing mean velocity. Among the effects of both serially correlated residuals and non-constant variance of residuals is that these: (1) preclude accurate calculation of error variance and prediction intervals, and (2) they result in bias in the prediction equation. Because the existing rating equations were not suitable to estimate the error in the discharges determined from this site, further analysis was done to determine an index-velocity rating that would allow the errors in the discharges to be estimated. The rating developed and presented in the following sections is intended to allow estimation of the accuracy of the discharge record for this site given the current instrumentation and rating measurements at the site. It is not intended as the rating for the site—rather it is hoped that this will provide guidance that will help the USGS plan further measurements and analyses to improve on and define the accuracy of this rating. The USGS is continuing to revise the measurement protocol and rating for this site to develop an index-velocity rating that will be used to calculate the discharge record for this site.

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[1] Based on the USGS quality-assurance plan for the Columbus Avenue AVM, May 27, 1998. The slope in the equation in the quality-assurance plan was for the inverse equation (V_{AVM} as a function of V_{mean}); equation 4.2-g has been corrected.

[2] More precisely, that the residuals from the regression are independent or not serially correlated. Values of the Durbin-Watson statistic outside the range $1.5 < d < 2.5$ indicate the residuals are likely serially correlated (Ott, 1992, p. 706).

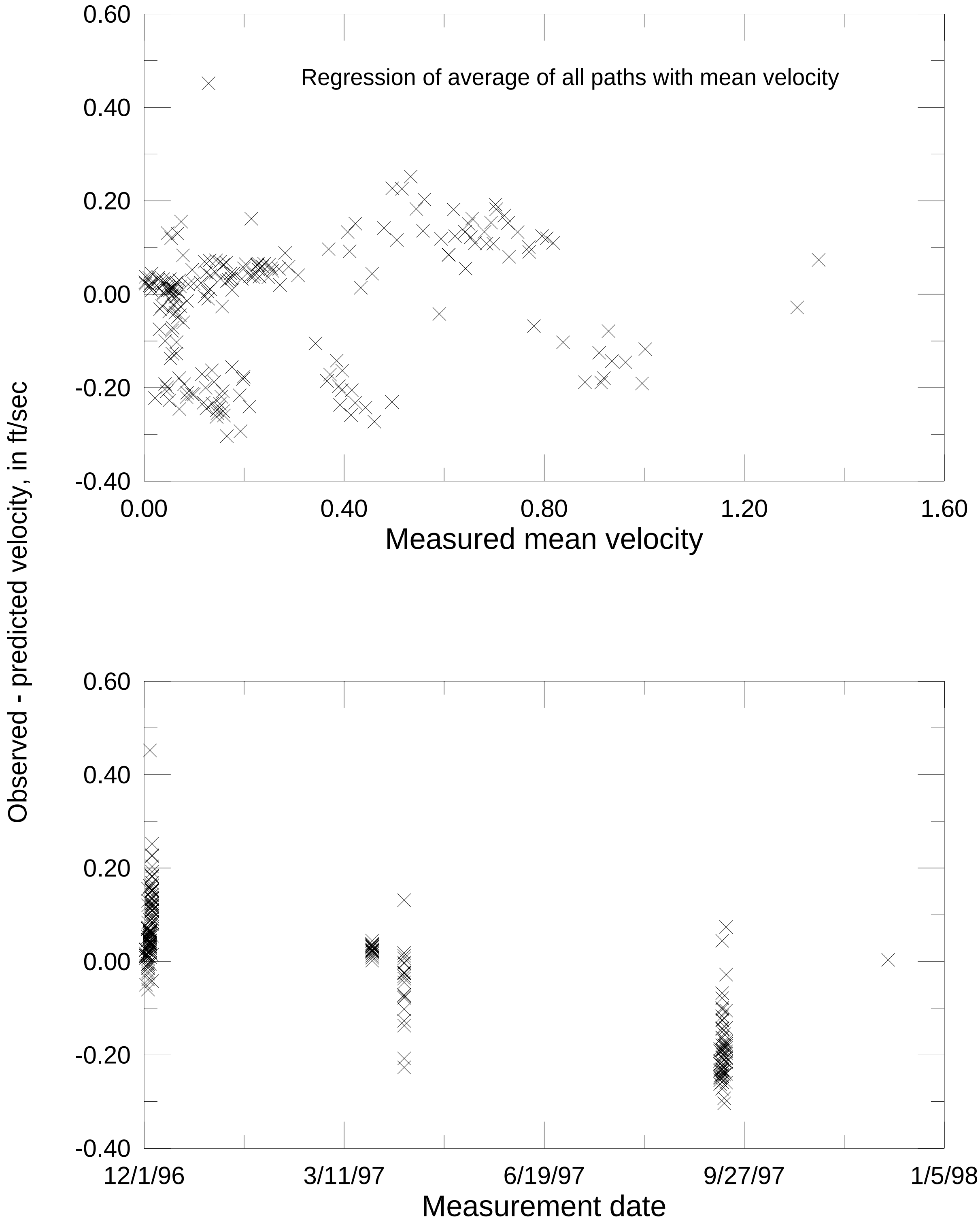

Figure 4.2-c - Graph showing differences between measured mean velocities and those predicted by rating based on average of all four AVM paths.

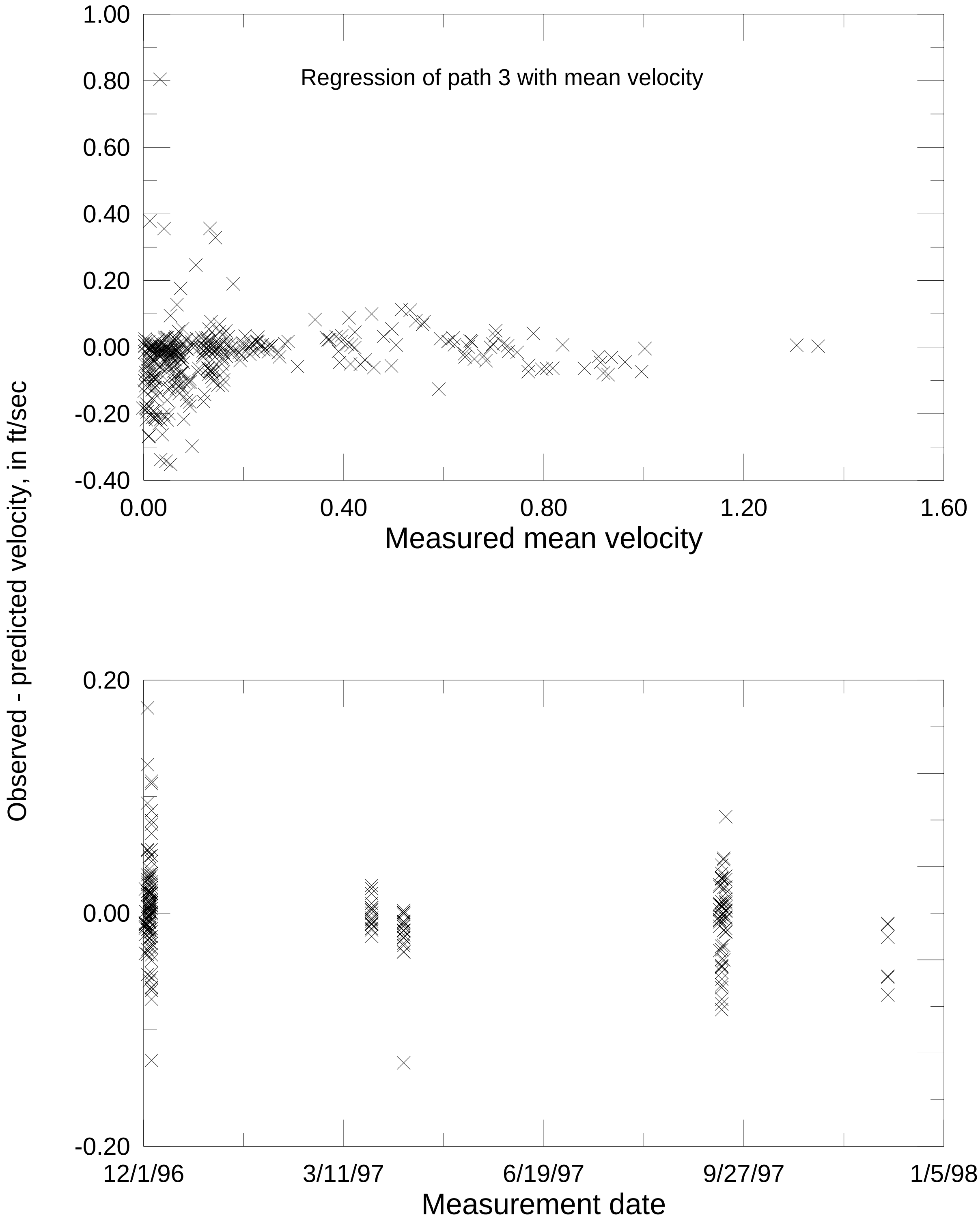

Figure 4.2-d - Graph showing differences between measured mean velocities and those predicted by rating based on AVM path 3.

Analysis of the time-series record of AVM velocities for this site indicated a great deal of 'noise' in the measurements. Graphical analysis of the data indicated an apparent signal overlain with noise as large a ±0.40 ft/sec (Figure 4.2-e). The data were filtered to attempt to remove the noise from the underlying signal. Data for each day of ADCP measurements, as well as for the 12 hours preceding and following that day were selected for filtering and analysis. A linear trend was fit to the data, and then subtracted from the time-series record to make the data 'stationary.' A low-pass filter was then used to remove frequencies greater than 0.5 hour^{-1}. The linear trend was then added back to the filtered record to produce a final estimate of the long-term 'true' AVM velocities with the high-frequency noise removed. This analysis was intended to provide data to examine the potential accuracy of the discharges at this site. If a similar approach is used to develop an operational rating for the site, a more thorough analysis is needed to: (1) determine if a filter is appropriate; (2) to select an appropriate filter; and (3) to quantify the error introduced by the filter. Figure 4.2-e shows the AVM velocities for September 16, 1997, the filtered AVM velocities, and the mean velocities from ADCP measurements.

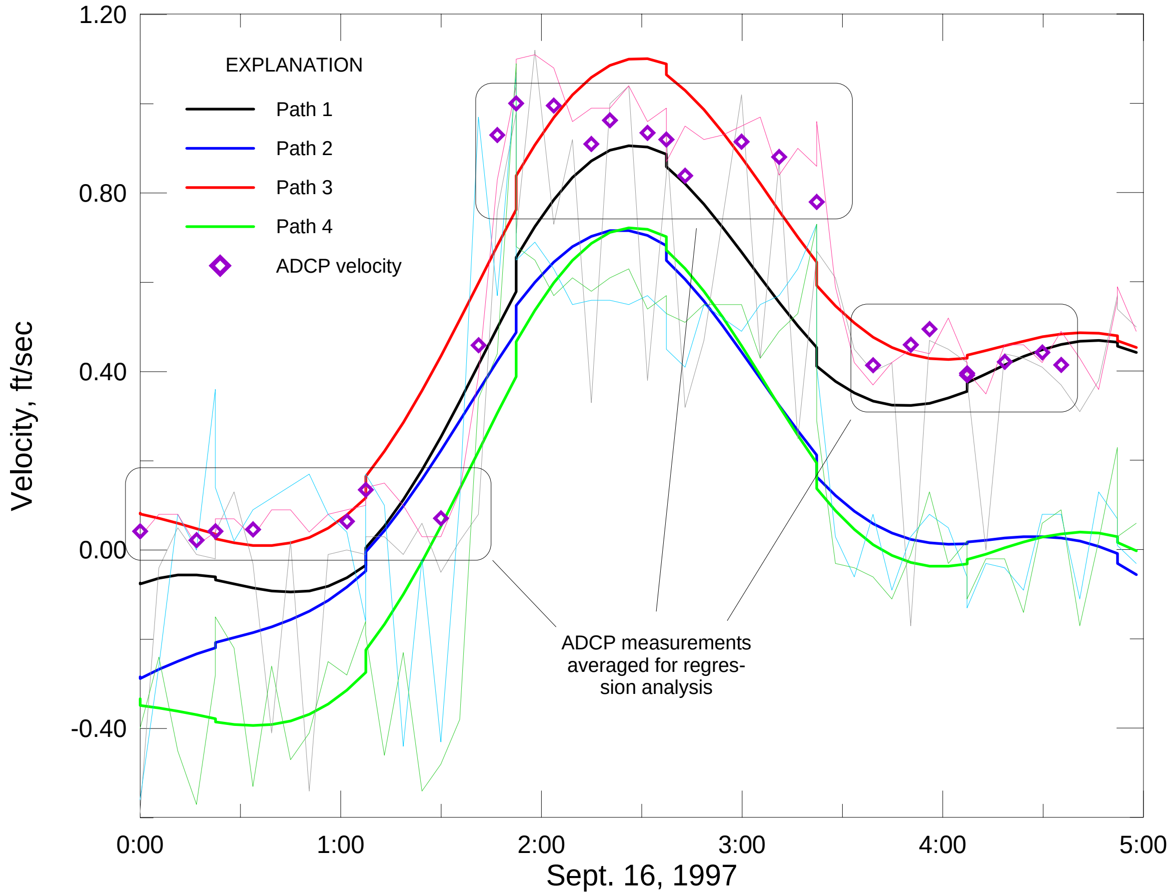

Figure 4.2-e - Graph showing measured and filtered AVM velocities and mean velocities from ADCP measurements at the Columbus Avenue AVM site, September 16, 1997.

The noise in the 5-minute AVM record at this site is significant. The filtering described above was done for every day with ADCP measurements. The RMS of the difference between the measured path velocities and the filtered path velocities for the entire data set was calculated as a measure of the noise in the data. The RMS differences for paths 1 through 4 were 0.211, 0.304, 0.158, and 0.221 ft/sec, respectively. This is compared to mean velocities for these paths of 0.030, -0.042, 0.062, and 0.049 ft/sec, respectively, for the same data. The median cross-sectional area of the channel is 4,700 ft^2. The median discharge from seventeen days that flow measurements were done was 360 ft^3/sec, which corresponds to a channel velocity of 0.08 ft/sec. Given the low velocities typical of this site, the magnitude of the noise in the AVM velocities often exceeded the mean velocity at this site. A method should be developed to filter the AVM record to improve the interpretation of these data.

Much of the noise in the AVM data may reflect the natural variability in the system caused by eddies, turbulence, and other effects in the flow at this site. In addition to the noise in the record, the relation among AVM paths varies over time. Flow reversals in the vertical have been observed during rating measurements at the site. These appear to be limited to low-flow conditions. These often had flow near the channel bed toward Lake Michigan while flow near the surface was in the downstream direction away from the Lake. The opposite condition has been observed on days with strong winds from the west and low-flow conditions in the river—in this case, flow near the bed was in the downstream direction, while flow near the surface was toward Lake Michigan. Figure 4.2-f shows the longitudinal component (west) of velocity measured by the ADCP for transect 558 on November 30, 1998. This is one of a series of measurements showing upstream flow (toward Lake Michigan) in the upper six to eight feet of the channel, and downstream flow for the rest of the depth. The average flow for 11 measurements between 12:15 and 14:30 was 230 ft^3/sec, with a mean channel velocity of 0.048 ft/sec. Depending on the depth at which the flow reversal occurs, it is possible for paths 3 and 4 to measure different velocities than paths 1 and 2. Figure 4.2-g shows measured and filtered AVM velocities for paths 1 and 2 were negative while those for paths 3 and 4 were positive during this measurement on November 30, 1998.

Other factors also can cause the relations among the four paths to change. Vertical gradients in water temperature have been measured at this site. It is possible for sharp temperature gradients to reflect the acoustic signals causing paths near the gradient to give erroneous velocity readings. Flows that are oblique to the channel could cause paths 1 and 3 to read different velocities than paths 2 and 4. Given the channel geometry, it is not likely that such flows would be sustained for any significant period.

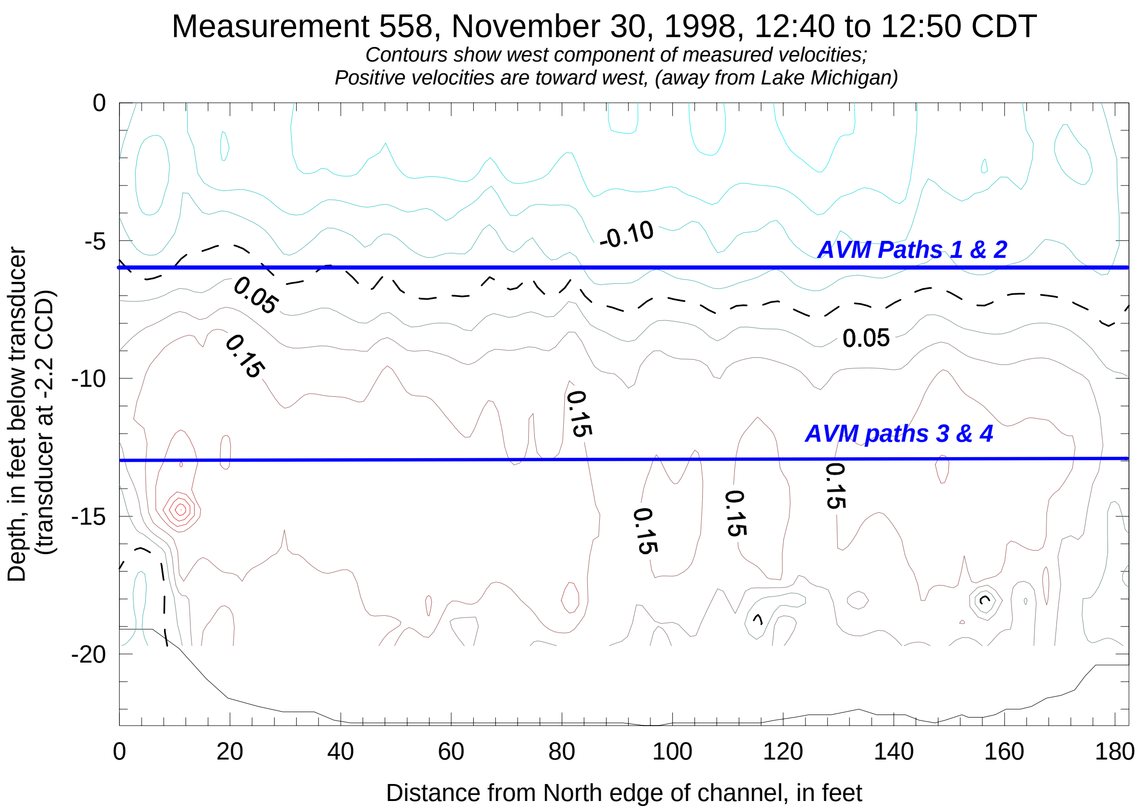

Figure 4.2-f - Graph showing west component of velocities from ADCP measurement 558 at the Columbus Avenue AVM site, November 30, 1998.

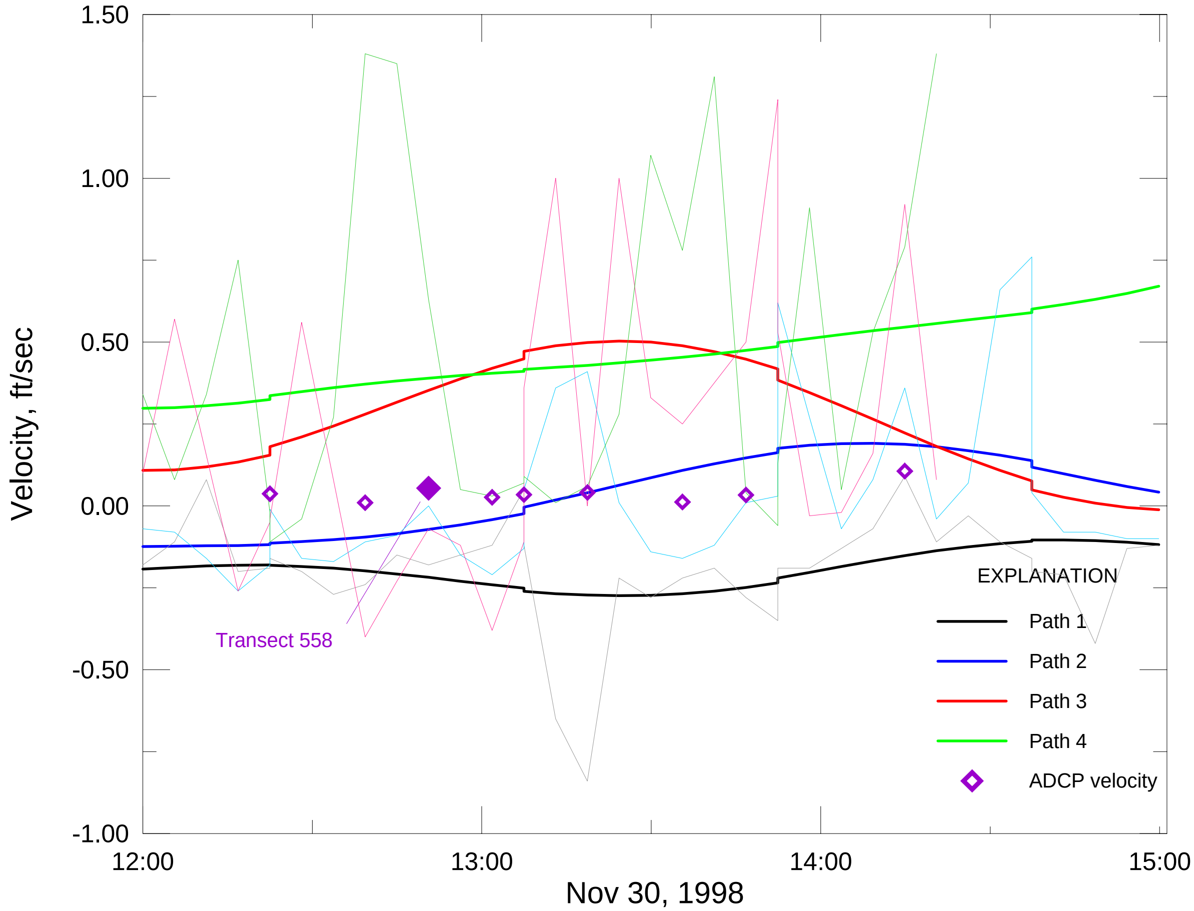

Figure 4.2-g - Graph showing measured and filtered AVM velocities and mean velocities from ADCP measurements at the Columbus Avenue AVM site, November 30, 1998.

Groups of replicate ADCP measurements were identified from time-series plots of ADCP and AVM velocities (for example, see Figure 4.2-e). The data from each group were averaged and the average velocities used for development of the rating equation. The rating-equation that provided the best fit to the mean velocities averaged AVM velocities from paths 1 and 3 and related this average to ADCP measurements. The index-velocity rating developed from this data set had a standard error of 0.047 ft/sec, which corresponds to a discharge of ±220 ft^3/sec. The index-velocity rating is a single estimate of the true relation between the AVM (index) velocity and the mean velocity in the channel. Any error in the rating is incorporated into every instantaneous discharge calculated from the rating, resulting in a bias in the calculated discharges (Gain, 1998; Sloat and Gain, 1995).

Dividing the measurements into different data ranges and developing separate ratings for each range may improve the accuracy of the rating. This is appropriate if (a) the relation is non-linear or (b) if the variance is not uniform over the data set. In addition to providing a better fit to the data and a more appropriate application of regression analysis, multiple ratings will reduce the overall bias in the record. While the error in each rating will still translate into a bias in the overall record, errors in different ratings may be opposite in direction, reducing the overall error in the annual discharge record.

A two-part rating was developed for the mean velocity based on the average of the AVM velocities measured from paths 1 and 3. The rating for the average of path 1 and 3 less than or equal to 0.12 ft/sec is:

$$V_{mean} = 0.473 \times V_{Avg1\&3} + 0.058 \qquad (4.2\text{-}i)$$

where $V_{Avg1\&3}$ is the average velocity from AVM paths 1 and 3.

For the average of path 1 and path 3 less than or equal to 0.12 ft/sec, the standard error is 0.028 ft/sec, which corresponds to a discharge of about ±130 ft3/sec.

The rating for the average of path 1 and 3 greater than 0.12 ft/sec is:

$$V_{mean} = 0.941 \text{ x } V_{Avg1\&3} \tag{4.2-j}$$

For the average of path 1 and path 3 greater than 0.12 ft/sec, the standard error is 0.051 ft/sec, which corresponds to a discharge of about ±240 ft3/sec. The overall error in the annual discharge using the two-part rating will be about ±190 ft3/sec.

The 'noise' in the velocity measurements and the accuracy of the index-velocity ratings will impose limits on the utility of data from this site for any real-time operational decisions. It has been proposed to use the data from this station to determine the leakage through the harbor walls and control structures during periods with no direct diversions. The calculated leakage would be used for real-time control of pumps to return the leakage to Lake Michigan. The noise in the data from velocity paths 1 and 3 (0.263 ft/sec) corresponds to an error in the calculated 5-minute discharges of approximately ±1,240 ft^3/sec. This error can be reduced by averaging many measurements together. A 4-hour average will have an error of ±180 ft^3/sec. Averaging, however, does not reduce the effect of bias, such as that from the rating. Leakage measurements will always fall in the low-flow range of the ratings. The bias for the low-flow rating is ±130 ft^3/sec, which is the limit of the accuracy of leakage measurements from the two-part rating described in this report.

Significant improvements should be possible in the accuracy of discharges determined for this site. The noise in the velocity signal from path 2 was over 40 percent greater than from the other upper path (path 1). Filtered data from paths 2 and 4 explained 45 and 52 percent of the variance in the mean velocities, respectively. In comparison, paths 1 and 3 explained 92 and 94 percent of the variance in the mean velocities, respectively. This indicates that improvements to paths 2 and 4 should be possible; these may improve the accuracy of the rating and long-term record.

In addition, the ratings developed to evaluate the accuracy of the record were based on using all the velocity measurements from paths 1 and 3. Given the flow characteristics of this site, especially at low-flow conditions (i.e., flow reversals, eddies, thermal gradients), the rating and record could potentially be improved if ratings are developed that use only AVM data that meet quality-assurance criteria. These criteria would have to be based on data recorded at part of each 5-minute measurement at the site. These quality-assurance data may include water temperature above and below each path, speed of sound or traveltime for each measurement, and measurement of the velocity profile over the entire depth of the channel.

4.2.2.3. Backup system

The backup system is a means to estimate flow at the CRCW for periods when the AVM is inoperative or is not functioning properly. The backup system for the AVM is a set of regression equations that relate the flow at the AVM to the head across the dam, the sluice-gate openings, and the lockages. The estimation equations described in this report are based on measurements of flow for four gate openings as well as five measurements of leakage. These equations were developed to present a possible format for the backup equations and to give a first approximation of the error to be expected from the backup equations. These equations are based on very limited data and are not intended for operational use. Much more data should be obtained to develop a set of backup equations for operational use.

The backup equations were developed by first developing an equation to estimate the leakage past the structure based on the headwater and tailwater elevations. Data provided to the Technical Committee included five sets of ADCP measurements for leakage conditions. After the equation to estimate leakage was developed, it was used to estimate the leakage for the sluice-gate measurements. The initial development of a backup equation was done using data collected for the North sluice gates from

November 17 - 30, 1998. Four sets of ADCP measurements were provided to the Committee for sluice-gate openings of 2, 4, 6, and 8 ft. Headwater and tailwater elevations for all the ADCP measurements were based on data collected by the USGS for gages installed on the south guide wall just west of the lock chamber. Table 4.2-b summarizes all the measurements used to develop these equations.

Table 4.2-b - Summary of sluice-gate and leakage measurements used to develop backup equations for the AVM at Chicago River at Columbus Avenue.

Date mm/dd/yyyy	Gate opening (ft)	Headwater elev. (CCD)	Tailwater elev. (CCD)	Head (ft)	Discharge Mean measured (ft^3/sec)	Standard deviation (ft^3/sec)	Coefficient of Variation (percent)
11/17/1998	Leakage	18.48	16.32	2.16	97	50	51.5%
11/18/1998	Leakage	17.94	16.44	1.50	79	70	88.6%
11/18/1998	Leakage	17.84	16.49	1.35	93	45	48.4%
11/19/1998	Leakage	18.32	16.49	1.83	107	111	103.7%
11/20/1998	Leakage	18.21	16.35	1.86	70	56	80.0%
11/18/1998	1.86	17.87	16.47	1.40	119	70	58.8%
11/19/1998	4.1	18.50	16.59	1.91	290	40	13.8%
11/19/1998	6.2	18.40	16.67	1.73	426	84	19.7%
11/20/1998	8.5	18.23	16.43	1.80	656	62	9.5%

Leakage was assumed to be through a vertical, rectangular orifice with a constant, but unknown, width extending the entire height of the structure. The theoretical equation for flow through a submerged orifice can be re-written for these assumptions as:

$$Q_L = C_L h_1 \sqrt{\Delta h} \tag{4.2-k}$$

where Q_L is the leakage discharge; C_L is the leakage coefficient; h_1 is the upstream depth of water above the gate sill; and Δh is the head difference across the structure.

Based on the five sets of measurements, the average leakage coefficient, C_L, is 3.75. The accuracy of this equation was estimated as ±110 ft^3/sec, based on the RMS of the standard deviations of the measured discharges.

The flow through the sluice gates is expected to be described by the theoretical equation for flow through a submerged orifice. The basic form of the equation is:

$$Q = C_{gs} B h_3 \sqrt{2g\Delta h} \tag{4.2-l}$$

where Q is the discharge; C_{gs} is the gate coefficient; B is the gate width; h_3 is the downstream depth of water above the gate sill; g is the acceleration of gravity, 32.2 ft/sec^2; and Δh is the head across the structure.

The following empirical equation was developed to estimate the gate coefficient, C_{gs}, based on the ratio of the gate opening to the downstream depth of water.

$$C_{gs} = 1.09\left(\frac{h_3}{h_g}\right)^{-1.08} \tag{4.2-m}$$

where h_g is the gate opening, and all other terms are defined above.

This equation has a standard error of –7.6 percent to +8.2 percent. Based on the limited measurements available, the estimated error in the discharge from the backup equations ranged from –123 to +130 ft^3/sec for the 1-foot gate opening to –298 to +392 ft^3/sec for the 8-foot gate opening.

As part of making these measurements in November, 1998, the actual gate opening was measured and compared with the gate opening from the dial indicators on the gate-operating mechanism, and with the gate opening indicated in MWRDGC's control room. The actual opening was measured by measuring the travel of the brass screw that moves the gate. Table 4.2-c lists the gate-opening measurements.

Table 4.2-c - Summary of measured and indicated openings for sluice gates at Chicago River Controlling Works, November 17-20, 1998.

Gate	Measured (ft)	Dial Indicator (ft)	Dispatcher (ft)
NORTH SLUICE GATES 11/18/1998 – *ALL GATES CLOSED*			
GATE 1	0.08	0.05	0.12
GATE 2	0.12	0	0.02
GATE 3	0.11	-0.1	0.02
GATE 4	0.135	0.05	0.12
SOUTH SLUICE GATES 11/18/1998 – *ALL GATES CLOSED*			
GATE 1	0.05	-0.1	0
GATE 2	0.06	0.05	0
GATE 3	0.045	0	0
GATE 4	0.05	0	0.04
NORTH SLUICE GATES 11/18/1998 – *2 FT NOMINAL OPENING*			
GATE 1	0.08	0.05	0.12
GATE 2	1.86	1.8	1.88
GATE 3	0.12	-0.1	0.02
GATE 4	0.05	0.05	0.12
NORTH SLUICE GATES 11/19/1998 – *ALL GATES CLOSED*			
GATE 1	0.08	0.05	
GATE 2	0.12	0	
GATE 3	0.11	-0.1	
GATE 4	0.13	0.05	
NORTH SLUICE GATES 11/19/1998 – *4 FT NOMINAL OPENING*			
GATE 1	0.08	0.05	
GATE 2	4.08	4	3.94
GATE 3	0.11	-0.1	
GATE 4	0.13	0.05	
NORTH SLUICE GATES 11/19/1998 – *6 FT NOMINAL OPENING*			
GATE 1	0.08	0.05	
GATE 2	6.23	6.1	5.96
GATE 3	0.11	-0.1	
GATE 4	0.13	0.05	
NORTH SLUICE GATES 11/20/1998 – *ALL GATES CLOSED*			
GATE 1	0.08	0.05	
GATE 2	0.15	0	
GATE 3	0.11	-0.1	
GATE 4	0.13	0.05	
NORTH SLUICE GATES 11/20/1998 – *8 FT NOMINAL OPENING*			
GATE 1	0.08	0.05	
GATE 2	8.54	8.3	7.99
GATE 3	0.11	-0.1	
GATE 4	0.13	0.05	

Linear regression of the measured and indicated sluice-gate openings for gate 2 of the north sluice gates indicated that the dial indicators underreported the opening by an average of 0.13 ft. This appears to be a calibration error. Linear regression of the measured and indicated sluice-gate openings for gate 2 of the north sluice gates indicated that the control-room indicators underreported the opening by about 5 percent. The sluice-gate openings indicated in MWRDGC's control room were measured by single-turn potentiometers attached to the gate-operating mechanism. The resistance of potentiometers will drift as the potentiometer ages. In addition, the resistance changes with ambient temperature. The Technical Committee recommends that the potentiometers be replaced with digital shaft encoders, which are not prone to the drift and temperature errors common to potentiometers. The Committee also recommends that the drive mechanism for the shaft encoders be changed to provide at least one complete turn of the encoder for each foot of gate travel; more turns per foot would be acceptable.

The USGS measured flows past the sluice gates in September 1997. Preliminary analysis of these data failed to yield an acceptable backup equation. These data were re-evaluated using the methods applied to the November 1998 data and correcting the sluice-gate openings for the errors observed in November 1998. Table 4.2-d summarizes the September, 1997 data.

Table 4.2-d - Summary of September, 1997 sluice-gate and leakage measurements used to develop backup equations for the AVM at Chicago River at Columbus Avenue.

Date mm/dd/yyyy	Gate opening (ft)	Headwater elev. (CCD)	Tailwater elev. (CCD)	Head (ft)	Discharge Mean measured (ft^3/sec)	Standard deviation (ft^3/sec)	Coefficient of Variation (percent)
9/16/1997	Leakage	--	--	4.12	236	--	--
9/18/1997	2	--	--	3.85	395	--	--
9/15/1997	4	--	--	3.80	646	--	--
9/18/1997	6	--	--	3.94	899	--	--
9/18/1997	10	--	--	3.82	1,770	--	--
9/16/1997	20	--	--	3.25	2,075	--	--
9/16/1997	30	--	--	3.36	4,415	--	--
9/18/1997	40	--	--	3.20	6,427	--	--

The headwater and tailwater elevations were not provided with the September, 1997 data. Since these are necessary to estimate the discharge coefficient, they were estimated from the head difference and the mean headwater and tailwater elevations for the 1998 data. When these data were included with the November, 1998 data, the leakage coefficient C_L, became 4.13. The empirical equation to estimate the gate coefficient, C_{gs}, based on the ratio of the gate opening to the downstream depth of water became.

$$C_{gs} = 0.78\left(\frac{h_3}{h_g}\right)^{-1.33} \tag{4.2-n}$$

where h_g is the gate opening, and all other terms are defined above.

Figure 4.2-h shows the values for the discharge coefficient, C_{gs} compared to the gate opening (h_3/h_g) for both sets of measurements.

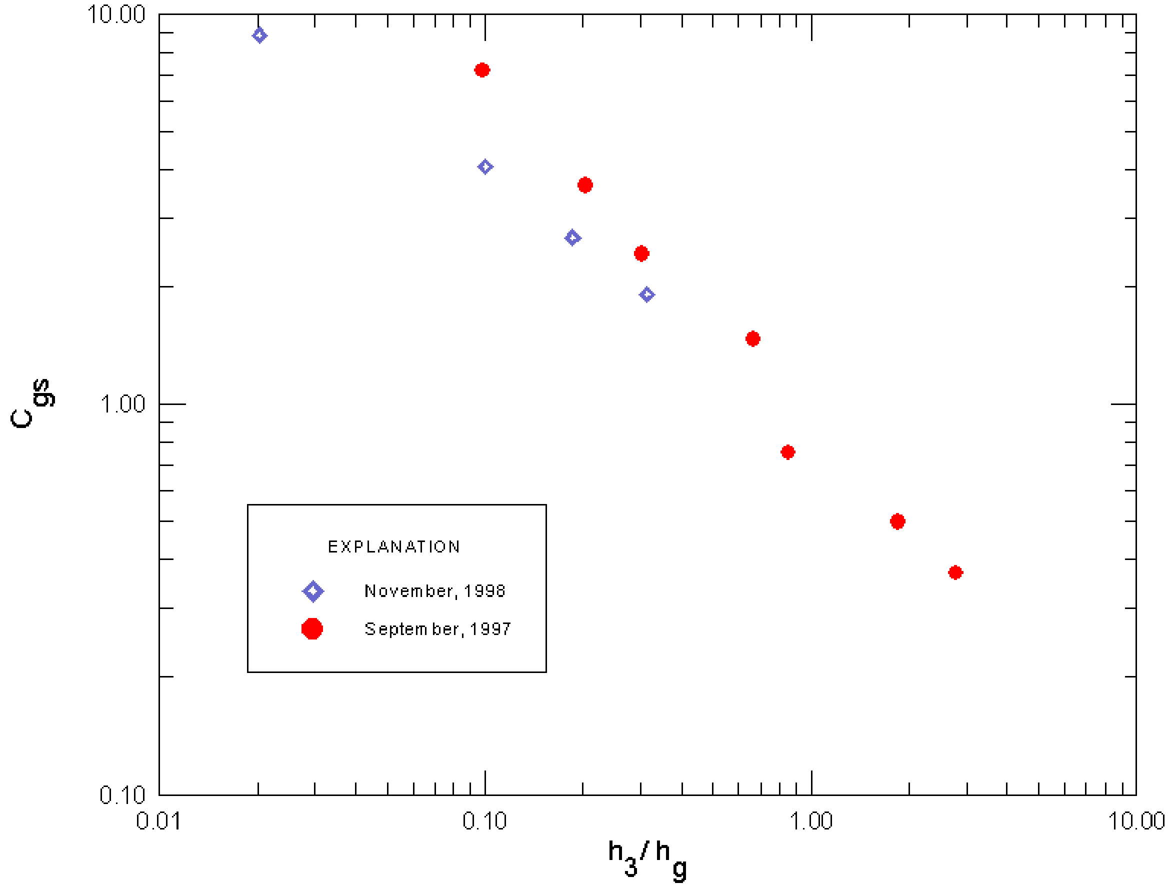

Figure 4.2-h - Discharge coefficients and gate openings for sluice-gates at the Chicago River Controlling Works.

This equation has a standard error of –32 percent to +47 percent. Based on the limited measurements available, the estimated error in the discharge from the backup equations ranged from –145 to +156 ft^3/sec for the 1-foot gate opening to –2,200 to +3,240 ft^3/sec for the 40-foot gate opening.

More detailed analysis of the error is not warranted, given the paucity of data for backup-equation development. The Committee recommends that more measurements need to be done to develop backup equations for this site. The Committee recommends that detailed records of gate opening, headwater, tailwater elevations, and lockages should be maintained to develop backup equations using the AVM record, once the AVM is operational. These records need to include which gates are open, how far each gate is open, and the times that gates are opened and closed and the times of lock-gate operation. Errors in the AVM record at low flows are likely to be large, relative to the magnitude of the flow. Therefore, the Committee recommends that ADCP measurements should be done to quantify the leakage and the sluice-gate flow for a variety of heads at smaller gate openings, rather than using the AVM record to develop the backup equations for low flows.

4.2.3. North Shore Channel at Wilmette (proposed AVM system)

4.2.3.1. Description

The 8-mile long North Shore Channel, along with a pumping station and small lock at Lake Michigan, were constructed in 1910. The pumps were used to pump Lake Michigan water into the North Shore Channel to convey wastewater to the Chicago River and the CSSC. The pumps were used until the completion of the TARP tunnel under the Chicago River. During the 1970's the lock gates were removed and replaced with a single sluice gate at the river end of the lock chamber. The sluice gate is used to allow Lake Michigan water to flow into the North Shore Channel to improve water quality. The pumps were removed and the pump bays sealed with steel plates in 1993.

The USGS measured the leakage from Lake Michigan into the North Shore channel in April and September 1993 (Oberg and Schmidt, 1994). The leakage in April, 1993 was 59 f^{t3}/sec ± 8 ft^3/sec. The leakage in September, 1993 was estimated to be less than 15 ft^3/sec. The USGS made a series of discharge measurements in December 9, 1997 to verify the sluice-gate rating for this site (K. Oberg, USGS, oral commun., September 18, 1998). Results from these measurements indicate that the sluice gate may not always be operating as indicated. Figure 4.2-i shows the measured flows for different gate openings, along with the measurement sequence. The first measurement with the gate open 0.5 ft showed no difference in the discharge from the measurement with the gate fully closed. The gate was then opened 1.5 ft and discharge measured for gate openings of 1.5, 1.0, and 0.5 ft. The discharge measured at a 0.5 ft gate opening following the larger gate openings is consistent with the discharges measured for the other gate openings. It appears that debris may have lodged near the gate sill when the gate was closed and caused the effective gate opening to be smaller than indicated until the gate was opened enough to flush the opening clean. This is supported by circumstantial observations from MWRDGS's water-quality monitoring in the North Shore Channel. Dissolved-oxygen monitors in the North Shore Channel at times do not indicate the improvements expected from addition of Lake Michigan water for small gate openings (K. Oberg, USGS, oral commun., September 18, 1998, referring to conversations with Mr. Irwin Polls, MWRDGC about this site).

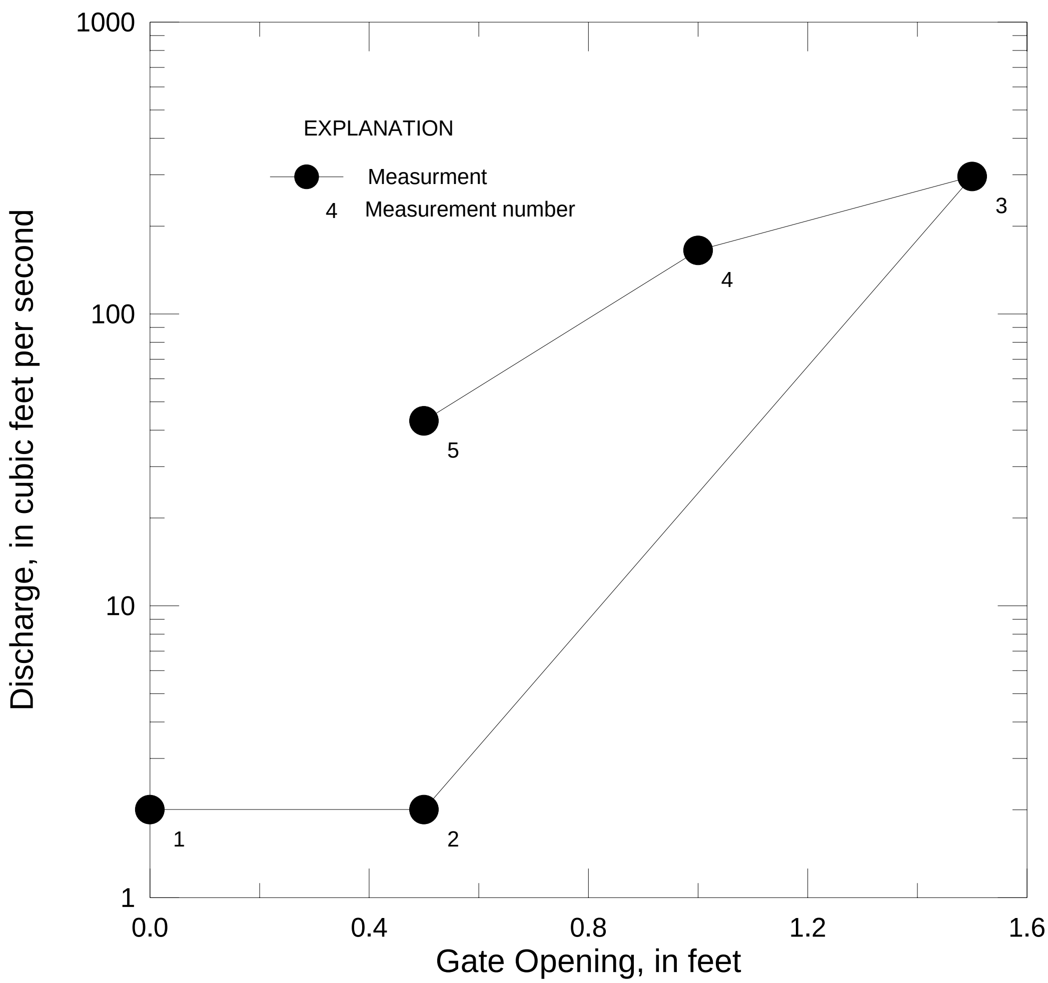

Figure 4.2-i - Discharges measured for different sluice-gate openings at the North Shore Channel at Wilmette, December 9, 1997.

4.2.3.2. Measurement errors

The USGS installed an AFFRA® AVM with a single acoustic path at this location in August, 1999. The AVM is located about ¼ mile downstream from the sluice gate. The index-velocity rating for this station will be developed based on ADCP measurements of the discharge for a variety of flow conditions. No data are presently available to evaluate the potential accuracy of the record from this station. The flows in the North Shore Channel at Wilmette are expected to typically result in small (less than 0.1 ft/sec) velocities and the flows may be affected by backwater from the discharge from the North Side wastewater treatment plant. The accuracy of the record is therefore expected to be similar to that from the AVM at O'Brien Locks and Dam. The Technical Committee recommends that a detailed evaluation of the accuracy of the flows measured at this site should be done and reviewed by the Fifth Technical Committee.

4.2.3.3. Backup system

The backup system is a means to estimate flow at the North Shore Channel at Wilmette for periods when the AVM is inoperative or is not functioning properly. The backup system for the AVM is a set of regression equations that relate the flow at the AVM to the head across the sluice-gate opening. The estimation equations described in this report are based on measurements of flow for three gate openings as well as two measurements of leakage. These equations were developed to present a possible format for the backup equations and to give a first approximation of the error to be expected from the backup equations. These equations were based on very limited data and were not intended for operational use. Much more data should be obtained to develop a set of backup equations for operational use.

The backup equations were developed by first developing an equation to estimate the leakage past the structure based on the headwater and tailwater elevations. Data provided to the Technical Committee included two sets of ADCP measurements for leakage conditions done in April and September 1993, and another measurement done in December 9, 1999. The April, 1993 measurement was excluded from the analysis because the pumps were removed and the pump bays sealed in July, 1993, and data indicated that this reduced the leakage significantly. The other two measurements were used to estimate an equation to estimate leakage based on the head across the structure. After the equation to estimate leakage was developed, it was used to estimate the leakage for the sluice-gate measurements. Four sets of ADCP measurements were provided to the Committee for sluice-gate openings of 0.5 (two measurements), 1, and 1.5 ft. The first measurement for the 0.5 ft gate opening was excluded from the analysis because of the problems described in the preceding paragraphs. Headwater and tailwater elevations were included with the 1993 data; the 1997 data included only the head across the structure. The headwater and tailwater elevations were estimated based on the September, 1993 elevations and the reported head.

Leakage was assumed to be through a vertical, rectangular orifice with a constant, but unknown, width extending the entire height of the structure. Based on the two sets of measurements the equation for leakage at the site is:

$$Q_L = C_L h_1 \sqrt{\Delta h} \qquad \text{(4.2-o)}$$

where Q_L is the leakage discharge; C_L is the leakage coefficient 0.13; h_1 is the upstream depth of water above the gate sill; and Δh is the head difference across the structure.

The flow through the sluice gates is expected to be described by the theoretical equation for flow through a submerged orifice.

The following empirical equation was developed to estimate the gate coefficient, C_{gs}, based on the ratio of the gate opening to the downstream depth of water.

$$C_{gs} = 3.3\left(\frac{h_3}{h_g}\right)^{-1.9} \qquad \text{(4.2-p)}$$

where all the terms are defined above.

This equation has a standard error of –12 percent to +13 percent. More detailed analysis of the error is not warranted, given the paucity of data for backup-equation development. The Committee recommends that more measurements need to be done to develop backup equations for this site. The Committee recommends that detailed records of gate opening, headwater, and tailwater elevations should be maintained to develop backup equations using the AVM record, once the AVM is operational. These records need to include the gate opening, and the times that the gate is opened and closed. Errors in the AVM record at low flows are likely to be large, relative to the magnitude of the flow. Therefore, the Committee recommends that ADCP measurements should be done to quantify the leakage and the sluice-gate flow for a variety of heads at smaller gate openings, rather than using the AVM record to develop the backup equations for low flows.

4.3. Water-Supply Pumpage

Water-supply pumpage accounts for about 80 percent of the Lake Michigan diversion to be measured under the proposed lakefront accounting. Based on the data provided to the committee (Wahlin and Replogle, 1998a, 1998b, 1998c; USACE, 1996b) and tours of the Evanston, Mayfair, Thomas Jefferson, and Jardine pumping stations, the coefficient of variation for the water-supply measurements is expected to range from 1.5 to 5 percent.

The USACE has initiated a series of quality-assurance reviews of the pumpage data for the major water-treatment (Evanston and Jardine) and pumping stations (Mayfair and Thomas Jefferson). The initial review was a review of 18 water-treatment facilities or pumping stations. This review gave a general overview of the metering, data-acquisition, and quality-assurance practices at each of these facilities. Table 4.3-a lists these 18 facilities and their approximate flows for water year 1993. The USACE is planning to visit and do a pre-evaluation of the minor stations.

Table 4.3-a - Summary of approximate 1993 flows from eighteen pumping stations or water-treatment facilities reviewed by the USACE (1998b).

[ft^3/sec, cubic feet per second; MGD, million gallons per day]

	Approximate 1993 Flow	
Pumping Station	(ft^3/sec)	Flow (MGD)
68th Street	108.0	69.7
Central Park Ave	178.0	114.8
Cermak	64.0	41.3
Chicago Ave	77.0	49.7
CLCJAWA	25.4	16.4
Evanston	74.3	47.9
Highland Park	16.9	10.9
Lakeview	78.0	50.3
Lexington Ave	109.0	70.3
Mayfair	195.0	125.8
Northbrook	9.2	5.9
Roseland	230.0	148.4
Southwest	154.0	99.4
Springfield Ave	172.0	111.0
Thomas Jefferson	89.0	57.4
Waukegan	13.5	8.7
Western Ave	141.0	91.0
Wilmette	20.8	13.4
Total flow	1755.1	1132.3

The initial review was followed by more detailed reviews of three of the facilities. These reviews include a description of the meters used, the calibration that is done for these meters, the backup systems for the meters and data, and a detailed error analysis for several short periods (3 to 17 weeks total) of data. These reviews were done by the USGS and the U.S. Department of Agriculture Water Conservation Laboratory. These three reviews were done to provide a protocol and format for subsequent reviews to be done at the rest of the water-treatment facilities and pumping stations.

4.3.1. Review of Quality-Assurance Reports on Pumping Stations

The USACE has initiated a series of quality-assurance reviews of the pumpage data for the major water-treatment and pumping stations. These reviews include a description of the meters used, the calibration that is done for these meters, the backup systems for the meters and data, and a detailed error analysis for several short periods (3 to 17 weeks total) of data. Overall, these reviews provide a good description of the accuracy of the pumpage measurements from these stations. Some of the assumptions in these reviews merit additional investigation. These include: (1) the assumption that the effect of corrosion and tuberculation (deposition) is negligible for the venturis; (2) the assumption that different types of venturis behave similarly, especially with regard to approach-length requirements; (3) the assumption that different upstream fittings can be approximated as a single 90° elbow; (4) the assumption that only the nearest upstream fitting affects the meter accuracy; and (5) assumptions about the direction of different biases in the calculations.

Two of the reviews (Evanston Water Works, Wahlin and Replogle, 1998a; and Mayfair pumping station, Wahlin and Replogle, 1998b) did not provide for degradation of the accuracy of the flow meters over time. The third review (Thomas Jefferson pumping station, Wahlin and Replogle, 1998c) listed an accuracy for 'a worn BID Dall flow tube,' without providing a reference. Since other studies indicate significant degradation in venturi accuracy over time (Phair, 1997), a thorough literature review about the long-term accuracy of venturi meters should be considered a minimal effort to address this. It may be necessary to design a random sampling of meters to estimate the effect of long-term accuracy of the venturi meters. It also would be informative to visually inspect, clean, and measure some of the meters, possibly combining this with 'before-and-after' flow measurements. Mr. Richard Figarelli of the Evanston Water Works indicated that they "never find wear in their finished venturis but do find corrosion of the pressure taps," (Richard Figarelli, Evanston Water Works, oral commun., October 27, 1998). Venturis at the Evanston Water Works have bronze throat liner, which may be why they do not show wear. Venturis at the Thomas Jefferson pumping station also have bronze throat liners, but this is not specified for the Mayfair pumping station. Anecdotal information from pumping station employees indicated that corrosion has never been a problem (B. Wahlin, USDA-ARS, Water Conservation Laboratory, written commun., October 11, 2000). The effect of a throat liner on venturi accuracy should be considered. The Committee recommends that the design of the quality-assurance reviews of the major water-treatment and pumping stations should include measurement of the effect of corrosion and tuberculation for representative meters.

Documentation about the behavior of some of the meters (the Simplex meters at the Thomas Jefferson pumping station and the Pratt meters at the Mayfair pumping station) appears to be missing or incomplete, and the meters were assumed to behave similarly to other meters that are described in the literature. Comparison of approach-length requirements for Dall flow tubes with those for Herschel venturis and Lo-loss flow tubes indicates a marked difference in the accuracy of these meters downstream from different fittings. Similarly, the Dall flow tubes have some sharp edges that result in a change in the discharge coefficient as the edges are rounded. It is not plain whether other meters have similar features. More background on the specific meters used at each facility should be included as part of these reviews. If the information on specific meters is unavailable an alternative would be to make a series of measurements with a calibrated meter to develop ratings for a representative sample of the undocumented meters.

Two of the reviews (Mayfair pumping station, Wahlin and Replogle, 1998b; and Thomas Jefferson pumping station, Wahlin and Replogle, 1998c) listed meters a short (compared to the recommended approach lengths) distance downstream from 23° elbows, 45° elbows, gate valves, and tees. These were all assumed to affect the measurement the same as a 90° elbow, and an error of ±0.5 percent was added for fittings within the approach length of the meter. Miner (1956) indicated that the error is a function of the distance from the fitting to the meter, and that the error and it's change with distance were different for different fittings. Miller (1989) and Miner (1956) also indicate that the direction of the bias error resulting from upstream fittings is known; this also should be accounted for in the analysis. Further literature review should be done to better document the effect of different upstream fittings on the meter accuracy.

Although expanders and reducers were not listed, the variety of meter sizes at the Mayfair pump station indicated that these also may be present. Miller (1989) and Miner (1956) indicate that expanders and reducers upstream from the meter affect the discharge coefficient, with reducers requiring nearly three times greater approach length than elbows for a Herschel venturi. The number and plane of elbows also should be reported. A single elbow or multiple elbows in the same plane distort the velocity profile but do not impart swirl to the flow; this effect is eliminated in a relatively short distance. In contrast, two elbows in different planes result in swirl, which can take significantly longer to dissipate to acceptable levels (over 150 diameters for Reynolds numbers in the range measured at these pumps, Miller, 1989, p. 5-38). Linford (1961, p. 123) reports that, errors exceeding 100 percent may result from using normal parallel approach lengths in situations where vortex flow occurs. Thus it is important that these reviews describe not only the distance to the nearest fitting, but also whether other fittings further upstream may affect the flow.

All of the systematic errors (biases) listed in the three reviews were presented as directionless. Miller (1989) and Miner (1956) indicate that the errors from upstream fittings, and wear of the venturis result in changes to the discharge coefficient of a know direction. Calculating the combined effect of such changes using the RMS method will result in the magnitude of the error being underestimated. Similarly, errors in the instruments used to calibrate the pressure transducers will result in a bias in each of the flow meters, and this bias will be the same direction for each flow meter. Thus, the RMS method should not be used to sum these effects for multiple meters, as the magnitude of the error will be underestimated.

4.3.1.1. Evanston

Wahlin and Replogle (1998a) reviewed the flow measurement and recording for the City of Evanston's water works. This plant supplies water to the Cities of Evanston and Skokie, and to the Northwest Water Commission, which serves Arlington Heights, Buffalo Grove, Palatine, and Wheeling. This plant has two Badger Lo-loss flow tubes (one 48-inch and one 60-inch) to measure the raw water (and returned filter backwash water). This plant has four Lo-loss flow tubes (two 36-inch, one 48-inch, and one 60-inch) and a 16-inch turbo meter to measure the finished water distributed from the plant. Backwash water is returned to the suction well for the raw-water pumps upstream from the raw-water meters. This flow is measured using three BIF Dall flow tubes (one 6-inch, one 20-inch, and one 30-inch). Pressure differentials from these meters are measured by Bristol-Babcock pressure transducers at six-second intervals and transmitted to a remote terminal unit (RTU). The RTU determines the hourly average flows and sums the 24 hourly flows to determine the daily volume. A backup system uses a discriminator, summator, and totalizer to monitor the total flow past each meter.

Meter installation errors are a common source of error in venturi-type meters. An appropriate length of straight pipe is required between the meter and any upstream bends or fittings. The length required depends on the specific type of meter and the number and type of fittings. The Dall flow tube is more sensitive to upstream fittings than the more common Herschel venturi tube. Wahlin and Replogle (1998a) indicate that all venturi-type meters at the Evanston Water Works have an appropriate length of straight pipe upstream from the meter. They indicate that there is both a tee and an elbow upstream from the 48-inch Lo-Loss tube, but do not indicate the planes of these fittings. Given the effect that swirl can have on the accuracy of flow tubes, the orientation of these fittings should be considered to determine whether this may affect meter accuracy.

Whalin and Replogle (1998a) identify several potential sources of random and systematic error in the flow calculations for the Evanston Water Works. One source of error is the error in the Amtek Mod-Cal multi-meter/transmitter module used to calibrate the Bristol Babcock pressure transducers. This is correctly reported as a bias error. Since the direction of this error is unknown, it is correct to use the RMS method to add it's affect to the other sources of error when determining the error for an individual meter. However, this error will have the same direction for each meter. Thus using the RMS method to sum the errors from multiple meters will underestimate the error, as it does not account for bias errors from the Amtek module being the same direction for all meters. This error is a very small percentage of the total flow (0.15 percent for the example given by Whalin and Replogle, 1998a) and any bias from using the RMS method should be small; however, this should be considered when summing flows from multiple meters. The example given by Whalin and Replogle (1998a) is for a single venturi, so the effect of this will not affect the calculated uncertainties.

The errors in the flows through the backwash meters are significantly larger than those from the raw-water meters. The backwash water enters the suction well for the raw-water pumps upstream from the raw-water meters. To avoid double-counting the backwash water, the flow from the backwash meters is subtracted from the raw-water meters. Whalin and Replogle (1998a) found that the errors in the flows adjusted for backwash are nearly identical to errors for the raw water, supporting the assumption that the backwash measurement errors have almost no effect on the overall uncertainty of the raw-water measurements.

The Evanston Water Works has a thorough quality-assurance plan for their water accounting. This includes daily comparisons among meters (raw versus finished water) comparison between volume and pumpage, and monthly calibration of secondary measurement devices (pressure transducers). They have capacity to route water through other measurement devices in the event of a failure, providing a backup for the measurement system in the event of any failure. The combination of raw- and finished-water meters provides a backup that is not available at the other pumping stations reviewed. The review by Wahlin and Replogle focused on the raw-water metering, which is Evanston's primary measurement. This is not consistent, however, with the measurement of finished water, which will be done for most of the lakefront diversion accounting. The finished-water volumes typically are one – two percent lower than the raw-water volumes. If finished water is to be used for the lakefront accounting, the analysis done by Whalin and Replogle (1998a) should be repeated for the finished-water meters.

4.3.1.2. Mayfair Pumping Station

Wahlin and Replogle (1998b) reviewed the flow measurement and recording for the City of Chicago's Mayfair pumping station. This station has six venturi-type flow meters--four Pratt venturi nozzles that were installed in 1992 and two BIF Dall flow tubes that were installed in 1954. Water from any of the station's seven pumps can be routed through any of the venturi-type meters. Pressure differentials from these meters are measured by Johnson-Yokogagwa pressure transducers and recorded at five-minute intervals by the Supervisory Control and Data Acquisition (SCADA) system and instantaneously by chart recorders. The SCADA system determines the daily average volume through any venturi. The chart recorders provide a backup measure of the daily flow through each venturi. Specifications for the Pratt flow tubes were not available, so specifications for the Dall flow tubes were used for all six meters. Wahlin and Replogle (1998b) provide a comprehensive review of the accuracy of flow metering at this station; however, some revisions to their analysis are warranted.

There are no wet calibrations done for this pumping station. The venturi meters are assumed to be operating within the manufacturer's specifications. This assumption is reasonable for the Pratt flow tubes, which have only been in service since 1992. The Dall flow tubes have been in service since 1954 and may require more detailed inspection. Phair (1997) reported that of over 200 flowmeters inspected, less than 20 percent had errors smaller than 2 percent. Of those with errors greater than 2 percent, approximately 27 percent were the result of tuberculation[3]. Tuberculation in the venturi inlet cone will increase the apparent flow, while tuberculation in the upstream piping will decrease the apparent flow. The construction of the venturis at the Mayfair pumping station is not specified; venturis at other stations (Wahlin and Replogle 1998a, 1998c) have bronze throat liners which are less subject to corrosion and tuberculation than steel. This would indicate that tuberculation is more likely upstream from the venturi, which would result in a bias toward underreporting the flow. Phair (1997) recommends that the inlet and throat of the venturi be cleaned, inspected, and accurately measured to prove the flows in venturi meters.

Wahlin and Replogle (1998b) also discuss the effect of rounding of the sharp edges of the groove for the pressure taps. The one-percent error reported by Miner (1956) from rounding of the upstream edge was to decrease the discharge coefficient. Because the direction of the bias error this introduces is know, the

[3] Tubercles are small growths or deposits on the pipe or venturi. Typically, these start with iron oxide or hydroxide, which forms a nuclei that materials such as calcium carbonate deposit on.

RMS method to determine the uncertainty is not correct. The RMS method is correct when summing uncertainties whose direction is unknown. The uncertainty should be the ±0.75 percent accuracy of the venturi minus the uncertainty from rounding of the sharp-edged groove, or an uncertainty of –1.75 percent to –0.25 percent. In addition to not accounting for the known direction of the error from rounding of the sharp edges, this error was not included in the calculations listed in table 6 of Wahlin and Replogle (1998b).

Meter installation errors are a common source of error in venturi-type meters. An appropriate length of straight pipe is required between the meter and any upstream bends or fittings. The length required depends on the specific type of meter and the number and type of fittings. The Dall flow tube is more sensitive than the more common Herschel venturi tube. Wahlin and Replogle (1998b) appear to reference information from ANSI standard 2530 and the American Society of Mechanical Engineers (ASME, 1971) *Fluid Meters*, as cited by Miller (1989). These standards have since been updated (ISO Standard 5167, 1980 and ASME MFC-3m(1985)). In addition, the required upstream lengths used are for standard Herschel venturi tubes, which require shorter lengths than the Dall flow tubes. Miner (1956) presented the error in the discharge coefficients for Dall flow tubes with three different β-ratios downstream from selected fittings. For the two Dall flow tubes, which are located 2.5 diameters downstream from fittings, Miner (1956, Fig. 7) shows that the discharge coefficient will be reduced by 2 percent. For the four Pratt flow tubes the discharge coefficient will be reduced by 0.06 percent to 1.2 percent (Table 4.3-b). Whalin and Replogle assume that a 45° elbow and a tee behave similarly to a 90° elbow. Information describing the change in discharge coefficient from a 45° elbow were not available; however, Miller (1989) shows that for a Low-loss flow tube preceded by a tee, approximately 50 percent more straight pipe is required upstream, or for the lengths of straight pipe similar to those at the Mayfair pump station, the discharge coefficient is about 1 percent smaller with an upstream tee rather than an upstream elbow. Whalin and Replogle do not indicate what fittings are upstream from each venturi, so the assumption of a 90° elbow being similar was used in subsequent analyses.

Miller (1989) and Miner (1956) indicate that expanders and reducers upstream from the meter affect the discharge coefficient, with reducers requiring nearly three times greater approach length than elbows for a Herschel venturi. Given the variety of venturi sizes and the capability to run any pump through any venturi, the system must contain some expanders and/or reducers. It should be stated whether these are sufficient distance from the meters to avoid affecting the discharge coefficient.

Table 4.3-b - Approach lengths and associated errors for venturi meters at the Mayfair pumping station.

Venturi Number	Type	Pipe diameter (in)	Throat diameter (in)	ß ratio	Approach length (ft)	Approach length (pipe diameters)	Change in discharge coefficient (percent)
1	Pratt	54	32.9	0.6092	34	7.6	-0.68%
2	Pratt	42	25.48	0.6059	64	18.3	-0.06%
3	Pratt	42	25.4	0.6059	21	6.0	-1.21%
4	Pratt	54	32.9	0.6092	28	6.2	-1.22%
5	Dall	48	27.4	0.5715	10	2.5	-1.96%
6	Dall	48	27.4	0.5715	10	2.5	-1.96%

The bias errors associated with the distance from upstream fittings is a reduction in the discharge coefficient, which results in the meter underreporting the flow. Since the direction of this error is known, it should be added directly to the other errors, rather than using the RMS method.

Miner (1956) reports that the discharge coefficient for Dall flow tubes becomes variable below a Raynolds number considerably higher than that at which a conventional venturi tube coefficient starts to vary (R_d>350,000 for Dall flow tubes, compared to R_d>4,000 for venturi tubes). Examination of the data listed by Whalin and Replogle for the Mayfair pumping station indicate that the Reynolds number is above 350,000 for flows greater than 12 MGD for all the flow tubes and that flows should always be well above this flow rate for this pumping station.

Whalin and Replogle (1998b) identify several potential sources of random and systematic error in the flow calculations for the Mayfair pumping station. One source of error is the error in the Ris DPG-700 pressure gage used to calibrate the Johnson-Yokogagwa pressure transducers. This is correctly reported as a bias error. Since the direction of this error is unknown, it is correct to use the RMS method to add it's affect to the other sources of error when determining the error for an individual meter. However, this error will have the same direction for each meter. Thus using the RMS method to sum the errors from multiple meters will underestimate the error, as it does not account for bias errors from the Ris DPG-700 pressure gage being the same direction for all meters. Using the data from Whalin and Replogle (1998b) for the first hourly reading on February 1, 1995 and correcting for only the bias from the Ris DPG-700 pressure gage changes the error from 1.06 MGD (0.92 percent) to 1.29 MGD (1.12 percent). This error is ±0.91 MGD for the systematic error for the three meters and ± 0.92 MGD systematic error for the Ris DPG-700 pressure gage; these are summed using the RMS method to determine the bias error of ±1.29 MGD for first hour's measurement. When the same analysis is done including the random error for each meter, the error is 2.71 MGD (2.4 percent) rather than the 2.61 MGD (2.27 percent) reported by Whalin and Replogle (1998b).

The error analysis for the first hour of data from February 1, 1995 was repeated using the errors and direction of errors indicated by Miner (1956). This indicated that the errors in the first hours data were –2.82 ± 2.56 MGD (-4.7 to –0.2 percent). Extrapolating these same errors to all the hourly data given in table 9 of Wahlin and Replogle (1998b) gives errors ranging from –4.22 to –0.68 percent to –4.93 to +0.03 percent. Applying the same errors to the daily average for February 1, 1995, gives an approximate error in the daily-average flow of –4.20 to –1.92 MGD (-3.4 to –1.5 percent). Applying these errors to the entire year would give an error in the annual volume of –3.3 to –1.6 percent.

The chart recorders at the Mayfair pumping station provide an independent check on the volume calculations, as well as a backup in the event of a failure of the SCADA system. Since these recorders use the same venturis and pressure transducers as the SCADA system, these do not provide an independent check on the flow measurements.

4.3.1.3. Thomas Jefferson Pumping Station

Wahlin and Replogle (1998c) reviewed the flow measurement and recording for the City of Chicago's Thomas Jefferson pumping station. This station has four pumps each of which pumps through a Simplex venturi-type flow meter. These venturi meters were installed in 1927. Pressure differentials from these meters are measured by Gould pressure transducers and recorded at five-minute intervals by the SCADA system. The SCADA system determines the daily average volume through each venturi. There are no chart recorders or other backup measure of the flows at this pump station. Based on information provided in Wahlin and Replogle (1998c, p. 1), the Simplex venturi tubes sound as if they are similar to the Dall flow tubes. The description of these tubes ("a variation of the classical venturi … designed to produce higher pressure differentials … designed to have a lower permanent pressure loss, a lower weight, and a shorter overall length") is the same as that for a family of "modified venturi tubes" that includes the Dall Flow tubes. However, these "modified venturi tubes" were introduced in the 1950's (Halmi, 1974a, 1974b; Miner, 1956). Since the Simplex venturis have been in service since 1927, it is not apparent whether the accuracy characteristics of classical Herschel venturis or Dall flow tubes is more appropriate for these meters.

There are no wet calibrations done for this pumping station. The venturi meters are assumed to be operating within the manufacturer's specifications. The Simplex flow tubes have been in service since 1927 and may require more detailed inspection. Phair (1997) reported that of over 200 flowmeters inspected, less than 20 percent had errors smaller than 2 percent. Of those with errors greater than 2 percent, approximately 27 percent were the result of tuberculation. Tuberculation in the venturi inlet cone will increase the apparent flow, while tuberculation in the upstream piping will decrease the apparent flow. The manufacturer specifications warn that tuberculation may affect the accuracy of these venturis, and Wahlin and Replogle (1998c) use the accuracy of a 'worn Dall flow tube' (no reference provided) of ±1.25 percent as an estimate of the accuracy of these venturis. Phair (1997) observed errors up to 30 percent, but did not indicate whether these were from tuberculation. Phair (1997) recommends that the inlet and throat of the venturi be cleaned, inspected, and accurately measured to prove the flows in venturi meters.

Wahlin and Replogle (1998c) do not indicate whether the Simplex flow tubes have the sharp edges of the groove for the pressure taps that the Dall flow tubes have. If these are not present, the potential bias from rounding of these edges will not be a factor. The report should indicate whether such edges are a part of these flow tubes.

Meter installation errors are a common source of error in venturi-type meters. An appropriate length of straight pipe is required between the meter and any upstream bends or fittings. The length required depends on the specific type of meter and the number and type of fittings. The Dall flow tube is more sensitive than the more common Herschel venturi tube. Wahlin and Replogle (1998b) appear to reference information from ANSI standard 2530 and the American Society of Mechanical Engineers (ASME, 1971) *Fluid Meters*, as cited by Miller (1989). These standards have since been updated (ISO Standard 5167, 1980 and ASME MFC-3m(1985)). In addition, the required upstream lengths used are for standard Herschel venturi tubes, which require shorter lengths than the Dall flow tubes. Miner (1956) presented the error in the discharge coefficients for Dall flow tubes with three different β-ratios downstream from selected fittings. If the Simplex flow tubes behave like Dall flow tubes, 14 pipe diameters, as listed in Wahlin and Replogle (1998c), will reduce the approach length error below 0.5 percent. If the Simplex tubes behave like Herschel venturi tubes or like Lo-loss flow tubes, three pipe diameters is all that is needed to reduce the errors below 0.5 percent. Table 4.3-c shows the errors in the discharge coefficient for these meters. Whalin and Replogle assume that a 23° elbow and a gate valve behave similarly to a 90° elbow. Information describing the change in discharge coefficient from a 23° elbow were not available; however, Miller (1989) shows that for a Herschel venturi preceded by a fully-open gate valve, approximately a 4.5 pipe diameters of straight pipe are required upstream. Miner (1956) shows that for a full-open gate valve about 3.7 to 5.2 diameters upstream, the increase in the discharge coefficient in around 0.5 percent. Whalin and Replogle do not indicate what fittings are upstream from each venturi, so the assumption of a 90° elbow being similar was used in subsequent analyses, which should provide a conservative estimate of the error. Since the report does not document whether the accuracy of the Simplex tubes downstream from fittings is more like Herschel venturis or Dall flow tubes, Table 4.3-c shows the change in discharge coefficients for both types of meters.

Table 4.3-c - Approach lengths and associated errors for venturi meters at the Thomas Jefferson pumping station.

Venturi Number	Type	Pipe diameter (in)	Throat diameter (in)	ß ratio	Approach length (ft)	Approach length (pipe diameters)	Change in discharge coef. (Dall) (percent)	Change in discharge coef. (Herschel) (percent)
1	Simplex	36	25	0.6944	11	3.7	-4.06%	<0.5%
2	Simplex	36	25	0.6944	14	4.7	-3.26%	<0.5%
3	Simplex	36	25	0.6944	14	4.7	-3.26%	<0.5%
6	Simplex	36	25	0.6944	11	3.7	-4.06%	<0.5%

The bias errors associated with the distance from upstream elbows is a reduction in the discharge coefficient, which results in the meter underreporting the flow. The bias errors associated with the distance from upstream gate valves is an increase in the discharge coefficient, which results in the meter overreporting the flow Since the direction of this error is known, it should be added directly to the other errors, rather than using the RMS method.

Miner (1956) reports that the discharge coefficient for Dall flow tubes becomes variable below a Raynolds number considerably higher than that at which a conventional venturi tube coefficient starts to vary (R_d>350,000 for Dall flow tubes, compared to R_d>4,000 for venturi tubes). Examination of the data listed by Whalin and Replogle for the Thomas Jefferson pumping station indicate that the Reynolds number is above 350,000 for flows greater than 9 MGD for all the flow tubes and that flows should always be well above this flow rate for this pumping station.

Whalin and Replogle (1998c) identify several potential sources of random and systematic error in the flow calculations for the Mayfair pumping station. One source of error is the error in the RiS DPG-700 pressure gage used to calibrate the Gould pressure transducers. This is correctly reported as a bias error. Since the direction of this error is unknown, it is correct to use the RMS method to add it's affect to the other sources of error when determining the error for an individual meter. However, this error will have the same direction for each meter. Thus using the RMS method to sum the errors from multiple meters will underestimate the error, as it does not account for bias errors from the RiS DPG-700 pressure gage being the same direction for all meters. Hourly data presented by Whalin and Replogle (1998c) are for a day when only venturi 4 was used; thus the effect of the bias from using the RiS DPG-700 to calibrate pressure gages for all four venturis cannot be determined.

An error analysis for the first hour of data from February 1, 1995 was done using both the errors and direction of errors indicated by Miner (1956) for Dall flow tubes and those indicated by Miller (1989) for Herschel venturis. This indicated that the errors in the first hours data were –2.51 ± 1.23 MGD (-6.0 to –2.1 percent) if the Simplex meters behave like Dall flow tubes, and –0.31 ± 1.23 MGD (-2.5 to –1.5 percent) if the Simplex meters behave like Herschel venturi tubes. Extrapolating these same errors to all the hourly data given in table 4 of Wahlin and Replogle (1998c) gives an approximate error in the daily-average flow of –3.31 to –1.67 MGD (-5.3 to –2.7 percent) if the Simplex meters behave like Dall flow tubes, and –1.13 to +0.51 MGD (-1.8 to +0.8 percent) if the Simplex meters behave like Herschel venturi tubes.. Applying these errors to the entire year would give a standard error in the annual volume of –5.0 to –2.4 percent if the Simplex meters behave like Dall flow tubes, and –1.8 to +0.8 percent if the Simplex meters behave like Herschel venturi tubes. It should be noted that these errors do not take account of the possible effects of tuberculation, which could significantly increase the errors in the reported flows for the Thomas Jefferson pumping station.

Since there are no chart recorders at the Thomas Jefferson pumping station, there is no independent check on the volume calculations nor is there a backup in the event of a failure of the SCADA system. The SCADA system runs on two parallel computers, with an additional computer at the pumping station maintaining three days of backup data. This should provide adequate backup to prevent loss of data. There is no independent check on the flow measurements at this pump station.

5. FOURTH TECHNICAL COMMITTEE'S RECOMMENDATIONS AND FINDINGS

1. To the committee's knowledge, the draft quality assurance plan (October 1988) has not been updated as recommended by the Third Technical Committee. The draft quality-assurance plan (October 1988) should be updated and finalized based on the present status of Lake Michigan diversion computational procedures and measurements (1999 conditions). Basic elements of the plan are as follows:

   - Develop documentation of measurements;
   - Develop documentation of methods of data collection;
   - Develop "Standard Operating Procedure" for calibration and verification of measurement components;
   - The QA plan should include the QA plans that have been established by the other agencies that collect data such as the USGS and the ISWS.
   - Provide a schedule to perform field evaluation of measurement sites;
   - Establish methods of verifying the accuracy of data used in diversion computation;
   - Maintain permanent records of measurements, verification tests, and other sources of diversion data;
   - Define criteria for the water budgets to flag water budgets that produce unacceptable results and therefore require reevaluation.
   - Compare budgets with previous years' budgets to identify long term bias.
   - Identify and describe any statistical tools, such as double-mass curves, that will be used to identify changes in the relations among components of diversion accounting.

2. The Lake Michigan Diversion Accounting Draft Manual of Procedures (USACE, 1998a) contains numerous QA provisions that apply to various components of diversion accounting. These QA provisions need to be an integral part of a comprehensive quality assurance plan. Check lists should be compiled that formalize the warnings that are described in the Lake Michigan Diversion Accounting Draft Manual of Procedures. For example, in the Manual of Procedures chapter for Hydrologic Simulation (HSPF) page 2, the resetting of the state variables. Other checks should include automated mass balance totals; for example comparing input rainfall to the total yearly rainfall from ISWS.

3. The Committee commends the USACE for the improved timeliness of the Water Year accounting reports since the third Technical Committee. The timeliness of publication of water-year diversion accounting reports should be maintained.

4. Before implementing lakefront accounting, a manual of procedures for lakefront accounting should be written. The manual should include a quality-assurance plan that includes automated checks of the input data. If possible, the manual should include a checklist that mandates checks at steps where potential errors can occur in the analysis.

5. At major steps in the diversion accounting process, a supervisory review should take place and documentation of the review should be maintained. The committee

emphasizes that the review and documentation are essential parts of the QA procedure.

6. The Lake Michigan accounting procedures should be modified to begin with an initial set of template files rather than begin with the previous year's files, which are copied and modified to represent the current year's data. The template files would have all data files initialized with the missing data value of –901. By initializing the data files with –901, data values that were erroneously not entered would be easily found because the model would not execute.
7. Values in tables 4 and 7 in the accounting reports for water years 1991 and 1992 contained errors which do not effect diversion. The committee recommends more detailed review of the accounting reports which could be attained through the supervisory review that was described in item 5.
8. The average annual deduction for Indiana water-supply pumpage that was discharged to the CSSC increased 34.7 percent corresponding to a change in the calculation procedures for this value. The impact of this change on the historical diversion record should be reviewed.
9. Results from statistical analyses of the six years of record considered in this review indicate that Budgets 9, 10, 11, and 13 may contain significant long-term biases. Biases in Budgets 9 and 10 may affect both the current accounting system and estimated runoff and consumptive-use values for lakefront accounting. The effect of these biases on historic accounting computations should be reviewed. Biases in budgets 11 and 13 will not affect current accounting, but will affect estimated runoff and consumptive-use values for lakefront accounting.
10. The total width measured by 59 ADCP Romeoville measurements from July 22, 1993 to October 24, 1995, averaged 2.1 percent less than the surveyed width of the cross section. It is recommended that the USGS investigate further to determine: (a) whether this error persists in more recent data; (b) whether this is an error in distance measurement or the result of ADCP compass errors; and (3) the effect of this on the discharge rating.
11. The regression analysis used to develop backup equations to estimate flows when the Romeoville AVM is not functioning properly should be repeated to develop new backup equations for periods when the turbine AVMs are the reported flows at Lockport.
12. Potential bias error in the annual mean discharge from the Romeoville AVM for the six years reviewed in this report is ±93 ft^3/sec.
13. Runoff from portions of the watershed is calculated from streamgage records using streamflow separation to subtract dry-weather flows from streamflow records. Since estimated records from these gaging stations are 'poor' and much of the estimated record is for winter low flows, care should be taken to exclude poor records from the estimation of dry-weather flows.
14. Overall, the design and operation of the 25-gage precipitation network should provide an accurate and complete record of the precipitation over the watershed. However, double-mass curves indicated four sites (sites 6, 14, 15, and 23) where the long-term catch compared to the other gages of the network changed coincident with moving or replacement of the gage. The effect of these changes should be evaluated, and their corresponding impact on diversion accounting reviewed. In addition,

records should be reviewed periodically to identify changes in the 'catch' of the gages.

15. The USGS is continuing to revise and update the instrumentation, rating, and backup equations for the AVM on the Calumet River at O'Brien Lock and Dam. The record from this station, through water year 1998, has not been published and is still considered 'Provisional' and subject to revision. The AVM velocities show significant noise and variation among paths. The accuracy of the mean annual discharge at this site, cannot be determined by the current records.

    - Discharge measurements used for rating analysis show a strong serial correlation. Consecutive discharge measurements for a fixed flow conditions should be grouped and averaged for rating analysis. Statistical tests for serial correlation should be a standard part of the regression analysis to determine the rating.
    - Measurements of the discharge in the shallow area near the left bank that cannot be measured by the ADCP should be done to verify and/or revise the method to calculate the flow through this area. These measurements should be done concurrently with ADCP measurements and should be done for a variety of flow, discharge, and sluice-gate conditions.
    - Backup equations should be developed to estimate flow for periods of missing AVM record based on the sluice gates. Equations should be developed in terms of present conditions and future construction. As part of this backup system, measurements of sluice gate opening should be improved.

16. The USGS is continuing to revise and update the instrumentation, rating, and backup equations for the AVM on the Chicago River at Columbus Avenue. The record from this station, through water year 1998, has not been published and is still considered 'Provisional' and subject to revision. The AVM velocities show significant noise and variation among paths. The accuracy of the annual mean discharge at this site, based on current records, is approximately ±190 ft^3/sec. The committee anticipates that the accuracy of the calculated discharges at this site should be improved from this value as a result of the continuing efforts to improve the instrumentation and discharge-calculation procedures. The committee has the following specific recommendations for the AVM at Columbus Avenue.

    - Steps should be taken to reduce the noise in the path velocities. This may involve upgrades to the AVM transducers and firmware, and also may involve moving paths 2 and 4 to the same plane as paths 1 and 3.
    - The effect of temperature gradients on the acoustical signal should continue to be investigated. If these prove to be significant, temperature probes should be installed to determine the location of the temperature gradient relative to the AVM paths.
    - Because of the common occurrence of flow reversals in the vertical, the USGS should continue to investigate use of an upward-looking velocity profiler to augment the data from the horizontal AVM paths. If this proves to improve the rating, such an instrument should be installed and used as part of the daily operation of this site.
    - Discharge measurements used for rating analysis show a strong serial correlation. Consecutive discharge measurements for a fixed flow conditions should be grouped and averaged for rating analysis. Statistical tests for serial correlation should be a standard part of the regression analysis to determine the rating.

- Because of the magnitude of the error, relative to the magnitude of low flows at Columbus Avenue, reflected by the record presented to the Committee, proposed real-time operational decisions based on the AVM record are severely limited.

- Backup equations should be developed to estimate flow for periods of missing AVM record based on the Chicago River Controlling Works gates. Equations should be developed in terms of present conditions and future construction. As part of this backup system, measurements of Lake Michigan and Chicago River stage and sluice gate opening should be improved.

17. The USGS is currently installing an AVM on the North Shore Channel at Wilmette, Illinois. This site may experience many of the difficulties encountered at Columbus Avenue and O'Brien Lock and Dam, and the Committee recommends:

    - Consecutive discharge measurements for a fixed flow condition should be grouped and averaged for rating analysis. Statistical tests for serial correlation should be a standard part of the regression analysis.

    - Backup equation should be development to estimate flow for periods at missing AVM record based on the position of the sluice-gate and the lake and channel stages. Measurements to develop this equation should be done with an ADCP. The lake and channel stage and gate-opening measurements should be verified as part of these measurements.

18. For lakefront accounting, the long-term average runoff from the diverted Lake Michigan watershed has been fixed at 800 ft^3/sec through the year 2020 as part of the mediation agreement. This runoff number was established as part of the mediation and has its basis from long-term simulation and streamflow separation of historical records. In order to re-evaluate this value in 2020, the capability to accurately simulate the hydrology of the watershed needs to be maintained. In particular, the Committee recommends the following:

    - For the purposes of analyzing historical record and long-term trends, errors in past modeling results should be considered that have been identified by this and previous committees as part of the 2020 review.

    - Continue to collect and compile data needed to simulate the hydrology of the watershed. This includes rainfall, groundwater, treatment plant flows, Des Plaines pump station flows, etc.

    - The committee recognizes the difficulty and expense associated with a field investigation of flows from the Des Plaines pumping station. Nevertheless, because this is a key calibration point in the simulation models and the accuracy of these flows will affect the accuracy of computed runoff and consumptive-use values, this committee concurs with the recommendation of the first three technical committees that a field investigation and quality-assurance review needs to be done for the flows from the Des Plaines pumping station. This needs to include bypass flows as well as pumpage.

    - For the TARP tunnel models, investigate the groundwater inflow into the tunnels. Also investigate the stormwater inflow into the TNET model by adjusting the closure elevations of the index drop shaft gates.

    - Quality-assurance reviews need to be done for the data used for major calibration points in the modeling. This includes treatment plant flows, pumping from the TARP system, as well as the Des Plaines Pump Station.

19. For lakefront accounting, the long-term average consumptive use of water pumped from Lake Michigan has been fixed at 168 ft^3/sec through the year 2010 as part of the mediation agreement. Based on a review of the available data, the Committee concludes that consumptive use cannot practically be determined directly. The committee therefore concludes that an indirect determination of consumptive use from a water budget analysis based on water-supply pumpage and treatment plant flow records and simulation results is consistent with best current engineering practice. This analysis would include measurement of treatment plant flows. In order to re-evaluate this value in 2010, the capability to accurately simulate the hydrology of the watershed needs to be maintained as described in item 18.

20. The accounting for Groundwater Pumpage Discharged to the Canal and Lake Michigan Pumpage not Discharged to the Canal (Columns 4 and 9 of the accounting report tables) is based on water-supply pumpage records. As such, these values do not consider consumptive use, although the Supreme Court Decree specifies sewage effluent for the accounting. The Committee therefore recognizes that the consumptive use should be subtracted from the water-supply pumpage records to determine the values for these columns in the Romeoville accounting. However, the Committee also recognizes that consumptive use varies widely among water suppliers and among geographic regions of the greater Chicago metropolitan area. In addition, the range of potential consumptive use values is not well defined. Finally, the imminent transition from Romeoville to Lakefront Accounting will preclude the need for this adjustment. Therefore, the Committee recommends that studies of consumptive use should focus on the need to re-evaluate the values of 168 ft^3/sec for Lakefront Accounting (see recommendation 19) rather than attempting to define values for the relatively small suppliers and short period that are reflected by Columns 4 and 9 of the Romeoville accounting tables.

21. The committee recognizes that, for portions of the system where both runoff and consumptive use are determined by simulation, errors in one of these components will tend to be offset by errors in the other, resulting in little net error in the diversion-accounting values. However, since runoff for some areas is calculated from streamflow separation rather than by simulation and since some of the consumptive use is outside the diverted Lake Michigan watershed, errors may not totally offset. The committee therefore recommends that consumptive use and unit runoff from areas where these terms are calculated independently be compared with those from simulated areas to ensure that the values are consistent for the entire system.

22. Water-supply pumpage accounts for about 80 percent of the measured components of Lake Michigan Diversion under the proposed lakefront accounting system. The USACE has initiated quality-assurance reviews of three of the water-supply facilities. These reviews were done to provide a protocol and format for subsequent review of the remainder of the water-treatment facilities and pumping stations. The reviews from the three prototype studies do not adequately document the accuracy of the pumpage records from these plants. The Committee has the following specific recommendations for analysis of the accuracy of water-supply pumpage.

    - More thoroughly investigate whether the discharge rating of the flow meters has been affected by wear and tuberculation.
    - Adequately document the discharge coefficients and approach-length requirements for meters based on manufacturer and literature specifications for

that particular type of meter, rather than using generalized coefficients for similar meters.

- If the information on specific meters is unavailable an alternative would be to make a series of measurements with a calibrated meters to develop ratings for a representative sample of the undocumented meters.
- Adequately document the effect of upstream fittings on meter accuracy, including the effect of multiple fittings, the effect of fitting and pressure tap orientation, the specific effect of different types of fittings, and the change in the effect with distance from the meter.
- Ensure that the direction of bias errors is identified, when possible, and properly accounted for in summing the total errors for a meter.
- Implementation of acoustic instrumentation to measure flows from the Jardine and Southside water treatment plants should be supported to provide a backup measurement for Chicago's water-supply pumpage.

23. The Committee recognizes the practical limitations of revising certified diversion flows. However, in light of the exceedance of the diversion amount as defined in the Supreme Court decree of 1980 and the various accounting errors that have been noted by this and previous committees, the historical diversion accounting should be reviewed.
24. The Technical Committee is concerned regarding the data viability during the initial part of the three-water-year transition period. The USGS is using state-of-the-art technology to measure the velocities and develop the ratings at these sites. The Technical Committee believes the accuracy of the record currently available for these sites does not reflect the potential of the current technology to measure flows at these sites. The Technical Committee does believe that the on-going refinements to the instrumentation at these sites and the continuing measurements with different instruments (e.g., upward-looking velocity profilers) may provide guidance for processing data already collected.
25. The Technical Committee recommends that the USGS use data from on-going measurements with different instruments to attempt to develop methods to screen or filter the data already collected. These methods should define the accuracy of the records thus developed, as well as the accuracy that is achieved with the refined instrumentation. This will allow the transition period to begin at some time between October, 1996 and when the instrumentation refinements are accepted as operational, while also providing the data needed to assess the technical acceptability of lakefront accounting.

6. APPENDICES

7. REFERENCES

American Society of Mechanical Engineers, 1971: Fluid meters, 6th Edition.

American Society of Mechanical Engineers, 1985, ASME MFC-3m (1985)

Ang, A.H.S., and Tang, W.H., 1975, Probability Concepts in Engineering Planning and Design; Volume I, Basic Principles: New York, Wiley, p. 409

Appell, G.F., Gast, J., Williams, R.G., and Bass, P.D., 1988, Calibration of Acoustic Doppler Current profilers: in Oceans '88, Proceedings of IEEE Ocean Engineering Society, Baltimore, MD, October 31 – November 1, 1988, P. 346-352

Barkau, 1991, Modeling of the Chicago Tunnel and Canal System. Prepared for Christopher B. Burke Engineering Ltd. as part of "Data Collection and Model Revisions." Prepared for the U.S. Army Corps of Engineers, Chicago.

Carter, R.W. and Anderson, I.E., 1963, Accuracy of Current Meter Measurements: Proceedings of the American Society of Civil Engineers, Journal of the Hydraulics Division: 8(HY4, Part 1), pp 105-115.

Crawford and Linsley, Digital Simulation in Hydrology: Stanford Watershed Model IV, Stanford University, Department of Civil Engineering Technology, Rep. 39, 1966.

Cunge, J.A., Hulls, F.M., and Verway, A., 1986, Practical Aspects of Computational River Hydraulics: Monographs and Surveys in Water Resources Engineering, University of Iowa, Iowa Institute of Hydraulics—unabridged reprint of work first published by Pitman Advanced Publishing Program in 1980, 420 p.

Dickenson, W.T., 1967, Accuracy of Discharge Determinations: Hydrology Papers, Colorado State University, Fort Collins, CO, U.S.A., No. 20, pp. 1-54.

Espey, W.H., Barnes, H.H., and Vigander, S., 1981, Findings of the First Technical Committee for Review of Diversion Flow Measurements and Accounting Procedures, October, 1981, U.S. Army Corps of Engineers, Chicago District, 110 p.

Espey, W.H., Barnes, H.H., and Westfall, D.E., 1987, Findings of the Second Technical Committee for Review of Diversion Flow Measurements and Accounting Procedures, November, 1987, U.S. Army Corps of Engineers, Chicago District.

Espey, W.H., Lara, O.G., and Barkau, R.L., 1994, Findings of the Third Technical Committee for Review of Diversion Flow Measurements and Accounting Procedures: 125 p., and 8 appendicies; *in U.S. Army Corps of Engineers, 1994, Lake Michigan Diversion Accounting Water Year 1993 Annual Report.*

Fulford, J.M., and Sauer, V.B.., 1986, Comparison of Velocity Interpolation Methods for Computing Open-channel Discharge: U.S. Geological Survey Water-supply Paper 2290, p. 139-144

Gain, W.S., 1998, Office of Surface Water Review of the Illinois District: Hydroacoustics: U.S. Geological Survey Water Resources Division written commun., 10 p.

Halmi, D., 1974a, Metering performance Investigation and Substantiation of the "Universal Venturi Tube" (U.V.T.): Part 1—Hydraulic Shape and Discharge Coefficient: Transactions of the American Society of Mechanical Engineers, Journal of Fluids Engineering, Paper No. 73-WA/FM-3, June, 1974, pp. 124-131

-----, 1974b, Metering performance Investigation and Substantiation of the "Universal Venturi Tube" (U.V.T.): Part 2—Installation Effect, Compressible Flow, and Head Loss: Transactions of the American Society of Mechanical Engineers, Journal of Fluids Engineering, Paper No. 73-WA/FM-4, June, 1974, pp. 132-138

Hart, E.D. and McGee, R.G., 1985, Lockport Powerplant Sluice Gate and Control Works Discharge Evaluation: U.S. Army Corps of Engineers Waterways Experiment Station Misc. Pump, HL-85-4

Harza Engineering, 1981, An Evaluation of Flow Measurements and Accounting Methods for Lake Michigan Diversion:

Herschy, R.W., 1970, The Evaluation of Errors at Flow-Measurement Stations: International Symposium on Hydrometry, Koblenz, International Association of Hydrological Sciences, Publication No. 99, pp. 109-131.

-----, 1975, The Accuracy of Existing and New Methods of River Gauging: Ph.D. thesis, Department of Geography, University of Reading, Reading, berkshire, England.

-----, 1978, The Accuracy of Current Meter Measurements: Proceedings, Institution of Civil Engineers (Great Britain), Part 2, 65: 431-437.

International Standards Organization, 1980, Standard 5167,

Johanson, R.C., Imhoff, J.C., Kittle, J.L., Donigian, A.S., 1984, Hydrological Simulation Program—Fortran (HSPF): User's Manual for Release 8.0: U.S. Environmental Protection Agency Reprot EPA-600/3-84-066, 767 p.

Kiefer and Associates, Inc., 1978, Flows Crossing the Lake Michigan Diversion Boundary in Indiana: Prepared for Illinois Department of Transportation, Division of Water Resources.

Linford, A., 1961, Flow Measurement and Meters: 2nd Edition, E. & F.N. Spon, Ltd., London, U.K., 430 p.

Lipscomb, S.W., 1995, Quality Assurance Plan for Discharge Measurements Using Broadband Acoustic Doppler Current Profilers: Open-File Report 95-701, 7 p.

Melching, C.S., and 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, 61 p.

Miller, R.W., 1989, Flow Measurement Engineering Handbook: 2nd Edition, McGraw-Hill, New York, NY, USA

Miner, I.I., 1956, The Dall Flow Tube: Transactions of the American Society of Mechanical Engineers, Paper No. 54-A-139, April, 1956, pp. 475-479

Morlock, S.E., 1996, Evaluation of Acoustic Doppler Current Profiler Measurements of River Discharge: U.S. Geological Survey Water-Resources Investigations Report 95-4218, 37 p.

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 Investigations Report 94-4112, 48 p.

Ott, R.L., 1992, An Introduction to Statistical Methods and Data Analysis: 4th Edition, Wadsworth Publishing, Belmont, CA, 1183 p.

Pelletier, P.M., 1988, Uncertainties in the Single Determination of River Discharge: A Literature Review: Canadian journal of Civil Engineering, Vol. 15, pp. 834-850.

Peppler, R.A., 1991a, 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, 87 p.

-----, 1991b, Continued Operation of a Raingage Network for Collection, Reduction, and Analysis of Precipitation Data for Lake Michigan Diversion Accounting: Water Year 1991: Illinois State Water Survey Contract Report 520, 65 p.

-----, 1993, Continued Operation of a Raingage Network for Collection, Reduction, and Analysis of Precipitation Data for Lake Michigan Diversion Accounting: Water Year 1992: Illinois State Water Survey Contract Report 551, 59 p.

-----, 1994, Continued Operation of a Raingage Network for Collection, Reduction, and Analysis of Precipitation Data for Lake Michigan Diversion Accounting: Water Year 1993: Illinois State Water Survey Contract Report 573, 60 p.

-----, 1995, Continued Operation of a Raingage Network for Collection, Reduction, and Analysis of Precipitation Data for Lake Michigan Diversion Accounting: Water Year 1994: Illinois State Water Survey Contract Report 585, 48 p.

Phair, D.A., 1997, Errors Occur often in Municipal Flowmetering: Process Engineering. v. 75, Sep. 1997, p. s9-12.

Rantz, S.E., and others, 1982a, Measurement and Computation of Streamflow: Volume 1. Computation of Discharge: U.S. Geological Survey Water Supply Paper 2175, 284 p.

----, 1982b, Measurement and Computation of Streamflow: Volume 2. Measurement of Stage and Discharge: U.S. Geological Survey Water Supply Paper 2175, p. 285-631.

Rust Environmental and Infrastructure, 1993, Diversion Accounting Update for the New 25-gage Precipitation Network: 25 p. and 8 appendicies; *in U.S. Army Corps of Engineers, 1994, Lake Michigan Diversion Accounting Water Year 1993 Annual Report, Appendix D.*

Simpson, M.R., and Oltman, R.N., 1992, 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.

Sloat, J.V. and Gain, W.S., 1995, Application of Acoustic Velocity Meters for Gaging Discharge of Three Low-Velocity Tidal Streams in the St. Johns River Basin, Northeast, Florida: U.S. Geological Survey Water-Resources Investigations Report 95-4230, 26 p.

Smoot, G.F., and Carter, R.W., 1968, Are Indivdual Current-Meter Ratings Necessary: Proceedings of the American Society of Civil Engineers, Journal of the Hydraulics Division: 94(HY2), pp 391-397.

U.S. Army Corps of Engineers, 1992, Lake Michigan Diversion Accounting Annual Report Water years, 1990-1992: Chicago District, p. 19 and 6 appendicies

-----, 1994a, Lake Michigan Diversion Accounting Water Year 1993 Annual Report: Chicago District, 15 p. and 8 appendicies.

-----, 1994b, HECDSS User's Guide and Utility Manuals: Hydrologic Engineering Center, p. 474

-----, 1995, Lake Michigan Diversion Accounting Water Year 1994 Annual Report: Chicago District, 135 p.

-----, 1996a, Lake Michigan Diversion Accounting Lakefront Accounting Technical Analysis: Chicago District, May 28, 1996, Draft

-----, 1996b, Preliminary QA/QC Field Investigations of Domestic Water Supply Metering Systems for Illinois' Major Direct Diverters of Lake Michigan Water: Chicago District Letter Report, 25 September, 1996, 24 p.

-----, 1997, Lake Michigan Diversion Accounting Water Year 1995 Annual Report: Chicago District, 142 p.

-----, 1998a, Lake Michigan Diversion Accounting Draft Manual of Procedures: Chicago District, *in press*.

-----, 1998b, Lake Michigan Diversion Accounting Water Year 1995 Annual Report: Chicago District, 73 p.

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

Wahl, K.L., 1977, Accuracy of Channel Measurements and the Implications in Estimating Streamflow Characteristics: Journal Research of the U.S. Geological Survey 5(6), pp. 811-814.

Wahlin, B.T., and Replogle, J.A., 1998a, Technical Review of Lake Michigan Water Withdrawals by the City of Evanston: U.S. Water Conservation Laboratory, Phoenix, AZ, U.S.A., 24 p., *in press.*

-----, 1998b, Technical Review of Lake Michigan Water Withdrawals by the Mayfair Pumping Station in the City of Chicago: U.S. Water Conservation Laboratory, Phoenix, AZ, U.S.A., 16 p., *in press.*

-----, 1998c, Technical Review of Lake Michigan Water Withdrawals by the Thomas Jefferson Pumping Station in the City of Chicago: U.S. Water Conservation Laboratory, Phoenix, AZ, U.S.A., 12 p., *in press.*

Westcott, N.E., 1996, Continued Operation of a Raingage Network for Collection, Reduction, and Analysis of Precipitation Data for Lake Michigan Diversion Accounting: Water Year 1995: Illinois State Water Survey Contract Report 601, 48 p.

-----, 1998, Continued Operation of a Raingage Network for Collection, Reduction, and Analysis of Precipitation Data for Lake Michigan Diversion Accounting: Water Year 1997: Illinois State Water Survey Contract Report 628, 51 p.