LAKE MICHIGAN DIVERSION

FINDINGS OF THE TECHNICAL COMMITTEE FOR REVIEW OF DIVERSION FLOW MEASUREMENTS AND ACCOUNTING PROCEDURES

Committee Members:

DR. WILLIAM H. ESPEY, JR., Chairman

HARRY H. BARNES, JR.

DR. SVEIN VIGANDER

Prepared for:

U.S. ARMY CORPS OF ENGINEERS
Chicago District
Chicago, Illinois

LAKE MICHIGAN DIVERSION

FINDINGS OF THE TECHNICAL COMMITTEE FOR REVIEW OF DIVERSION FLOW MEASUREMENTS AND ACCOUNTING PROCEDURES

Committee Members:
Dr. William H. Espey, Jr., Chairman
Harry H. Barnes, Jr.
Dr. Svein Vigander

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

October 1981

TABLE OF CONTENTS

Section		Page
	LIST OF TABLES	vi
	LIST OF FIGURES	vii
	ACKNOWLEDGEMENTS	viii
	EXECUTIVE SUMMARY	ix
1.0	INTRODUCTION	1
1.1	PURPOSE AND SCOPE	1
1.2	HISTORY OF THE LAKE MICHIGAN DIVERSION	3
1.3	COMPONENTS OF DIVERSION	5
2.0	ACCOUNTING PROCEDURES	9
2.1	LOCKPORT MEASUREMENTS	10
2.1.1	Turbines	10
2.1.1.1	Computation of Discharge	13
2.1.1.2	Evaluation of Rating Tables	14
2.1.1.2.1	Model Tests	14
2.1.1.2.2	Generators	16
2.1.1.2.3	1936 Test	16
2.1.1.2.4	1937 Test	17
2.1.1.2.5	1977 Tests	17
2.1.1.2.6	1979 Tests	18
2.1.1.2.7	Dye-Dilution Tests	19
2.1.1.3	Evaluation of Verification Tests	20
2.1.1.4	Recommendations	24
2.1.2	Exciters	26
2.1.3	Lockport Powerhouse Sluice Gates	27
2.1.3.1	Model Study	27
2.1.3.2	Rating Analysis	30
2.1.3.3	Rating Tables	30
2.1.3.4	Conclusions	35
2.1.3.5	Recommendations	35

TABLE OF CONTENTS (Cont'd)

Section		Page
2.1.3.6	Leakage	36
2.1.3.6.1	Leakage Tests	37
2.1.3.6.2	Evaluation	37
2.1.3.6.3	Recommendations	38
2.1.4	Lockport Controlling Works	40
2.1.4.1	Discharge Rating	41
2.1.4.2	Rating Evaluation	43
2.1.4.3	Computation of Discharge	47
2.1.4.4	Leakages	47
2.1.4.5	Recommendations	48
2.1.5	U.S. Government Locks	49
2.1.5.1	Lockages	49
2.1.5.2	Leakages	50
2.1.5.2.1	Leakage Tests	50
2.1.5.2.2	Evaluation of Leakage Tests	51
2.1.5.2.3	Lock Leakage Accounting	52
2.1.5.2.4	Recommendations	57
2.2	INDUSTRIAL DIVERSIONS	58
2.3	DOMESTIC PUMPAGE	59
2.3.1	Ground Sources - Lake Michigan Watershed	61
2.3.2	Ground Sources - Outside the Lake Michigan Watershed	62
2.3.3	Domestic Pumpage from Indiana and Wisconsin	62
2.3.4	Evaluation of Domestic Pumpage Accounting	64
2.3.5	Recommendations	65
2.3.6	Contribution of Lake Michigan to Groundwater	67
2.4	STORM RUNOFF AND INFILTRATION FROM THE ILLINOIS WATERSHED	67
2.4.1	Infiltration	68
2.4.1.1	Evaluation	70

TABLE OF CONTENTS (Cont'd)

Section		Page
2.4.1.1.1	Upper Des Plaines Pumping Station Discharge Measurement	70
2.4.1.1.2	Computation of Daily Discharge	70
2.4.1.1.3	The Computation of Domestic Pumpage	71
2.4.1.1.4	Infiltration and Inflow Factors	71
2.4.1.2	Recommendations	71
2.4.2	Summit Conduit	72
2.4.2.1	Depth of Flow Recording	75
2.4.2.2	Depth of Flow-Discharge Relation	76
2.4.2.3	Evaluation	77
2.4.2.4	Recommendations	78
2.4.3	Des Plaines River Watershed South of the Main Channel	79
2.4.4	Sewer System 13A (Part of the TARP System)	79
2.4.5	O'Hare Water Reclamation Plant	81
2.5	HYDRAULIC REPORT - COMPUTATIONAL PROCEDURES	82
3.0	WILLOW SPRINGS ROAD ACOUSTICAL VELOCITY METER	85
4.0	TARP - IMPACT ON THE DIVERSION ACCOUNTING PROCEDURES	88
5.0	CONCLUSIONS AND RECOMMENDATIONS	90
5.1	CONCLUSIONS	90
5.1.1	Dynamics of the Diversion System	93
5.1.2	Evaluation of the Diversion Components	94
5.2	RECOMMENDATIONS	98
5.2.1	Master Plan for Diversion Accounting	98
5.2.2	Quality Assurance	99
5.2.3	Alternatives to Measurement at the Lockport Facilities	101
5.2.4	Lockport Turbines	102
5.2.5	Lockport Sluice Gates	103

TABLE OF CONTENTS (Concluded)

Section		Page
5.2.6	Lockport Controlling Works	104
5.2.7	Lockport Lock Leakage	104
5.2.8	Industrial Diversion	105
5.2.9	Domestic Pumpage	105
5.2.10	Storm Runoff and Infiltration from the Illinois Watershed	106
5.2.11	Summit Conduit	106
5.2.12	Des Plaines Watershed South of the Main Channel	107
5.2.13	Sewer System 13A	107
6.0	REFERENCES	108
	APPENDIX A - QUALIFICATIONS OF COMMITTEE MEMBERS	
	APPENDIX B - SCHEDULE OF COMMITTEE ACTIVITIES	
	APPENDIX C - MINUTES OF COMMITTEE WORKSHOPS	

LIST OF TABLES

Table		Page
1	Average Annual Flow at Lockport	12
2	Lockport Powerhouse Sluice Gates Operational Gate Opening Combinations	31
3	Lockport Powerhouse Sluice Gates MSD Rating Tables Gate Combination Option 2	31
4	Lockport Powerhouse Sluice Gates MSD Rating Tables Gate Combination Options 4, 5, and 6	32
5	Lockport Powerhouse Sluice Gates MSD Rating Tables Gate Combination Option 7	34
6	Comparison of Constant Leakage Discharge with Averaged Leakage-Lockport Lock	54
7	Evaluation of Key Components of Lake Michigan Diversion	95
8	Proposed Alternatives to the Lake Michigan Diversion Flow Measurements and Accounting Procedures	97

LIST OF FIGURES

Figure		Page
1	History of Diversion from Lake Michigan at Chicago	4
2	Location Plan - Lake Michigan Diversion at Chicago	6
3	Components of Diversion from Lake Michigan at Chicago	7
4	The Metropolitan Sanitary District of Greater Chicago Hydraulic Report for September, 1980	11
5	Lockport Powerhouse Sluice Gate Leakage	39
6	Lockport Controlling Works - Gate Seat and Canal Bed Definition Sketch	46
7	U.S. Government Lock, Lockport Leakage Test, 1977-1981	55
8	Location Plan - Lockport Lock and Dam, Sewer System 13A, Summit Conduit, and Des Plaines River Watershed South of the Main Channel	73
9	Detailed Location Plan - Summit Conduit	74
10	Location Plan - Hart Ditch Gage	80

ACKNOWLEDGEMENTS

During the course of the Committee's work, a number of individuals and organizations were extremely helpful in assisting the Committee to accomplish its tasks.

The Committee gratefully acknowledges the invaluable help of Mr. Neil Fulton and Mr. Daniel Injerd, Illinois Department of Transportation, and Mr. William Eyre, Mr. Ram Koduri, and Mr. Subhash Patell, Metropolitan Sanitary District of Greater Chicago (MSD), who provided information and assistance in the Committee's inspection of the various MSD facilities involved in the Lake Michigan Diversion. The Committee would also like to express its appreciation for the assistance of the U.S. Army Corps of Engineers, Chicago District, and specifically, Mr. Jerry Stadler, Mr. Larry Dunbar, Mr. Steve Klawans, Mr. Donald Dressel, and Mr. Stan Boc for their contribution in providing numerous reports and data, and arranging and assisting the Committee in the various workshops and field trips. The Committee also recognizes the assistance of Ms. Maryann Sumi and Dr. Kenneth Potter, State of Wisconsin, for providing information. In addition, the Committee acknowledges Mr. Gary Guhl for his invaluable assistance in coordinating the preparation of this report, Ms. Linda Henderson for her word processing support, and Ms. Vera Massaro for graphics preparation and cover design.

EXECUTIVE SUMMARY

The Committee has made an evaluation of the techniques and procedures for determination of Lake Michigan Diversion with respect to the best current engineering practices and scientific knowledge as stipulated by the 1980 Supreme Court Decree. This evaluation has been made within the limitations of time, resources and information made available to the Committee. In its evaluation, the Committee was aware of the importance of the Lake Michigan Diversion to Illinois' mandated leadership for management, development, and conservation of water resources of the metropolitan region.

Based on a casual observation, the measurement and accounting of diversion appear to be relatively simple in concept. However, the measurement of flow at Lockport is at best a summation of complex components which are synthesized by a variety of hydraulic and hydrologic techniques that have been developed and have evolved over a long period of time. The Committee traced back development of some of these techniques more than three decades only to find an explanation less complete than hoped for, or worse, none at all. This was not surprising, considering that the sole purpose for much of the information sought was to comply with requirements of the Court, and within that context, address conditions within the diversion system that have changed significantly over the years.

Although the Decree specifically limits the measurement of diversion water to Lockport, there is one aspect of the proceedings leading to the Decree that should not be ignored. The Illinois proposal to the Supreme Court centered on the assignment of a constant value to the runoff from the diverted watershed. A constant value would have provided the greatest flexibility in efficient management of the water allocation. The Committee concluded from its review of the testimony for the Court that there was no significant disagreement among the interested parties with respect to the principle. The adoption of a numerical value was rejected because of a lack of concensus among the parties, not because of insurmountable differences over Lakefront vs. Lockport measurement sites. The potential value in terms of operational effectiveness, represented by an average watershed runoff, is important and should not be ignored. Ideally, the techniques and procedures adopted for the measurement and accounting of diversion flow will be an integral part of Illinois planning and management strategy.

During the course of the study, the Committee was made keenly aware that the measuring and accounting process lacked credibility. Some of this is possibly due to a lack of a complete understanding and familiarity with the problems associated with the computation of the Diversion. Many of the issues concerned with credibility stem from inconsistencies in quality assurance. However, the review of the diversion computational program has to be viewed with respect to the priority that has been established by the State of Illinois. Limiting funding of the diversion computations has clearly effected the overall quality of the program.

The Lockport flow components are deficient in practically every respect. The basic fluid principles, physical laws, and data requirements, for these hydraulic components are widely understood and a judgement of adequacy is relatively straightforward. However, judgements for some of the non-diversion, and runoff and infiltration components could not be made quite so decisively. This was especially true for methods based on hydrologic similarities, extrapolations, and indices. The opinion of the Committee regarding the state-of-the-art evaluation of the flow measurement techniques and the computational procedures currently used for diversion reporting are summarized in this report.

In the Committee's view every component is deficient with respect to quality assurances. Generally, these deficiencies cover the full range of elements, from a simple flow measurement to the final endorsement of activities during the 5-

year accounting periods. The restoration of credibility can be achieved largely through acceptable quality assurance programs, third party technical review, and improved communication among the interested parties.

The probable error in the computed flow was estimated for each of the major flow components. The error is the cumulative effect of deficiencies, and its value is usually fixed. However, the turbine, exciter, and leakage component errors tend to increase with time. This is important because these components account for about 80 percent of the Lockport flow, the turbines accounting for about 78 percent alone. Clearly, the turbines represent the diversion component for which flow measurement errors are of the greatest significance. Also, a probability of a biased undermeasurement of the total flow measured is evident. Therefore, recommendations for implementation should be focused on the turbine rating first, the sluices second, the Controlling Works third, etc.

Considerable efforts have been made to obtain reliable measurements of the flow at Lockport, particularly in 1977 and again in 1979, for comparison with those being computed by the Metropolitan Sanitary District of Greater Chicago (MSD). The Detroit District, USCE, performed both sets of channel measurements. The results of the two sets of measurements are somewhat at odds. In 1980, the Detroit District reexamined the data sets individually and comparatively in hopes of explaining the disparities. They finally concluded that there was insufficient information for drawing conclusions on the basis of either set. In retrospect the Committee concurs and attributes these disparities to two principal causes. First, the field procedures did not provide adequate consideration for the unsteady flow conditions that existed in the channel reach. Second, while the two field exercises certainly could be used for comparison with MSD computed flows, both were inadequate for the purpose of verifying or calibrating the turbine generators discharge rating.

The Committee proposes that the present turbine generators be recalibrated on the basis of field measurements periodically on or before and after significant repairs if reliable flow figures are to be expected. Two techniques are available for calibrating. The more accurate and expensive utilizes a special hoist structure and supporting frame for cosine-component current meters to traverse the flow passing the scrollcase bulkhead slots. The other technique is a standard channel section current-meter measurement. The expected accuracy of the bulkhead slot measurement is 1 to 2 percent, compared to 3 to 5 percent for the channel measurements. A preferred alternative to the turbine generator rating is the installation of Winter-Kennedy piezometer taps to each turbine's scrollcase. The advantages of a direct turbine rating are improved stability, reliability, and reduced calibration costs.

The Committee also proposes that a study be undertaken to evaluate an acoustical velocity meter (AVM) system as an alternative to Lockport for the measurement of total flow. If found to be feasible, the AVM system offers potential for flow information, in real time at a 1 percent level of accuracy at a lower annual cost. The Committee has made a cursory review of the Willow Springs Road AVM experiment conducted in the late 1960's. It is the Committee's considered opinion that it would be unfortunate indeed if the future potential of the AVM to the diversion program should be discounted on the basis of the failure of the Willow Springs Road AVM experiment.

Recommendations are made for needed improvements to virtually all components. However, those dealing with the turbines are considerably more costly. In addition to specific recommendations of alternatives to the present accounting procedures, the Committee recommends that a master plan for the management of the Lake Michigan Diversion program be developed. The plan should be in accord with the Supreme Court Decree and should assure a level of accuracy for the diversion flow record that is consistent with best current engineering practices. As an integral part of this master plan, it is recommended that an "Operational Procedure Manual", delineating specific technical procedures, be developed.

Future priorities and demands for water allocations, changing diversion into and from the basin, watershed dynamics, and an important but aging waterway are a few of the problems that will pose challenges to the diversion program during the next forty years. Without attempting to define the future needs of the program several needs are clearly obvious. These include; an evaluation of alternatives to the Lockport measurement system, an expansion of the monitoring system for the measurement and determination of the hydrologic response of Chicago Tunnel and Reservoir Plan (TARP) and other modifications to the Lake Michigan Watershed.

TARP, by capturing nearly all the combined sewer overflow, will greatly enhance the river and canal water quality, reducing or eliminating the need for dilution water diverted from the lake. The large reservoirs to be constructed as part of the TARP plan will almost eliminate the requirement for navigation makeup water by release of stored water when needed. Overall, the TARP system will permit a more efficient allocation of Lake Michigan water for domestic use, reducing the demand for ground-water sources which are rapidly being depleted.

The impact of TARP on the diversion program will be manifested in two ways. First, the computations, procedures and factors for the determination of domestic pumpage, and infiltration and inflow discharges will need to be reassessed for the installation of each new element to the system. Second, the increased storage provided by the tunnels and reservoirs will reduce the magnitude of storm hydrographs at Lockport. This will result in a higher percentage of the annual flow being discharged through the turbines, and conversely less flow through the powerhouse sluices and the Controlling Works.

Operation flexibility is essential for a dynamic program with a projected time-frame of 40 years.

Equally obvious is that the diversion program should become an integral part of a real-time waterway system operational model capable of optimizing the use of Lake Michigan water.

The diversion measurement and accounting procedures are an essential part of the State's water allocation management plan. One important aspect of this plan deals with the collection and dissemination of hydrologic data. Hydrologic data are collected by a number of federal, state, and local agencies for a variety of needs such as flood control, regulatory compliance, program mission, as well as accountability for diversion. During efforts to obtain information, the Committee was continuously impressed with the difficulties that it assumed to be the result of two causes: first, the lack of coordination among data base systems, and second, that often the data retrieval was not an important consideration in the development of the data systems. It was inevitable that the Committee would develop views on the whole matter of data adequacy and availability. Essentially these views suggest the need for a comprehensive water data management system that would include: (a) data collection network evaluation (existing networks) and design (future networks); and (b) a central data storage and retrieval facility. The goals of the comprehensive water data management system should be to anticipate future data needs, promote the acquisition, provide direction necessary to insure uniformity, consistency, and make water resources information available in a timely fashion.

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The amendment to the 1967 U.S. Supreme Court Decree for the Lake Michigan Diversion at Chicago, Illinois was adopted by the Court on December 1, 1980. The pertinent provisions of the modified Decree include extending the period for determining the running average diversion rate allowable by the State of Illinois from five years to 40 years and changing the beginning of the accounting year from March 1 to October 1.

The modified Decree also provides that the Chief of Engineers, U.S. Army Corps of Engineers (USCE) appoint a three-member committee (hereinafter referred to as the Committee) to evaluate the current procedures, and to recommend appropriate changes for measurement and accounting of the diversion using the best current engineering practice and scientific knowledge. At least every five years, the USCE shall reconvene such a Committee to report on the method of flow measurement and the accounting procedures.

The services to be rendered by the Committee under the scope of work include a review of current diversion-related measurement techniques at the Lockport control structure, and other pertinent locations, to determine if the best current engineering practice and scientific knowledge is being applied. This determination has been made in compliance with the stipulations of the 1967 Supreme Court Decree with the 1980 modifications and to include the following basic elements:

- analysis of current diversion-related measurement techniques and accounting procedures;

- evaluation of these techniques and procedures to determine whether the best current engineering practice and scientific knowledge are being used;

- recommendation of appropriate revisions within the legal constraints of the Decree; and

- preparation of draft and final reports.

Through the insight gained in field inspections, review of engineering data, technical workshops, and engineering analysis, the Committee has evaluated the current accounting procedures and has developed recommendations based on:

- examination of the current method of computation and accounting for the diversion;

- evaluation of the pertinent factors having an effect on the diversion computation; and

- consideration of alternative methods of computing diversion using state-of-the-art hydrologic and hydraulic methods.

From the above information, the committee has included in this report a summary of the existing and proposed accounting procedures addressing:

- the degree of accuracy that can be achieved for the diversion computation under the recommended plan; and

- the approximate cost of implementing the recommended plan.

Based upon the recommendation of the Committee, the USCE will determine the best current engineering practice and scientific knowledge for computing the Lake Michigan Diversion at Chicago.

1.2 HISTORY OF THE LAKE MICHIGAN DIVERSION

The current diversion of water from Lake Michigan at Chicago by the State of Illinois began in 1900 with the completion of the Sanitary and Ship Canal by the Metropolitan Sanitary District of Greater Chicago (MSD) as illustrated in Figure 1. In 1922, the State of Wisconsin successfully sought an injunction to bar the State of Illinois from diverting Lake Michigan water. In 1925, the U.S. Supreme Court overturned the injunction, and diversion was allowed at an average annual rate of 8,500 cubic feet per second (cfs). A 1930 Decree authorized the State of Illinois and the MSD to divert Lake Michigan water, in addition to domestic pumpage according to the following schedule of upper limits:

- an average annual rate of 6,500 cfs, on and after 1 July 1930;
- an average annual rate of 5,000 cfs, on and after 30 December 1935; and
- an average annual rate of 1,500 cfs, on and after 31 December 1938.

The 1967 Decree limited the diversion, including domestic pumpage, to an average of 3,200 cfs over a five-year running accounting period. The first accounting period began March 1, 1970 and ended on February 28, 1975. During this period, the average diversion was 3,183 cfs. The sixth and latest accounting period began March 1, 1975 and ended February 29, 1980. During this period, the average diversion was 3,044 cfs.

The U.S. Supreme Court amended its 1967 Decree on December 1, 1980. The amendment changes, in part, the provisions of the 1967 Decree that prevented the State of Illinois from effectively utilizing and managing the 3,200 cfs of Lake Michigan water which had been allocated previously by the Court. The amendment provides:

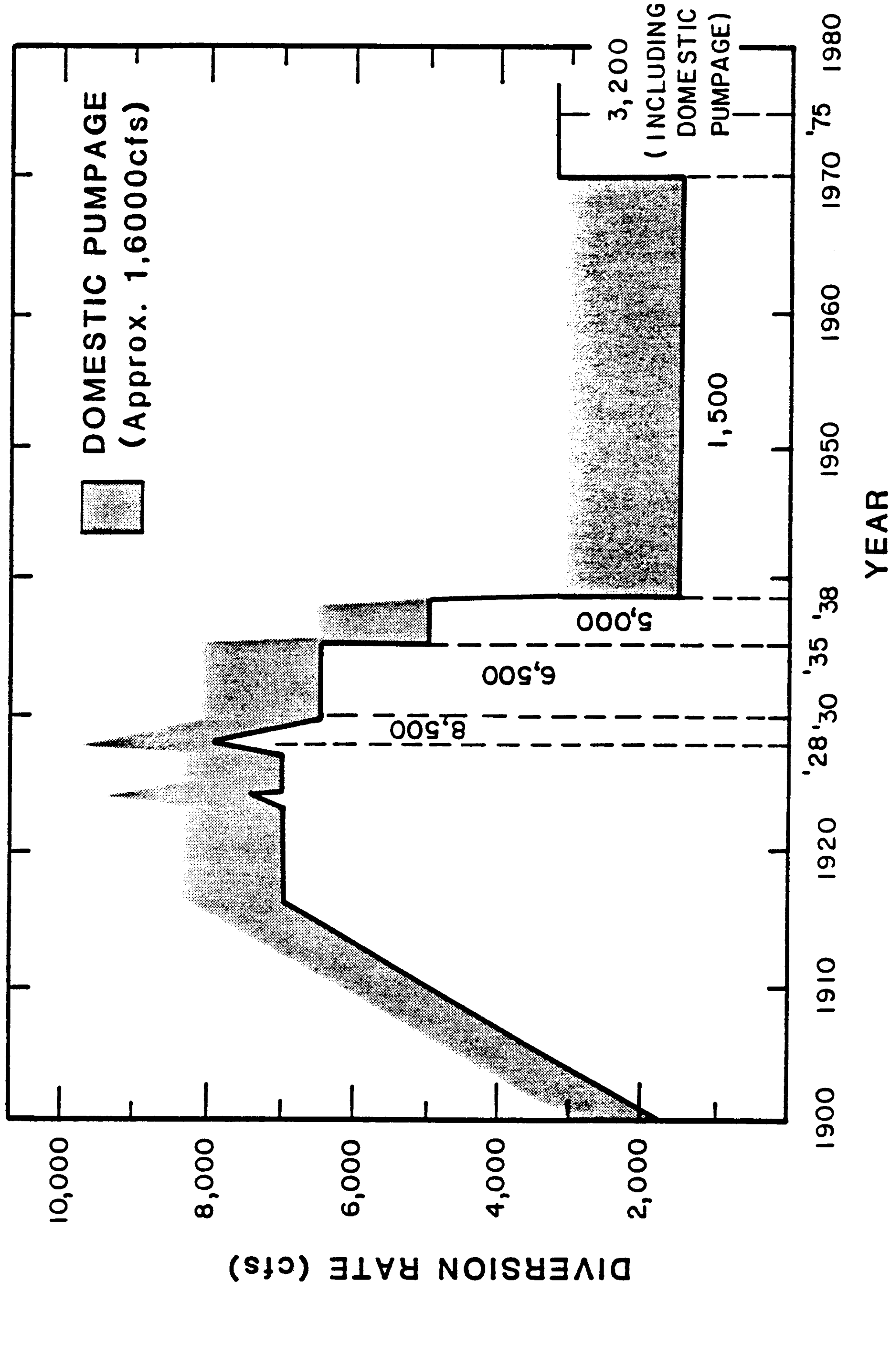

FIGURE 1 HISTORY OF DIVERSION FROM LAKE MICHIGAN AT CHICAGO

- 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;

- changing the beginning of the accounting year from March 1 to October 1;

- a limit on the average diversion in any accounting year to 3,680 cfs, except for an average diversion of 3,840 cfs in any 2 accounting years within a 40-year period; and

- a limit on the cumulative algebraic sum of the average annual diversions minus 3,200 cfs during the first 39 years to 2,000 cfs-years.

1.3 COMPONENTS OF DIVERSION

The geographic area of concern is illustrated in Figure 2 which shows part of Lake Michigan, the diverted watershed, the canal system, and the location of the major hydraulic structures.

The primary components of the Lake Michigan Diversion, illustrated in Figure 3, and described below along with the approximate contribution of each component to the total 3,200 cfs diversion (in percent) 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 (53 percent);

- storm runoff discharged from the diverted watershed area of Lake Michigan, draining to the river and canal system in the greater Chicago area (17 percent); and,

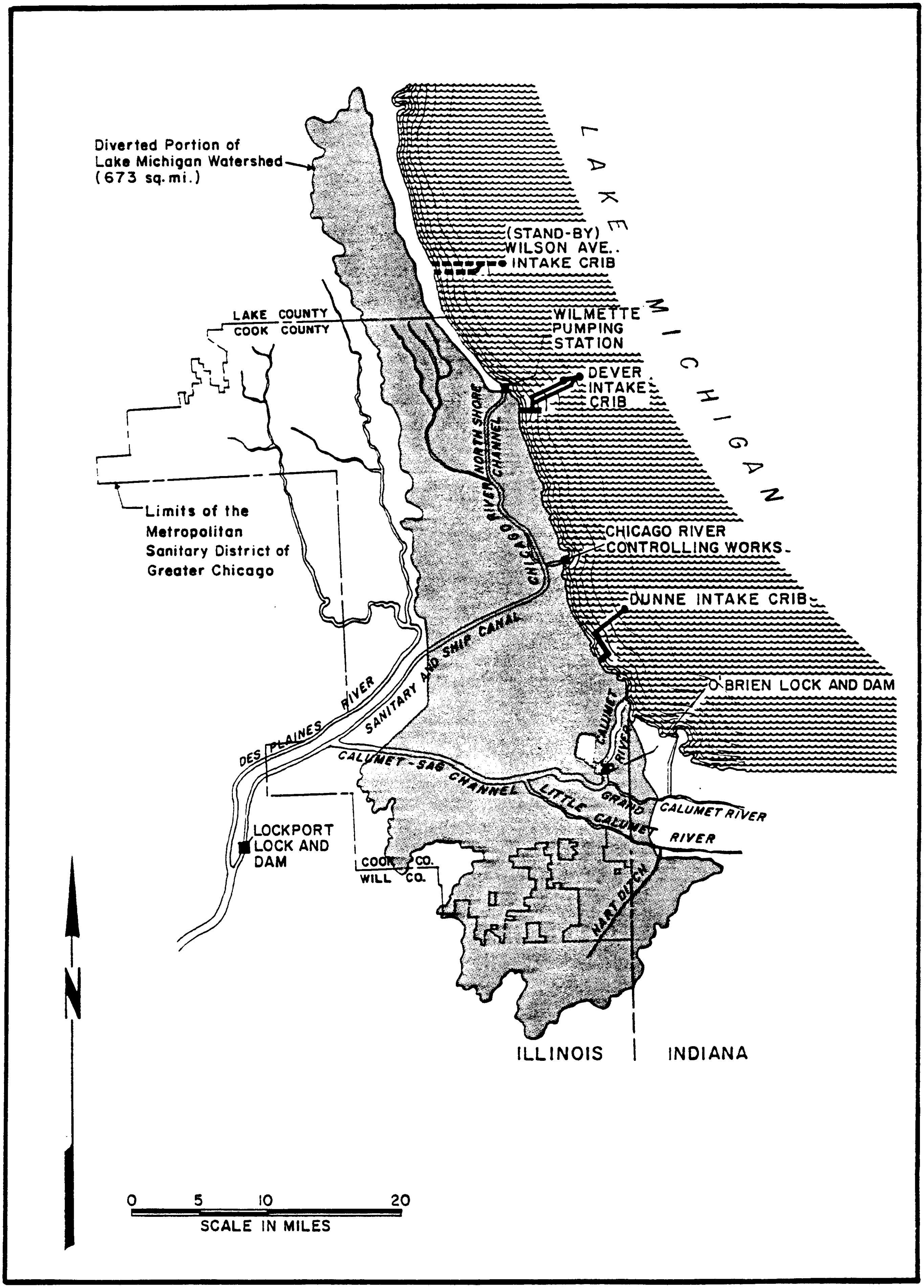

FIGURE 2 LOCATION PLAN - LAKE MICHIGAN DIVERSION AT CHICAGO

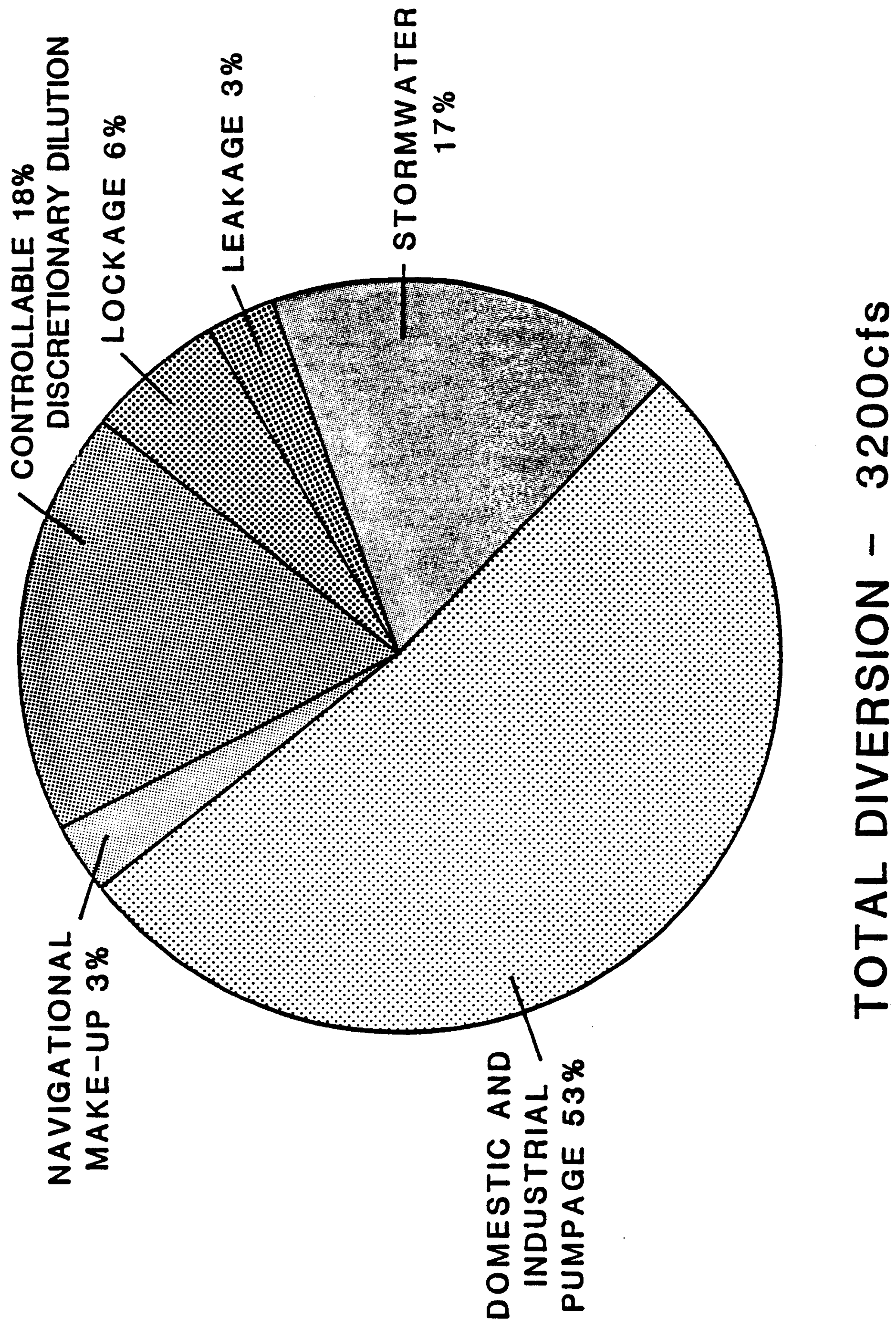

FIGURE 3 COMPONENTS OF DIVERSION FROM LAKE MICHIGAN AT CHICAGO

- water entering directly from Lake Michigan into the river and canal system in the greater Chicago area (30 percent). This component consists of the following three parts:

  - water required for lockages at the Chicago River Controlling Works and the Thomas J. O'Brien Lock (6 percent);
  - leakages occurring at the Chicago River Controlling Works, O'Brien Lock and Dam and Wilmette Pumping Station (3 percent); and
  - water taken in for navigational make-up and discretionary purposes at the Chicago River Controlling Works, O'Brien Dam and Wilmette Pumping Station (21 percent).

2.0 ACCOUNTING PROCEDURES

The Illinois diversion from Lake Michigan has historically been measured at Lockport. Prior to the construction of the control structures at the Lakefront, Lockport was the only practical point of measurement. Water flowing past Lockport originates in the Chicago River system, the Calumet River system, and various small drainage areas once in the Des Plaines River watershed and now diverted into the navigation channels. Domestic sewage effluent from the MSD treatment plants also flows past Lockport, and includes inflow and infiltration into the combined sewer system that drains the diverted watershed and portions of the Des Plaines River Watershed.

The flow chargeable to diversion is determined by taking the total flow measured at the Lockport control structure, adding the diversion flows bypassing Lockport, and deducting all flows which enter the canal system which are not chargeable to diversion. Nonchargeable flows would include estimates of infiltration and runoff from nondiverted watershed areas, domestic pumpage from Indiana and Wisconsin, and domestic ground-water pumpage from Illinois. Nonchargeable flows account for about 10 percent of the total flow measured at Lockport (1).

The computation of the Lake Michigan diversion is compiled from a variety of periodic reports by MSD for the State of Illinois and summarized in the official monthly hydraulic report submitted to the Chicago District USCE for review and approval. Data for some of the parameters used in the accounting process are telemetered directly to MSD Central Control Office as input for the real-time management of water levels in the canal system. These data also provide a basis for MSD to check, in part, the validity of the Lockport powerhouse operator's daily report readings.

The monthly hydraulic report (Figure 4) consists of a 19 column summary of the total flow measured and recorded daily at Lockport, estimates of flow from nondiverted watershed areas, and domestic pumpage which are deducted from the total flow to calculate the total diversion. Domestic pumpage chargeable to the City of Chicago and other surrounding communities diverting Lake Michigan water is then substracted from the total diversion to arrive at the direct diversion and storm runoff flows.

In the following, each element of the accounting procedure will be reviewed in terms of its representativeness as a measure or index, the calibration techniques, computational procedures, and recommendations for improvement.

2.1 LOCKPORT MEASUREMENTS

The discharge measured at Lockport consists of flow through two Kaplan hydraulic turbines, two hydraulic exciter turbines, nine powerhouse sluice gates, lockages, seven Controlling Works sluice gates, and leakage through the various components of the facility.

2.1.1 Turbines (Column 1 of the Hydraulic Report)

About 78 percent of the average annual flow at Lockport (Table 1) is passed through the number one and two turbines (1). The turbines, 127-inch diameter adjustable blade Kaplan runners with vertical axis rated at 8,500 horsepower, were placed in operation in November 1935. The intakes, scrollcases and draft tubes, are constructed on the site of the original numbers 1 and 2 horizontal axis turbines at the west end of the powerhouse.

The discharge ratings for the two turbines are taken to be identical. The rating curves dated January 1936 are based on laboratory model tests performed in 1934 by the S. Morgan Smith Co. A 16-inch diameter model runner was tested under

COL. EXPLANATION

1. Total flow through hydroelectric generators nos. 1 and 2 including leakage(s) through the turbine(s) not in operation.
2. Flow through hydroelectric exciters as computed from rating chart.
3. Total flow through sluice gates at powerhouse as computed from rating curves.
4. Total flow through sluice gates at Controlling Works as computed from rating charts.
5. Leakage through lock gates.
6. Discharge through U.S. government lock as computed from head and area.
7. Total industrial and sludge dilution withdrawals bypassing Powerhouse and Controlling Works.
8. Total flow in the Canal at Lockport.
9. The total amount of domestic pumpage from ground sources in the Lake Michigan watershed, except to the extent that any such ground sources are supplied by infiltration from Lake Michigan, by the State of Illinois and its municipalities, political subdivisions, agencies, and instrumentalities the sewage effluent derived from which reaches the canal.
10. The total amount of domestic pumpage from ground and surface sources outside the Lake Michigan watershed the sewage effluent derived from which reaches the canal.
11. The total amount of domestic pumpage from all sources by municipalities and political subdivisions of the States of Indiana and Wisconsin the sewage effluent derived from which reaches the canal.
12. The total estimated storm runoff and infiltration from the upper Illinois River watershed reaching the canal.
13. Water diverted by Illinois, with the consent of the United States, into Lake Michigan from any source outside the Lake Michigan watershed.

4. Sum of columns 9-13 inclusive.
5. The amount of water diverted into the Sanitary and Ship Canal directly from Lake Michigan and as storm runoff from the Lake Michigan watershed. (Column 8 less column 14)
6. The amounts of domestic pumpage from the lake and any such ground sources supplied by infiltration from Lake Michigan by the state and its municipalities, political subdivisions, agencies, and instrumentalities the sewage and sewage effluent derived from which enters the canal above Lockport.
7. The amount of water diverted into the Sanitary and Ship Canal directly from Lake Michigan and as storm runoff from the lake Michigan Watershed. (Column 15 minus column 16)
8. The amounts of domestic pumpage from the lake by the State and its municipalities, political subdivisions, agencies, and instrumentalities the sewage and sewage effluent derived from which reaches the Illinois waterway, bypassing Lockport.
9. The total amount of water diverted from Lake Michigan by the State of Illinois and its municipalities, political subdivisions, agencies and instrumentalities. (Column 15 plus column 18)

THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO HYDRAULIC REPORT FOR SEPTEMBER, 1980 ALL DATA IN CFS

DAY	TURBINES 1 2	EXCITERS	SLUICE GATES POWER HOUSE	SLUICE GATES CONTR. WORKS	LEAKAGES	LOCKAGES	INDUSTRIAL & OTHER	TOTAL FLOW AT LOCKPORT	DOMESTIC PUMPAGE GRND. SRCE. LAKE MICH. WSHD.	DOMESTIC PUMPAGE OUT SIDE LAKE MICH. WSHD.	IND. & WISC.	STORM RUNOFF & INFILTRATE FROM ILL. RIVER WSHD.	DIV. INTO LAKE MICH	TOTAL DEDUCTIONS	TOTAL THRU LOCKPORT / TOTAL DIVR. ENTR. CANAL	DOM. THRU LOCKPORT / LAKE MICH. PUMP. ENTR. CANAL	DIRECT DIVR. / DIRECT DIVR. & STORM RUNOFF FROM LAKE MICH.	DOM. PUMP FROM LAKE BY ILL. BY-PASS CANL	TOTAL DIVR. / TOTAL DIVR. FROM LAKE MICH. BY ILL.
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
1	3953.9	60.8	3703.3	0.0	113.8	360.3	0.7	8192.6	59.9	47.1	14.7	577.2	0.0	698.8	7473.8	1520.9	5972.9	47.0	7540.8
2	4173.4	58.2	5.2	0.0	118.3	332.0	0.7	4687.9	59.9	47.1	14.7	339.0	0.0	460.6	4227.3	1692.6	2534.7	47.0	4274.3
3	3575.1	39.6	5.2	0.0	115.7	426.9	0.6	4163.1	59.9	47.1	14.7	198.9	0.0	320.5	3842.6	1723.5	2119.1	47.0	3889.6
4	3923.2	32.8	457.6	0.0	105.8	393.9	0.5	4913.6	59.9	47.1	14.7	140.6	0.0	262.2	4651.5	1723.5	2928.0	47.0	4698.5
5	4180.8	33.2	5.2	0.0	116.3	395.9	0.9	4732.2	59.9	47.1	14.7	121.3	0.0	242.9	4489.3	1725.1	2764.2	47.0	4536.3
6	4159.3	33.0	5.2	0.0	128.5	332.1	0.7	4658.9	59.9	47.1	14.7	92.6	0.0	214.2	4444.7	1694.2	2750.5	47.0	4491.7
7	4163.0	35.3	781.7	0.0	117.3	357.0	0.6	5454.9	59.9	47.1	14.7	200.4	0.0	322.0	5133.0	1496.1	3636.9	47.0	5180.0
8	4134.0	57.9	1140.6	0.0	117.2	296.2	1.0	5746.9	59.9	47.1	14.7	300.0	0.0	421.6	5325.4	1664.8	3660.6	47.0	5372.4
9	4119.3	58.7	5.2	0.0	119.7	327.8	0.7	4631.3	59.9	47.1	14.7	221.5	0.0	343.1	4288.3	1644.6	2643.7	47.0	4335.3
10	4117.7	48.5	5.2	0.0	126.2	294.4	0.4	4592.4	59.9	47.1	14.7	190.1	0.0	311.7	4280.7	1616.8	2663.9	47.0	4327.7
11	4154.1	31.1	5.2	0.0	121.2	392.1	0.7	4704.4	59.9	47.1	14.7	155.7	0.0	277.3	4427.2	1626.1	2801.1	47.0	4474.2
12	4165.2	33.5	42.8	0.0	115.2	360.7	0.6	4718.0	59.9	47.1	14.7	195.9	0.0	317.5	4400.5	1604.4	2796.1	47.0	4447.5
13	3932.3	59.2	648.1	0.0	116.3	390.8	0.8	5147.5	59.9	47.1	14.7	130.7	0.0	252.3	4895.2	1575.0	3320.2	47.0	4942.2
14	4313.5	58.7	5.2	0.0	106.1	361.8	0.7	4846.0	59.9	47.1	14.7	88.8	0.0	210.4	4635.7	1477.6	3158.1	47.0	4682.7
15	4104.3	58.6	5.2	0.0	93.9	427.4	0.5	4689.9	59.9	47.1	14.7	135.7	0.0	257.3	4432.6	1564.2	2868.4	47.0	4479.6
16	3954.4	58.2	2719.3	307.5	106.2	354.3	0.5	7500.6	59.9	47.1	14.7	230.9	0.0	352.5	7148.1	1579.7	5568.4	47.0	7195.1
17	4043.5	23.3	3087.4	70.0	109.8	353.6	0.4	7687.9	59.9	47.1	14.7	580.4	0.0	702.0	6986.0	1571.9	5414.1	47.0	7033.0
18	4099.9	11.4	5.2	0.0	110.4	333.1	0.4	4560.4	59.9	47.1	14.7	418.8	0.0	540.4	4020.1	1592.0	2428.1	47.0	4067.1
19	4100.3	11.0	5.2	0.0	109.0	455.0	0.6	4690.1	59.9	47.1	14.7	264.0	0.0	385.6	4304.5	1602.9	2701.6	47.0	4351.5
20	4107.3	30.8	5.2	0.0	115.3	322.1	0.5	4581.1	59.9	47.1	14.7	208.1	0.0	329.7	4251.4	1578.1	2673.3	47.0	4298.4
21	4087.8	31.3	5.2	0.0	108.0	480.4	0.7	4713.3	59.9	47.1	14.7	92.0	0.0	213.6	4499.7	1533.3	2966.4	47.0	4546.7
22	4113.0	32.3	5.2	0.0	114.8	454.4	0.8	4720.5	59.9	47.1	14.7	89.9	0.0	211.5	4509.0	1649.3	2859.7	47.0	4556.0
23	4120.3	31.3	5.2	0.0	110.9	421.2	0.4	4689.2	59.9	47.1	14.7	178.9	0.0	300.5	4388.7	1579.7	2809.0	47.0	4435.7
24	4116.9	32.3	5.2	0.0	116.8	354.4	0.3	4625.9	59.9	47.1	14.7	143.5	0.0	265.1	4360.9	1575.0	2785.9	47.0	4407.9
25	4143.7	32.1	5.2	0.0	119.9	386.0	0.3	4687.2	59.9	47.1	14.7	109.5	0.0	231.1	4456.1	1571.9	2884.2	47.0	4503.1
26	4047.5	32.3	5.2	0.0	128.7	291.4	0.3	4505.4	59.9	47.1	14.7	106.4	0.0	228.0	4277.4	1551.8	2725.6	47.0	4324.4
27	4098.2	32.4	5.2	0.0	105.9	417.0	0.5	4659.3	59.9	47.1	14.7	80.6	0.0	202.2	4457.1	1508.5	2948.6	47.0	4504.1
28	4049.7	32.3	5.2	0.0	111.9	453.2	0.4	4652.7	59.9	47.1	14.7	49.7	0.0	171.3	4481.4	1462.1	3019.3	47.0	4528.4
29	4044.4	32.3	5.2	0.0	112.8	420.0	0.7	4615.4	59.9	47.1	14.7	56.6	0.0	178.2	4437.2	1601.3	2835.9	47.0	4484.2
30	4022.0	31.4	5.2	0.0	113.1	455.4	0.5	4627.6	59.9	47.1	14.7	72.4	0.0	194.0	4433.6	1650.8	2782.8	47.0	4480.6
MEAN	4077.5	38.5	423.2	12.6	114.2	380.1	0.6	5046.6	59.9	47.1	14.7	192.3	0.0	313.9	4732.6	1598.6	3134.0	47.0	4779.6
MAR	2556.5	26.7	27.1	0.0	104.1	337.8	0.3	3052.5	53.5	74.0	13.1	295.9	0.0	436.4	2616.1	1511.3	1104.8	37.6	2653.7
APR	2740.6	29.6	126.7	0.0	108.8	357.6	0.3	3363.6	53.1	73.5	13.0	327.7	0.0	467.4	2896.2	1506.8	1389.5	40.5	2936.8
MAY	2009.3	26.3	331.7	0.0	103.7	394.2	0.4	2865.7	57.6	66.0	14.1	126.1	0.0	263.8	2601.9	1605.9	996.0	36.0	2637.9
JUN	2976.8	33.2	408.6	32.0	109.3	347.6	0.6	3908.1	63.8	62.7	15.6	262.9	0.0	405.0	3503.0	1704.0	1799.1	42.8	3545.8
JUL	3327.0	32.6	302.4	128.5	113.6	429.5	0.8	4334.3	67.6	70.4	16.6	75.9	0.0	230.4	4103.9	2028.8	2075.1	41.1	4145.0
AUG	3408.7	34.1	578.2	31.6	110.4	393.5	0.7	4557.3	63.8	60.8	15.6	179.6	0.0	319.8	4237.4	1811.8	2425.6	47.9	4285.3
SEP	4077.5	38.5	423.2	12.6	114.2	380.1	0.6	5046.6	59.9	47.1	14.7	192.3	0.0	313.9	4732.6	1598.6	3134.0	47.0	4779.6
OCT	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
NOV	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DEC	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
JAN	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
FEB	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
MEAN	3013.8	31.5	314.0	29.3	109.1	377.2	0.5	3875.4	59.9	64.9	14.7	208.6	0.0	348.1	3527.3	1681.0	1846.3	41.8	3569.2

AVE. FROM MARCH 1st TO LAST DAY OF THIS MONTH = 3569 CFS AVE. FROM START OF 5 yr PERIOD THRU LAST DAY OF THIS MONTH = 3096 CFS

Checked by: Date 3/20/81 Approved: Date Jun 3 '81 Approved for State of Illinois by: Date 6/4/81

FIGURE 4

THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

HYDRAULIC REPORT FOR SEPTEMBER, 1980

TABLE 1 AVERAGE ANNUAL FLOW AT LOCKPORT[1/]
(1961 - 1979)

	Total Flow at Lockport						
		Powerhouse		U.S. Gov. Locks		Controlling	Industrial
Year	Turbines	Sluice Gates	Exciters	Lockages	Leakages	Works	Diversions
1961	2719	47	31	379	37	54	144
1962	2850	17	31	390	37	8	140
1963	2752	41	28	365	37	4	132
1964	2714	68	27	369	38	2	125
1965	2657	187	28	366	37	16	122
1966	2624	139	26	377	38	35	124
1967	2629	155	26	382	37	22	125
1968	2637	209	27	371	38	20	120
1969	2759	107	27	355	37	12	103
1970	2631	357	28	353	37	90	26
1971	2023	949	24	363	38	29	26
1972	2857	94	32	365	38	339	25
1973	2486	0	29	383	38	421	25
1974	2245	421	26	363	37	170	20
1975	1863	744	26	379	109	116	23
1976	2290	344	27	352	105	44	22
1977	2154	515	23	341	58	62	16
1978	2251	599	26	292	83	52	7
1979	2548	616	28	319	101	157	1
Avg.	2510	295[2/]	27	361	52	88	70

[1/]Source: Reference 1.
[2/]Zero flow in 1973 not included.

heads from 8 to 10 feet with homologous draft tube and scrollcase included. The manufacturer provided results of the model tests in terms of reduced shaft horsepower and turbine efficiency as functions of reduced speed, gate opening, and turbine blade angles. To use these turbine characteristics for flow measurement, MSD computed a set of turbine rating curves based on the model data. The rating curves give the discharge as a function of gross head and generator output. The gross head is taken as the difference between the measured headwater and tailwater surface elevations. These elevations are measured with tapes attached to floats in stilling wells connected to outside piezometers located in positions homologous to those used in the model. However, the headwater piezometer was placed in the unit 1 intake bay between the trashrack and the scrollcase. Therefore, debris on the trashracks and the associated headloss will cause an error in the headwater reading used for unit 2. The magnitude of the error depends on the operational status of turbine 1 and debris accumulations on the trash racks. The acceptance of the turbines by the MSD was contingent on a field check of the model calibration. Current meter measurements were made in the canal in 1936 to check the turbine discharge rating (2).

2.1.1.1 Computation of Discharge

The turbine discharge is calculated every 30 minutes by the Lockport operator who reads the gross head and the generator output, and determines the flow from the rating curves. The generator output is taken as the reading of the kilowatt meters when the power factor is adjusted to unity by changing the excitation voltage. Periodic corrections are made to create agreement between the kW and the kW-hr meters, neither of which are apparently calibrated, and flows are computed and reported on the basis of the adjusted kW readings.

2.1.1.2 Evaluation of Rating Tables

2.1.1.2.1 Model Tests

The turbine model tests, preparation of the rating tables, and current meter verification tests were conducted in accordance with the state-of-the-art and applicable codes for the period 1934-1936. However, by 1981 standards, the methods used and accuracies obtained can not be considered outstanding and perhaps not even acceptable. For example, the original Moody step-up formula for Kaplan turbine model efficiencies is no longer in general use; it is recognized that this formula tends to predict too high prototype efficiences. The best efficiency of the model, which was 85.9 percent, scales up to 92.7 percent using the original Moody formula as MSD did in preparing the rating tables. The modern version of the Moody formula would give 90.7 percent, and the current practice of various USCE districts of applying only 2/3 of the step-up would compare favorably with the frequently used Hutten step-up formula which would yield about 89.7 percent for the Lockport turbines. Thus there could be an uncertainty in the prototype discharge rating of up to 3 percent due to model test interpretation and changes in the state-of-the-art since 1934.

The discharge rating for the prototype turbine should agree reasonably well with the model-determined rating at the time of installation. However, once in operation, the turbine rating and efficiency can be expected to change gradually over time as the turbine components become worn and tolerances are exceeded. The rate of deterioration of the components depends largely on the abrasive characteristics of the suspended materials transported by the flow and operations under cavitating condition. Hence, it is necessary to restore the eroded and pitted surfaces of the turbine periodically to regain lost efficiency. As a general rule, it is virtually impossible to fully regain lost efficiency through repairs. Insitu restoration by welding and grinding results in irregularities in the profile smoothness, and clearances, of the surfaces. In addition, there are mechanical losses in the bearings and changes in the angular relationship between the wicket gates and turbine blades due to cable stretch and wear in the Kaplan mechanism.

All these changes tend to decrease the turbine efficiency and increase the discharge for a given head and generator output. For example, the effect of age could typically reduce the efficiency to some 82 to 85 percent of its original value, which corresponds to an increase in flow of about 10 percent. For example, a 1/8-inch tip clearance increase (not an unusual magnitude) would pass an additional 5-10 percent of water flow. Since these changes are not random and independent, and tend to concur, one could have a change equal to the sum of the parts, namely a possible maximum of 18-23 percent increase in flow above that reported on the basis of the rating tables.

These changes in turbine efficiency will depend on the frequency and degree of maintenance and repair, and could be expected to differ for the two units. Therefore, the errors made in using a common rating curve is also different for the two units. In fact, the following table of major repairs, supplied by MSD, illustrate the point that the efficiencies of the two units should be expected to differ:

Maintenance and Repair Record
Lockport Kaplan Turbines

1936 Installation

1971 Replaced shaft seal rings on Unit 1

1975 Replaced shaft seal rings on Unit 2

1975 Unit 2: Overhauled wicket gate linkages. Welded cavitation damage on the runner and draft tube liner.

1977 Allis Chalmers rebuilt unit 1, which included removal and repair of the runner and replacement of the draft tube liner.

1980 Overhauled both governors

Therefore, it would be appropriate to use individual headwater piezometers and rating curves for each of the two units.

Wicket gates, when closed, tend to leak, and this leakage flow tends to increase as gate linkages wear. The amount of leakage is not measured nor accounted for in the hydráulic report. As can be seen from the above table, the leakage rates can be expected to be different for the two units. This gate leakage should be evaluated and taken into account during turbine rating tests.

2.1.1.2.2 Generators

The preparation of the rating tables require a knowledge of the hydraulic, mechanical, and electrical efficiencies of the turbo-generator set in order to relate flow and generator output (kW). The hydraulic or turbine efficiency is taken from the model tests. Mechanical efficiencies represent losses through bearing friction and windage losses. These are generally small, but no accounting was found by the Committee. The electrical or generator efficiency is usually fairly accurately known through heat rate tests. The efficiency depends on the power factor, which can be varied by varying the field excitation voltage. The rating curves were developed for a 95 percent power factor, while the turbines have apparently always been operated at 100 percent power factor. These discrepancies introduce an error of less than 1 percent in the flow rating.

The generator output meters (kW) should be periodically calibrated and certainly before turbine rating tests. The Committee found no records that this has been done.

2.1.1.2.3 1936 Test

The description of the 1936 tests, performed by the USCE, Chicago District, is relatively simple and straightforward. A total of nine measurements operating alone, and seven with both running. The current meter measurements were made at the upstream side of Romeo Bridge. One measurement was made each day, usually requiring 4 to 5 hours for completion. The report of the 1936 tests is very brief and contains little detail with respect to the particulars of the field data. Nevertheless, the report reflects a clear understanding of purpose and the hydraulic problems associated with the plan for the field tests.

The results of the discharge measurements indicated that the turbine discharge was about 5 percent larger than indicated by the rating curve. Furthermore, it was noted that the Kaplan mechanism for one of the runners was set to other than the best pitch for maximum efficiency.

2.1.1.2.4 1937 Test

The descriptions of the May 1937 measurements are contained in a series of letters exchanged between the USCE Chicago and Detroit Districts. Thirteen current-meter measurements were made at the Romeo Bridge section used for the 1936 test. The measured discharge, adjusted for channel storage and Lockport leakage was about 3 percent greater than the computed discharge through the turbines.

The fact that measurements (2) were made with blade angles different from normal suggests that the anomaly in blade pitch had been corrected.

2.1.1.2.5 1977 Tests

The current-meter measurements for the 1977 tests were quite elaborate compared to those made in 1936. A series of 23 measurements were made during the period May 24 to June 1. Each consisted of simultaneous current-meter measurements upstream and downstream from the Lockport structure. The upstream data were taken at the Lemont Bridge. The downstream measurements were made in the tailrace of the powerhouse.

The tests and analytical results carried out by the Detroit District USCE are described in the report dated July 1977 (3). The report's findings were inconclusive. The measured flows at both locations were significantly greater than the MSD computed flows for Lockport. The differences that existed between the upstream and downstream measurements, most of which agreed within 5 percent, were attributed to "the influence of various inflows and outflows along the ten-mile reach from Lemont to Lockport, as well as the effects from Brandon Road's lock-filling activity, approximately five miles downstream from Lockport (3)."

The data presented in the report suggests, based on the regression analyses, that the actual flows at Lockport could possibly be some 600 to 750 cfs greater than the calculated flows. However, the conclusion in the report was that further study was necessary to determine the reliability of the USCE measurements.

2.1.1.2.6 1979 Tests

The 1979 testing program (4), also conducted by the Detroit District USCE, during the period August 21 to September 17, was quite similar to the 1977 tests with the following exceptions:

- The location for the upstream measurements was moved to a point approximately one-half mile upstream of the Lockport powerhouse.
- An in-situ current meter was placed in the powerhouse tailrace.
- An electromagnetic current meter was extensively used to establish direction of flow in each measurement panel.

The significant findings of the report for the 1979 tests, dated January 22, 1980 (4), are summarized as follows:

- The upper pool discharge measurements are accurate to ±5 to 6 percent, which are normal acceptable tolerance limits for this type of open-channel stream gaging.

- The powerhouse tailrace discharge measurements are accurate to ±10 percent at best, which is not considered within acceptable tolerance limits.

- From the above two, the upper pool measurements alone should be compared with the calculated outflow at Lockport as supplied by the MSD. The powerhouse tailrace measurements should be used for supporting data only.

- The upper pool measurements generally agree with the calculated outflow as supplied by MSD.

In 1980, the Detroit District (5) reported a comparative analysis of the 1977 and 1979 measurements in an effort to explain the disparity in findings between the two reports. The analysis concluded that there was an insufficient basis for comparison due to a multitude of disparate conditions, and the results from the two studies should not be compared.

2.1.1.2.7 Dye-Dilution Tests

Concurrently, but independent of the 1979 tests by the Detroit District, the USCE Waterways Experiment Station (WES) team conducted a series of measurements during the period August 28-30, 1979. The measurements, using dye dilution techniques, were made on the flow passing through turbines 1 and 2. The report of the tests (6), dated January 16, 1980, concluded that the accuracy of the computed results could not be quantified. This was because nonstandard analytical procedures were necessary when the tests failed to achieve complete mixing of the dye solution.

2.1.1.3 Evaluation of Verification Tests

The report of the 1936 tests is a brief and straightforward account of the field procedures, analytical techniques, and conclusions. The report is essentially devoid of the kinds of data and detail required for a quantitative evaluation of the tests as a verification of the ratings for turbines 1 and 2. Nevertheless, the report reflects a comprehensive understanding of the complex setting for the verification of the turbines. Although not stated explicitly, the report conveys an appreciation and recognition of the unsteady, gradually varied flow problem in the reach below Romeo Bridge and the problems associated with measuring at a site removed from the Lockport facility.

The report points out that the blade angle of the runner and the wicket gate opening setting relationship for one of the new turbines was not set so as to give maximum efficiency. Also pointed out was that the generators of turbines 1 and 2 were operated at 100 percent power factor. However, the discharges were obtained from curves developed for a 95 percent power factor. It was recommended that no allowance be made for this until further measurements were made in the spring of 1937.

The rating curves provided to the Committee are dated January 1936. It is assumed that the currently used rating has not been adjusted for the 100 percent power factor presently used.

This oversight introduces a bias which is of little consequence on a daily or even yearly basis. However, over a 40-year period, this alone would represent a biased error of more than 10 percent of the Court's allowable cumulative algebraic excess of 2,000 cfs-years.

The significance of the less than optimum blade angle-gate setting relation has not been determined, but the reported discharge correction factors for the individual turbine tests were +7.6 and +4.3 percent. The average discharge correction factor, based on nine measurements, was +5.1 percent.

The 1936 tests were state-of-the-art for the time, and the results are considered reliable.

In May 1937, 13 current-meter measurements were made at the Romeo Bridge section. These measurements, after adjustments for channel storage and leakage, indicated flow through the turbines to be about 3 percent greater than the rating curve discharge. A summary table of the measurements provided by USCE noted that all tests were made with generator loads at 100 percent power factor. Also noted was that measurements on May 26 were made with the blade angles 2 degrees flatter, and steeper than normal for comparisons of runner efficiency with blage angles other than normal. Although the comparison analysis was not available, a cursory examination of the summary table indicates an efficiency ranging from 85 to 88 percent for normal blade angle settings and 1 to 4 percent less for the flatter and steeper angles.

The Detroit District USCE reports for the 1977 and 1979 measurements are considered together (3) (4). Both sets of measurements have been examined in some detail by the Detroit District and other interested parties. The data sets have been examined individually and comparatively in hopes of explaining the disparities between the sets and perhaps drawing conclusions based on one, to the exclusion of the other. The Detroit District concluded that there was an insufficient basis for drawing conclusions on the basis of either set.

A "Plan of Study", agreed to by the several parties, is mentioned in the Detroit District's report of the 1979 discharge measurements but is not described in any detail as such. There are indications that the objective of the Detroit District's participation may have been less comprehensive than intended and needed. For example, the introduction for the 1979 report states, "... measure the flow in the

Chicago Sanitary and Ship Canal at Lockport ..." and to compare these measured outflows with those being computed by the Metropolitan Sanitary District ..." (4). The 1977 report contains essentially the same statement in the introduction. The activities described in both reports are consistent with the introductory statements (3). Unfortunately, however, these statements fall short of the objective that was needed; namely, to conduct field tests to verify the discharge-head-kilowatt calibration for turbines 1 and 2.

With an objective to verify the turbine calibration, a great deal of attention should have been given to the assurance of appropriate observation and documentation of the hydraulic and mechanical performance of the turbines, resulting electrical outputs, water levels, leakage, etc., associated with the powerhouse operations. The same importance and attention should have been given to the U.S. Lock and the Controlling Works concerning operations, water level and leakages.

The most serious technical shortcoming of the 1977 and 1979 measurements was the failure to completely deal with the hydraulics of the canal as an unsteady flow problem.

The 1977 report explains the selection of the Lemont Bridge section, 10.4 miles upstream from Lockport, because of "... accessibility and upstream removal from backwater effects, ..." The report concludes that the differences in discharges measured simultaneously at Lockport and Lemont "... can be attributed to the influence of various inflows and outflows along the ten mile reach .... "(3). These influences may be real, but recognizing that an average rate of change in stage of 0.1 foot per hour in the reach has a storage equivalent of about 300 cfs is a more compelling argument.

The upstream measuring site for 1979 was moved to a point 2,500 feet upstream from Lockport. This move reduced the magnitude of storage effects due to unsteady flow, but unfortunately a water-stage recorder at the site was ruled out for lack of a suitable location.

An analysis of the stability of the upper pool is contained in Appendix J of the 1979 report (4). The readings from the Canal West and Penstock gages during measurements in the upper pool are the basis for the analysis. Differences in the gage readings suggested that one of three distinct configurations existed in the water surface immediately upstream of the powerhouse/lock complex. First was a water surface sloping downstream toward the penstocks. Second, a water surface sloping upstream away from the penstocks, and third was a level water surface. Generalizations were made about the water surface configurations, but the anomaly was not explained.

It is interesting to note that of the discharge measurements made when only one of the turbines was running, the water surface sloped downstream when turbine 1 was running, and sloped upstream when turbine 2 was running. This would be consistent with the earlier conclusion that the headwater elevation piezometer is located in the forebay of turbine 1 only.

The upper pool stability analysis concluded that the measurements could be influenced by change in channel storage, but made no adjustments due to the extreme difficulty in quantifying the changes. Paradoxically, adjustments to the cross-sectional properties for the upper pool measurements were made on the basis of interpretation of the Canal West gage readings.

Both studies recognized that the Brandon Road locking operation would affect the tailwater measurements, but neither made use of the water-level record of the lower lock gage at the measuring section. No lockages were made at Brandon Road during the 1979 tests.

The main problem with measuring at the tailwater is the inherent instability of the flow pattern downstream from the abrupt expansion of flow exiting the powerhouse. This would be true even for steady flow through the turbine. However, the turbine discharge was probably varying slightly, but more or less continuously, due to minor fluctuations in head, wicket gate opening, and blade angle caused by electrical load demand changes.

The dye-dilution measurements were unsuccessful primarily because adequate mixing of the dye was not achieved in its passage through the turbine and draft tube. Better results could have been obtained with a multipoint injection and sampling a cross section downstream at a point above the lower end of the lock wall.

The committee agrees that the 1977 and 1979 data do not provide sufficient information to define the discharge-head-kilowatt output rating for turbines 1 and 2. Nevertheless, the open channel current-meter measurement is considered to be an acceptable method for measuring the total flow at Lockport.

Both the USCE North Pacific Division and the TVA have developed and used current-meter systems to verify ratings of turbines of installation and type similar to Lockport. Measurements are made with a horizontal frame upon which is mounted several cosine-component current meters, covering the width of each bulkhead slot at the scrollcase entrance. The frame is lowered to the bottom and raised with constant speed while the current-meter readings are totalized. With appropriate corrections for wall effects, temperature effects, meter calibration, frame interference, and other minor adjustments, accuracies of about 1.5 percent on discharge can be expected (7).

2.1.1.4 Recommendations

The Committee recommends that the two Kaplan turbines be individually rated in the field at periodic intervals (for example every five years) and whenever major system changes occur that may affect the accuracy of the previous rating. For this purpose, each unit should be equipped with its own individual headwater and tailwater piezometers.

The rating could be performed by either of two methods:

- o a traverse of cosine-component current meters in the scrollcase bulkhead slots; or

- o a current meter traverse in the canal.

The cosine-component current-meter method will require the highest capital cost, but probably offers the best long range economy, accuracy, and convenience. Briefly the rating method consists of the following components:

- o a horizontal frame which slides up and down in the scrollcase bulkhead slots and which supports a number (10 to 12) of cosine-component current meters (Ott, Neyrpic or others);

- o hoisting machinery which allows the accurate vertical positioning and constant velocity movement of the current meter frame;

- o individual current-meter rating curves including quantification of temperature effects, wall effects, cosine-component efficiency, and turbulence or frame interference effects;

- o accurate prorating measurements of the dimensions of each of the velocity test planes; and

- o a data reduction or flow integration analysis.

During the rating tests all efforts must be exerted in an attempt to maintain steady conditions of the flow in the canal, the headwater and tailwater surface elevations, and the generator load. To this end, the wicket gates should be blocked in locked position for each gate opening tested. Other gate operations, lockages, and spills should be avoided as far as possible.

A series of measurements must be made of the variables affecting the turbo-generator unit performance and all of these should be made with calibrated instrumentation with recorded calibration factors. These measurements should include: headwater and tailwater elevations, generator output (kW), wicket gate opening, turbine blade angle, power factor, and water temperature.

A preferred alternative to the continued use of the generator output (kW) and head for the determination of discharge would be based solely on the characteristics of the turbine scrollcase. A direct turbine discharge rating requires the installation of Winter-Kennedy piezometer taps in the scrollcase of each turbine. The piezometer tap pressure can be read from a simple water manometer, or preferably a recording and integrating meter, calibrated with discharge.

The principle advantage of the turbine rating (differential pressure and discharge) over the turbine-generator rating (kW, head, and discharge) is that the troublesome time-varient errors characteristic of the latter are avoided. This is because the stability of the turbine rating is primarily dependent on the integrity of the piezometer tap orifice-plate assembly. Another important advantage is that the recommended rating procedure can be simplified considerably.

2.1.2 Exciters (Column 2 of the Hydraulic Report)

The two turbine driven exciters discharge an average of about 27 cfs which accounts for only about 1 percent of the long-range average discharge at Lockport (Table 1). The flow from the exciters is determined from rating curves based on turbine characteristics given by the vendor, and therefore, may suffer from most of the same possible inaccuracies as does the turbine discharge determination. Therefore, a 10 to 20 percent error in flow measurement at the exciters is possible. Moreover, this bias is generally an underestimate of the flow.

The Committee recommends that a stoplog slot current-meter rating be performed for the exciter turbines in a manner similar to that previously recommended for the power turbines.

2.1.3 Lockport Powerhouse Sluice Gates (Column 3 of the Hydraulic Report)

A total of nine sluice gates are utilized in the powerhouse to discharge flows at the Lockport facility when the capacity of the two turbines is exceeded. The sluice gates are located in bays 3, 4 and 7 of the powerhouse. The original hydraulic turbines were removed, and the turbine chambers were structurally modified to close the number 4 draft tube in each bay. Three screw-stemmed sluice gates, each 9 feet wide by 14 feet high, were installed in the upstream stoplog slots of each bay. The sluice gates are an integral part of the flow-regulating scheme for the Lockport facility. Generally, flow is discharged through the sluice gates only when floods are forecasted and during storm periods.

The annual average flow through the sluice gates is 214 cfs or 6.4 percent of the total average annual flow at Lockport (Table 1). The gates are usually opened for short periods of time during flood operations. However, during these periods the rate of flow through the gates ranges from 10 to over 100 times the average annual flow rate for the sluice gates.

The discharge through the sluice gates is computed using tables prepared by MSD on the basis of a scale model study. The tables are entered with the appropriate headwater elevation and gate opening to determine discharge. The discharge rating for the sluice gates has not been verified by field measurements.

2.1.3.1 Model Study

The model study was conducted in the University of Illinois Hydraulic Laboratory by Bruce J. Muga as partial fulfillment towards a Master of Science Degree in Civil Engineering (8). The study was supervised by Professor J. C. Guillou and funded by MSD. Mr. Don Brown, Hydraulic Engineer for MSD was the contact man for the test work and furnished all prototype data.

The laboratory tests were performed in a 1:20 scale model of one bay. The physical dimensions and characteristics of the model were homologous to the prototype except at the entrance. There the trash rack was omitted, and the width of the forebay was limited to the width at the scrollcase entrance and was therefore not representative of the approach flow to the trash racks.

The trash rack was omitted from the model because the investigators could not satisfy the Reynolds number criterion for the rack. However, two approaches could have been taken to fully account for the hydraulic effect of the rack:

- o by performing the model tests with a simulated (not geometrically scaled) model trash rack that would satisfy the Reynolds criterion; or
- o lacking a simulated rack, by analytically accounting for its presence in the reduction and scale-up of the model data.

The principal investigator offered no explanation for limiting the forebay width in the model to the width of the turbine chamber. He noted that the single-bay model could not determine the effect of operating adjacent bays concurrently, but because the width of the model forebay was limited, the simultaneous operation of adjacent chambers approaches the model condition more closely than the operation of a single chamber. Failure to properly model the full width of the forebay and the entrance geometry raises doubt as to the applicability of the model data to the prototype without field verification. More specifically, this doubt stems from the following considerations:

- o The round-nosed piers separating each bay extend some 15 to 20 feet upstream from the sluice gate section and forms a width constriction between forebay and turbine chamber. The effect of the constriction on the discharge coefficient, which is not reflected in the model, depends on stage and configuration of the sluice gate and turbine operations.

- Whenever floods are forecast for the watershed, the sluices are operated to provide a greater hydraulic gradient and additional storage by lowering the water level in the canal system. Drawdown is usually accomplished by increasing the discharge at Lockport by opening the powerhouse sluice gates. When this takes place, the forebay may be drawn down to an elevation of -12 ft City of Chicago Datum (CCD). The lip of the sluice gate is at elevation -14.4 ft CCD and recessed in the roof of the forebay. This results in a h/b ratio of only 1.17 of head to gate opening. It is virtually certain then that a free-water surface condition develops under the sluice gates and flow control shifts upstream to the section formed by the leading edge of the large round-nosed piers. This phenomenon will likely occur when the head to gate opening ratio falls below a value of 1.5, which corresponds to an upstream water-level elevation of -7.5 ft CCD. When the elevation falls below this value, the model-determined tables are technically inappropriate.

- The steel framing on the leading edge of the interior piers between sluice gates, which supported the original turbine bay gates, is still in place. The framing has a predictable, but negligible effect, which is not accounted for in the model study.

The above factors plus the absence of trash racks in the model combine to produce a bias in the rating tables for the prototype. Each factor tends to increase energy losses and therefore, results in reported discharges being greater than actual.

2.1.3.2 Rating Analysis

The results of the Muga analysis are expressed in prototype units and presented in graphical and tabular format as is commonly done in model testing, but the report does not contain the measurements and observations of the individual tests in model scale units.

The head-discharge relation for each model sluice gate is defined at each quarter-open position from closed to fully open with all possible combinations of quarter-open positions of the other two gates. The discharge characteristics of each combination was defined for headwater elevations, ranging from 0 to -10 feet, and for tailwater elevations between -33 and -39 feet, CCD.

From these data, MSD adopted an operation schedule in which the sluice gates are either fully open or fully closed. Therefore, the number of operational gate opening combinations for the three gates in each bay, A, B, and C, is reduced from over 100 to only 8, and these are shown in Table 2.

2.1.3.3 Rating Tables

The MSD has developed rating tables from the Muga report which are used for determining flow through the sluice gates for operational and accounting procedures.

The rating table for each of the gate combination options and corresponding data from the Muga study are given in skeleton form in Tables 3 and 4. A comparison of the curves with the source data reveals differences that are usually minor, but nevertheless unexplained.

For the case of a single gate fully open, the rating table for option 2 is used. Options 1 and 3 are not used in the sluice gate operations. The rating table

TABLE 2
LOCKPORT POWERHOUSE SLUICE GATES
OPERATIONAL GATE OPENING COMBINATIONS[1]

Gate Combination Options	Gates A	Gates B	Gates C
1	Open	Closed	Closed
2	Closed	Open	Closed
3	Closed	Closed	Open
4	Open	Open	Closed
5	Closed	Open	Open
6	Open	Closed	Open
7	Open	Open	Open
8	Closed	Closed	Closed

1/Source: Reference 8.

TABLE 3
LOCKPORT POWERHOUSE SLUICE GATES
MSD RATING TABLES
GATE COMBINATION OPTION 2[1]

	Discharge, cfs			
		Muga Curve		
Canal Elev CCD	Rating Table (option 2)	B-515[2] B open (option 2)	B-155[2] A open (option 1)	B-552[2] C open (option 3)
0	2865	2880	2760	2800
-2	2735	2700	2630	2670
-4	2560	2530	2490	2500
-6	2370	2360	2300	2270
-8	2170	2170	2040	2050
-10	1965	1979	1740	1800

1/Source: Reference 8.
2/Notational code used by Muga (8). The code B-552 is apparently in error and probably should have been B-551.

TABLE 4

LOCKPORT POWERHOUSE SLUICE GATES
MSD RATING TABLES
GATE COMBINATION OPTIONS 4, 5, and 6[1]

	Discharge, cfs				
		Curve		Curve	
Canal Elev CCD	Rating table (option 6)	B-151[2] A & C open (option 6)	Rating table (options 4 & 5)	B-115[2] A & B open (option 4)	B-511[2] B & C open (option 5)
0	5845	5800	5710	5600	5700
-2	5630	5600	5440	5370	5440
-4	5320	5280	5130	5060	5110
-6	4940	4890	4770	4700	4760
-8	4495	4460	4400	4310	4390
-10	3990	4000	4000	3900	4000

1/Source: Reference 8.
2/Notational code used by MUGA (9). The code B-552 is apparently in error and probably should have been B-551.

for option 2 and the corresponding values for curves in the Muga study are given in Table 3.

The slightly higher discharge of option 2 over options 1 and 3 is probably due to the symmetrical alignment of the open gate with the draft-tube openings and webs. The expected discharge for options 1 and 3 in the model should be identical.

The ratings for options 4, 5, and 6 are shown in Table 4.

The slightly higher discharge for the option 6 rating is due to the symmetrical alignment of the gates with respect to the draft-tube openings.

When all three gates of a bay are open, the rating shown in Table 5 is used. In the model study, the headwater-discharge relation for three gates fully open was affected by variations in the tailwater elevations. Values corresponding to Muga curves for selected tailwater elevations are shown.

Although the discharges in the rating tables for options 2 and 6 are generally higher than the Muga curves, the difference is less than 1 percent. However, for all three gates open, the Muga data clearly indicates that the discharge is influenced by the tailwater elevation. For reasons not apparent, a rating table is used which does not account for the influence of tailwater. As can be seen in Table 5 for option 7, the rating table values correspond to a Muga tailwater of about -38 feet.

The tailwater effects defined by Muga are consistent with the fact that the cross-sectional flow area provided by three sluice gates fully open is greater than the minimum cross-sectional area of the draft-tube opening.

TABLE 5
LOCKPORT POWERHOUSE SLUICE GATES
MSD RATING TABLES
GATE COMBINATION OPTION 7[1]

Canal Elev CCD	Rating Table (option 7)	Discharge (cfs) Muga Curve B-111			
		TW39[2]	TW37[2]	TW35[2]	TW33[2]
0	8385	8470	8400	8160	7950
-2	8250	8360	8240	8000	7800
-4	8035	8100	8000	7770	7600
-6	7710	7740	7680	7490	7390
-8	7260	7300	7250	7170	7040
-10	6735	6750	6740	6700	6630

[1]Source: Reference 8.

[2]Tail-water gage height.

2.1.3.4 Conclusions

The exclusion of the trash rack and a representative forebay in the model study introduces a bias in the computed discharge ratings for the prototype gates. As a result, the discharge indicated by the rating table could be from 0 to perhaps 15 percent greater than the actual flow when one or two gates are open and the headwater elevation is higher than -7.4 feet, CCD.

When three gates are open and the headwater is lower than -7.4 feet, the magnitude of the error varies with the tailwater elevation. The prediction of the error for this condition is complicated by virtue of the fact that the rating assumes fairly constant tailwater elevation.

2.1.3.5 Recommendations

Field verification of the ratings for flow through the powerhouse sluice gates is not recommended. Field verification would require the establishment of a range of steady flow conditions over a range of water surface slopes to represent the headwater and tailwater elevations, and gate opening conditions encountered during the operational phase. This would entail a dedication of flow in the whole canal system for the verification effort to the exclusion of other operational interests. Considering the small percentage of the annual flow discharged through the sluice gates, the time, manpower, cost, water supply, and operational inconveniences required to conduct adequate field verification measurements would be difficult to justify.

It is recommended, however, that as a minimum the following steps be taken:

- o Reanalyze the data collected in the Muga model study giving due consideration to the deficiencies in the model study. This would include: (a) adjustments for head losses due to the presence of the trash rack in the prototype based on the experimental work of Kirschmer (9), Spangler (10), and Fellenius (11), and others; (b) adjustments for head losses due to the width contraction formed by the forebay and turbine chamber geometry; (c) determination of flow controls for the range of headwater, gate opening, and tailwater based on the model data, theoretical considerations and the considerable experimental work cited in the literature; and (d) development of theoretical ratings for those conditions where the Muga ratings are inappropriate.

- Consider the development of new operating rules for the powerhouse sluice gates using partial opening of gates. Expanding the gate opening combination options will provide a means to minimize the impact of uncertainties in the application of the model data to the Lockport sluice gates.

The Committee feels that, if the recommended analytically generated new rating curves are inconclusive, the only means to resolve the rating uncertainties would be to perform a new scale model study. The model study should be performed by a well-known, experienced hydraulic laboratory and take into consideration the discussions made herein.

2.1.3.6 Leakage

The sluice gates are fitted with J-type rubber seals on the sides and a 4 inch x 6 inch timber on the bottom of each gate to minimize leakage. Periodically, measurements are made to determine the leakage discharge as part of the continuing effort to improve the accuracy of reporting diversion.

2.1.3.6.1 Leakage Tests

Measurements of leakage through the sluice gates are made by volumetric techniques after sealing the three draft-tube openings with water-tight bulkheads.

Each chamber is reported to have a cross-sectional area of 2,034 square feet which is constant with depth. As the water rises in the chamber, water-level increments of 1-foot are read from a staff gage and the time is noted to the nearest second. The staff is set to an arbitrary datum.

A minimum of three water-level increments are observed and timed. The leakage discharge is computed by dividing the water volume for a 1-foot increment by the average of the times taken for the 1-foot incremental changes.

2.1.3.6.2 Evaluation

The most recent leakage tests were made February 28, March 1, and June 13, 1979, for pits nos. 3, 4, and 7, respectively. The report of the test concludes:

- Leakages of 3.2, 4.4, and 4.8 cfs, respectively, occur through the sluice gates of pit nos. 3, 4, and 7.

- The test data showed no unusual dispersion or scatter.

- The variations of head causing the leakage, due to normal operating conditions in the canal, was assumed to be negligible.

The analytical procedure using the average time interval for a 1-foot rise in the water level in the chamber tends to underestimate the leakage rate. If, in fact, the leakage occurs near the bottom of the gates, then the procedural error will increase as the number of water-level increments is increased.

Leakage through the sluice gate will vary with the head in the canal and will function hydraulically as an orifice with a free jet discharge. However, when the draft-tube openings are blocked, water rises in the chamber creating a submerged orifice effect. Leakage is a function of the differential head and will decrease as the water level in the chamber increases. Consequently, the discharge should be computed for each time-volume increment and plotted against depth. This is illustrated in Figure 5 where the 1979 test data have been plotted.

Extrapolation of eye-fitted straight lines to zero depth indicate the leakage rates for pits 4 and 7 were probably 5 to 10 percent greater than reported. This of course depends on the staff gage datum with respect to the chamber floor.

Equally important, the plot clearly indicates something is amiss with the test for pit no. 3. It appears that the leakage rate increased during the third increment (5 to 6 feet). In any event, it seems probable that the leakage through the sluice gates in pit no. 3 was in the range of 4 to 5 cfs rather than 3.2 cfs.

On the basis of this graphical interpretation, the total leakage through the sluice gates was about 15 percent greater than reported for the 1979 tests.

2.1.3.6.3 Recommendations

- The field procedures should specify that the datum of the staff gage for measuring depth of water in the chamber be set equal to the gate sill elevation.

- The leakage rate should reflect operational conditions rather than a contrived test condition. Consequently, the analytical

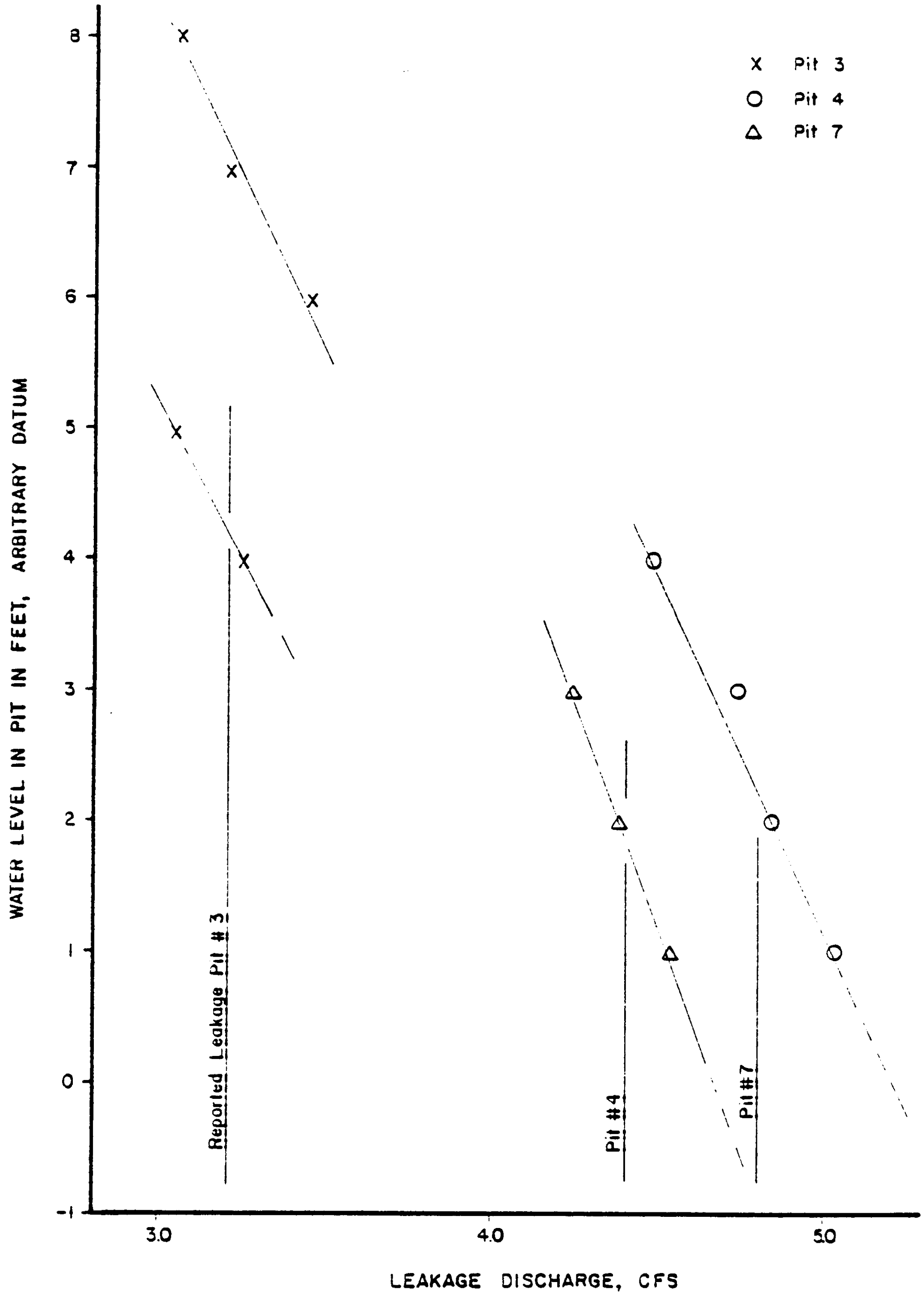

FIGURE 5 LOCKPORT POWERHOUSE SLUICE GATE LEAKAGE

procedures should be consistent with the hydraulic characteristics of the leakage through the gate. The leakage rate should be computed for each incremental depth with proper account for the correct differential head.

- The most important aspect of accounting for leakage through the powerhouse sluice gates is to ensure that the gates are properly closed at the conclusion of sluicing operations. Because of the usually heavy debris load during floods, it is recommended that the sluice gates be inspected following each use to make certain that no debris is preventing proper sealing.

- In the accounting procedures, the results of leakage tests become effective immediately and are used until subsequent tests are made. This procedure will lead to consistent bias error if the leakage rate is changing monotonically with time. (This fact is discussed in more detail under lock leakage paragraph 2.1.5.2). Therefore, it is recommended that the calculations be altered to adjust the leakage rate values reported between leak tests each time a new leak rate value is obtained. A linear interpolation scheme will suffice.

- It is also recommended that semi-annual inspections of the sluices be made to visually ascertain changes in their condition and in the leakage rate. Written inspection reports should be filed and used to determine the need for new leak rate tests.

2.1.4 Lockport Controlling Works - (Column 4 of the Hydraulic Report)

The average annual discharge through the Controlling Works is about 88 cfs or 2.6 percent of the total flow at Lockport (Table 1). The Controlling Works are located on the right bank of the canal about 2 miles upstream from the Lockport powerhouse. The Controlling Works structure, built in 1900, consists of seven sluice gates which are 30 feet wide by 16 feet high. During floods, the Controlling Works gates can be opened to divert flow from the Sanitary and Ship Canal into the Des Plaines River. In the operational mode, gates are raised one at a time to the fully opened position or else clear of the water surface (12).

2.1.4.1 Discharge Rating

The computation of discharge through each gate is based on a rating prepared by MSD and revised in 1947 (13). The information available to the Committee does not describe the development of the rating. The supporting computations suggest that the rating was developed on the basis of a special application of the general orifice equation and the principle of specific energy. Accordingly, the discharge for each gate opening is:

$$Q = C_1 A \sqrt{2\,g\,(H)} \tag{1}$$

The MSD computation substitutes V_H for (H) in eq. (1):

$$Q = C_2 A \sqrt{2g\,V_H} \tag{2}$$

which is appropriate for the special case when the orifice is submerged. For the Controlling Works, A is the product of the gate-opening width, b, and the depth of flow, d, at some section along the gate sill rather than the area of the gate opening. Implicitly C_1 and C_2 are not equal since H is the upstream energy head and V_H is the velocity head at depth, d, on the gate sill.

A trial-and-error procedure was used to compute discharge assuming various combinations of depth and velocity head, whose sum equals the total head, h, which is the elevation of the water surface minus the elevation of the gate sill at the section where the depth, d, is taken. The computed discharges for each value of the total head, h, were plotted against the assumed velocity heads. A smooth curve was fitted to the points to determine the velocity head (and depth) yielding the maximum discharge. This procedure was repeated for selected head values throughout the range of water levels expected in the canal.

Apparently the revision of the rating in 1947 used the same trial-and-error technique that was used earlier except that:

- the coefficient C_2 was valued at 0.9 rather than 0.86; and

- for reasons unexplained, the coefficient was varied from 0.9 to 0.97 in the range of canal elevations from -8.0 to -10.0 feet CCD.

Documentation explaining the revision is not available. However, it is presumed that the changes in C_2 were made on the basis of the previously mentioned 1947 field measurement. The computation technique has some legitimacy, but its reliability depends on the validity of C_2. In the absence of supporting evidence, the basis for the coefficient is a matter of speculation.

The use of a trial-and-error computational procedure is rather circuitous considering that the fundamental hydraulic principle of minimum specific energy provides a direct solution. In the case of the Controlling Works rating, this simply means that the maximum discharge for a given specific energy is assumed to occur when the velocity head is equal to half the hydraulic depth where:

$$h = d + \frac{d}{2} \tag{3}$$

and

$$h = d + \frac{v^2}{2g} \tag{4}$$

since $$v = \sqrt{gd} \tag{5}$$

then $$q = dv \tag{6}$$

where q is the discharge per unit width. Values of maximum discharge, q, and the canal water level, represented by h, and v as velocity at depth d can be computed for each assumed depth. Equation 3 is rearranged and substituted in equation 2 to provide a direct solution of discharge for each assumed canal level:

$$Q = C_2 b \frac{2h}{3} \sqrt{2g \frac{h}{3}} \tag{7}$$

2.1.4.2 Rating Evaluations

The reliability of the rating is dependent on several considerations:

- o The engineering basis for the coefficient C_1. The Committee was unable to secure information concerning the basis or verification of the coefficient.
- o The reference elevation for the rating is -16.0 feet CCD, at the downstream sill for the gate opening. This is about 1 foot lower than the upstream sill.
- o The canal bed in the vicinity of the structure is indicated to be approximately the same as the upstream sill.
- o The influence of the large mooring piers on flow approaching the gates has not been determined.
- o Possible submergence effects from the Des Plaines connecting channel tailwater are unknown.

- The influence of multigate configuration on the discharge characteristics of a single gate has not been determined.

- The constancy of the Controlling Works canal gage as an indicator of the effective head for the various gate-opening configurations and water levels in the canal is unknown.

A more acceptable approach to the computation of flow through the Controlling Works would address the hydraulic geometry characteristics of the structure in a straightforward manner.

The sluice gates are normally operated in a fully raised position and the structure functions as either a sluice gate or a broad-crested weir. If the canal water surface is in contact with the face of the gate, flow is controlled by the submerged portion of the gate. Discharge would be computed as flow through sluice gates.

The indicated operating rules for Lockport call for the powerhouse gates to be opened before opening the Controlling Works gates. This suggests that the Controlling Works gates function as sluices only during the few minutes required to raise the gate free of the water. After that point in time, discharge is controlled by the gate sills and abutment piers. The structure would function as a broad-crested weir with rounded end contractions. Weir flow would be computed using the discharge equation:

$$Q = Cb\, H^{3/2} \tag{8}$$

where C is a coefficient, b is the weir width, and H is the total energy head referred to the weir crest.

The discharge coefficient in equation 8 is described as a function of the following dimensionless ratios:

$$C = f\left(\frac{h}{L}, \frac{R}{h}, \varepsilon_1, \varepsilon_2, \frac{b}{B}, \frac{r}{b}, \frac{h_3}{h}\right) \tag{9}$$

where h and h_3 are the upstream and downstream depths referred to the weir crest, L is the length of the weir, R is the radius of rounding of the upstream face, r is the radius of rounding of the side abutments, B is the effective width of the approach flow, ε_1 and ε_2 are the slopes of the upstream and downstream faces of the weir.

Implicit in the consideration of the structure as a broad-crested weir is that the gate seats are raised above the elevation of the canal bed. This may not be the case. A sketch, Figure 6, from a drawing of the structure, believed to have been part of the construction plans, indicates gate seat and canal bed to be at the same elevation. For this case, the structure would function hydraulically as a contracted terminal sill of zero height.

A cursory examination of the discharge coefficient for a low broad-crested weir 9 feet in length indicates values ranging from about 3.6-3.4 to 3.2-2.7 with a lowering of the water level in the canal from 0 to -10 feet, CCD. These values, for simplicity, ignore the possible effects of canal velocities, the mooring piers, and tailwater submergence. However, it seems likely that the water level in the canal would already be lowered considerably before the Controlling Works sluice gates are opened during a drawdown operation. Consequently the upper limits of the discharge coefficient are more likely to be 3.3-3.0 rather than 3.6-3.4.

If, on the other hand, as suggested by Figure 6, the structure indeed acts as sill of zero height the discharge coefficient is no longer a function of the parameters $\frac{h}{L}$, $\frac{R}{h}$, ε_1 and ε_2 in equation 9 above. Its limiting value would be 3.09 before adjustments for the remaining parameters.

For the special case of critical state of flow it can be shown that the coefficient in equations 7 and 8 is equal to 3.09. This is equivalent to a value of 1.0

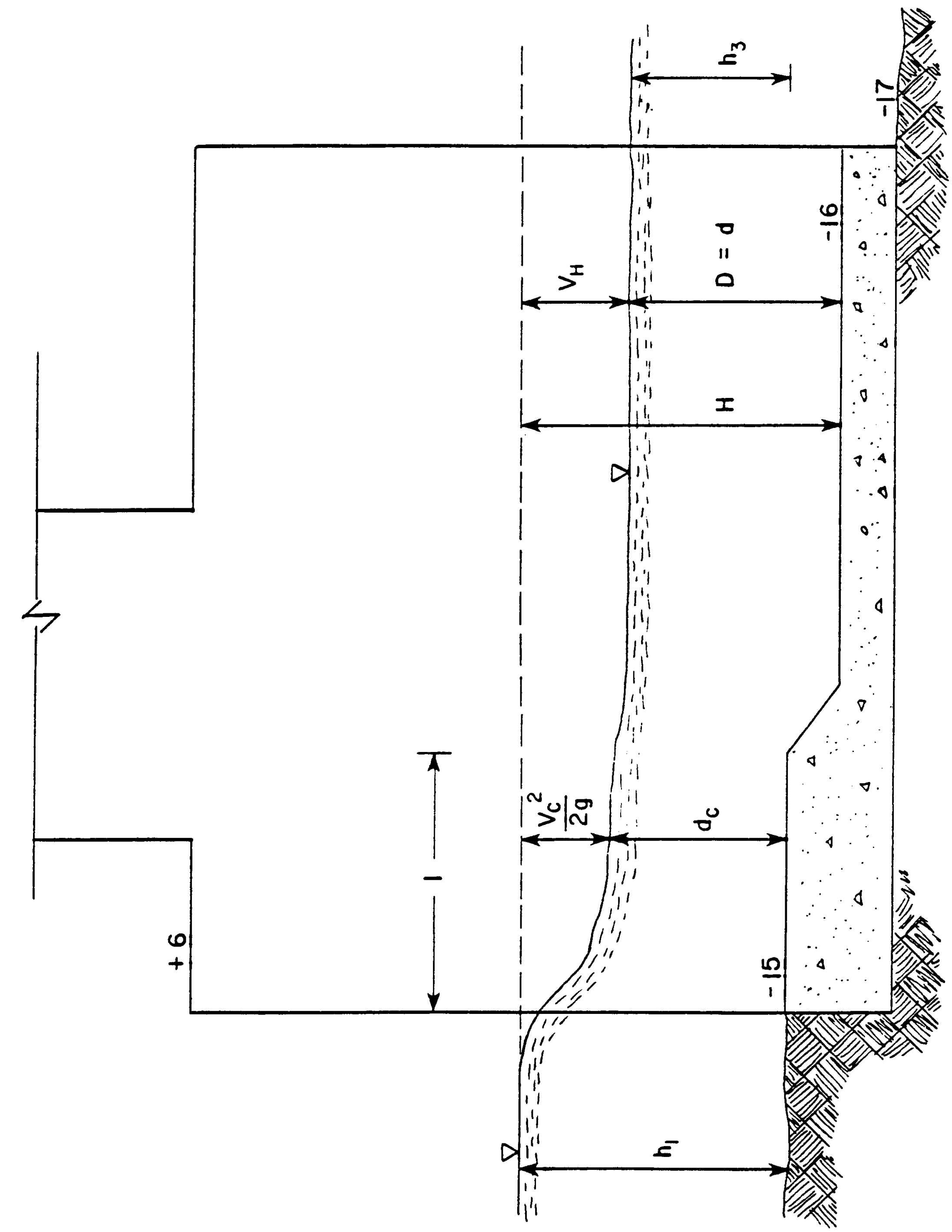

FIGURE 6 LOCKPORT CONTROLLING WORKS - GATE SEAT AND CANAL BED DEFINITION SKETCH (DATUM, FEET, CCD)

for the coefficient in equation 2. Conversely, the equivalent broad-crested weir coefficient (equation 8) used for the 1947 rating (equation 1) is 2.78. This suggests that discharge determined from the 1947 rating could possibly be on the order of 10 percent or more too low.

The Committee observed flow through gates 6 and 7 during its inspection of the structure on July 8, 1981. On the basis of this limited observation the Committee is of the opinion that the reliability of the computed flow through the structure is dependent upon a better definition of the approach conditions in the canal and possible tailwater submergence in the connecting channel to the Des Plaines River.

2.1.4.3 Computation of Discharge

Discharge through individual gates is determined by applying recorded water levels to a rating table at 30-minute intervals. The discharges are summed for the time periods in which flow occurred, and averaged for the entire day.

The procedure for computing discharge during the time when the gate is in the process of opening and closing should be examined for the purpose of eliminating any systematic errors.

2.1.4.4 Leakages

The Committee observed various leakages at the Lockport Controlling Works during two visits. During the first visit significant leakage was observed under gate number 7, and other gates were leaking, but somewhat less. During a subsequent visit, no significant leaks were observed. Leakage at these gates is not accounted for in the diversion procedures.

2.1.4.5 Recommendations

- The discharge rating should be completely reanalyzed on the basis of the existing structural features, hydraulic characteristics, and operating rules. This will require a field survey to provide:

  - geometry of the gate sills and canal at the entrance to the structure;
  - location of the mooring piers;
  - canal depths in the approach area; and
  - geometry of the downstream connecting channels to the Des Plaines River.

- Field observations and water level measurements should be made during discharge operations to assess:

  - the adequacy of the existing gages for the purpose of defining the effective head;
  - the effect of the mooring piers on the head and approach flow;
  - the effect of submergence on the rating; and
  - the effect of gate-opening configuration.

- The Controlling Works gates should be inspected following each use to make certain that the gates are properly closed and that no debris is preventing proper sealing. The inspector may have to return to the Controlling Works some time after the cessation of operation when the tailwater conditions are such that a meaningful leakage observation can be made. The written daily log should reflect each gate operation including attestation of inspection for leakage.

- Any consideration for a model study should be reviewed with respect to the results of the engineering analysis. The feasibility of a model study seems unlikely because of the relatively small percentage of the annual flow discharged through the structure, model cost, and an uncertain submergence effect.

2.1.5 U.S. Government Locks (Column 5 and 6 of the Hydraulic Report)

2.1.5.1 Lockages

The average annual discharge through the lock operation for the period 1961-74 is 370 cfs which is about 11 percent of the total flow at Lockport (Table 1).

The lockage discharge is computed on the basis of information provided to MSD by the USCE lockmaster. The lockmaster's daily log contains the time of day for each filling and emptying of the lock and corresponding readings of the headwater and tailwater gage recorders.

The volume of water discharged each time the lock is emptied from the 110 feet x 600 feet chamber is computed as the product of the cross-sectional area (74,725 square feet) and the difference in water level reflected by the headwater and tailwater gages. The volumes are summed for each day's lockages and converted to equivalent cfs-day.

The techniques for measurement and computation of lockages are satisfactory. However, supporting computations for the lock area should be documented. Also, steps should be taken to assure that water-level gages function properly.

2.1.5.2 Leakages

Leakage at the lock accounts for about 1 percent of the total average annual flow at Lockport (Table 1), or about 37 cfs. Leakage problems arise from the gradual or sudden deterioration of the sealers for the sides, sills, and miter edges of the lock and culvert gates. Normally, the leakage rate can be expected to increase with time or otherwise remain constant. Of course, the leakage rates can vary following each lockage depending on the compression pressures on the gates. As a general case, leakage will be concentrated along the sill and lower edges where the hydrostatic pressure is greatest.

Leakage flow rates have been measured on a semiannual basis for at least the past several years.

2.1.5.2.1 Leakage Tests

A leakage test usually consists of four volumetric measurements in which the change in lock-chamber water level is measured with time. Two measurements are made with the lock filled to the headwater level and two with the lock emptied to the tailwater level.

The USCE advises that each measurement requires from 15 to 20 minutes. Water-level change in the lock is determined by means of a portable platform equipped with a float-cable counterweight assembly. The platform is mounted astride the upstream stop-log slot for the lock emptying gate in the west wall (river wall). The platform accommodates a vertical scale backing for the float cable. The position of the float-level indicator on the cable is marked by hand on the scale backing at the beginning of the measurement and at selected time intervals during the measurement period. For example, during the test of August 5, 1981, the water level was recorded at 0, 2, 4, 5, 10, 15 and 20 minutes during the initial measurement and 0, 5, 10, and 15 minutes for the second measurement for each lock condition.

The leakage rates reported are the average of the two measurements for each condition, lock full or lock empty. The leakage tests are reported as "Results of Leakage Tests at U.S. Lock, Lockport" (14).

2.1.5.2.2 Evaluation of Leakage Tests

The report of the leakage tests do not provide enough information for a rigorous evaluation. However, it appears that the field techniques and analytical procedures should be rated marginal. From an examination of the leakage reports, one may conclude that the field measurements and reporting of same are carried out in a somewhat perfunctory manner. Sometimes even the minimal procedural requirements for the tests are ignored. In some cases, only a single measurement was made or one of the two measurements would be discarded but not repeated. The time periods for individual measurements ranged from 10 to 30 minutes for observation of water-level changes of 0.3 to 2.8 feet.

The average difference in measured leakage for all two-measurement tests reported during the period 1977 to 1981 was 10 percent for full lock conditions, with differences between measurements for a single test ranging from 0 to 36 percent. The average difference for empty lock leakage was 4 percent for the period, with a 1 to 11 percent difference in measurements of the same test.

Computations supporting the analysis of the leakage test data are not available. However, it appears that the task of determination of leakage rates was reduced to a simple form that ignores:

- o leakage during the leakage test measurements; and
- o the variations and differences in water levels upstream and downstream during and between measurements.

A cursory examination of the August 1981 test data suggests the leakage rate for the lock-full condition may be underestimated, because of these omissions, by as much as 75 percent. Similarly, the leakage rate for the lock-empty condition would be underestimated by about 15 percent. In addition the leakage analysis ignores the fact that the leakage rate varies in some manner with the square root of the head acting on the gate.

Overall, the lock leakage tests with respect to field procedures, analytical techniques, and overall quality assurance, are not considered to be representative of acceptable engineering practice.

2.1.5.2.3 Lock Leakage Accounting

The computation of lock leakage discharge is based on the following assumptions:

- the lock-empty and lock-full leakage rates are constant between leakage tests;
- leakage is continuous and alternates between the two rates with each lockage operation; and
- the leakage rates are independent of the headwater and tailwater elevations.

Leakage rates become effective on the day determined and remain in effect as constant values until a subsequent test is made. This procedure is appropriate provided there is equal probability that future leakage rates will decrease as well as increase. Intuitively, the leakage rate is expected to increase with time.

The practice of assuming a constant leakage rate between tests for the purpose of daily accounting is acceptable provided annual adjustments are made to the leakage discharge to account for errors in the assumption. In the absence of evidence to the contrary, the most reasonable assumption is that the leakage rate varies linearly between tests.

Leakage test data are plotted in Figure 7 to illustrate the assumed constant rates. The dashed lines reflect the gradually varied rate assumption. The areas defined by the solid and dashed lines, when summed and divided by two, would be an estimate of the volume difference in the assumptions. This is also an indication of the bias in the constant rate assumption.

A comparison of these assumed leakage rates is made in Table 6. For simplicity, the total times for the lock full and empty are assumed to be equal.

On an annual basis, the error is insignificant, but if it is a biased error, it should not be ignored. For example, if the data for the 3½-year period is extrapolated to the 40-year period specified by the decree, the cumulative underestimated leakage would be 172 cfs-years. The apparent error, considered to be biased, amounts to almost 10 percent of the maximum cumulative 2,000 cfs-years excess permitted by the Decree.

Errors resulting from the current practice of assuming a constant leakage rate can be easily corrected by an adjustment to the annual average leakage discharge.

Leakage is also assumed to be continuous with time. The computation procedure is based on an assumption that the leakage rate changes instantaneously to the alternate rate at the beginning of each lockage operation.

A lockage operation, either upstream or downstream, begins when the second of the lock gates is closed and the lock chamber filling, or emptying is

TABLE 6 COMPARISON OF CONSTANT LEAKAGE DISCHARGE WITH AVERAGED LEAKAGE - LOCKPORT LOCK

Test Date (1)	Time Period (Yrs) (2)	Test Leakage (cfs) (3)	Constant Leakage (cfs) (4)	Constant Leakage for Period (cfs-yr) (5) (2x4)	Average Leakage (cfs) (6) (3+4)/2	Average Leakage for Period (cfs-yr) (7) (2x6)	Cumulative Difference (cfs-yr) (8) (5-7)
				LEAKAGE DISCHARGE			
				LOCK FULL			
1977							
21 Oct	-	30.7					
1978							
21 Apr	.5	39.9	30.7	15.35	35.30	17.65	-2.30
27 Oct	.5	38.3	39.9	19.95	39.10	19.55	+0.40
1979							
27 Apr	.5	34.9	38.3	19.15	36.60	18.80	+0.35
31 Aug	.33	47	34.9	11.52	40.95	13.51	-1.99
1980							
1 May	.67	40.0	47	31.49	43.50	29.15	+2.34
2 Oct	.42	67.6	40	16.67	53.80	22.60	-5.93
1981							
3 Apr	.42	92.9	67.6	28.39	80.25	33.7	-5.32
		CUMULATIVE DIFFERENCE FOR LOCK FULL					-12.45
				LOCK EMPTY			
1977							
21 Oct	-	105.5					
1978							
21 Apr	.5	136.8	105.5	52.75	121.15	60.58	-7.83
27 Oct	.5	144.0	136.8	68.40	140.40	70.20	-1.80
1979							
27 Apr	.5	144.6	144.0	72.0	144.30	72.16	-0.16
31 Aug	.33	148	144.6	47.72	146.30	48.28	-0.56
1980							
1 May	.67	166.6	148	99.16	157.30	105.39	-6.23
2 Oct	.42	168.3	166.6	69.97	167.45	70.33	-0.36
1981							
3 Apr	.42	171.6	168.3	70.69	169.95	71.38	-0.69
		CUMULATIVE DIFFERENCE FOR LOCK EMPTY					-17.63
		AVERAGE CUMULATIVE DIFFERENCE FOR 3½ YEARS					-15.04

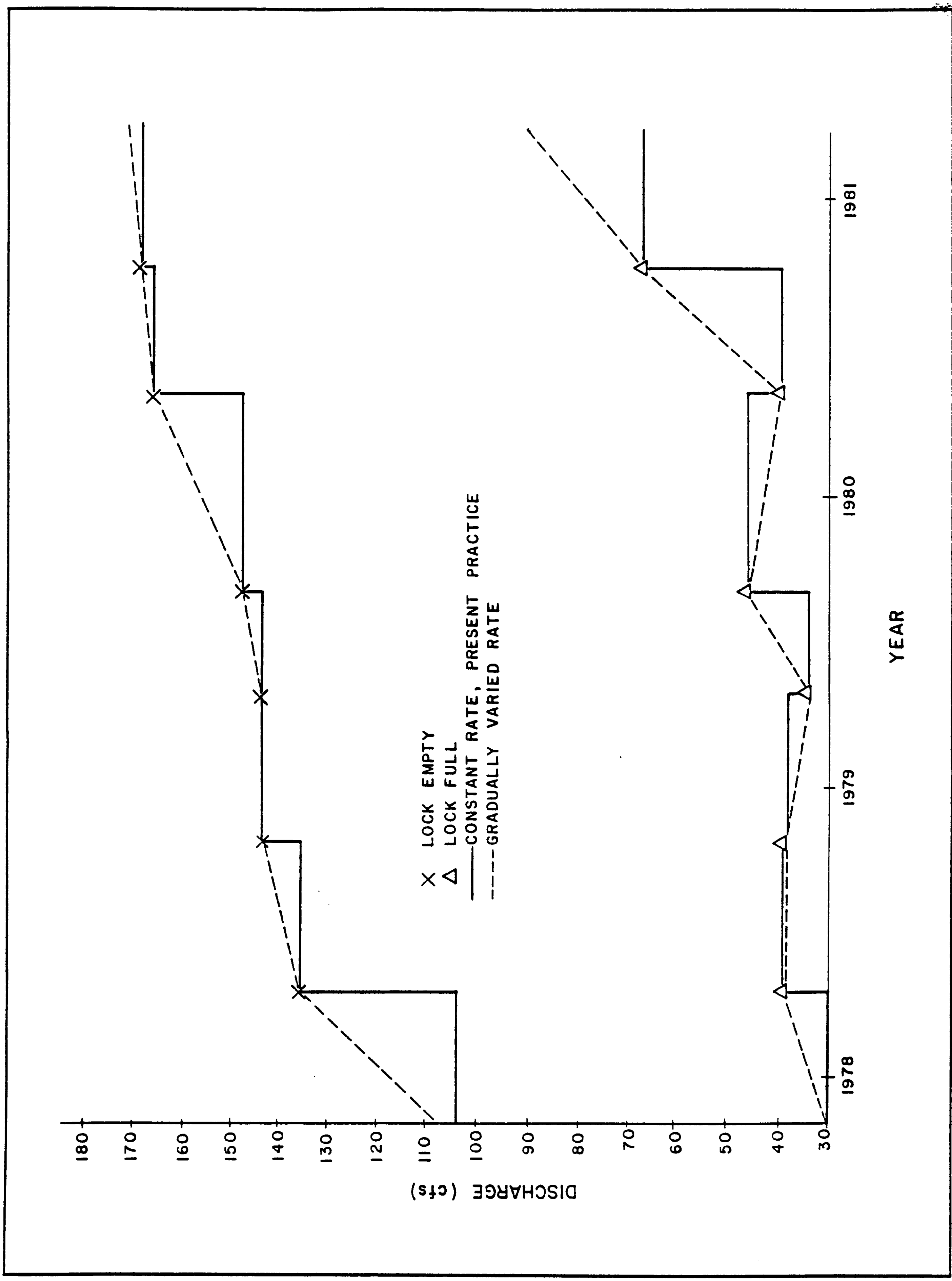

FIGURE 7 U.S. GOVERNMENT LOCK, LOCKPORT LEAKAGE TEST 1977- 1981

started. At the moment of closure the leakage rate is zero through the second gate since the differential head is zero. The leakage through the second gate will commence and increase with the head, created by the changing water level in the lock, to the alternate constant leakage rate.

The leakage through the first gate, which was in a closed position initially, was at a constant rate. However, the leakage rate becomes, in effect, zero with the beginning of the operation since the leakage flow goes either to lock storage or lock discharge with the filling or emptying of the lock chamber.

Consequently, the leakage discharge during lockage operations is always overestimated. A cursory analysis, more consistent with the hydraulic characteristics of the lock, suggests an error of about 19 cfs based on 16 lockages per day. Expressed another way this would amount to about 750 cfs-year for a 40-year period assuming April 1981 leakage rates.

Errors resulting from the assumption that leakage rates are independent of the headwater and tailwater levels depend on the levels represented by the leakage tests, and the characteristics of water level fluctuations. For example, based on a reanalysis of the leakage test data for September 12, 1979, the leakage discharge through the upstream gates will vary from 158 cfs to about 130 cfs for upstream water levels ranging from 0 to -10 feet, CCD. Leakage discharge through the downstream gates will vary from 69 cfs to about 60 cfs for a 10 feet range in differential water levels. The report for the September 1979 lock leakage test shows the leakage rates through the upstream and downstream gates to be 148 cfs and 47 cfs, respectively.

The leakage analysis is based on a cross-sectional area larger than the lock chamber and which is apparently constant for any depth. The method and accuracy of this determination has not been confirmed.

2.1.5.2.4 Recommendations

A lock leakage test plan should be developed to provide an accurate determination of leakage rates based on reliable data and analysis. The objective of the plan should be good quality assurance procedures with a goal to improve credibility for the measurement techniques and accounting for leakage flow through the lock. The plan must recognize that leakage during the test is inherent, and leakage rates are dependent on the upstream and downstream water levels. The plan should include these features:

- Standard water-level measuring equipment components; i.e. float, cable, tape, recorder, reference elevations, timer, etc. should be specified. Dependable equipment is essential.

- Each measurement of the test should be an independent sample. In the event lockages are not made between measurements, the gates should be repositioned for each measurement. (For example, the testing sequence would be: lock condition full/leakage measurement/reposition upstream gate/dewater lock/reposition downstream gate/leakage measurement/dewater lock/reposition downstream gate/fill lock chamber/reposition upstream gate/leakage measurement. This sequence should be repeated to obtain a minimum of three consistent data sets for each lock condition.)

- The water level should be measured at intervals sufficiently frequent to define any water-level oscillations in the lock chamber. Ideally, the water level would be measured near midpoint in the lock. As a practical matter, a point near one end of the lock may be used. A stop-log slot may be used to dampen the oscillation, but the importance of definition is not diminished. A continuous record of the water level during the measurement is recommended.

- Each measurement should also contain the headwater and tailwater elevations, as well as the water level elevation in the lock at the beginning and end of each measurement.

- The report of leakage test results should be comprehensive enough to reflect accurately and favorably on the quality of the information determined.

- The leakage test data should be analyzed to account for leakage during the test and to define the leakage rate as a function of the effective water levels.

The procedures for computing lock leakage discharge should be revised to account for the biased errors resulting from the assumptions that leakage is both constant and continuous, and for variations in upstream and downstream water levels. It should be noted that all of the previous turbine rating verification attempts, based on measurements in the upper pool, probably are in error to some degree because of the inherent errors in the present methods for determining and computing leakage through the navigation lock.

2.2 INDUSTRIAL DIVERSIONS (Column 7 of the Hydraulic Report)

Data compiled by USCE (1) indicates a dramatic decrease in industrial diversions over the period of record. An average diversion of 144 cfs was reported for 1961 but has decreased steadily to 1 cfs in 1979, resulting in a 20-year average of 70 cfs.

In the 1976 USCE reevaluation of methodology for monitoring the Lake Michigan diversion components, three industrial pumping stations were identified. These are: the Argonne National Laboratory (ANL), Union Oil, and Texaco Oil (2). The water is used for cooling, boiler feed water make-up and processing. Since the withdrawals are not returned to the canal above Lockport, they must be accounted for in the monthly hydraulic report, being added as a part of the total Lockport flow. The water withdrawn is metered through venturi-type meters, and the data is sent to the MSD every two weeks.

In the 1980 Hydraulic Report Computation (15) only two such withdrawals were mentioned: ANL and sludge dilution water. At present, both Union Oil and Texaco Oil discharge back to the Sanitary and Ship Canal. At ANL, located north and east of Lemont, Illinois, water is withdrawn from the Sanitary and Ship Canal and is discharged to the Des Plaines River, and is treated as diversion. Meter data on this withdrawal is sent to the MSD monthly; quantities are reported in million gallons per day.

Sludge dilution water, shipped to Fulton County Illinois with MSD sludge, is reported only if the quantity exceeds 0.5 cfs. This method was established through discussions between the MSD and the USCE.

Industrial diversion, in recent years, have comprised a very small percentage of the total flow at Lockport and have little impact on the diversion accounting procedure. Nevertheless, the Committee recommends that flow metering devices be kept in calibration to document the validity of the flow measurements. Record of the periodic calibration tests should be forwarded to the responsible agency and be made available for inspection by other parties to the diversion.

2.3 DOMESTIC PUMPAGE (Column 9-11 of the Hydraulic Report)

A number of communities, public and nonpublic entities in the MSD system obtain raw water for domestic and other purposes from ground water rather than from the City of Chicago. The pumpage reports from these sources are used to determine the amounts of these nondiversion pumpages that are discharged into the canal system and subsequently subtracted from the Lockport flow. Domestic pumpage averages 189 cfs or about 5 percent of the average annual flow at Lockport and accounts for about 54 percent of the deductible nondiversion component (1).

The domestic pumpage reported in columns 9, 10, and 11 of the Hydraulic Report consists of three components:

- ground-water sources within the Lake Michigan Watershed;
- ground-water sources outside the Lake Michigan Watershed; and
- pumpage originating from Indiana and Wisconsin.

Each community that discharges into the MSD system is assigned a factor that is representative of the portion of its domestic pumpage discharged into the canal system. Most communities have a factor of 1.0 since all their sewage is included in the Lockport discharge. The Indiana communities of East Chicago, Hammond and Whiting have been assigned a factor less than 1.0 as explained in Sec. 2.3.3.

Domestic pumpage from ground-water sources is composed of two parts, public and nonpublic. At the beginning of each year, public domestic ground-water pumpage reports are received by MSD from the villages, sanitary districts, and utilities whose discharge reaches the canal system.

Nonpublic ground-water users report voluntarily their annual pumpage to the Illinois State Water Survey. These users include quarries, commercial, industrial, and institutional users, irrigators, and subdivisions. Nonpublic ground water pumpage is reported by section, township and range and separated by its origin as being from either the Des Plaines River Watershed or the Lake Michigan Watershed. At the present, nonpublic ground-water pumpage averages about 40 cfs per year (15). Of this amount, approximately 50 percent is allocated to each watershed. A coefficient of 0.887 is applied to the total nonpublic ground-water pumpage to yield the estimated percentage of the total reaching the MSD system waterways. Documentation on how this coefficient was determined was not furnished to the Committee. However, as related in a personal communication (16) this factor is an estimate of that portion of the nonpublic ground-water pumpage in Cook County that reaches Lockport and is a drainage area ratio, not a consumptive use factor. By area, 11.3 percent of Cook County is outside the diversion area. Therefore, the factor 0.887 is applied to the total nonpublic ground-water pumpage reported for Cook County.

The monthly computations of domestic pumpage from public and nonpublic sources are based on a composite of the annual pumpage reports for the previous year. The mean pumpage rates for the previous year are adjusted for seasonal variability on the basis of monthly pumpage rates for the City of Chicago. However, the monthly adjustment factors are based on the monthly Chicago values averaged over the previous 5 years rather than just the previous year's Chicago pumpage.

There is, in addition to the nonchargeable flows discussed below and in Section 2.4, a provision in the Hydraulic Report for reporting diversion into Lake Michigan. Water that might conceiveably be diverted by Illinois into Lake Michigan from a source outside the Lake Michigan watershed with the consent of the United States Government is included under Column 13. However, at present there are no sources that fall into this category.

2.3.1 Ground Sources - Lake Michigan Watershed (Column 9 of the Hydraulic Report)

This component of the domestic pumpage consists of water which enters the canal system above Lockport. This includes pumpage quantities within the Lake Michigan watershed drawn by communities not receiving water from Lake Michigan. Included in these quantities are public and nonpublic ground-water pumpage.

2.3.2 Ground Sources - Outside the Lake Michigan Watershed (Column 10 of the Hydraulic Report)

Like the domestic pumpage component within the Lake Michigan Watershed, this component is water that gets into the canal system above Lockport. This component includes public ground-water pumpage figures for the entire previous year which is reported by the towns and villages along with nonpublic ground-water pumpage.

2.3.3 Domestic Pumpage from Indiana and Wisconsin (Column 11 of the Hydraulic Report)

This component of the domestic pumpage originates in Indiana and enters the MSD canal system above Lockport via the Grand Calumet River. Under the 1967 Supreme Court Decree, this pumpage is not chargeable to the State of Illinois and, therefore, is deducted from the flow at Lockport.

The communities of East Chicago, Hammond, and Whiting withdraw their water supply from Lake Michigan and discharge sewage effluent to the Grand Calumet River.

A portion of the sewage effluent from these three communities (Hammond also supplies three other Indiana communities) reaches the MSD system and passes Lockport while the remainder drains to the Indiana Harbor Canal and then into Lake Michigan. Factors to be applied to the water withdrawn from Lake Michigan by these communities to compute the portions passing Lockport are:

Village	Factor[1]
East Chicago	0.10
Hammond	0.25
Whiting	0.25

[1]/Provided by MSD, apparently based on field observations, however, no documentation was supplied.

The following describes the channel conditions and how these factors were determined (15).

The sewage effluent from Whiting, Hammond, Highland, Griffith, and Munster is treated by the Hammond Sanitary District and discharged into the Grand Calumet River through an outfall structure located approximately 200 feet east of Columbia Ave. in Hammond. The sewage effluent from East Chicago, Indiana enters the Grand Calumet River near Indianapolis Blvd. in East Chicago.

The Grand Calumet River heads east of Gary and flows westward, picking up a great deal of industrial process water which is obtained from Lake Michigan and discharged into the river. In East Chicago the Indiana Harbor Canal is connected to the river and provides direct access to Lake Michigan. Most of the flow originating to the east of the junction with the canal is reportedly discharged into Lake Michigan. From the junction with the Indiana Harbor Canal westward the river gradient is essentially flat to the hydraulic summit of the stream in the vicinity of Columbia Avenue. In this reach the river is sluggish and the direction of flow is subject to reversal depending upon wind and lake level fluctuations. The East Chicago and Hammond Sewage Treatment Plants discharge into this reach of the river and consequently can flow either eastward to Lake Michigan via the canal or flow westward and pass Lockport.

A stream gage station was established on the Grand Calumet River at Hohman Avenue, about 1½ miles west of the Hammond effluent outfall (15). This gage was operated during the period 1966 through 1970 to determine the quantity of effluent entering the MSD system. The average flows at Hohman Avenue and the average discharge from Hammond Treatment Plant during this period were used to determine that approximately 25 percent of the flow entered the MSD system. Presumably the estimate that 10 percent of the East Chicago flow goes westward was based on observation. During the time period used to determine the percentages of pumpages reaching the MSD system, Lake Michigan never exceeded

+1.00 feet CCD. Documentation on the time when these percentages were established was not available to the Committee.

2.3.4 Evaluation of the Domestic Pumpage Accounting

Domestic pumpage from ground-water sources is not considered to be Lake Michigan water. As such, the domestic pumpage entering the canal system above Lockport is a deductible for computing the diversion flow. Therefore, the computational process, which ignores consumptive use of domestic pumpage, results in the maximum possible deduction. As a consequence, a too large deduction leads to a computed diversion discharge that is too small. A State of Illinois study (17) indicates that a reasonable consumptive use is approximately 10 percent of domestic pumpage. This suggests that the deduction is overestimated and, conversely diversion is underestimated by about 10 percent for domestic pumpage. This results in a biased error of approximately 19 cfs which when accumulated over the 39-year accounting period, represents approximately 37 percent of the allowable 2,000 cfs-years excess. In terms of the annual diversion of 3,200 cfs, 19 cfs represents approximately a +0.6 percent error.

The explanation for the factor 0.887 which is applied to nonpublic ground-water pumpage is understood but not fully appreciated. It appears that ground-water pumpage in Cook County, but which is outside the MSD service area could be readily identified, excluded, and thus eliminate the need for the factor.

The domestic pumpage from Indiana (Column 11 of the Hydraulic Report) crosses the Lake Michigan Watershed as sewage effluent from the Hammond and East Chicago treatment plants on the Grand Calumet River. The procedure for estimating these flows and the percentage entering the canal system are discussed in Section 2.3.3. This component consists of the domestic pumpage return flows only and is not concerned with stormwater inflow and infiltration. Although the consumptive use aspect of the domestic pumpage is not specifically addressed, it is nevertheless implicit since the coefficients applied to the East Chicago, Hammond and Whiting domestic pumpage are believed to be based on measurements and observations of the effluent flows.

2.3.5 Recommendations

The Committee recommends:

- Reconsideration of the domestic pumpage factors to determine the portion of flow from the East Chicago and Hammond sewage treatment plants, passing Lockport.

The 10 and 25 percent factors which are applied to the East Chicago and Hammond pumpages are questioned in the absence of supporting analysis and documentation. During the Committee's visit to the area all discharge from the plants appeared to be flowing westward. This is consistent with observations in March 1964 by the Indiana Flood Control and Water Resources Commission when Lake Michigan was extremely low.

The Grand Calumet River, from Lake Michigan at the Indiana Harbor Canal, to its mouth, represents a hydraulic potential of about 2 to 4 feet. The effectiveness of the potential head to cause flow from Lake Michigan toward the Cal Sag Channel depends on the magnitude and location of withdrawals from and discharges to the system, hydraulic capacity of the channel, channel gradients, and constrictions such as the Columbia Avenue crossing.

An analysis of the data cited by Keifer (18) and the USGS data of 1954-56 and 1964 should provide an adequate basis for developing reliable factors for the East Chicago and Hammond pumpages.

- Development of a procedure for estimating the annual net flow across the hydraulic summits of the Grand and Little Calumet rivers.

The Keifer report (18) describes the formation of a "summit" on the Grand Calumet River that was caused by the construction of the Indiana Harbor Canal. Similarly a "summit" has formed on the Little Calumet River just east of Hart Ditch that was caused by the construction of Burns Ditch which connects the Little Calumet River to Lake Michigan. The documentation for the establishment of these hydraulic summits, and subsequently the drainage boundary to determine what portion of the Calumet River Basin is effectively within the diverted watershed, have not been examined.

The idea of establishing a boundary for the diverted watershed across the Calumet Basin is appropriate. However, the appropriateness of the assumption that the channels of the Grand and Little Calumet rivers at their respective summits act as hydraulic boundaries is a moot question. It is a fact that flow can occur across these boundaries in either direction depending on hydrologic, as well as hydraulic conditions. The hydraulic summit presents the problem of determining the net flow, and direction, across the boundary. If the net flow is not zero, an appropriate adjustment should be made to the diversion flow. The problem of determining the net flow across the hydraulic summit is complicated by the fact that the hydrologic and hydraulic conditions which cause the flow can also change because of developmental changes in the Calumet Basin.

Flows based on the proposed procedure would also reflect the effects of consumptive use and storm runoff.

- o Data be provided on the calibration and accuracy of the domestic pumpage flows and that periodic reports of maintenance and calibration of the flow metering devices be submitted to the responsible agency and be made available for inspection by other parties.

2.3.6 Contribution of Lake Michigan to Groundwater

A comparison of Lake Michigan water levels and ground-water levels in the shallow aquifer along the lakeshore in Lake and Cook Counties, Illinois indicates that, at present, ground-water is being discharged into Lake Michigan (19). However, with large concentrations of shallow-aquifer pumpage near the lake shore, it is possible to reverse the hydraulic gradient and induce lake water to contribute to the shallow aquifer (20). This induced flow is referred to as induced recharge. For this phenomenon to occur, two conditions must be met:

- a hydraulic gradient must exist where hydraulic heads consistently decrease from the lake to the aquifer; and

- a continuous hydraulic connection must exist between the lake and ground-water systems, establishing a path along which the lake water can flow.

At this time pumpage along the lake shore in the Chicago vicinity is not great enough to induce appreciable quantities of water from Lake Michigan (19).

2.4 STORM RUNOFF AND INFILTRATION FROM THE ILLINOIS WATERSHED (Column 12 of the Hydraulic Report)

Stormwater runoff and infiltration from an area of 217 square miles of the Des Plaines River Watershed is discharged into the MSD canal system, is measured at Lockport, is not diversion, and therefore is deducted from the Lockport flow in the computation of Lake Michigan diversion. Several techniques, direct and indirect, are used to determine these flows from the Des Plaines River Watershed. The accounting procedure for runoff and infiltration is complicated by the fact that it also accounts for a special, but interim situation in which Des Plaines River Watershed runoff is diverted into the MSD canal system.

The runoff and infiltration component consists of five parts:

- Infiltration - Flow from a 140 square mile area of the Upper Des Plaines River Watershed that reaches the canal system by infiltration and inflow to the MSD sewer system.

- Summit Conduit - Surface runoff from an area of 5.4 square miles in the Des Plaines River Watershed (village of Summit) which passes through a conduit beneath the Des Plaines River directly into the MSD system.

- Des Plaines River Watershed South of the Main Channel - Runoff from a 67.0 square mile area of the Des Plaines River Watershed that is south of the Sanitary and Ship Canal cutoff from the Des Plaines River by the canal.

- Sewer System 13 A - An operating portion of TARP intercepts and provides temporary storage for a 4 square mile area of the Des Plaines Watershed in the vinicity of La Grange and Brookfield.

- O'Hare WRP - The inflow record for the O'Hare WRP is used to estimate the infiltration component for the O'Hare service area that is discharged to the Des Plaines River.

2.4.1 Infiltration

The infiltration and inflow component from the 140 square mile Upper Des Plaines River Watershed averages 90 cfs. The I&I flow is computed on the basis of the Upper Des Plaines Pumping Station flow record, I&I factors and data for old and new sewered areas, and population per capita use data.

The Upper Des Plaines Pumping Station intercepts runoff from a 36.7 square mile sewered area and lifts the flow to a higher part of the sewer system. Flow through the station is measured by means of an orifice plate in the discharge line for each of the 26,928 GPM pumps. A recording and integrating meter for each pump is driven by an air medium impulse differential pressure from the orifice plate taps. Discharge is recorded on 7-day 12-inch circular charts. The flow intergrater provides direct digital reading of volume in units of 1,000 cubic feet.

Daily discharge is determined from an analysis of the recorder charts where the product of flow rate and pump running time is averaged over 24 hours. The intergrating meter provides a basis for checking the discharge computed from the 7-day charts.

The infiltration and inflow factor for the pumping station service area is based on an analysis of the 1970-74 Hydraulic Report which indicated a value of 0.85 mgd/square mile for old sewered areas. The Committee understands that this analysis was based on measurements from a 25 square mile old sewered segment of the service area. The I&I factor for new sewered areas was based on a MSD study of new sewers in the villages of Elk Grove, Streamwood, and Orland Park. The value of 0.24 mgd/square mile was determined from a combined area of 20 square miles.

The daily I&I flow for the pumping station service area is computed by subtracting the domestic pumpage from the total measured flow. The domestic pumpage is estimated on the basis of the 1971 population and a per capita use of 135 gpd. The daily I&I flow for the upper Des Plaines Basin entering the MSD system is computed as the product of the pumping station I&I flow times the ratio of the drainage area for the basin to that of the pumping station served, adjusted for differences in effective sewerage. This ratio was recomputed in 1980 on the basis of updated maps and the value of 4.18, determined in 1975, was changed to 3.39.

Because consumptive use is neglected and domestic pumpage is based on population and average per capita pumpage, the I&I figure is underestimated. Since I&I is not diversion, an under estimation of I&I results in a too small deduction from the flow at Lockport.

2.4.1.1 Evaluation

2.4.1.1.1 Upper Des Plaines Pumping Station Discharge Measurement

The orifice plate-recording meter is an excellent device for the measurement of flow provided certain qualifying conditions are adhered to. For example:

- o The orifice plates, pressure taps, and the conduit walls immediately upstream and downstream from the orifice should be maintained in essentially the same condition as when installed.

- o The orifice recording and integrating meters should be maintained in calibration over their full range.

The Committee is of the opinion that efforts to satisfy these requirements are not adequate.

2.4.1.1.2 Computation of Daily Discharge

The copies of the recording meter charts reviewed indicate that the interpretation of the pen trace may lack the necessary attention and precision; namely, improper pen setting, absence of comparisons of computed and weekly integrated flow, and failure to use subdividing techniques when flow changes rapidly and frequently.

2.4.1.1.3 The Computation of Domestic Pumpage

The domestic pumpage, which is subtracted from the pumping station discharge to determine the daily I&I flow, is not consistent with the population and per capita data given on the computation form.

2.4.1.1.4 Infiltration and Inflow Factors

The rate of infiltration depends on the length of sewer, the area served, the soil and topographic conditions, number of connections, groundwater levels, precipitation, and years of service. The factor of .24 mgd/square mile used for new sewers, based on a study by MSD of sewers in three villages, is within the range of expected values (21). The use of the data is appropriate, however the interpretation is inconsistent with the concept that the infiltration rate varies with the size of the service area. The I&I factor of 0.85 mgd/square mile determined for the 25 square mile old sewered portion of the pumping plant service area may be a legitimate value. However, it is almost twice the recommended (21) infiltration design allowance. The use of this relatively high factor for old sewers to extrapolate the pumping station I&I flow to the larger area, in the absence of supporting documentation is suspect. Because I&I is an essential component to diversion accounting, its determination should be based on good reliable data.

A compilation listing 57 reports of inflow and/or infiltration studies for sewer systems within the MSD service area has been completed (22). A summary review is made for 18 of the studies.

2.4.1.2 Recommendations

The following recommendations are made to improve the reliability of the computed infiltration and inflow from the upper Des Plaines River Watershed:

- Perform, document, and report periodic inspections and necessary maintenance on the orifice system.

- Make periodic checks and reports on the calibration of the recording and integrating meters.

- Develop procedures for the interpretation and analysis of the weekly recorder charts to insure consistent and reliable discharge computations.

- Reconsider the adequacy of the I&I factors used for the pumping station and the Upper Des Plaines Watershed.

  - Review pertinent information available for I&I studies in the Chicago area and the subject area in particular.

  - Consider the merit of establishing index areas in the watershed where I&I, population, and domestic pumpage could be monitored periodically (5-year intervals for example) to provide a basis for evaluation and improvement of the I&I component.

- Develop reliable estimates of consumptive uses that can be applied to domestic pumpage for the computation of return flows.

2.4.2 Summit Conduit

The average annual flow through the Summit Conduit, located in Figure 8 and detailed in Figure 9, for the period 1961-1974 was 11 cfs, or about 3 percent of the deductible component (1).

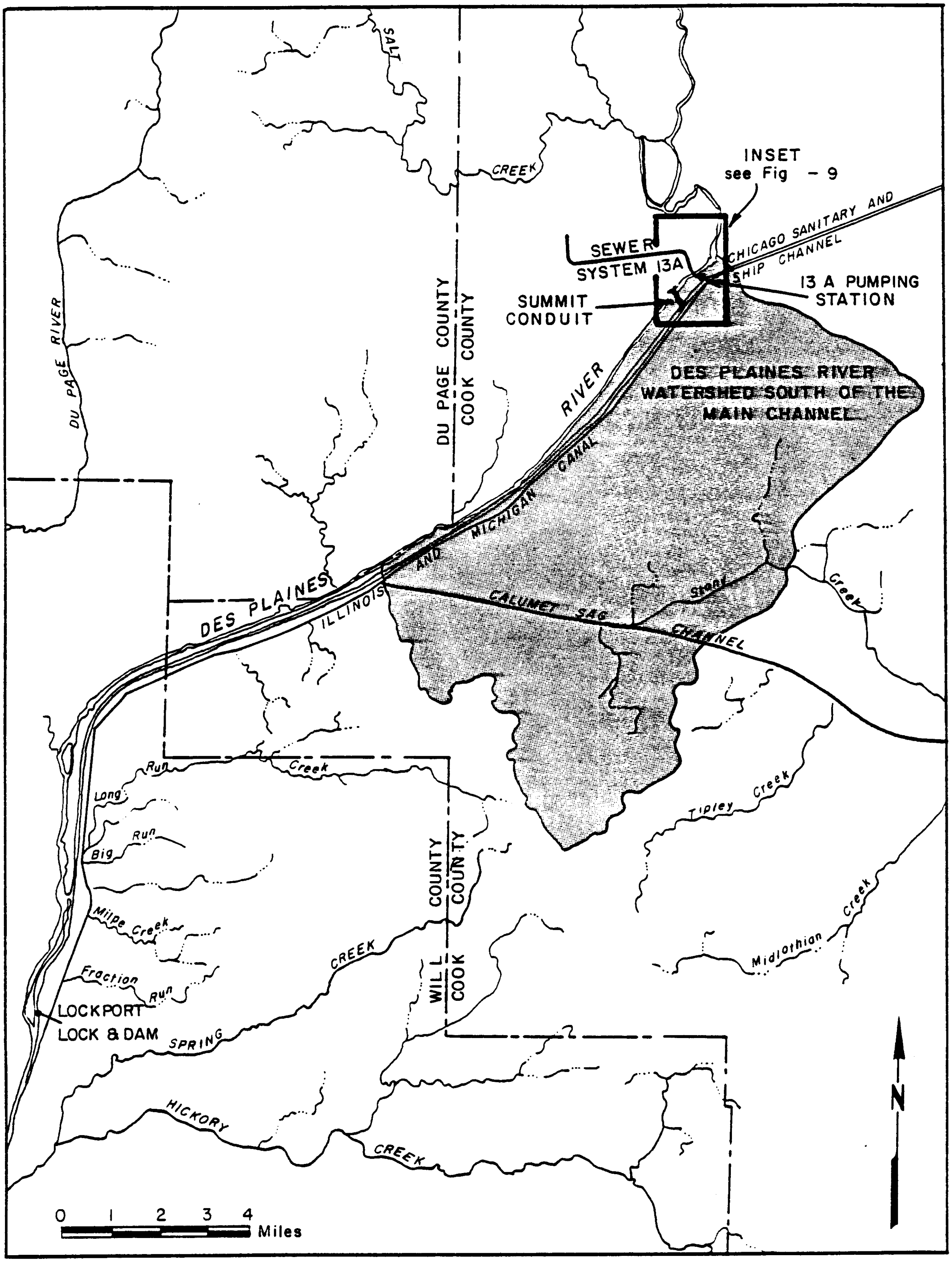

FIGURE 8 LOCATION PLAN - LOCKPORT LOCK AND DAM, SEWER SYSTEM 13A, SUMMIT CONDUIT, AND DES PLAINES RIVER WATERSHED SOUTH OF THE MAIN CHANNEL

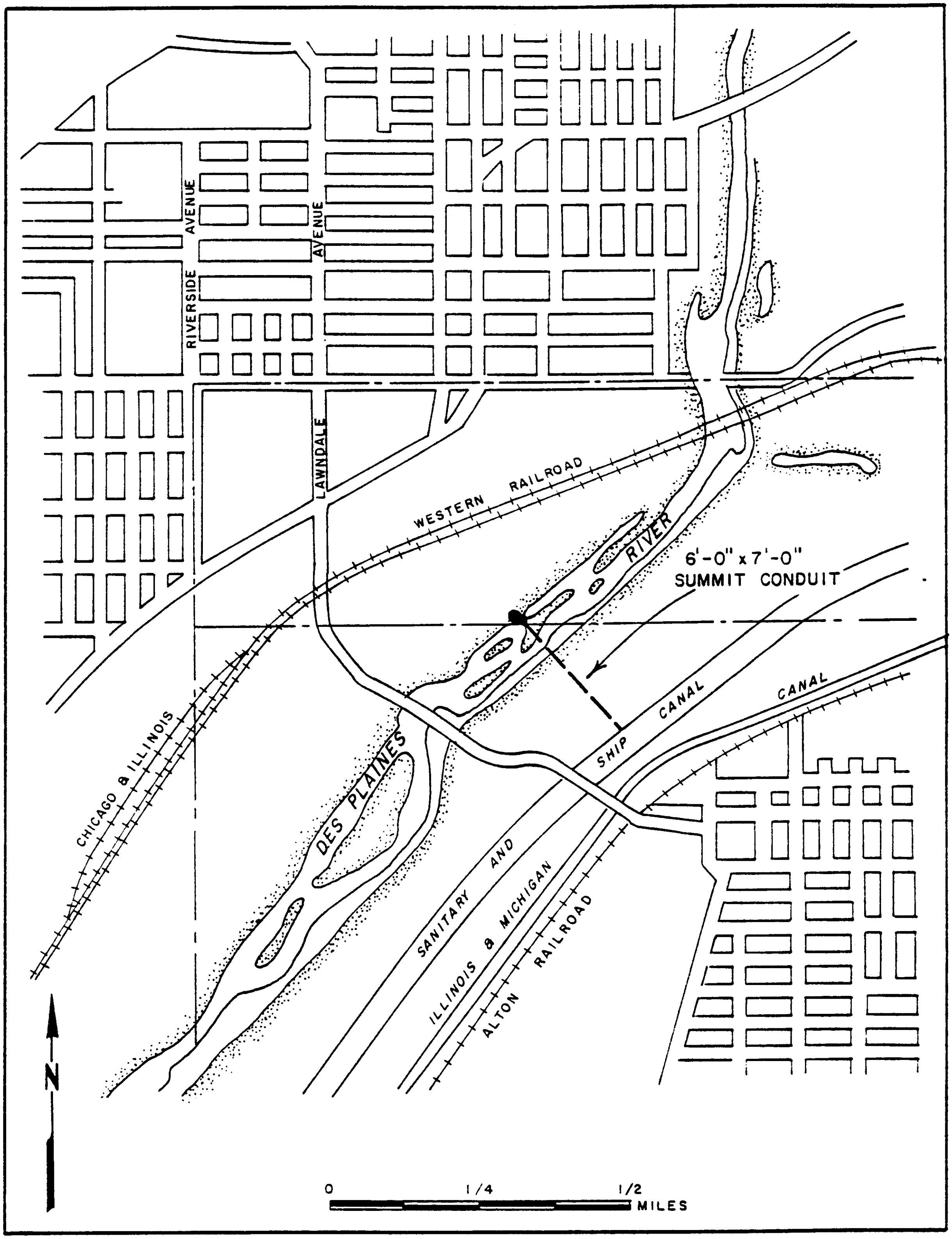

FIGURE 9 DETAILED LOCATION PLAN - SUMMIT CONDUIT

In 1910, a levee was constructed in the flood plain of Des Plaines River to prevent overflow into the McCook and Countryside area during flood periods in the Des Plaines River. Drainage of local runoff from the 5.4 square mile area protected by the levee is conveyed to the main channel of the Chicago Sanitary and Ship Canal via the Summit Conduit.

The flows in the Summit Conduit are derived from the Des Plaines River Watershed. However, since the Summit Conduit flow is included in the Lockport flow, deductions are applied to the Lockport flows in the amount of the discharges of Summit Conduit on a daily basis.

2.4.2.1 Depth of Flow Recording

Flow in the Summit Conduit is determined by applying a depth of flow measured at the entrance to the 6 x 7 foot conduit to a known depth-velocity relationship. The depth of water is measured by bubbling compressed air through a small pipe at the invert of the conduit. The air pressure, which is proportional to the hydrostatic depth, is converted to an electrical time pulse and transmitted by leased phone line to a recorder at MSD's headquarters.

During the Committee's visit to the gage, several observations were made that have a direct bearing on the reliability of flows computed for the conduit:

- o The bubbler orifice is normally anchored at the entrance of the pipe section to the invert. It is highly improbable that hydrostatic pressure occurs at this point except perhaps at fairly shallow depths. (Pressure-depth relation must be calibrated in the field.)

- o Several feet of the lower end of the bubbler pipe apparently had broken loose from its anchor clamps and was bent downstream along the conduit sidewall. Since the precise elevation of the bubbler orifice must be known and fixed in position, all depth observations that have been made since the orifice was dislocated are invalid.

- A large steel pipe, about 3.5 inches in diameter, had been positioned vertically at the centerline of the entrance to the circular section to house a conductance sensor. The lower end of the pipe apparently had been bent downstream by floating debris during a period of high flow. It is suspected that the pipe has had an adverse effect on the relationship between the water pressure and true depth of flow at the bubbler orifice. However, this pipe has since been removed.

2.4.2.2 Depth of Flow-Discharge Relation

The conduit is defined as a circular 6-foot concrete pipe modified by a 90-degree triangle section tangent at lower quarter points of the pipe. The bottom of the triangular portion is rounded out on a 7-inch radius to provide a maximum depth of 7 feet when the pipe flows full.

The relation between depth of flow and discharge is described (23) as being developed through velocity measurements. However, the rating table apparently was developed on the basis of hydraulic computations using the well known Manning equation:

$$Q_d = \frac{1.486\ AR^{2/3}S^{1/2}}{n}$$

where Q_d is the discharge for a given depth, A is the flow area, R is the hydraulic radius, S is the energy gradient, and n is the coefficient of roughness for the conduit.

The rating, computed by the Manning equation, assumes the water surface and energy gradient are equal to the conduit slope for any depth of flow. The relation of the hydraulic properties, Q, A, R, and V with depth of flow are computed for selected depths and developed graphically as a percentage of the full-pipe value.

Current-meter measurements of discharge are made periodically, by MSD, to check the rating. A copy of two current-meter measurements made on March 8, 1979, was provided to the Committee.

As a general procedure the velocity is measured at several points, typically at one-half foot intervals in the vertical at the centerline of the conduit. The observed point velocity is assumed to be constant for the full width of conduit segment represented by the observation. This practice is contrary to the widely accepted principle that the lines of equal velocity tend to be concentric with the conduit boundary. This practice is almost certain to result in computed discharges greater than actual.

A cursory review of the March 9, 1979, measurement notes suggests the discharge computation is in error. If the observed flow depth and the indicated depths of the current meter are correct, then the flow area for the two measurements should be 14.46 square feet rather than 12.97 square feet. The recomputed discharges for the measurements are +8 and +11 percent, rather than -0.1 and +0.1 percent different from the theoretical rating.

2.4.2.3 Evaluation

Much of the engineering analysis related to the determination of flow through the Summit Conduit was made more than three decades ago. Although not documented, most of the effort in recent years seems to have focused on problems related to the transmission of data from the gage stage sensor to the Waterways

Control Center at MSD. Overall, the continued use of somewhat archaic hydraulic engineering practices and field operations by rote seems pervasive. An opinion that the level of reliability and credibility for the Summit Conduit data has deteriorated is a manifestation of the nonstandard conditions at the gage, questionable computation procedures, and the casual manner in which field measurements are made.

The location of the depth sensor just inside the entrance to the conduit is probably the most difficult position to obtain a meaningful or reliable measure of discharge. Verification of the depth sensor performance at this location is difficult under most conditions and virtually impossible during some conditions of flood flow.

Essentially, every aspect of the gaging operation at the Summit Conduit can be improved by the use of modern hydraulic principles and procedures aimed at acceptable levels of accuracy and quality assurance.

2.4.2.4 Recommendations

The Committee recommends:

- Consideration be given to referencing discharge to a water surface elevation upstream from the entrance to the conduit.
- If the depth of flow reference is to be continued, the bubbler orifice be moved downstream to a zone of hydrostatic pressure conditions.
- Reference elevations and staff gages be established to provide independent water surface elevations or gage heights.
- Current-meter verification measurements contain enough point velocity observations to define the velocity distribution in the measurement cross section.

2.4.3 Des Plaines River Watershed South of the Main Channel

This component of the deduction included in Column 12 represents runoff from a 67.0 square mile area of the Des Plaines River Watershed that lies south of the Sanitary and Ship Canal (Figure 8). It is not possible, nor practical, to measure the runoff from this area that is intercepted by the Cal Sag Channel and passes Lockport. Instead a gaged area of Hart Ditch (Figure 10), which is physically similar, is used as an analogy. The U.S. Geological Survey (USGS) has gaged the flow of Hart Ditch at Munster, Indiana since 1974. The drainage area above the gage is 70.7 square miles.

The two areas are reported to be hydrologically similar, however, a quantification of these similarities was not available. A cursory examination of runoff data for gages in the general area suggests that Hart Ditch is a reasonable choice. The operation of the USGS Hart Ditch gage and the analysis of its record is subject to periodic quality assurance reviews.

Although no changes are proposed the Committee does recommend that parameters indicative of urbanization for the two basins be measured and documented. This is recommended because assurances cannot be made that conditions influencing runoff in either basin will remain unchanged.

2.4.4 Sewer System 13A (Part of the TARP System)

Sewer system 13A, located on Figure 8, is a completed operational portion of the TARP system. TARP 13A serves an area of about 4 square miles and provides combined sewer overflow relief during storms to the Des Plaines River Watershed villages of La Grange, Brookfield, and Lyons. Subsequently, the collected overflows are pumped to the West-Southwest STW for treatment and discharge to the MSD system. Each of the two 5000 GMP pumps is equipped with standard electromagnetic flow recording devices.

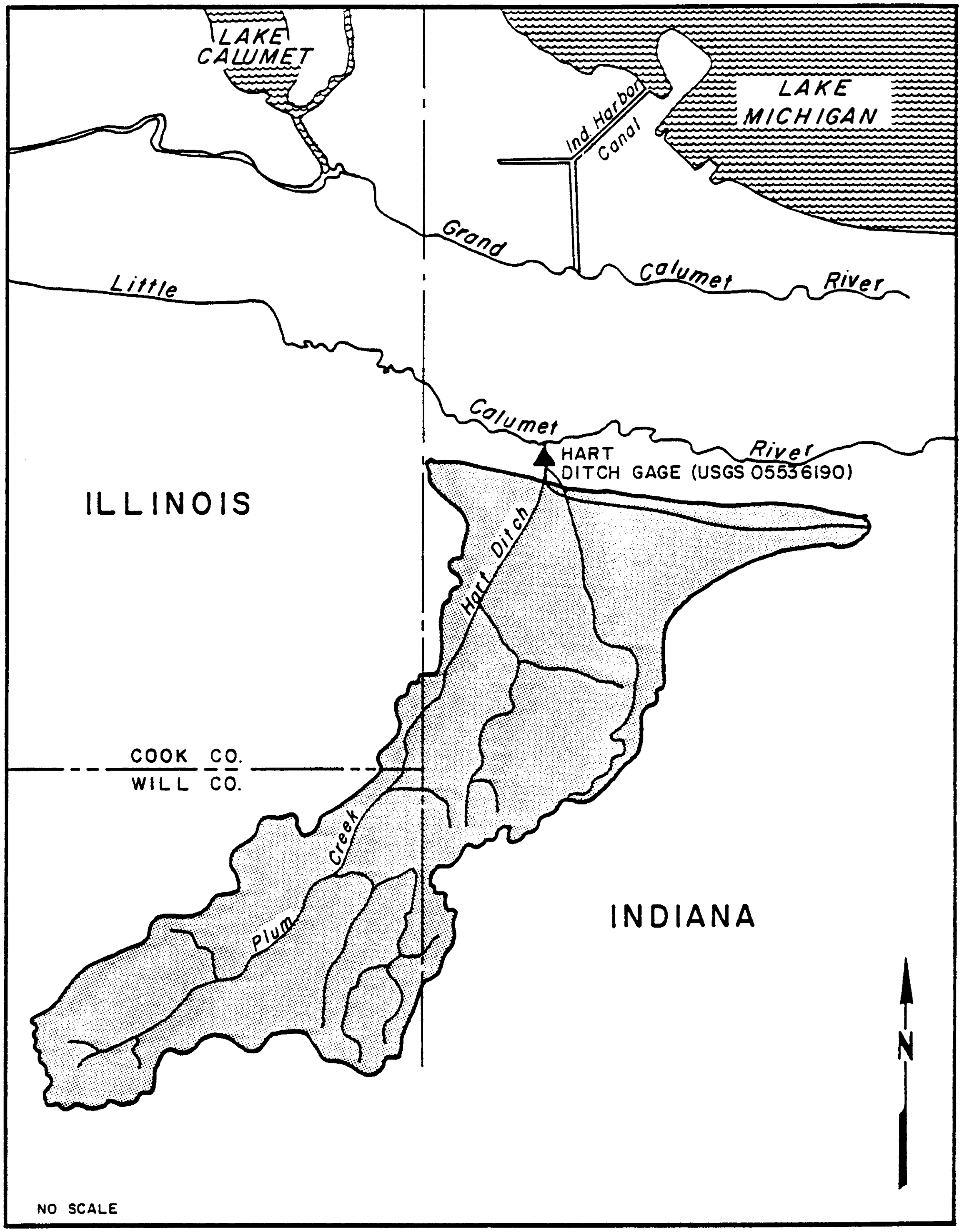

FIGURE 10 LOCATION PLAN - HART DITCH GAGE

Pumping records are sent to the MSD once each month. Flows recorded are not chargeable to diversion, and are deducted from the total flow measured at Lockport. In the event of overflow into the canal at the pump station, water levels are recorded allowing calculation of the overflow discharge rate. To date no such overflow has occurred.

The Committee was not provided documentation of the flow meter calibration although it is understood that the flow meter output is periodically compared to the pump characteristics supplied with the pumps by the pump manufacturer. This practice is state-of-the-art, and these periodic comparisons should be documented.

2.4.5 O'Hare Water Reclamation Plant (WRP)

The O'Hare WRP will eventually serve the entire 58.2 square mile O'Hare Basin. Flow from the 9.2 square mile combined sewer area and the 49 square mile separately sewered area in the Des Plaines River Watershed will be received through the TARP tunnels. The O'Hare WRP discharges into Willow-Higgins Creek which is tributary to the Des Plaines River.

The O'Hare WRP went into operation in May of 1980. Until that time, sanitary flows, including I&I from the O'Hare Basin, were conveyed to the North Side STW. The effluent from that plant is measured at Lockport. The I&I from this area is deducted from the measured Lockport flows, and are determined by applying a factor to the measured flow at the Des Plaines pumping station (Section 2.4.1). Once the O'Hare WRP becomes fully operational, its total tributary area will be excluded from consideration in the Lake Michigan Diversion accounting process, except for any Lake Michigan water used as domestic pumpage which will be included as a diversion component..

In the interim, a portion of the wastewater from the O'Hare Basin will continue to be treated by the North Side STP. This portion is unknown. In the transition period, the following procedure is being used:

- Computations of deductions claimed for flows from Des Plaines River Watershed remain unchanged (Column 10 and 12).

- Flow to the O'Hare WRP, reported on a daily basis, are deducted from Column 10 and 12 using an estimated ratio of 2 to 1, domestic pumpage to I&I, to arrive at the amounts subtracted.

The procedure outlined appears to be a reasonable approach to account for deductions in domestic pumpage and I&I outside the diversion area.

2.5 HYDRAULIC REPORT - COMPUTATIONAL PROCEDURES

Lake Michigan Diversion is compiled and summarized by MSD in the official monthly Hydraulic Report (Figure 2) which is submitted to the State of Illinois and the Chicago District USCE for review and approval.

The reports consist of a 19 column summary of the total flow measured and recorded daily at Lockport and estimates of flows from nondiverted watershed areas and ground-water pumpage which are deducted from the total flow to calculate the total diversion. Domestic pumpage chargeable to the City of Chicago is then subtracted from the total diversion to arrive at the direct diversion and storm runoff.

Columns 1-7 of the Hydraulic Report are totalled in column 8 for the "Total Flow at Lockport."

Columns 9-11 are deductions for domestic pumpage for those communities that discharge a portion of their sewage effluent into the MSD system and get their water from sources other than the City of Chicago. Column 12 is a compilation of storm runoff and infiltration which under natural conditions, prior to construction of the Chicago Sanitary and Ship Canal, flowed into the Illinois River, but now enters the Canal above Lockport and must be deducted from the total flow measured, because it is not diversion.

Column 13 is presently not used.

Column 14 is the total of Columns 9 through 13, "Total Deductions."

Column 14 is subtracted from Column 8 and the results are reported in Column 15 which represents the total diversion, including storm runoff, reaching the Sanitary and Ship Canal from the Lake Michigan Watershed.

Column 16 and 17 are the two parts that make up column 15 and do not offer any additional information needed for diversion calculations. Column 16 is the "Lake Michigan Pumpage Entering the Canal" and consists of water pumped by the City of Chicago and surrounding villages. Column 17, derived by subtracting Column 16 from Column 15, "Direct Diversion and Storm Runoff from Lake Michigan" is water diverted into the Sanitary and Ship Canal from Lake Michigan and storm water from the diverted Lake Michigan Watershed.

Column 18 lists the domestic pumpage received from the City of Chicago by the Village of Libertyville, and sewage treated by the North Shore Sanitary District and discharged into the Des Plaines River Watershed so that it by-passes Lockport and is not included in the total flow at Lockport, but, nonetheless, chargeable to Illinois as diversion.

Finally, Column 19 of the Hydraulic Report represents the total diversion from Lake Michigan and is the sum of Column 15 and 18.

The committee has reviewed the analysis of the MSD computer program, "REPORT", provided in the briefing material and titled Hydraulic Report Documentation (24). The Hydraulic Report Documentation proceeded through the diversion accounting manually to verify the computer results. The date chosen for verification was September 16, 1980. The Committee was satisfied in its review that the computer program "REPORT" adequately performs the computational steps as outlined.

The Committee has made a number of recommendations for improvement of Lockport measurement. Their implementation would necessitate some changes in the computational steps of the accounting procedure.

3.0 WILLOW SPRINGS ROAD ACOUSTICAL VELOCITY METER

In 1967, MSD contracted for the installation of an acoustical velocity meter (AVM) in the Sanitary and Ship Canal near Willow Springs Road. The AVM system, a Westinghouse leading-edge flow meter, was intended to provide the flow component for a water-quality monitoring station at the site.

The Willow Springs AVM is not a component of the Lake Michigan diversion measuring or accounting process. However, the AVM, which is sometimes called an ultrasonic flow meter, was recommended (25) as an alternative to the existing Lockport measurement system. Because of this, together with MSD experience and views of the Committee, a discussion of the Willow Springs Road AVM is in order.

The AVM transducer path is located 475 to 650 feet downstream from the Willow Springs Road bridge in a straight reach of channel in cut rock, near rectangular in shape, about 160 feet wide and 25 feet deep. The four-path velocity transducers are aligned at an angle of about 45 degrees with the flow and set at depths of about 4½, 8, 14, and 22 feet.

Acceptance of the meter by MSD was contingent upon confirmation of performance based on the agreement with manual flow measurements. Accordingly, MSD engaged the Detroit District USCE to make current-meter discharge measurements for this purpose. A series of 102 measurements were made at the bridge during the period January 28 to February 5, 1970 (26). The AVM system failed to meet the performance requirements and the contract was terminated. Consequently, the system was never activated as an operational unit.

A comprehensive assessment of the AVM performance is not available. However, a general knowledge of the state-of-development of the AVM prior to and since 1969, discussions with MSD personnel, and the USCE Detroit District's report provide insight for future consideration of this measurement technique.

There is agreement among those experienced with this particular first generation AVM that it was not a field-proven operational instrument for open channels during the 1967-69 period. The general experience during that time was that every installation was somewhat of a bread-board exercise. The Willow Springs equipment consisted primarily of separate function boards made up of transistor circuitry. The board functions were linked by hardwire to perform the required operations. This provided a flexible system configuration, but one difficult to work with and troubleshoot.

The almost continuous attention required to deal with system malfunctions experienced by MSD was fairly typical of the times for the particular AVM. The problem was compounded by the frequent spurious signal output when the system was operating.

The USCE report to MSD concluded (26):

- o The flows recorded by the Westinghouse AVM deviated from the current-meter measurements by 16.4 percent for at least 80 percent of the measurements. (MSD acceptance criteria called for agreement within 5 percent for 80 percent of the cases.)

- o The flows from the AVM were lower than the measurements in 99 of 101 measurements. Deviations of the AVM flows from those measured ranged from +11.9 percent to -68.9 percent.

- o Two tests, made January 31 and February 5, 1970, of quasi-direct velocity comparisons of the AVM and current-meter methods indicated that the AVM mean velocities were slightly higher (0.02 to 0.06 fps) than the velocities measured by current meter. The corresponding AVM discharges were consistently lower. This discrepancy suggests that the apparent errors in flow were due to built-in integration errors.

Considering the essentially uniform channel conditions and the arrangements with Westinghouse for special line-velocity output, the close agreement between the AVM and current-meter velocities is not surprising. Nevertheless, the results of the two tests do not warrant drawing any conclusions with respect to the general reliability of the Willow Springs system.

In 1981, the state-of-development of an acoustical velocity meter, based on total travel time techniques and leading-edge signal detection, has advanced significantly with the introduction of the integrated circuit and silicon memory chips. There are about 100 AVM's in operation in open channels with path lengths greater than 10 meters. There are numerous AVM systems operating in channels and conduits with shorter paths. Most of the AVM systems have been installed during the past decade at key locations where the importance of flow accuracy justified this relatively expensive system.

During the past few years, considerable interest and research have produced new equipment and a better understanding of the problems associated with the design and operation of AVM systems. The net effect is the emergence of new systems that provide an alternative for the measurement of flow which is competitive in terms of accuracy, reliability, and versatility. The fact that these advances have been accompanied by significant reductions in cost make the AVM a viable consideration for a greatly expanded field of flow measurement installations. The frequent presence of stationary and moving barges in the Chicago Sanitary and Ship Canal could cause difficulties for the successful operation of a modern AVM flow metering system. However, these difficulties can be minimized with proper site conditions and docking restrictions.

4.0 TARP - IMPACT ON THE DIVERSION ACCOUNTING PROCEDURES

The Chicago Tunnel and Reservoir Plan (TARP) consist of 120 miles of tunnels, collecting the flow from 640 sewer overflow points, from an area of 375 square miles. The objectives of the plan are:

- elimination of storm runoff in the Chicago River entering Lake Michigan;
- elimination of basement and viaduct flooding; and
- treatment of the discharge from the combined sewers entering the Illinois Waterway.

For funding purposes the TARP plan is divided into two phases. Phase I includes tunnels, connecting sewers and pumping stations. Included in Phase II are storage reservoirs and some auxiliary tunnels needed to convey large storm flows.

TARP, by capturing nearly all the combined sewer overflow, will greatly enhance the river and canal water quality, reducing or eliminating the need for dilution water diverted from the lake. The large reservoirs to be constructed as part of the TARP plan will almost eliminate the requirement for navigation makeup water by release of stored water when needed. Overall, the TARP system will permit a more efficient allocation of Lake Michigan water for domestic use, reducing the demand for ground-water sources which are rapidly being depleted.

An indepth assessment of the impact of TARP on Lake Michigan Diversion is beyond the scope of the Committee's study. Nevertheless, it is apparent that TARP is already a factor in the diversion program. Its influences will change as each new element of the system is put into operation.

The impact of TARP on the diversion program will be manifested in two ways. First, the computations, procedures, and factors for the determination of domestic pumpage, and infiltration and inflow discharges will need to be reassessed for the installation of each new element to the system. Second, the increased storage provided by the tunnels and reservoirs will reduce the magnitude of storm hydrographs at Lockport. This will result in a higher percentage of the annual flow being discharged through the turbines, and conversely less flow through the powerhouse sluices and the Controlling Works.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The Committee has made an evaluation of the techniques and procedures for determination of Lake Michigan Diversion with respect to the best current engineering practices and scientific knowledge. This evaluation has been made within the limitations of time and resource constraints. To an extent, the report reflects the fact that some information pertinent to the study was not always available, adequate, nor timely.

The State of Illinois has the mandate to provide leadership for management, development, and conservation of water resources of the metropolitan region. Consequently any set of recommendations for the measurement and accounting of the diversion should be consistent with Illinois goals as well as the Supreme Court Decree.

Although the Decree specifically limits the measurement of diversion water to Lockport, there is one aspect of the proceedings leading to the Decree that should not be ignored. The Illinois proposal to the Supreme Court centered on the assignment of a constant value to the runoff from the diverted watershed. A constant value would have provided the greatest flexibility in efficient management of the water allocation. The Committee concluded from its review of the testimony for the Court that there was no significant disagreement among the interested parties with respect to the principle. The proposal was rejected simply because the parties could not agree on the value of the average runoff, not because of insurmountable differences over Lakefront vs. Lockport measurement sites. The potential value in terms of operational effectiveness represented by an average watershed runoff is too important to be ignored for an indefinite future. Ideally then, the techniques and procedures adopted for the measurement and accounting of diversion flow will be an integral part of Illinois planning and management strategy.

The Committee performed a cursory water budget analysis of the Chicago Sanitary and Ship Canal for the period of record calendar year 1970 through 1980. The inpetus for this analysis was information presented by the State of Wisconsin and a report by the Northeastern Illinois Planning Commission (NIPC) suggesting a bias in the Lockport flow. The USCE, Chicago District, provided assistance in this analysis. The water budget analysis was performed in order to further explore the possibility that the Lockport measurement underestimates the flow. The water budget analysis considered the following components: a) lake-front diversions, b) stream flow from the North Branch of the Chicago River and Little Calumet River, c) sewage treatment plant discharges, d) industrial and other point sources, e) estimate of combined sewer overflows, and f) estimate of storm runoff from minor tributaries. Data for periods of low flow from combined sewers and minimal discretionary diversions at the lake-front were utilized to minimize the possible errors associated with these components. The result of this preliminary analysis suggest that the total flow at Lockport is underestimated for measured Lockport flows less than approximately 4,000 cfs. However, this tentative result must be viewed in light of the possible errors associated with the components of the water budget analysis and the preliminary nature of the analysis.

Because the diversion measurement and accounting procedures are an essential part of the State's water allocation management plan, there is one important aspect of diversion that is not within the scope of the Committee's work. This deals with the collection and dissemination of hydrologic data. Hydrologic data are collected by a number of federal, state, and local agencies for a variety of needs such as flood control, regulatory compliance, program mission, as well as accountability for diversion. During efforts to obtain information, the Committee was continuously impressed with the difficulties that it assumed to be the result of two causes: first, the lack of coordination among data base systems, and second, that often the data retrieval was not an important consideration in the development of the data systems. It was inevitable that the Committee would develop views on the whole matter of data adequacy and availability. Essentially these views suggest the need for a comprehensive water data management system that would include: (a) data collection network evaluation (existing networks) and design (future networks); and (b) a central data storage and retrieval facility. The goals of the comprehensive water data management system should be to anticipate future data needs, promote its acquisition, provide direction necessary to insure uniformity, consistency, and make water resources information available in a timely fashion.

The measurement and accounting of diversion water is fairly simple in concept. However, the measurement of flow at Lockport is at best a summation of complex components which are synthesized by a variety of hydraulic and hydrologic techniques developed over a long period of time. The development of some of the techniques was traced back more than three decades only to find an explanation less complete than hoped for, or worse, none at all. This was not surprising considering that the sole purpose for much of the information sought was to comply with requirements of the Court, and within that context, address conditions that have changed substantially over the years.

During the course of the study the Committee was made keenly aware that the measuring and accounting process lacked credibility. Some of this is possibly due to a lack of a complete understanding and familiarity with the problems associated with the computation of the Diversion. Many of the issues concerned with credibility stem from inconsistencies in quality assurance. The problem of credibility can be put in perspective by paraphrasing the divergent views held. On the one hand is the view that it will be impossible to account for diversions at Lockport once TARP is completed, while the other holds that measurement in the vicinity of Lockport can easily be accomplished.

The Committee recognizes that effective quality assurance is an important factor in judging credibility. There is an awareness of the popularly held belief that the appearance and general state of repair of a facility reflects an attitude toward maintenance that is indicative of interest and concern for matter of accuracy and reliability as well. An attitude of "laissez faire" seems to prevail at times among some of the participating parties concerning these matters.

An essential cornerstone for restoring credibilty to the diversion program will be the USCE's role in carrying out its responsibility to provide continuous supervision, direction, and examination of the measuring device calibration, measurements, data gathering, and computations. The annual report, on the diversion measurements and computations, required of the USCE is, among other things, a certification that the diversion program is in compliance with the Supreme Court Decree.

The findings and opinions of the Committee are in contradiction with the conclusions of the USCE's accounting report for the period ending February 29, 1980 certifying the adequacy of the diversion measurement.

5.1.1 Dynamics of the Diversion System

The amount of water diverted from Lake Michigan is measured and accounted for at Lockport, which is located approximately 35 miles from the shore of Lake Michigan. Because the measurement is not at the lake front, a complicated series of deductions and additions to the total flow at Lockport is made in order to determine the amount of diversion. As changes in the Diversion system are made, such as the O'Hare WRP, the accounting computations are revised. In effect, the diversion system is a dynamic process. Future allocation of Lake Michigan water to suburban areas outside the diverted watershed is projected and will further increase flows that by-pass Lockport. The TARP system, which is now under construction, will also result in changes in the diversion system, and therefore, in the computation. Moreover, the flattening of storm hydrographs at Lockport resulting from TARP operation will shift the use of discharge devices from Controlling Works and sluices toward the turbines, thereby increasing the relative importance of accurate turbine discharge measurements. As the various phases of TARP are completed, the diversion computations will have to be revised. The ultimate construction status of TARP will dictate the impact of TARP on the diversion and nondiversion computations.

5.1.2 Evaluation of the Diversion Components

The Committee's evaluation of the current status of the components of Diversion are summarized in Table 7. The adequacy of methodologies used for measurement, computation, accounting, and quality assurance for each component is rated with respect to state-of-the-art practices. The yes and no field simply reflects the Committee's judgement that a particular methodology is, or is not, fully equivalent to the best engineering practices in concept and application. For the purposes of the tabular summary, no attempt has been made to quantify the deficiency for methods that fall short of full equivalency.

The Lockport flow components are deficient in practically every respect. The basic fluid principles, physical laws, and data requirement for these hydraulic components are widely understood and a yes/no judgement of adequacy is relatively straightforward. However, judgements for some of the nondiversion, and runoff and infiltration components could not be made quite so decisively. This was especially true for methods based on hydrologic similarities, extrapolations, and indices. Perhaps the most revealing aspect of the table is the Committee's view that every component is deficient with respect to quality assurances. Generally, these deficiencies cover the full range of elements, from a simple flow measurement to the final endorsement of activities during the 5-year accounting periods.

In addition, the probable error in the computed flow has been estimated and shown in Table 7 for each of the Lockport flow components. The error is the cumulative effect of deficiencies in the four methodological elements, and its value is usually fixed. However, the turbine, exciter, and leakage component errors tend to increase with time. This is important because these components account for about 80 percent of the Lockport flow. Efforts to determine the time variations of

TABLE 7
EVALUATION OF KEY COMPONENTS OF LAKE MICHIGAN DIVERSION

Report Section	Element of Diversion	State-of-the-Art Evaluation[1/] Flow Determination Measurement Concept	Measurement Technique Application	Computation/ Data Reduction	Flow Accounting Application to Diversion	Quality Assurance Calibration/ Documentation Procedures	Accuracy Estimated Probable Error (Percent)	Percent of[2/] Total Flow at Lockport	Relative[2/] Importance of Error (Percent)
	TOTAL FLOW AT LOCKPORT								
2.1.1	Turbines	No	No	No	Yes	No	-15	78.0	71.5
2.1.2	Exciters	No	No	No	Yes	No	-15	0.8	0.7
2.1.3	Sluice Gates	Yes	No	Yes	Yes	No	+15	6.4	5.9
2.1.3.6	Leakage	Yes	No	No	No	No	-15	-	-
2.1.4	Controlling Works	No	No	No	No	No	-30	2.6	4.8
2.1.5	Locks								
2.1.5.1	Lockages	Yes	Yes	Yes	Yes	No	±2	11.0	1.3
2.1.5.2	Leakages	Yes	No	No	No	No	±25	1.1	1.6
2.2	Industrial Diversion	Yes	Yes	Yes	Yes	No	±5	2.7	0.9
	NON-DIVERSION								
2.3	DOMESTIC PUMPAGE								
2.3.1	Ground Sources Lake Michigan Watershed	Yes	Yes	Yes	Yes	No			
2.3.2	Ground Sources Outside Lake Michigan Watershed	Yes	Yes	Yes	Yes	No	±15	5.6	5.1
2.3.3	Indiana and Wisconsin Watershed	Yes	Yes	Yes	Yes	No			
2.4	STORM RUNOFF AND INFILTRATION ILLINOIS WATERSHED								
2.4.1	Upper Des Plaines Watershed I&I								
	Upper Des Plaines Pumping Station	Yes	Yes	Yes	Yes	No	±5		
	New Sewers	Yes	No	Yes	Yes	No	±30	2.7	4.9
	Old Sewers	No	Yes	No	Yes	No	±50		
2.4.2	Summit Conduit	Yes	No	Yes	Yes	No	±50	0.3	0.9
2.4.3	Des Plaines Watershed South of the Main Canal	Yes	*3/	*3/	Yes	No	±20	1.9	2.3
2.4.4	Sewer System 13A	Yes	Yes	Yes	Yes	No	+2	-	-
2.4.5	O'Hare WRP (Interim)	Yes	Yes	Yes	Yes	No	±50	-	-

1/ A "no" signifies that one or more aspects of the methodology is not state-of-the-art in concept and/or application.
2/ Based on average annual flows of diversion components for the period of 1961-1974.
3/ Insufficient information to evaluate the state-of-the-art.

the leakage errors have been only partially successful. The estimated probable error for the turbines is based on a general synthesis of turbine experiences rather than Lockport measurements. To put this error estimate into perspective, the level of effort involved in improving the measurement accuracy should be in line with the relative importance of the diversion component and its impact on the total diversion figure. To express this, the estimated probable error was evaluated in terms of the relative importance of error expressed in terms of the average annual flow at Lockport. Clearly, the Lockport turbines represent the diversion component for which flow measurement errors are of the greatest significance. Also, the numbers show a probability of a biased under-measurement of the total flow. Second, the Lockport sluices and Controlling Works discharges and the domestic pumpage and I&I flows are of about equal significance.

Table 8 lists estimates of the improvements of flow measuring accuracy that can be achieved by adopting the Committee's recommendations and estimates of the first costs of doing so. The adoption of a specific recommendation to improve the flow measuring accuracy of a particular diversion component depends upon the following factors:

- the importance of that particular diversion component to the Diversion system;

- the magnitude of the improvement in flow metering accuracy that can be achieved; and

- the cost of performing the changes and adopting the recommendations.

Taking these factors into account, the Committee concluded that if any of the alternative recommendations are chosen for implementation, the effort should be focused on the turbine rating first, the Controlling Works second, the Lockport sluices third, the Lockport lock leakage fourth, the I&I estimation fifth, etc.

TABLE 8

PROPOSED ALTERNATIVES TO THE LAKE MICHIGAN DIVERSION FLOW MEASURMENTS AND ACCOUNTING PROCEDURES

Measurement Component	Improved Accuracy (Percent)	Estimated Cost Range (K$)
Quality Assurance Program[1/]	-	-
Lockport Turbines	From 15 - 25 to 1.5 - 5	80-150
Lockport Powerhouse Sluice Gates	From 15 to 5	20-80
Lockport Controlling Works	From 30 to 5	20-80
Lockport Lock - Leakage	From 25 to 10	10-20

[1/] This program will permit acceptance of stated accuracies.

5.2 RECOMMENDATIONS

5.2.1 Master Plan for Diversion Accounting

The Committee recommends a master plan for the management of the Lake Michigan Diversion program. The plan should be in accord with the Supreme Court Decree and the State of Illinois water allocation plans and should address the following points:

- Insure a level of accuracy for the diversion flow record that is consistent with best current engineering practices (about 2-5 percent).

The required accuracy can be insured by adopting recommendations which deal with the measurements and accounting for each of the diversion components.

- Restore credibility to the diversion program.

The restoration of credibility can be achieved largely though acceptable quality assurance programs, third party technical review, and improved communication among the interested parties.

- Provide operational flexibility to deal with the watershed dynamics.

Operational flexibility is essential for a dynamic program with a projected time-frame of 40 years.

Future priorities and demands for water allocations, changing diversions into and from the basin, watershed dynamics, and an important, but aging waterway are a few of the problems that will pose challenges to the diversion program during the next 40 years. Without attempting to define the future needs of the program several needs are clearly obvious. These include; an evaluation of alternatives to the Lockport measurement system, and expansion of the monitoring system for the measurement and determination of the hydrologic response of TARP and other modifications to the Lake Michigan watershed. Equally obvious is that the diversion program should become an integral part of a real-time waterway system operational model capable of optimizing the use of Lake Michigan water.

5.2.2 Quality Assurance

The Committee recommends:

- the development of an "Operational Procedure Manual" delineating specific technical procedures to be used in the diversion compilations as an integral part of the Master Plan for the diversion program;

- the initiation of a quality assurance program which is more formal than the current practice, and which should include written, checked, and filed records of:

  - all major and important minor maintenance and repair to flow control structures, flow metering devices, valves, gates, seals, etc.;

  - independent checks of rating curves and calibration factors; and

  - periodic recalibration of all flow metering devices.

Such records as these would show that the flow metering devices are kept well maintained and calibrated, and that the values of flows measured are as accurate as they should be. These records would also improve mutual credibility among the parties concerned with Lake Michigan Diversion.

Significant improvement in diversion measurement accuracy can be expected when trained people adhere to well planned quality assurance procedures. Unavoidable errors are easier to spot and to correct, and people gain confidence in their work.

- The USCE annual report on the diversion should reflect the processes leading to certification of the diversion flow. Among other things, it should describe:

    - activities undertaken during a year by any of the parties dealing with the calibrations of the measuring devices, measurements, data collection and analysis, as part of the diversion program;

    - the USCE's activities in regards to supervision, and direction, and audit;

    - details of program review pertinent to identification of deficiency, corrective measures, and technical evaluation; and

    - projects and studies needed, to be initiated, or planned to insure continued compliance to the Decree.

It is difficult to estimate the cost of planning and implementing such a program, but from the experience of the Committee, this cost is often partially offset by the improved quality of the product or operation.

5.2.3 Alternatives to Measurement at the Lockport Facilities

The Committee recommends:

- o A study to evaluate alternatives to the Lockport facilities for the measurement of diversion flow.

The most promising alternative measurement technique is considered to be the acoustical velocity meter system. The evaluation of the AVM system should include:

- o A reconnaissance of the canal downstream from the Cal-Sag junction for the selection and assessment of potential sites. (A preliminary cursory investigation reveals that a good site may be difficult to find.)
- o Determine availability of shore easements and docking restrictions for prospective sites.
- o Develop specifications and contractual arrangement for the acquisition and installation of the AVM system.
- o Initiate a testing period of the AVM system to evaluate its performance as an alternative or supplement to the Lockport powerhouse measurement components.

The potential advantages of an AVM system in terms of accuracy, reliability, operational simplicity, and costs over the present measurement method merits the most serious consideration.

5.2.4 Lockport Turbines (Section 2.1.1.4)

The Committee recommends:

- o Each turbine be equipped with its own headwater and tailwater elevation gages which should be located as described in the original turbine contract.

- o Each turbine be rated periodically, for example every 5 years, and immediately before and after important repair or alterations. The ratings could be performed by either of the following methods:

  - Cosine-component current-meter traverses placed in the scrollcase bulkhead slots. It is recommended that the North Pacific Division USCE assist with possible lease of equipment and trained personnel (1.5-2 percent on Q).

  - Current-meter traverses placed in the canal at a location near the one used in 1979 (4) (3-4 percent on Q). An experienced consultant should be employed to review the test plans and to supervise the measurement and data reduction.

The Committee feels that the bulkhead slot measurements would require much higher capital investment in equipment and personnel training than the canal measurements. However, the former method would also offer flexibilities and other advantages that might make it more attractive than the latter in the long run.

- o Serious consideration be given to replacing the present turbine-generator-head flow measurement system with the more direct turbine-pressure metering system utilizing the Winter-Kennedy piezometer taps.

5.2.5 Lockport Sluice Gates (Section 2.1.3.5)

The Committee recommends:

- The rating tables for the computation of discharge through the sluice gates be reexamined to:

  - explain the discrepancies between the MSD rating and the model study data; and

  - adjust the approximate model ratings to reflect the influence of the approach conditions and trash rack which are not reflected in the model study.

- A physical scale model study of the sluices could be performed by a competent laboratory including only the most useful gate operating schemes, but identifying properly tailwater effects, approach flow effects, the onset of free discharge under the gates, and the effects of operation of adjacent bays. The model study should not be undertaken without a critical review of the proposed test plan. This alternative is less cost effective than the analytical one, and is recommended only if the latter proves unsuccessful.

- The procedures discussed in Section 2.1.3.6.3 be developed and adopted to properly measure and account for sluice gate leakage. In addition:

  - Inspect the gates following each sluicing operation to make certain that the gates are properly seated. The inspection should be documented in the daily log.

- Conduct tests whenever the leakage is determined to exceed some allowable limit.

5.2.6 Lockport Controlling Works (Section 2.1.4.5)

The Committee recommends:

o New theoretical rating curves be developed (Section 2.1.4.5).

o A physical scale model study of the gates be performed by a competent laboratory and include the effect of the canal approach flow, the mooring cells, the tailwater, and the operation of adjacent gates. The model study should not be undertaken without a critical review of the proposed test plan.However, the Committee feels that this alternative is less cost effective than the previous and should be adopted only if the former is unsuccessful.

5.2.7 Lockport Lock Leakage (Section 2.1.5.2.4)

The Committee recommends:

o Written procedures and specifications be developed and adopted to properly measure and account for lock leakage. Briefly, these procedures are:

  - leakage test (Section 2.1.5.2.4);

  - leakage accounting between tests (Section 2.1.4.2.3); and

  - leakage accounting during lock filling and emptying (Section 2.1.5.2.3).

5.2.8 Industrial Diversion (Section 2.2)

The Committee recommends:

- Flow metering devices be maintained in calibration to document the validity of the flow measurements.

- Records of the periodic calibration tests be forwarded to the responsible agency and be made available for inspection by other parties to the diversion.

5.2.9 Domestic Pumpage (Section 2.3.5)

The Committee recommends:

- domestic pumpage flows be documented and periodic reports of maintenance and calibration of flow metering devices be forwarded to the responsible agency and made available for inspection by other parties to the diversion;

- determination of the source and validity of nonpublic ground-water pumpage factor;

- reconsideration of the factors applied to East Chicago, Hammond and Whiting domestic pumpage; and

- development of procedures for estimating the net annual flow across the hydraulic summits of the Grand and Little Calumet Rivers.

5.2.10 Storm Runoff and Infiltration from the Illinois Watershed (Section 2.4.1.2)

The Committee recommends:

- institution of the procedures for periodic inspection maintenance, calibrations, and data reduction for the Upper Des Plaines Pumping Station flow metering equipment and records;

- reevaluation of the I&I factors on the basis of available I&I studies in the MSD service area, and consideration for establishing index areas for the purposes of future modification; and

- development of consumptive use estimates for application to domestic pumpage in the computation of return flow.

5.2.11 Summit Conduit (Section 2.4.2.4)

The Committee recommends:

- consideration be given to referencing discharge to a water surface elevation upstream from the entrance to the conduit;

- the bubbler orifice be moved downstream to a zone of hydrostatic pressure conditions if the depth of flow reference is to be continued;

- reference elevations and staff gages be established to provide independent water surface elevations or gage heights; and

- current-meter measurements contain enough point velocity observations to define the velocity distribution in the measurement cross section.

5.2.12 Des Plaines Watershed South of the Main Channel (Section 2.4.3)

The Committee recommends:

- The measurement and documentation of land use parameters in this area of the Des Plaines and Hart Ditch watersheds that are indicative of urbanization.

5.2.13 Sewer System 13A (Section 2.4.4)

The Committee recommends:

- The periodic comparisons of flow meter output and pump characteristics be documented.

6.0 REFERENCES

1. USCE, Chicago District, Lake Michigan Diversion, Reevaluation of Methodology for Monitoring Diversion Components, August, 1976.

2. USCE, Chicago District, Report on Calibration of Lockport Turbine Units, January 21, 1937.

3. USCE, Detroit District, "Chicago Sanitary and Ship Canal Discharge Measurements at Lemont, Illinois and Lockport Powerhouse Tailrace, Lockport, Illinois from 25 May through 1 June 1977," July 1977.

4. USCE, Detroit District, "Chicago Sanitary Ship Canal Discharge Measurements at Lockport, Illinois from August 21, through September 17, 1979," January 1980.

5. USCE, Detroit District, "Comparative Analysis of the 1977 and 1979 Chicago Sanitary and Ship Canal Discharge Measurements made at or near Lockport, Illinois," May 1980.

6. USCE, Waterways Experiment Station, "Lockport Powerhouse Discharge Measurements," Memorandum for Record, Vicksburg, Mississippi, January 1980.

7. Public Utility District No. 1 of Chelan County, "Rocky Reach Dam Turbine Ratings by the Current Meter Method," Columbia River Turbine Discharge Rating Program, Wenatchee, Washington, December 1969.

8. Muga, Bruce J., "The Lockport Sluice Gate Model Study," Thesis submitted in partial fulfillment of the requirements for the degree of Masters of Science in Civil Engineering, University of Illinois, Urbana, 1961.

9. Kirschmer, Otto, "Untersuchungen uber den Gefallsverlust an Rechen (Studies on the head loss through a rack)," Mitteilungen des hydraulischen instituts den technischen Hochschule Munchen, No. 1, pg. 21-41, Munich, 1926.

10. Spangler, J., "Untersuchungen uber den verlust an rechen bei schrages tustromung (studies on the head loss through a rack inclined to stream flow)," mitteilungen des hydraulichen Instituts der technischen Hochschule munchen, no. 2, pp. 46-60, Munich, 1928.

11. Fellenius, W., "Undersokingar betroffande fallforluster i skyddsgrinder uid vattenkraftanloggninger (studies of head loss through racks in power plants)". Transactions, Hydraulic Institute of the Royal Technical University of Stockholm, No. 5, 1927.

12. USCE, Chicago District, Inspection report National Dam Safety Program Lockport Powerhouse and Controlling Works, July 1979.

13. USCE, Chicago District, One Page Lockage Report, Written Communication - MSD to USCE, Briefing material - Technical Committee for Review of Diversion Flow Measurements and Accounting Procedures, NCCPE-HS, July 24, 1981.

14. MSD, "Results of Leakage Tests at US Lock, Locport" Semi-annual test report.

15. MSD, "Hydraulic Report Computations, " 1980.

16. Stadler, Gerald, Chief, Hydrology and Hydraulics Branch, USCE Chicago District, Personal Communication, September 3, 1981.

17. Fulton, Neil, IDOT, Division of Water Resources, Personal communication.

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

19. Schicht, Richard, J., Illinois State Water Survey, Letter dated November 21, 1978.

20. Cherkauer, D.S., Zvibleman, B., "Hydraulic Connection between Lake Michigan and a shallow ground-water aquifer." Vol. 19, No. 4 - Ground Water - July-August 1981.

21. Metcalf and Eddy, Inc., Waste Engineering, McGraw-Hill Book Company, New York, 1972.

22. USCE, Chicago District, "Review of Reports on Inflow/Infiltration in the MSD Service Area," Compiled by Jeffrey Frauenfelder, Hydrology of Hydraulic Branch (NCCPECH), 28 August 1981.

23. USCE, Summit Conduit, Section A Volume IV, Technical Comittee for Review of Diversion Accounting Procedures, NCCPE-HS, February 19, 1981.

24. MSD, "Hyudraulic Report Documentation," Metropolitan Sanitary District of Greater Chicago, 1981.

25. U.S. Supreme Court, Direct Testimony, State of Wisconsin Witness William W. Durgin, Department of Justice, Madison, Wisconsin, October Term, 1966.

26. USCE, Detroit District, "Chicago Sanitary and Ship Canal Discharge Measurements at Willow Springs from January 21 to February 6, 1970", November 1970.

LAKE MICHIGAN DIVERSION

FINDINGS OF THE TECHNICAL COMMITTEE FOR REVIEW OF DIVERSION FLOW MEASUREMENTS AND ACCOUNTING PROCEDURES

APPENDIX A

QUALIFICATIONS OF COMMITTEE MEMBERS

Qualifications of Mr. Harry H. Barnes, Jr. as a member of the Lake Michigan Diversion Technical Committee. 1/

Mr. Barnes' educational background includes a B.S. (1950) in Civil Engineering from Mississippi State University and graduate study in Water Resources Engineering (1965-66) from John Hopkins University. Mr. Barnes has served on numerous professional national and international technical committees on hydrologic and hydraulic engineering issues from 1959 to the present. Among these he has served as the chairman of the United States Committee to the International Standards Organization for liquid flow measurements, rapporteur on standardization for the World Meteorological and Organization Commission for Hydrology, and the ASCE task force on flow measurements. Mr. Barnes is a member of the National Society of Professional Engineers and a fellow in the American Society of Civil Engineers. Mr. Barnes is a registered professional engineer in the State of Virginia. Mr. Barnes is currently a private consultant in water resources and was formerly (1973-1980) Chief, Surface Water Branch, Water Resources Division, U.S. Geological Survey, Reston, Virginia.

Mr. Barnes was responsible for providing national leadership for the entire Water Resources Division program in the field of surface-water hydrology. He provided policy, technical leadership, guidance, and participation in applied research in hydrology and hydraulics, technical standards, and operational techniques for all organizational units involved in the national surface-water program. Mr. Barnes also established standards for the collection, analysis, and reporting of basic information related to the source, quantity, movement, availability, conservation, and characteristics of water resources. He initiated the first national study to analyze the effects of urbanization on the magnitude and frequency of floods.

While Chief of the Water Resources Division of the Tennessee District of the USGS, Mr. Barnes was responsible for the scientific, technical, and administrative direction of the cooperative program with local, county, state, and other Federal agencies for water resources investigations in Tennessee. He was responsible for stimulating, promoting, developing, and maintaining a comprehensive program of water-resources investigations for the state with local and other Federal agencies. He also served as coordinator for programs with the Tennessee Valley Authority for the planning, development, and management of cooperative water resources investigations in the Tennessee River basin portions of the States of Alabama, Georgia, Kentucky, North Carolina, Tennessee, and Virginia. Coordination with the Nashville and Memphis Districts of the Corps of Engineers for the Survey Districts of Arkansas, Kentucky, Mississippi, and Missouri, was also accomplished during this time.

Throughout his career, Mr. Barnes has been involved in the documentation and analysis of surface water data.

1/ As prepared by the Corps of Engineers.

Qualifications of Dr. Svein Vigander as a member of the
Lake Michigan Diversion Technical Committee. 1/

Dr. Vigander's educational background includes two years of mechanical engineering studies (1956) at Schou's Institute of Technology, Oslo, Norway; a B.S. (1958) and a M.S. (1959) in Mechanical Engineering with a major in fluid mechanics from Purdue University and a Ph. D. (1965) in Engineering Mechanics from the University of Kansas. Dr. Vigander has served on several water resources technical committees as a member or chairman from 1969 to the present. Dr. Vigander is a member in the American Society of Civil Engineers, American Society of Mechanical Engineers and International Association for Hydraulic Research. As an active member in these societies, Dr. Vigander has served on several technical committees such as the ASCE Hydraulic Division's national task force committee on flow measurement. Dr. Vigander's current position is as Head, Fluid Systems - Physical Analysis Group with the Tennessee Valley Authority Engineering Laboratory. Dr. Vigander has been in this position since 1980.

Dr. Vigander's post graduate work majored in fluid mechanics and his fifteen years of work as a research engineer at the TVA Engineering Laboratory has continued in this field. At the Laboratory he has been involved with the development and design of projects involving ducted gas flow models; physical hydraulic structure scale models specifically for the structural development and discharge rating of spillways and sluices and for filling and emptying systems for navigation locks; flow induced vibration; rotating fluid machinery; unsteady flow; multi-phase flows; flow measurements and calibration; field measurements and cavitation.

He has also participated in acceptance and performance testing of hydraulic turbines and pump turbines including the measurements of efficiency and discharge.

1/ As prepared by the Corps of Engineers

LAKE MICHIGAN DIVERSION

FINDINGS OF THE TECHNICAL COMMITTEE FOR REVIEW OF DIVERSION FLOW MEASUREMENTS AND ACCOUNTING PROCEDURES

APPENDIX B

SCHEDULE OF COMMITTEE ACTIVITIES

SCHEDULE OF ACTIVITIES

TASK NO.		COMPLETION TIME AFTER CONTRACT AWARD IN WEEKS
1.	Workshop No. 1 (5 Days)	1
2.	Workshop No. 2 (4 Days)	5
3.	Workshop No. 3 (4 Days)	10
4.	Committee Member Submit Their Findings To Chariman For Consolidation Into Draft Report.	12
5.	Workshop No. 4 (Closed) Committee Members Review Draft Report.(2 Days)	14
6.	Workshop No. 5 Chairman Submits Draft Report To Corps Of Engineers. (5 Days)	18 (2 OCT 81)
7.	Workshop No. 6 Committee Provides Final Report To The District And To All Parties For Review And Comments. (2 Days)	21 (27 OCT 81)
8.	All Parties Provide Comments To District.	26 (7 NOV 81)
9.	Workshop No. 7 District Provides Comments To Committee Members For Review; Discussion Of Comments And District's Recommendations; Formulation Of Plans For Implementation Of Recommendation. (4 Days)	28 (11 DEC 81)
10.	Committee Submits Addendum To District Addressing Comments In Task No. 9.	32 (8 JAN 82)

LAKE MICHIGAN DIVERSION

FINDINGS OF THE TECHNICAL COMMITTEE FOR REVIEW OF DIVERSION FLOW MEASUREMENTS AND ACCOUNTING PROCEDURES

APPENDIX C

MINUTES AND AGENDA OF COMMITTEE WORKSHOPS

NCCPE-HS

17 June 1981

MEMORANDUM FOR RECORD

SUBJECT: First Workshop Meeting with the Technical Committee for Review of Lake Michigan Diversion at Chicago, Illinois

1. The first workshop meeting was held in Chicago from 9 June 1981 to 12 June 1981 and was of an organizational nature for committee members only. A letter dated 28 May 1981 was sent to all of the parties concerning the details of the workshop. A copy of one of the series of letters sent to the parties is included as inclosure 1. No comments were received from any of the parties concerning the nature of the workshop. A copy of the final agenda for the workshop is included as inclosure 2. A list of participants and associated activities is included as inclosure 3. Participating in the workshop were the following individuals:

Committee Members	Corps of Engineers
Dr. William Espey	Mr. Gerald Stadler, Chief, Hydrology & Hydraulics Branch
Mr. Harry Barnes	Mr. Stephen Klawans, Mathematican, Hydrology & Hydraulics Branch
Dr. Svein Vigander	Mr. Donald Glondys, Hydraulic Engineer, Hydrology & Hydraulics Branch
	Mr. Stanley Boc, Hydraulic Engineer, Hydrology & Hydraulics Branch
	Mr. Lawrence Dunbar, Hydraulic Engineer, Hydrology & Hydraulics Branch
	Mr. Frank Kozak, Attorney, North Central Division

2. General discussions were held in the afternoon on 9 June between the committee members, Mr. Gerald Stadler, Mr. Stephen Klawans and Mr. Donald Glondys. The committee members were presented with volume six of briefing documents detailing the concepts and computations involved in the preparation of the monthly hydraulic report by the Metropolitan Sanitary District. The dates for future workshops were finalized at this time as follows: (a) Workshop two will be held in Chicago from 6 July 1981 to 10 July 1981 inclusive. (b) Workshop three will be held in Chicago from 10 August 1981 to 14 August 1981 inclusive. (c) Workshop four will be held in Austin, Texas from 31 August 1981 to 2 September 1981 inclusive.

3. In the morning and early afternoon on 10 June, Mr. Stephen Klawans gave an in-depth presentation on the preparation of the monthly hydraulic report. Each column of the report was examined in detail and the methods of computation were discussed. The presentation by Mr. Klawans was a description of the methods used by the Metropolitan Sanitary District in preparation of the report. Discussion of the first eleven columns was completed on June. In

SUBJECT: First Workshop Meeting with the Technical Committee for Review of Lake Michigan Diversion at Chicago, Illinois

In addition, Mr. Stanley Boc gave a presentation on the leakage tests at the Lockport lock and Mr. Larry Dunbar gave an in-depth talk on the Lockport Powerhouse and Controlling Works including history of flow measurements.

4. On 11 June, a field trip was taken by the committee members, Mr. Gerald Stadler, Mr. Larry Dunbar, Mr. Stanley Boc and Mr. Donald Glondys to the Lockport Powerhouse facility for the purpose of informal inspection. The powerhouse and controlling works were visited at this time.

5. In the morning on 12 June, the tentative agenda for workshop two was arranged between the District and the committee members. Pertinent administrative matters were also handled at this time. Mr. Klawans concluded the discussion of the eight remaining columns on the hydraulic report.

FOR THE COMMITTEE:

DR. WILLIAM H. ESPEY
Chairman

FOR THE CORPS:

Stephen M. Klawans

STEPHEN M. KLAWANS
Mathematician, Hydrology & Hydraulics Branch

GERALD J. STADLER
Chief, Hydrology & Hydraulic Branch

3 Incl
As stated

ENCLOSURES NOT INCLUDED IN THIS REPORT

NCCPE-H

TECHNICAL COMMITTEE FOR REVIEW OF
DIVERSION FLOW MEASUREMENTS
AND
ACCOUNTING PROCEDURES

AGENDA
WORKSHOP SESSION I

9 June 1981

1:30-3:00	General Introduction Administration for the Workshop Update the Briefing Packet
3:00-3:30	Coffee Break
3:30-4:30	History of Diversion Hydraulic Characteristics of the Canal System, Components of Diversion

10 June 1981

8:00-9:30	Preparation of the. Monthly Hydraulic Report (Lockport Powerhouse and Controlling Works)
9:30-10:00	Break
10:00-11.30	Preparation of the Monthly Hydraulic Report (deductibles and domestic uses)
11:30-1:00	Lunch
1:00-2:30	Corps Measurement Program at Lockport Powerhouse
2:30-3:00	Coffee Break
3:00-4:30	Other flow studies at Lockport

NCCPE-H

11 June 1981

8:00-4:30	Field Trip to Lockport Powerhouse, Lock and Controlling Works

12 June 1981

8:00-12:00	Conclusions, Summarization, and Recommendations for future workshops
12:00-1:00	Lunch
	Departure

NCCPE-HS

20 July 1981

MEMORANDUM FOR RECORD

SUBJECT: Second Workshop Meeting with the Technical Committee for Review of Lake Michigan Diversion at Chicago, Illinois

1. The second workshop meeting was held in Chicago from 6 July 1981 to 10 July 1981. A letter dated 22 June 1981 was sent to all of the parties which detailed the plans for the workshop and asked the parties to specify the role they wanted to have during this period. A copy of one of the series of letters sent to all of the parties is included as inclosure 1. The State of Wisconsin was subsequently the only party which expressed a desire to make a presentation to the committee members. Therefore, the Wisconsin presentation was scheduled for the third workshop. A copy of the final agenda for the workshop is included as inclosure 2. A list of participants and associated activities is included as inclosure 3.

2. A meeting was held at the Illinois Department of Transportation in the afternoon on 6 July. The purpose of the meeting was to familiarize the technical committee with the state allocation program and current IDOT activities. Mr. Daniel Injerd gave a detailed presentation on the state allocation program for Lake Michigan water. Criteria for allocations were discussed as well as projections of water demand. Mr. Robert Sasman of the Illinois State Water Survey then gave a presentation on groundwater use. Mr. Injerd followed with a discussion of current activities maintained by the State of Illinois. It was emphasized that complete cooperation will exist between the state's consultant, Harza Engineering, and the technical committee. An MFR on this meeting is included as inclosure 4.

3. A meeting was held at the Metropolitan Sanitary District on 7 July. The purpose of the meeting was to familiarize the committee members with the facility and to give them a broader insight into the procedures used by the Metropolitan Sanitary District in monitoring diversion. Mr. William Eyre gave a formal tour of the Water Control Center. Mr. Subash Patel gave a presentation on data collection and data handling and processing. The hydraulic report was reviewed, providing further reinforcement to the materials received by the committee during the first workshop. Mr. Jim Tzakis gave a talk on the computer facilities and the function of the computer program. Mr. Tzakis explained how data is entered into the terminal and answered questions from all present. Mr. Jim Krawchuk then gave a presentation on the Metropolitan Sanitary District gaging network. Mr. Gerald Stadler discussed with Mr. William Eyre the procedure for transmitting information to the committee members. Mr. Eyre related that he felt the best procedure would be for the committee members to request information from the Corps of Engineers. The Corps would then contact the Metropolitan Sanitary District for assistance in providing the needed information to the committee members. Essentially, it was determined that the Corps would act as the intermediary between the technical committee and the Metropolitan Sanitary District. The committee expressed the desire to meet with the Metropolitan Sanitary District at the next workshop for further and more extensive interrogation. The Metropolitan Sanitary District

offered its complete cooperation in dealing with the technical committee. An MFR on the 7 July 1981 meeting is included as inclosure 5.

4. A field trip was taken by MSD personnel, IDOT personnel, Corps of Engineers personnel and the technical committee on 8 July. On this excursion, the Chicago River Lock and Controlling Works pumping station 13A and the Summit Conduit were viewed. In addition, the Lockport Powerhouse and Controlling Works were revisited for further observation. An MFR detailing the events of this trip is included as inclosure 6. An additional field trip was taken by IDOT personnel, Corps of Engineers personnel and the technical committee on 9 July. This trip included a visit to O'Brien Lock and Dam and the Grand Calumet River area in Indiana. Stops were made at the Hammond Sanitary District and the East Chicago Sanitary District to view the outfall locations. The Hart Ditch at Munster and Little Calumet at Munster gages were also visited during this trip. An MFR detailing the events of this trip is included as inclosure 7.

5. A session was held on 10 July with the technical committee. Mr. Stadler reviewed with the committee members the events of the week. In addition, Mr. Stadler finalized the District's obligations to the committee in preparation for the next workshop. Mr. Larry Dunbar gave a slide presentation on the Tunnel and Reservoir Plan.

FOR THE COMMITTEE:

DR. WILLIAM H. ESPEY
Chairman

FOR THE CORPS:

STEPHEN M. KLAWANS
Mathematician, Hydrology & Hydraulics Branch

GERALD J. STADLER
Chief, Hydrology & Hydraulics Branch

7 Incl
As stated

ENCLOSURES NOT INCLUDED IN THIS REPORT.

NCCPE-H

TECHNICAL COMMITTEE FOR REVIEW OF
DIVERSION FLOW MEASUREMENTS
AND
ACCOUNTING PROCEDURES

AGENDA
WORKSHOP SESSION II

MONDAY 6 July 1981

Illinois Department of Transportation (IDOT)

IDOT MEETING

1:00-2:00	State Allocation Program
2:00-3:00	Groundwater Aquifer
3:00-4:00	IDOT Current Activities

TUESDAY 7 July 1981

Metropolitan Sanitary District of Greater Chicago Presentation

9:00-9:30	Water Control Center Tour
9:30-10:00	Data Collection System
10:00-10:30	Data Handling & Processing
10:30-11:30	ADP Facilities
11:30-1:00	LUNCH
1:00-2:00	Hydrologic Index Stations

NCCPE-H

TUESDAY 7 July 1981

2:00-3:00	Instrumentation Gages
	Lockport Powerhouse Gages
	Lockport Controlling Works
	Summit Conduit
	Pumping Station 13A
	Des Plaines Pumping Station
3:00-3:30	Sonic Measuring Devices
3:30-4:00	Miscellaneous

WEDNESDAY 8 July 1981

Field Trip (8:30-4:30)

8:30	Leave Hotel
9:30-10:00	Pumping Station 13A
10:15-11:00	Summit Conduit
11:30-12:30	LUNCH
12:30-3:30	Lockport Powerhouse & Controlling Works
4:30	ETA Hotel

NCCPE-H

	THURSDAY	9 July 1981
	Field Trip (8:30-4:30)	
8:15	Leave Hotel	
9:15-10:15	O'Brien Lock and Dam	
	Grand Calumet River Reconnaissance	
10:15-10:45	Calumet Avenue Divide	
11:00-11:15	Indianapolis Blvd. Bridge	
11:15-11:30	Kennedy Ave. Bridge	
11:30-11:45	Cline Ave. Bridge	
12:00-1:30	LUNCH	
	Little Calumet River Reconnaissance	
1:45-4:00	Little Calumet River, Hart Ditch at Munster Gage	
4:30	ETA Hotel	
	FRIDAY	10 July 1981
8:15	Leave Hotel	
9:00-10:00	Chicago River Controlling Works	
10:45-11:30	Wilmette Controlling Works	
11:30-12:30	LUNCH	
2:00-3:00	Upper Des Plaines Pumping Station	
4:00	ETA Hotel	

NCCPE-HF 13 July 1981

MEMORANDUM FOR RECORD

SUBJECT: Technical Committee for Review of Lake Michigan Diversion Workshop #2 field trip to various diversion locations

1. On Wednesday 8 July 1981, the following individuals met at the Chicago River Controlling Works (CRCW) for the subject mission:

Gerald Stadler	NCCPE-H
Larry Dunbar	NCCPE-HF
Donald Dressel	NCCPE-HF
Harry Barnes	Technical Committee
Svein Vigander	Technical Committee
William Espey	Technical Committee
William Eyre	MSDGC
Subhash Patel	MSDGC
Daniel Injerd	IDOT

2. The CRCW is located at the mouth of the Chicago River on Lake Michigan. It prevents unrestricted flow of Lake Michigan from entering into the canal and river system. CRCW is composed of a navigation lock, a fixed dam and two sets of four 10 foot by 10 foot sluice gates. The sluice gates are used to regulate flow into or, in the event of very high river levels, out of the river system. The MSDGC personnel presented a tour of CRCW including the operations control room. During the tour, MSDGC personnel answered question posed to them by the three committee members. At the committee's request, the operation of a routine lockage was performed. Leakage of Lake Michigan water into the Chicago River was observed at the hinges of the lock gates. MSDGC personnel stated that they plan on resealing the hinges in the near future. Upon completion of the tour Mr. Dunbar returned to the Corps' office.

3. From CRCW we proceeded to the 13A Southwest Pump Station located on the right bank of the Sanitary and Ship canal just North of I-55 bridge. Inclosure 1 is a description of the 13A pump station from a previous field trip MFR. A tour of the 13A pump station was lead by Mr. Dick Heil from MSDGC. The drainage area for the 13A pump station is located within the Des Plaines River Watershed and therefore the storm water flow pumped by 13A is deducted from the Lockport flow during the Lake Michigan Diversion accounting procedure. Mr. Heil stated that after the Mainstream Phase I TARP system becomes operational in 1984, the 13A pump station will be shut down permanently.

4. After inspecting the emergency spillway for 13A we drove south approximately 1000 feet until we arrived at the outlet of the Summit Conduit. The Committee members climbed down the bank to inspect the 6 foot by 7 foot concrete conduit. After inspecting the outlet, we proceeded to the location of the inlet of the Summit Conduit. The drainage area for the conduit is approximately 5.4 square miles of the Des Plaines River Watershed. The flow through the conduit is a

deductible item from the diversion report. The depth of the flow through the pipe is measured by a stage recording gage, which is connected to MSD's control center located in their office downtown. The committee members observed that a vertical pipe coming from the stage recording gage was bent 90 degrees and was not touching the water surface. Also, the orifice pipe from the gage was not securely connected and was moving back and forth in the flow.

5. After lunch, we proceeded to the Lockport Powerhouse which is maintained and operated by MSDGC. Mr. Larry O'Brien of MSDGC conducted a tour of the powerhouse for the group. The two 8,500 BHP vertical turbines with generators rated at 6,000 KW at 37.5 feet of head and 2,160 cfs of flow were in operation at the time of our visit. Also, there is two inoperable horizontal turbines located in the powerhouse. The two vertical turbines are located in Bays 1 and 2. Bays 3 and 4 no longer house turbines but are equipped with three 9 foot by 14 foot sluice gates each. During a previous visit, leakage was observed in bay 4 coming from the three sluice gates. In the interim period MSDGC has resealed the gates in bay 3 and 4. The committee was allowed to climb down into bay 4 in order to inspect the three sluice gates for any evidence of leakage. They observed that leakage was not present at any of the three gates. After the bay area was cleared of personnel, the middle sluice gate was opened full. The two other sluice gates in bay 4 were then opened full while concurrently the middle gate was closed. After the gates were opened full for a few minutes the MSDGC personnel began to close them. A problem arose when one of the sluice gates refused to close due to debris being caught at the bottom of the gate. The other two gates had closed properly and no leakage through these gates was evident.

6. Our final stop of the day trip was at the Lockport Controlling Works located two miles upstream of the Lockport Lock. The controlling works consists of seven 30 foot by 20 foot vertical lift sluice gates which allow flow from the Sanitary and Ship Canal to be diverted into the Des Plaines river which parallels the canal at this point. The elevation of the sluice gate sills is at -15.0 ft - CCD. During their last visit on 11 June 1981, the committee observed leakage through the gates. However, during this field trip no leakage through the gates was observed by the committee. Mr. O'Brien from MSDGC proceeded to lift gate number 6 and after gate number 6 was opened gate number 7 was lifted. The gates were then closed and again the committee check for leakage through gates 6 and 7, but none was observed.

7. A number of photographs were taken at each the above locations.

Donald Dressel
Hydraulic Engineer

Gerald Stadler
Chief, Hydrology and Hydraulics Branch

NCCPE-HS 20 July 1981

MEMORANDUM FOR RECORD

SUBJECT: Field Trip to O'Brien Lock and Dam, Hammond Sewage Treatment Plant, East Chicago Sewage Treatment Plant, Grand Calumet River Area and Little Calumet River Area

1. On 9 July 1981, the following individuals met for the purpose of undertaking the subject field trip:

Gerald J. Stadler	Corps of Engineers
Jim Mazanec	Corps of Engineers
Frank Rupp	IDOT
Dr. William Espey	Technical Committee
Mr. Harry Barnes	Technical Committee
Dr. Svein Vigander	Technical Committee

The purpose of the trip was to familiarize the committee members with the area and to allow the committee an opportunity to gather information for use in their study on Lake Michigan Diversion at Chicago. A briefing packet which included photographs of the Grand Calumet River, detailed maps of the area and diagrams pertaining to the subject treatment plants and O'Brien Lock and Dam was provided to the committee members. During the course of the trip, Corps personnel briefed the committee members on various past, current and future hydrology and hydraulics activities in both the Grand Calumet and Little Calumet River basins.

2. A stop was made initially at O'Brien Lock and Dam. The committee members met with the assistant lockmaster, Mr. Art Aylmer, to discuss various matters of interest. The committee's questions on gage locations and gage types were addressed by Mr. Aylmer. In addition, the data collection process was reviewed. The committee members observed the lock and controlling gate mechanism and photographs were taken before proceeding.

3. Stops were made to view the Grand Calumet River at various locations in Illinois and Indiana. The first significant view of the river was obtained at Hohman Avenue. There was evidence of a gage at Hohman Avenue and photographs were taken at the site. Additional views of the river were obtained at Calumet Avenue and Indianapolis Blvd. Photographs of the river were taken at Indianapolis Blvd. Stops at Burnham Avenue, Sohl Avenue and Columbia Avenue provided views of the river.

4. Stops were made to view the Hammond Sewage Treatment Plant and the East Chicago Sewage Treatment Plant for informational purposes. The Hammond Sewage Treatment Plant is located just north of the intersection of Columbia Avenue and the Grand Calumet River. The plant and industrial operations coordinator is Mr. Stewart Roth. The design load of the plant is 48 MGD and daily information on plant flows is provided to the State of Indiana and Region five of the

NCCPE-HS
SUBJECT: Field Trip to O'Brien Lock and Dam, Hammond Sewage Treatment Plant, East Chicago Sewage Treatment Plant and Grand Calumet River Area

United States Environmental Protection Agency in Chicago twenty days after the end of each month. No information is submitted to MSD or the State of Illinois. It should be noted that the current accounting procedure assume 25 percent of the plant effluent reaches Lockport. The outfall locations into the Grand Calumet River were observed by the group. The East Chicago Sewage Treatment Plant is located just north of the intersection of Indianapolis Blvd. and the Grand Calumet River. The operations supervisor for the wastewater division is Mr. Peter Baranyal. Mr. Baranyal provided the following facts and information about the plant: (a) The design load of the plant is 20 MGD with a maximum load of 25 MGD. (b) The city of Whiting has no official sewage treatment plant. Sewage from the city of Whiting is sent to the Hammond Sewage Treatment Plant for treatment. The outfall location into the Grand Calumet River was observed. Additional observations were made of the bypass structures at the plant. Daily information on plant flows is provided to the State of Indiana and U.S.E.P.A.

5. The trip was continued east along the Grand Calumet River to the United States Ship Canal and then proceeded to the Little Calumet River. Stops were made to view the Hart Ditch at Munster gage and the Little Calumet River at Munster gage.

Steve Klawans
STEVE KLAWANS
Mathematician,
Hydrology and Hydraulics Branch

GERALD J. STADLER
Chief, Hydrology and
Hydraulics Branch

Incl 7

NCCPE-HS 17 August 1981

MEMORANDUM FOR RECORD

SUBJECT: Third Workshop Meeting With The Technical Committee for Review of Lake Michigan Diversion at Chicago, Illinois

1. The third workshop meeting was held in Chicago from 10 August 1981 to 14 August 1981. A letter dated 31 July 1981 was sent to all of the parties which detailed the plans for the workshop. A copy of one of the series of letters sent to all of the parties is included as Inclosure 1. A copy of the final agenda for the workshop is included as Inclosure 2. A list of participants and associated activities is included as Inclosure 3.

2. A public workshop was held at the Corps of Engineers office in the afternoon on 10 August. During this time, the State of Wisconsin gave its presentation to the committee members on the current and proposed flow network at the Lockport Powerhouse. Five major points were discussed as follows: (a) measurement of flows through the powerhouse (b) 1979 Corps study indicating undermeasurement of flows at Lockport (c) causes of undermeasurement of flows at Lockport (d) errors in diversion accounting (e) recommendations. The presentation was given by Ms. Maryann Sumi and Mr. Kenneth Potter. The committee members asked for a written summary of the presentation by the State of Wisconsin. The State of Wisconsin stated that it would provide a summary to the committee in the near future.

3. A field trip was taken by the committee members, Corps personnel and MSD personnel to view the Wilmette Control Structure in the morning on 11 August. The facilities were inspected at this time. In addition, the stage and flow record system was reviewed. Gate operation was also observed. A committee working session for general discussion was held in midmorning and an additional session was held at the headquarters of the Metropolitan Sanitary District in the afternoon. The purpose of the session at the Metropolitan Sanitary District was to obtain information on a series of questions compiled by the committee pertaining to diversion measurements.

4. In the morning on 12 August, the committee attended a presentation given by Keifer Engineering to the Chicago District concerning the Tunnel and Reservoir Plan (TARP). A field trip was then taken by the committee members, Corps personnel and MSF personnel to view and inspect the Upper Des Plaines Pumping Station. The Upper Des Plaines Pumping Station is an index station by which inflow and infiltration for the entire Des Plaines Watershed reaching the canal system is determined. Personnel from the Metropolitan Sanitary District gave a presentation on the operation of the three parallel pumps as well as the flow measurement and recording system. A committee working session was held in the afternnon, where a discussion of the changes in hydraulic report format and computations ensued. Several hydraulic reports from recent years were studied in order for the committee members to obtain a fuller appreciation of the changes in accounting procedures.

5. The 13th of August was devoted to a field inspection of the canal system by boat. Representatives from the Chicago District, MSD, IDOT, and the 3 member committee were present. The inspection trip began at 0800 hours Burnham Harbor, proceeding north to the Chicago River Lock, into the Sanitary and Ship Canal, to Lockport, back to the Cal-Sag Channel, to the O'Brien Lock, into the lake and terminating at Burnham Harbor at approximately 2000 hours. During the trip the representatives of MSD pointed out sites of interrest to the committee, such as the Willow Springs Ultrasonic measuring site, the Lockport Power House and others. Many discussions of the canal system were held and the trip provided to the 3-member committee and the others a better understanding of the entire system.

6. A final committee working session was held on 14 August. General discussions were held between the Corps and committee members, including a summary of data to be collected and furnished the committee at their request and the making of plans for the fourth workshop meeting.

FOR THE COMMITTEE:

DR. WILLIAM H. ESPEY
Chairman

FOR THE CORPS:

Stephen M. Klawans

STEPHEN M. KLAWANS
Mathemantician, Hydrology & Hydraulics Branch

GERALD J. STADLER
Chief, Hydrology & Hydraulics Branch

3 Incl
As stated

ENCLOSURES NOT INCLUDED IN THIS REPORT.

NCCPE-H

5 August 1981
Revised - 7 August 1981

TECHNICAL COMMITTEE FOR REVIEW OF
DIVERSION FLOW MEASUREMENTS
AND
ACCOUNTING PROCEDURES

AGENDA
WORKSHOP SESSION III

Monday		10 August 1981
	State of Wisconsin Presentation	
1:30-3:30	Current and Proposed Flow Network at Lockport Powerhouse	
3:30-4:00	Miscellaneous	
Tuesday		11 August 1981
9:00-9:30	Wilmette Control Structure	
10:30-11:30	Committee Working Session	
11:30-12:30	Lunch	
1:30-4:30	Metropolitan Sanitary District Working Session	
Wednesday		12 August 1981
8:00-10:00	Committee Working Session	
11:00-12:00	Upper Des Plaines Pumping Station	
12:00-1:00	Lunch	
1:00-4:00	Committee Working Session	
Thursday		13 August 1981
8:00-6:00	Boat Trip on Canal System	
	Lunch (on Boat)	
	Boat Trip on Canal System	
Friday		14 August 1981
9:00-12:00	Committee Working Session	
12:15	Workshop III Concludes	

NCCPE-HS

5 October 1981

MEMORANDUM FOR RECORD

SUBJECT: Fourth Workshop Meeting With The Technical Committee for Review of Lake Michigan Diversion at Chicago, Illinois

1. The fourth workshop meeting was held in Austin from 31 August 1981 to 4 September 1981. A copy of the final agenda for the workshop is included as Inclosure 1.

2. The purpose of the workshop session was to allow the committee members to work privately in order for a draft report to be assimilated. Committee working sessions were held on 31 August, 1 September and 2 September. Joint meetings between the committee members and Chicago District personnel were held on 3 September and 4 September. The joint meetings provided the necessary time for an open discussion of the draft report.

FOR THE COMMITTEE:

DR. WILLIAM H. ESPEY
Chairman

FOR THE CORPS:

Stephen M. Klawans
STEPHEN M. KLAWANS
Mathematician, Hydrology & Hydraulics Branch

GERALD J. STADLER
Chief, Hydrology & Hydraulics Branch

AGENDA

WORKSHOP NO. 4
August 31 - September 4, 1981

LAKE MICHIGAN DIVERSION
Technical Committee for
Review of Diversion Flow
Measurements and Accounting Procedures

MONDAY 31 August 1981

1:00 p.m.	A) Discuss scope and project schedule B) Review draft report outline C) Include new material and initiate revisions D) Assignment of specific report sections to each Committee member (editing)
5:00 p.m.	Adjourn

TUESDAY 1 September 1981

8:30 a.m.	Individual Working Sessions
12:00 p.m.	Lunch
1:00 p.m.	Committee Working Session
4:30 p.m.	Review progress - New assignments
5:00 p.m.	Adjourn

WEDNESDAY 2 September 1981

8:30 a.m.	Individual Working Sessions
12:00 p.m.	Lunch
1:00 p.m.	Committee Working Session (The goal of this session is to have a revised draft by adjournment)
5:00 p.m.	Adjourn (USCE Chicago District Arrival)

THRUSDAY 3 September 1981

8:30 a.m.	Joint Meeting - Committee and USCE
	Open Discussion of Draft Report
12:00 a.m.	Lunch

AGENDA
Workshop No. 4

LAKE MICHIGAN DIVERSION

THURSDAY Cont'd.

1:00 p.m.	Individual Work Sessions Assignments and Revision to Draft
5:00 p.m.	Adjourn

FRIDAY 4 September 1981

8:30 a.m.	Review Draft Revisions Continue Preparation of Revised Draft Report
12:00 p.m.	Lunch
1:00 p.m.	Departures

NCCPE-HS 5 October 1981

MEMORANDUM FOR RECORD

SUBJECT: Fifth Workshop Meeting With The Technical Committee for Review of Lake Michigan Diversion at Chicago, Illinois

1. The fifth workshop meeting was held in Austin from 28 September 1981 to 2 October 1981. A copy of the final agenda for the workshop is included as inclosure 1.

2. The purpose of the workshop session was to allow the committee members to work privately in order for the draft report to be refined and revised accordingly. Committee working sessions were held on 28 September, 29 September and a portion of 30 September. Joint meetings were held on 30 September, 1 October and 2 October. The joint meetings provided the necessary time for an open discussion of the draft report.

FOR THE COMMITTEE:

W H Espey

DR. WILLIAM H. ESPEY
Chairman

FOR THE CORPS:

Stephen M. Klawans

STEPHEN M. KLAWANS
Mathematician, Hydrology & Hydraulics Branch

GERALD J. STADLER
Chief, Hydrology & Hydraulics Branch

AGENDA

WORKSHOP NO. 5
September 28 - October 2, 1981

LAKE MICHIGAN DIVERSION
Technical Committee for
Review of Diversion Flow
Measurements and Accounting Procedures

Monday 28 September 1981

1:00 p.m.	Review Revised Draft (No. 3)
	Initiate Discussion
5:00 p.m.	Adjourn

Tuesday 29 September 1981

8:30 a.m.	Draft Review and Assignment of Specific Report Sections (editing)
12:00 p.m.	Lunch
1:00 p.m.	Committee Working Session, Produce Draft (No. 4)
5:00 p.m.	Adjourn

Wednesday 30 September 1981

8:30 a.m.	Committee Working Session
12:00 p.m.	Lunch (USCE Chicago District Arrival)
1:00 p.m.	Review Supplemental Information and Water Budget Data
5:00 p.m.	Adjourn

Thursday 1 October 1981

8:30 a.m.	Joint Review and Revision of Draft (No. 4)
12:00 p.m.	Lunch
1:00 p.m.	Prepare Final Report
5:00 p.m.	Adjourn

Friday 2 October 1981

8:30 a.m.	Discussion of Final Report
12:00 p.m.	Lunch
1:00 p.m.	Departures