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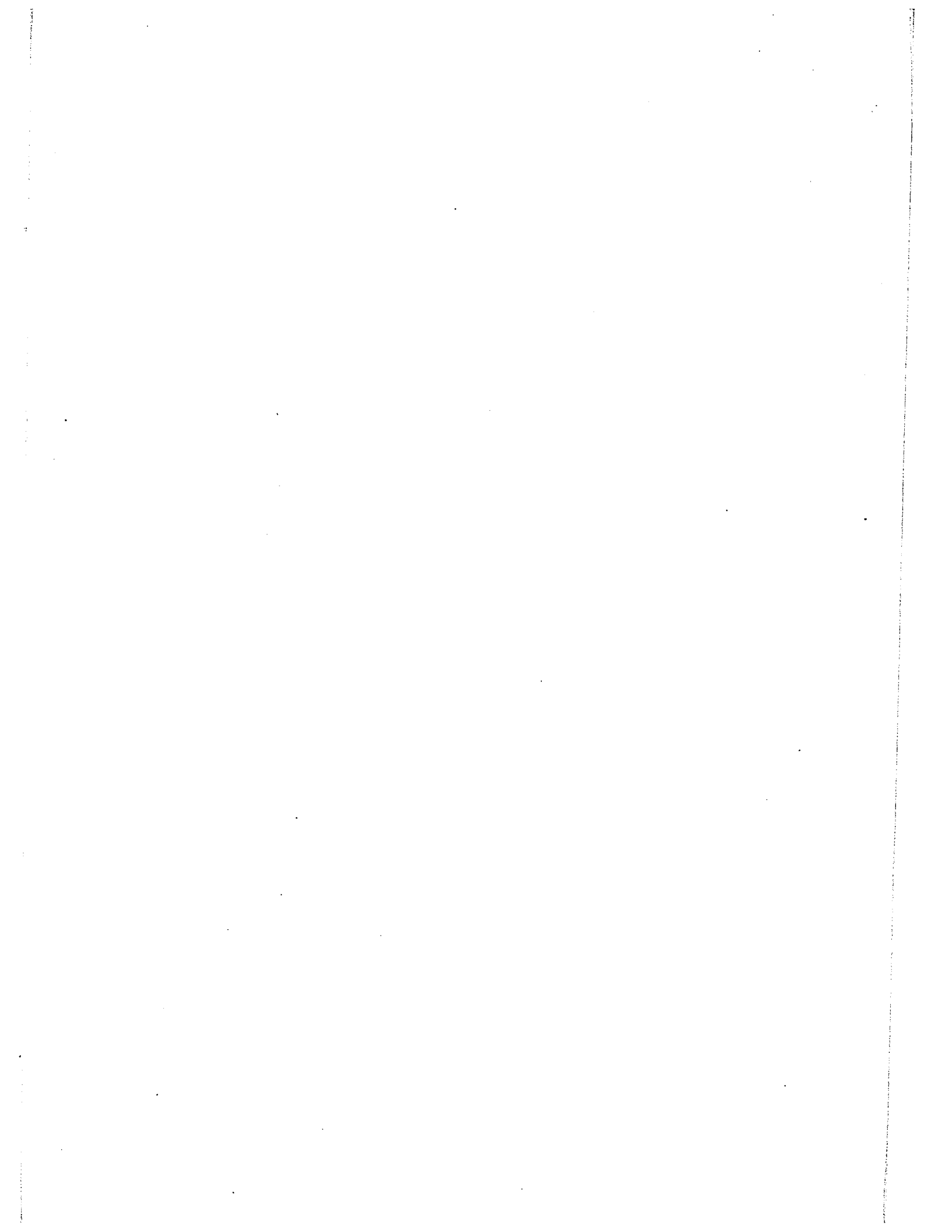
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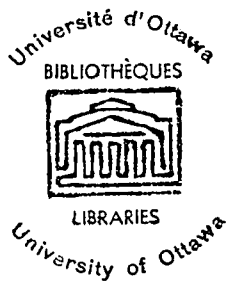
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A SYSTEMS APPROACH TO  
GREAT LAKES REGULATION

by

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Submitted in partial fulfillment  
of the requirements for the degree of  
Master of Engineering



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RESUME

The objective of this study is to investigate the application of simulation models and mathematical programming techniques to the regulation of the levels and outflows of the Great Lakes. The simulation model combines a method of generating synthetic supplies with a routing model for the lakes and their connecting channels. The mathematical programming method used is a single reservoir dynamic programming algorithm extended to the four-lake system by an iterative technique.

Based on experimentation with the methodology, it is apparent that the complexity of the problem is such as to necessitate the systems approach. It was found that the dynamic programming model is suitable as a preliminary tool in designing operating rules for the Great Lakes System, and that the simulation model used adequately represents physical conditions.

TABLE OF CONTENTS

<u>Chapter</u>		<u>Page</u>
1	INTRODUCTION	1
2	REVIEW OF LITERATURE	3
	Optimization of Reservoir Operating Rules	3
	History of Regulation Studies	7
	Present Operational Regulation Plans	8
	Purpose and Scope of Studies	10
3	SYSTEM DESCRIPTION	14
	Great Lakes System	14
	Description	14
	Geology	15
	Land Use and Natural Resources	16
	Regulatory Works	17
	Present Water Uses	20
	Hydrology of the System	22
	Available Data	22
	Runoff	22
	Net Basin Supply	26
	Winter and Weed Retardation	27
	Flow-Level Model	28
	Supply Synthesis	31
	Supply Forecasting	33

Table of Contents (Cont'd.)

<u>Chapter</u>		<u>Page</u>
4	THEORETICAL DEVELOPMENTS	35
	Maximization of Economics Benefits	35
	Approach	35
	Dynamic Programming	35
	Dynamic Programming Applied to Reservoir Operation	37
	Extension to a Four Lake System	41
	Computational Efficiency	45
	Reservoir Operating Rules	49
	Overall Approach	49
	Operation	49
	Design	55
5	DISCUSSION AND CONCLUSIONS	57
	BIBLIOGRAPHY	60
	ACKNOWLEDGEMENTS	62

LIST OF TABLES AND ILLUSTRATIONS

<u>Figure</u>		<u>Page</u>
1	Organization of Study	12
2	Land and Water Areas	14
3	Runoff Map	24
4	Mean Basin Land Runoff	25
5(a)	Flow-Level Model	29
5(b)	Detailed Routing Through Lakes Michigan-Huron, St. Clair, and Erie	29

## INTRODUCTION

In recent years, methods of managerial decision making have undergone rapid changes. These changes have largely been the result of an increasing acceptance of systems analysis, and operations research. While early applications of these techniques were mainly in the defence and aerospace industries, recently there has been an increased willingness to accept them in the solution of civilian problems. Water resource planning is one of the areas in which these new managerial methods are having a substantial impact.

There are several reasons for the adoption of systems analysis techniques in water resource planning. Firstly, the increasing complexity of recent studies has made it difficult, if not impossible to apply traditional approaches. The increased complexity is due in part to a growing demand for basin wide and regional planning, whereas previously, many fragmented political constituencies were the ultimate arbitrators of water resource use. Also, traditional concerns such as hydro-electric power, flood control, etc. are now having to compete with other aspects, such as recreation and water quality management. Secondly, it is now generally recognized that the overall system design is likely to be better by using the mathematical modelling and computing approach of operations research, than that achieved by conventional techniques. Lastly, and perhaps most important is the rapid development of computing machinery and its widespread availability.

This report describes applications of mathematical programming and simulation methods to a joint Canada-United States study of the water resources of the Great Lakes. Several aspects of the Great Lakes studies are discussed including previous regulation studies,

current operating policies, a description of the physical system, and a brief discussion of the hydrology of the basin. Also described are various computer programs used in current regulation studies including a flow-level model, a method of generating hydrologic inputs, and a forecasting model. The main emphasis is placed on methodology being used to optimize reservoir operating rules based on economic criteria. The adequacy of the modelling techniques is discussed.

## REVIEW OF LITERATURE

### Optimization of Reservoir Operating Rules

Although both design and operating aspects will be discussed in this report, the main emphasis will be placed on reservoir operation. Reservoir operation requires rules to indicate the amount of water which should be released at any point in time. While extensive research has been reported on designing water resource systems for increased economic efficiency, only recently has the problem of applying economic criteria to the derivation of rules for reservoir operation received much attention. Economically derived operating rules can be used to maximize benefits from an existing facility, or in planning new water resource development schemes.

In order to derive a set of operating rules based on economic inputs, it is necessary to specify the exact basis on which the best solution is to be judged. This must be a single index that can be expressed as a function of either reservoir levels or outflows. This index is referred to as the "objective function". Important intangible factors which cannot readily be expressed as part of the objective function can be handled by constraining the solution to take them into account. These limitations, as well as other physical limits are referred to as constraints, or operational criteria in this report.

With the advent of the electronic computer, mathematical optimizing procedures are being used ever increasingly by water resource planners. These procedures fall into three basic groups: linear programming or some nonlinear adaptation thereof; dynamic programming; and gradient methods. The objective of any optimizing procedure is to define a policy ie. to assign particular values to those aspects of the system which are subject to some degree of control. The controllable or partially controllable

inputs to the system are referred to as decision variables, while the variables describing the system proper at any time and place are known as state variables.

There is no general agreement amongst water resource analysts as to which optimizing technique is most appropriate for deriving operating rules. Gradient methods are undoubtedly the most powerful, and the most versatile. A good summary of the application of gradient methods in water resource planning was presented by Beard (1967). Beard concluded that "the complexity of water resource problems is growing faster than our ability to devise models that will yield satisfactory solutions". Because of this complexity, he was somewhat pessimistic about the use of some of the more sophisticated mathematical techniques, and stated that the most promising approach appears to be in rapidly converging gradient procedures based on successive approximations.

While most water resource systems investigations include some use of gradient methods, attempts to introduce other techniques are increasing because a) there is no way of knowing if the solution reached by gradient methods is the highest point on the overall response surface, or only the top of a local optimum and b) other techniques are generally more efficient computationally. Hall and Dracup (1970) presented a good summary of linear programming and its adaptations for water resource planners. The main postulate of linear programming is that all constraints can be expressed as simple algebraic equations and inequalities. Because of linearity, the optimizing set of decisions must be a set which is also a "corner" of the policy space defined by the constraints. The maximum value of the objective function will be that point for which the "surface" defined by the objective function is farthest from the origin and still

includes at least one feasible policy. A linear programming solution to the reservoir operating problem is available in the book by Maass et al (1962).

More recently, Loucks (1968) developed a stochastic linear programming model for a single reservoir subject to serially correlated net inflows. In addition to defining operating policies, Louck's solution also provides information on the probability distributions of volumes and discharges that may result from the policies.

The main advantage of the linear programming approach is that standard computer programs are available and can be used with a minimum of effort. On the other hand, water resource problems are typically nonlinear, and not directly solvable by linear methods. Nonlinear deterministic problems can conceptually be solved by nonlinear mathematical programming. The solution is obtained by minimizing a separable loss function subject to a set of equality and non-negativity constraints. However, Young (1968) experimented extensively with nonlinear programming as applied to reservoir operation, and concluded that the approach is not practical. He suggested that dynamic programming is generally most suitable for the operational problem.

In the dynamic programming approach for a single reservoir, the decision variables are the releases to be made in each time period, and the state variables are discrete storages at any point in time. In the deterministic case, it is assumed that a perfect foreknowledge of net inflows is available, and a set of outflows is chosen in such a way as to maximize the total return over a particular period of record. Several investigators have presented algorithms for the deterministic solution (Hall and Buras 1961, Hall and Roefs 1966). Reservoir releases from the

deterministic solution can be used as a basis for generalizing operating rules.

One weakness in this approach is that the perfect forecast of inflows will be inherent in the operating rules. The rules will therefore be based to some extent on "hindsight", and may tend to overestimate future benefits. To avoid this problem, stochastic dynamic programming formulations are available (Butcher 1968). Unfortunately, this approach is computationally expensive.

Young (1968) presented a useful and efficient compromise between the deterministic and stochastic alternatives. He suggested developing operating rules using a deterministic solution over long sequences of synthesized inflows. The resulting rules could then be tested over different sequences than those used in their development, thus arriving at a realistic estimate of probable future benefits. The algorithm and general overall approach suggested by Young are being used extensively in the Great Lakes studies. Young's single reservoir solution has been extended to the four lake system with an iterative technique, which is described in detail later in this report.

### History of Regulation Studies

Some of the earlier Great Lakes regulation studies were described in a paper by Patterson and Lawhead (1968). It might be useful to summarize their discussion at this time to enable the reader to understand the evolution of regulation methodology which has taken place.

The first feasibility type of regulation study was carried out on Lake Superior by a team of consulting engineers in 1911. They devised a set of rules for regulating the flows of the St. Marys River. However, the actual control structure which was built differed somewhat from that envisioned by the consultants, so that their plan was superceded upon completion of the works.

In 1920, a report was submitted to the U.S. Congress which discussed regulation as a means of obtaining better navigation depths on the lakes. This report apparently referred to a document dated 1900 which discussed the possible regulation of Lake Erie.

One of the more notable early studies in the field of Great Lakes regulation was prepared by J.R. Freeman in 1926. His work was carried out for the Chicago Sanitary District and therefore concentrated on the effect of the Chicago Diversion. Nevertheless, he did discuss regulation prospects at some length and concluded that it would be beneficial to raise both maximum and minimum water levels. Although it is unlikely that this conclusion would be acceptable under present conditions, some of Mr. Freeman's interpretations of natural phenomenum in the region were remarkably accurate.

The first plan of regulation which was actually put into operation was designed by L.C. Sabin, U.S. member of the Lake Superior Board of Control. Lake Superior was operated according to the Sabin

Rule from 1921 until 1941. In 1941, the control works were modified somewhat and a plan designated Rule P-5 replaced the Sabin Rule. This plan, which was used until 1951 increased minimum flows for the benefit of the power interests to the extent possible without serious detriment to other interests. In 1951, a plan designated the Rule of 1949 was introduced to take into account increased supplies of water resulting from the Ogoki and Long Lac Diversions from the north. The rule of 1949 has been used since 1951, with minor modifications introduced in 1955. This plan will be discussed in more detail later in the section on current regulation methodology.

During planning for the St. Lawrence Seaway in the early 1950's, the Canadian Department of Transport studied regulation possibilities for Lake Ontario, and published preliminary conclusions in 1952. In 1957, the Lake Ontario Board of Engineers submitted a report outlining the development of governing criteria to be used as a guide in further development of regulation plans for Lake Ontario. Regulation of Lake Ontario actually began in 1960 under supervision of the St. Lawrence River Board of Control. The first plan, designated 1958 - A was used from 1960 to 1962 at which time it was replaced by plan 1958-C. The present operational plan 1958-D, which was introduced in 1963 will be discussed in more detail later.

The first economically oriented studies of Great Lakes regulation were published by the U.S. Corps of Engineers in 1965. For the first time, an attempt was made to define feasibility in terms of benefits and costs.

Studies currently being carried out by the International Joint Commission are along similar lines to those Corps studies, but introduce international implications, and more advanced computerized technology.

Present Operational Regulation Plans

At the present state of development of the system, Lake Superior is regulated by the Lake Superior Board of Control, and Lake Ontario by the St. Lawrence River Board of Control. Presented below are brief descriptions of the actual regulation methodology being utilized on these lakes.

- a) Lake Superior - The Rule of 1949 has been in operation on Lake Superior since 1951, and was modified in 1955 in an attempt to obtain improved results. According to the modified rule of 1949, an initial outflow is selected for any given month depending on the mean lake elevation for the previous month. The initial outflow is used to determine the gate setting which provides a flow closest to the desired outflow. Outflow limits are applied in order to protect the various interests. The limits are:
1. An absolute maximum outflow limited by the works. That is, 65,000 cfs through the power and navigation canals plus all 16 gates of the control structure at the head of the rapids open. The flow through the 16 gates varies from 35,000 cfs at a lake elevation of 599.5 to 59,000 cfs at an elevation of 602.0.
  2. A winter maximum outflow of 85,000 cfs. During the winter of 1916-17 the St. Marys River carried a discharge of 108,000 cfs and severe ice jams formed, having an adverse effect on the river levels at Sault Ste. Marie. Later in the same winter, a flow of 86,000 cfs was maintained with no appreciable trouble due to ice jams. This led the International Lake Superior Board of Control to impose a maximum flow limitation of

85,000 cfs for the months December through March.

3. Minimum outflows of 55,000 cfs in the winter months and 58,000 cfs in the summer months. These limitations have been set mainly for the protection of the power interests. A maximum of about 68,000 cfs could be used for power production.
  4. A physical limitation of no gate movement during the winter months. This is not a limitation specified in the regulation plan but is necessitated by the fact that the hand-operated gates are frozen in place for the winter months.
- b) Lake Ontario - The present operational regulation plan for Lake Ontario, 1958-D, is a rule curve approach, with strict outflow limitations. An initial outflow estimate is based on water levels in the antecedent time period, and a supply indicator (forecast of future supplies). Outflow limits are applied to the initial outflow estimate in order to protect the downstream interests. Regulation is in accordance with the IJC Orders of Approval of 1952 and 1956 which are generally followed in setting outflow limits.
1. The outflow must be large enough so as not to increase the frequency of low levels in Montreal Harbour during the navigation season.
  2. The maximum outflows during the winter period are severely limited because of the difficulties of winter operation. Present maximums are 220,000 cfs in January, 240,000 cfs in the first quarter of February, 260,000 cfs in the second and third quarters of February and 280,000 in the fourth quarter of February and March.

3. The regulated outflow from Lake Ontario during the annual spring break-up period must not be so large as to worsen flooding conditions in Montreal Harbour and downstream.
4. The Lake Ontario winter minimum outflow limits are such as to provide maximum dependable power. Present minimum allowable flows are 210,000, 210,000, 207,000 and 204,000 for December-March respectively.
5. The levels of Lake Ontario are regulated for the benefit of the property owners on the lake shore by reducing the extremes of stage. Specific level limits have been given in the IJC Orders of Approval.
6. Channel excavations in the International Rapids Section were designed to provide stipulated limiting depths and velocities for navigation. The maximum outflow during the open water season is therefore dependent on the level in this section, but has an absolute maximum of 310,000 cfs.

#### Purpose and Scope of Studies

A description of the scope of the studies was made public in an interim report by the International Joint Commission in July of 1968. The Terms of Reference for the study as quoted in the report request that the Commission should "study the various factors which affect the fluctuations of these water levels and determine whether in its judgment action would be practicable and in the public interest from the points of view of both governments for the purposes of bringing about a more beneficial range of stage for, and improvement in: a) domestic water supply and sanitation, b) navigation, c) water for power and industry,

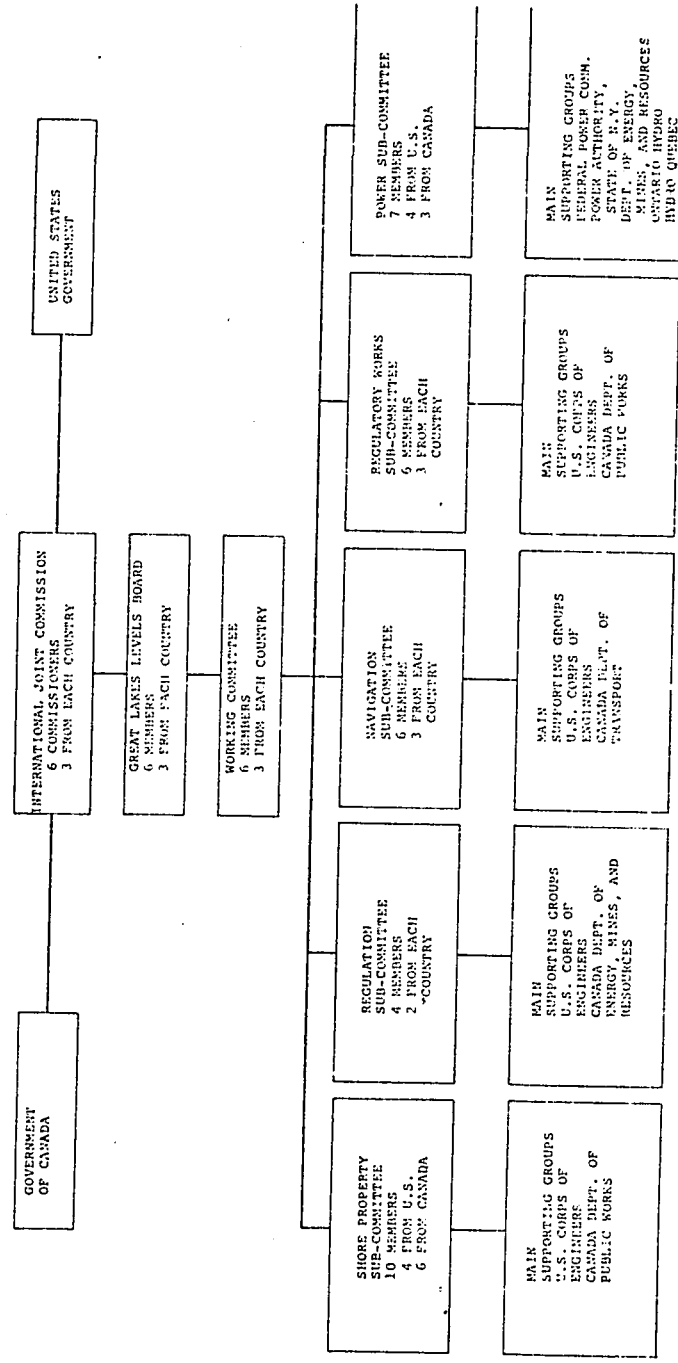


FIGURE 1 - ORGANIZATION OF STUDY

d) flood control, e) agriculture, f) fish and wildlife, g) recreation and h) other beneficial public purposes." As is obvious from the Terms of Reference quoted above, there is a diversity of interests affected by variations in the levels and flows of the Great Lakes. In order to cope with the complexity of the problem, a systems approach is being used. There were several reasons why a systems approach was not taken earlier. One of these was related to the lack of adequate computer facilities to handle such a complex problem. Another important reason was that the economic significance of the relationships between different interests was not defined.

The organization of the current studies was presented in a paper by Pentland, Rosenberg and Cavadias (1968) and is reproduced as Figure 1 in this report. Shore property interests are being studied by the Shore Property Subcommittee. The main supporting agencies in this group are the U.S. Corps of Engineers and the Canadian Department of Transport. The power entities (Power Authority of the State of New York, Ontario Hydro, and Quebec Hydro) along with the U.S. Federal Power Commission and the Canadian Department of Energy, Mines and Resources are involved in power evaluations. The Regulatory Works Subcommittee comprised mainly of members from the U.S. Corps of Engineers, and the Canadian Departments of Public Works and Energy, Mines and Resources supplies cost data on proposed regulatory works, or extensions to existing works. Input from these four Subcommittees is used by the Regulation Subcommittee (Canadian Department of Energy, Mines and Resources and U.S. Lake Survey) to carry out detailed regulation studies. It is expected that the current studies will be completed in 1973, and therefore all methodology is still in a state of development.

SYSTEM DESCRIPTION

Great Lakes System

Description

The Great Lakes form one of the largest systems of fresh water in the world, having a total water area of about 95,000 square miles. The drainage basin constitutes the major part of the St. Lawrence River system and has an area of about 295,000 square miles above the outlet of Lake Ontario. Figure 2 below summarizes the land and water area of the individual lake basins.

Figure 2 - Land and Water Areas (Square Miles)

WATER SURFACE AREA	LAKE SUPERIOR	LAKE MICHIGAN-HURON	LAKE ST. CLAIR	LAKE ERIE	LAKE ONTARIO
a) U.S.	20600	31400	162	4980	3460
b) Canada	11100	13900	268	4930	3880
c) Total	31700	45300	430	9910	7340
LAND AREA					
a) U.S.	16900	61600	1020	18000	12600
b) Canada	32400	34700	3780	4720	10900
c) Total	49300	96300	4800	22700	23500
TOTAL AREA					
a) U.S.	37500	93000	1180	23000	16100
b) Canada	43500	48600	4050	9650	14800
c) Total	8100	142000	5230	32600	30900

The volume of Lake Superior is 2858 cubic miles, nearly 60% of the total of the system. The lake has an average depth of 474 ft., and a maximum depth of about 1330 ft. Maximum dimensions are about 350 miles in length, and 160 miles in width. The mean water level is 600.4 feet above mean sea level, and the range of stage over a hundred-year period has been in the order of four feet. The outflow of Lake Superior reaches Lake Huron via the St. Marys River.

Hydraulically, Lakes Huron and Michigan are considered as a single lake since there is no measurable fall in the Straits of Mackinac which connect them. The volumes of the two lakes are 824 and 1180 cubic miles respectively. Lake Huron has maximum and average depths of 750 and 189 feet while similar depths for Lake Michigan are 924 and 279 feet; Maximum lengths and widths of the two lakes are 206 x 183 and 307 x 118 miles respectively. The long-term mean elevation of Lake Michigan-Huron has been about 578.5 feet above mean sea level, with a range of about 6 feet over the period of record (1900-1968). The Lake Huron outflow is through the St. Clair River to Lake St. Clair (mean elevation 573.6) after which it passes through the Detroit River to Lake Erie.

In terms of volume, Lake Erie is by far the smallest of the Great Lakes (116 cubic miles) having an average depth of only 62 feet. Maximum dimensions of the lake are 241 x 57 miles, the mean level is about 570.6 feet above sea level, and the maximum variation in levels has been in the order of 6 feet. The majority of the outflow of Lake Erie reaches Lake Ontario via the Niagara River, with the remainder passing through the Welland Ship Canal, and the New York State Barge Canal System.

Lake Ontario, although the smallest of the lakes in area has about three times the volume of Lake Erie (393 cubic miles). The average depth of Lake Ontario is 283 feet, and its maximum dimensions are 193 and 53 miles. The maximum recorded monthly variation in the stages has been about 6 feet, about a long-term mean elevation of 244.6 feet above sea level.

#### Geology

Geologists are in general agreement that the Great Lakes Region at some time in the past consisted of river basins and valleys draining

to the south and east. The present system is a heritage of the ice age which reformed the face of the northern part of the continent. Huge glaciers deepened the basins and left great piles of debris on the southern rim, as well as depressing the earth's crust in the region by their enormous weight. As the ice masses retreated, the old drainage channels were blocked by glacial debris, and melting ice filled up the deepened basins before finding new outlets. At that time, the lakes were much larger than they are at present, and there is evidence that ancient rivers flowed south from Lake Superior to the Mississippi system, and east to the Hudson River. The system gradually evolved to its present state with the outflow travelling in a generally north-easterly direction by way of the relatively youthful St. Lawrence River.

The Precambrian shield dominates the area north of Lake Superior, Huron and Ontario. Within this area, glacial deposits are slight and the bedrock surface has vertical relief as high as 300 ft. The sparse glacial deposits have, however, blocked much of the drainage resulting in the creation of numerous lakes, swamps and muskegs. The area as a whole is rough, strewn with erratic boulders and generally forested.

Further south, glacial drift deposits dominate the landscape. Undulating till plains produce vertical relief of 10 to 50 feet and belts of end moraine result in local relief of 30 to 100 feet or more. Extensive outwash plains exist, underlain by sand and gravel. Wet lands are common in the till plains where post-glacial erosion has generally been insufficient to destroy the glacial microrelief and develop integrated drainage.

#### Land Use and Natural Resources

The Canadian portion of the Great Lakes basin lies entirely within the Province of Ontario. The United States basin includes all of the

State of Michigan, and parts of Minnesota, Wisconsin, Illinois, Ohio, Pennsylvania and New York. Large urban areas have grown along the shorelines including Toronto, Hamilton, Windsor, Chicago, Milwaukee, Detroit, Toledo, Cleveland, Buffalo and Rochester. Population within the basin is presently greater than 30 million.

In the United States portion of the basin, about 48% of the land is forested. Of the remainder, 33% is cropland, 6% is pasture and 13% is non-agricultural (including urban, commercial, industrial, transportation and small water areas). In the Canadian basin, 68% of the area is forested and the remaining 32% is cleared, urbanized, or small water areas. Open water and marshes make up about a third of the non-forested area, leaving much less productive agricultural land in the Canadian portion of the basin than in the United States portion.

As a general rule, the better agricultural areas are in the more southerly parts of the basin. The Indiana and Ohio portions of the basin and the lower part of the Michigan peninsula are in the fertile corn belt. Intensive dairy farming is prominent in most parts of the basin. The lands in Ontario bordering on Lake Erie have become notable tobacco-producing areas, and important fruit-growing areas exist because of the moderating effects which the lakes have on the climate near the lakeshore.

The proximity of an abundance of natural minerals to the relatively inexpensive lake transportation is an important factor of the North American economy. There is an abundance of iron ore and limestone near the shores of the upper lakes, and high-quality coal within 200 miles of the southerly lake ports. In addition to iron ore, the upper Great Lakes region produces large quantities of nickel as well as some copper, cobalt, silver, gold, platinum, zinc and uranium. In the lower Great Lakes region,

the more prominent resources are the non-metallics such as limestone, salt, sand and gypsum, and fuels such as petroleum and natural gas.

#### Regulatory Works

In the early nineteenth century, the Great Lakes basin supported fewer than 300,000 persons. The area now supports a population more than 100 times that number, a population which has had a significant influence on the environment. The most publicized of these influences has been in the water quality area. This subject will not be discussed in this report, because it is essentially unrelated to the level-flow problem being considered. Man has also had an influence on the supply of water reaching the lakes by tributary regulation, deforestation, urbanization, and increasing consumptive use of water. These influences still have a relatively small effect on the levels and flows of the lakes, and will not be discussed further at this time. Some of these factors along with the effect of diversions will be considered later in the section on associated hydrologic studies.

Man's most important influence on the Great Lakes in terms of levels and flows has been construction of works in the connecting channels. A brief history of this work for each of the connecting channels is presented below:

- a) St. Marys River - In 1914 the International Joint Commission issued orders of approval for the use of the St. Marys River for power development on both sides of the River. This included construction of a gated structure (compensating works) above the rapids, as well as power diversion facilities. Navigation locks have also been built, and the lower river enlarged for navigation purposes. Since 1921, the outflows of Lake Superior have been fully regulated.
- b) St. Clair and Detroit Rivers - The outflow of Lake Michigan-Huron through the St. Clair and Detroit Rivers is unregulated at present. The natural outflows of Lake Michigan-Huron depend not only on the elevation of Lake

Huron, but also to a minor extent on the levels of Lake Erie since there is no natural control section in the St. Clair or Detroit Rivers. Therefore, in the consideration of any works or channel improvements in this reach, the effect of Lake Erie levels may be an important factor.

Considerable dredging in the St. Clair River has already been carried out for navigation purposes. Dredging has been particularly extensive at the head of the St. Clair River where it leaves Lake Huron. Much of it has been through earth, clay and boulders; but in the lower Detroit River long channel sections have been cut through rock. From 1933-37, the dredging provided the 25-foot navigation channel, and in 1962 the 27-foot channel was completed. Between 1904 and 1928 large amounts of sand and gravel were dredged from the St. Clair River near Point Edward for use by private contractors. Much of the material dredged from the Detroit River has been replaced in a different part of the river to partially offset the lowering of water levels caused by its removal.

c) Niagara River - Lake Erie discharges its water through the Niagara River, which has a 325 foot drop to Lake Ontario. Niagara Falls provides a huge source of hydro-electric power. Canadian plants have a total capacity of 3,190,000 horsepower, while the U.S. plants have a capacity of 2,930,000 horsepower. A submerged weir above the Horseshoe Falls was replaced by a new control structure completed in 1963. This structure with 18 sluiceway gates extends from the Canadian shore to Tower Island, just across the International boundary. Excavation in the Horseshoe Cascades has provided a more uniform distribution of flow over the Falls. To avoid ice problems, the power companies have placed an ice boom across the head of the Niagara River each winter since 1964.

In addition to power diversions, about 7000 cfs of the Niagara River flow is diverted through the Welland Canal to Lake Ontario through the two

De Cew Falls Plants which began operating in 1901 and 1943. The total flow of the Niagara River is uncontrolled at present.

d) St. Lawrence River - The outflow from Lake Ontario has been fully controlled since 1958 by the works constructed as part of the St. Lawrence Seaway and Power Project. These works include a dam in the vicinity of Iroquois Point, a dam in the Long Sault Rapids between Barnhart Island and the United States shore and two powerhouses, one on either side of the international boundary at the foot of Barnhart Island. The installed capacity of the two powerhouses is about 2,400,000 horsepower. Navigation facilities include a lock and canal on the Canadian shore to by-pass the Iroquois control dam, and a canal and two locks on the United States shore to by-pass the Long Sault Dam and Barnhart Island Powerhouses.

The Lake St. Francis section extends from the head of Lake St. Francis near Cornwall to Coteau Rapids, a distance of about 30 miles. The total fall in this reach is about 83 feet. This section includes the Beauharnois canal, a combined power and navigation channel.

#### Present Water Uses

Although consumptive use of water in the Great Lakes Basin is small, the cumulative effect is becoming significant. Consumptive use is defined for the purposes of this study as that portion of the water withdrawn or withheld from the Great Lakes for internal purposes and not returning or transmitted to the lakes. Therefore, the water lost to the lakes in consumptive uses is no longer available because it has been incorporated into products and crops, used in industrial processes, consumed by man or livestock, or has been otherwise expended. The effect of consumptive use on lake levels and outflows is a general

lowering of levels, and an eventual decrease in outflow equal to the amount of water consumed.

Consumptive use is considered under four main categories and the methodology is discussed in very general terms below:

1. Thermal electric power generation - Consumptive use by power is that quantity of water which is lost to the system as a result of evaporation of a portion of the cooling water. A modern thermal-electric plant may have an overall efficiency of 40%, and the remainder of the energy is dissipated as heat. From a consideration of energy production, efficiency, etc. an estimate can be made of how much cooling water is lost by evaporation.

2. Irrigation - Consumptive use by irrigation can be estimated from a consideration of climatic factors, and acreage under irrigation. The maximum amount of water which can be used usefully can be calculated by subtracting actual regional evaporation from potential evaporation as estimated by Penman's or some other method. This will set an upper limit on this use, and a consideration of licenses, withdrawals, etc. can be used to estimate the proportion of this maximum which is actually used at the present time.

3. Industry - Water use based on industrial production depends entirely on the level of industrial activity in the area. From a consideration of actual production of steel, copper, magnesium, paper, petroleum, textiles, rubber and chemicals, and theoretical consideration of unit rate of consumption for each of these products, a reasonably accurate estimate of industrial consumptive use can be made.

4. Municipal and Rural - This estimate is perhaps the least accurate since the ratio of consumptive use as a proportion of withdrawals differs markedly among cities with similar populations. Many factors must be considered such as climate, economy, urbanization, water distribution facilities, etc.

## Hydrology of the System

### Available Data

In general, the data used in the regulation studies are provided by an international committee known as the Coordinating Committee on Great Lakes Basic Hydraulic and Hydrologic Data. This Committee is responsible for developing and coordinating data on water levels, river flows, and physical characteristics of the Great Lakes System. The recorded data required in the actual regulation studies are beginning-of-period lake levels, outflows, and diversions into or out of the Basin. The period of record being used is 1900-1967. Additional data required in hydrology studies of the basin (i.e. stream-flow and climatological records) are available from the data collecting agencies in the region.

### Runoff

This section is based on a paper by Pentland (1968). Some of the conclusions reached in the paper may be useful in understanding the general hydrology of the region. In the investigation, runoff maps were constructed for the basin for each month, using more than 250 hydrometric stations. The average physical and climatic features of several of the major tributaries were related to long-term average runoff to explain the reasons for the areal distribution of runoff. The paper is summarized below:

a) Preparation of runoff maps -

Runoff maps were based on the thirty-year period 1935-64. Base stations having all thirty years of record were given most weight in drawing lines of equal runoff. Subsidiary stations (15 years or more) were given somewhat less weight, and reference stations (5 - 15 years) were considered only in areas where no base or subsidiary stations were

available. Average unit runoff was plotted at the centroid of each drainage basin. The annual map was prepared first with reference to precipitation records, and monthly maps prepared with reference to the annual map. The annual map is presented in this report as figure 3.

b) Area-runoff distribution graphs -

Since only unregulated streams were used for the runoff maps described above, the maps represent essentially natural runoff. An analysis of these maps was carried out to produce graphs showing the relationship between unit runoff, and percentage of area with more than each level of runoff. It is expected that these graphs may be useful in studying future effects of changing land use, urbanization, and increasing consumptive use. The graphs have already been used to study the effect of tributary regulation on the seasonal distribution of supplies.

c) Total basin runoff -

Precipitation generally increases from west (30 inches in the Lake Superior Basin) to east (35 inches in the Lake Ontario Basin) in the Great Lakes Basin. The areal distribution of annual mean runoff is affected by this factor, as well as several others such as the seasonal distribution of the precipitation, and the north-south temperature gradient. For example, Lake Erie runoff is substantially less than on the Lake Superior basin, even though it receives 15% more precipitation. This is because snow accumulation is a highly efficient source of runoff, and the Lake Erie Basin receives only about 60% as much snowfall as the Superior basin. Evaporation losses are also higher from the Lake Erie Basin.

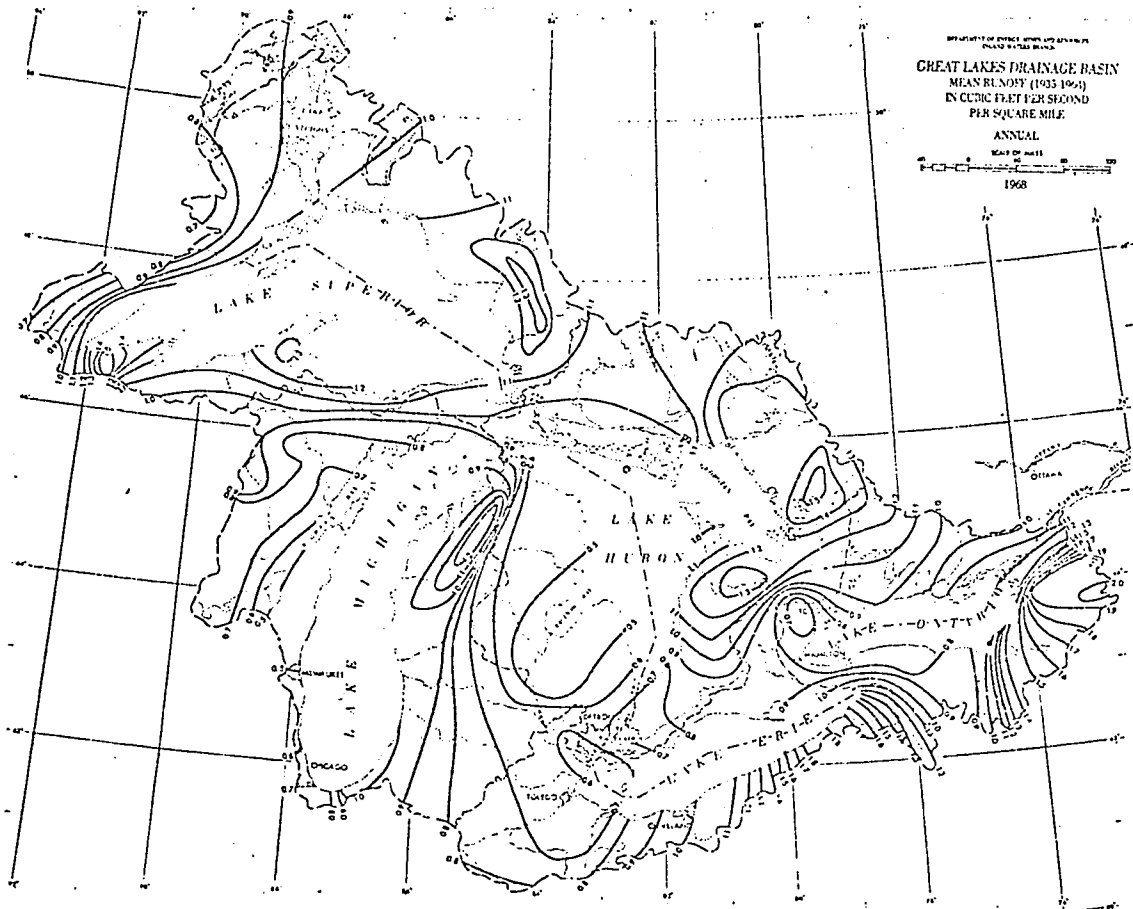


Figure 3

d) Seasonal runoff distribution -

Figure 4 below summarizes the mean monthly runoff from each of the lake basins:

Figure 4

Mean Basin Land Runoff (1935-64)

(cfs/square mile)

	<u>Ontario</u>	<u>Erie</u>	<u>Huron</u>	<u>Michigan</u>	<u>Superior</u>
January	1.14	1.23	0.62	0.75	0.43
February	1.15	1.37	0.67	0.72	0.36
March	2.58	2.19	1.43	1.16	0.54
April	3.07	1.89	2.64	1.72	1.95
May	1.48	0.97	1.70	1.17	2.75
June	0.69	0.58	0.94	0.80	1.66
July	0.45	0.29	0.65	0.57	0.99
August	0.35	0.20	0.44	0.49	0.60
September	0.36	0.16	0.46	0.54	0.67
October	0.54	0.26	0.61	0.60	0.77
November	0.84	0.46	0.86	0.69	0.85
December	1.03	0.75	0.83	0.60	0.64

Basin storage is the single-most important factor affecting seasonal runoff distribution. Naturally, the most important source of storage is in the snowpack, with less important factors being soil moisture and natural reservoirs. An example of the importance of the snowpack could again be observed by comparing the Lakes Superior and Erie basins. Average winter temperatures are about 15°F higher in the Lake Erie Basin with a result that the Lake Erie basin has virtually no snowpack by the end of March, while the Lake Superior basin still has an average snowpack of 16 inches. This contributes to a much lower peak

runoff on the Lake Erie basin, and a lag of about 2 months between the peaks on the two basins. Evapotranspiration rates are high in the May-August period throughout the basin which contributes to a rapid runoff recession. A sharp drop in evapotranspiration rates in October and November cause increasing runoff even though precipitation is generally decreasing.

e) Areal runoff distribution -

In order to study the reasons for areal variability of runoff, climatic and physical characteristics of 50 major tributaries were studied and related to runoff. These factors were precipitation, evaporation, temperature, snow as a percentage of total precipitation, position with respect to the lake, mean elevation, land slope, channel slope, free water surface, and drainage density. As a result of these studies, it was concluded that there are two main reasons for areal runoff variability in the basin:

1. Lake effect-moisture is picked up by the prevailing winds moving in a generally easterly direction over the lakes. The saturated air masses lose part of their moisture in passing over the land to the lee of the lakes.

2. Latitude - The north-south temperature and radiation gradients greatly influence evapotranspiration losses, and the percentage of precipitation in the form of snow is important because additional energy is required for losses from a frozen surface.

Net Basin Supply

Net basin supply can best be described with the classical water balance equation:

$$\Delta S = P + R + U - E + I - O \pm D$$

where:

$\Delta S$  = change in storage in the lake

P = precipitation on the lake surface

R = runoff from the drainage basin

U = groundwater contribution

E = evaporation from the lake surface

I = inflow from an upstream source

O = outflow from the lake through its natural outlet

D = artificial diversions into or out of the lake

With data and techniques presently available, exact values of precipitation, land runoff, groundwater inflow, and lake evaporation cannot be determined accurately. However, these terms can be combined into a single term called net basin supply (NBS) and estimated from measured values of storage, inflow, outflow and diversions with the relationship

$$\Delta S = NBS + I - O \pm D$$

Net basin supply represents the water which the lake receives from precipitation on both its surface and its own drainage basin, minus evaporation from the lake surface. It has the same value irrespective of diversions into or out of the lake, being modified only by the factors that pertain to and affect the natural local supplies of the individual lake basins.

#### Winter and Weed Retardation

Since Lakes Superior and Ontario are regulated, and the regulated condition is used for a basis of comparison, no estimation of ice or weed retardation of the outflows of these two lakes is required for routing purposes. However, retardation estimates are required for

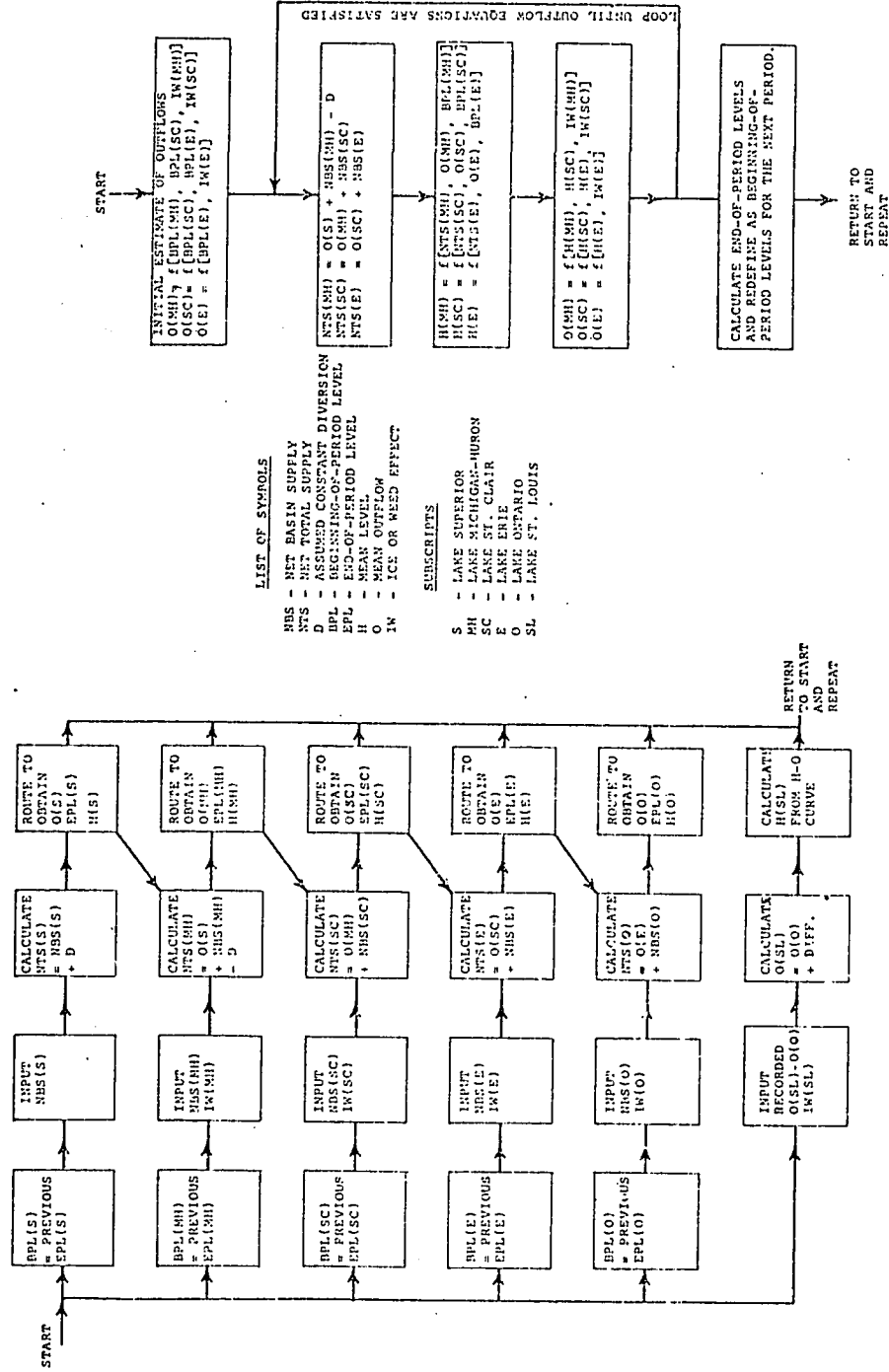
routing supplies through Lakes Michigan-Huron, St. Clair and Erie.

Ice retardation in the St. Clair River can be estimated by subtracting recorded flows from the corresponding discharge based on an open water stage-discharge relationship for the Harbour Beach-Grosse Pte. reach. Separate relationships must be used for each period of channel stability (i.e. between periods of dredging). Similarly, winter retardation in the Detroit River can be calculated as the difference between the flow calculated from the Grosse Pte. - Cleveland relationship and the recorded flow.

Small amounts of ice and weed retardation also exist in the Niagara River at the outlet of Lake Erie. Since these values are reasonably small, long-term average values can be assumed for each month.

#### Flow Level Model

Obviously, all recorded data incorporate the effects of changes in the regimen of the lakes and their connecting channels which have occurred over the study period (1900-1967). The principal changes have been man-made, and consisted of changes in the amount of diversion into and out of the Basin, alterations in the configuration of the connecting channels, and the erection of control structures at the outlets of Lakes Superior and Ontario. In order to permit a consistent evaluation of the effects of various regulation plans, it is necessary to develop a set of lake levels and outflows which reflect a constant or fixed regimen in the System over the study period. The constant regimen selected is essentially the present physical conditions and regulation methods, with a few minor exceptions. The flow-level model used to route the supplies through the system with the fixed regimen was described by Pentland, Rosenberg, and Cavadias (1968), and is demonstrated in the flow chart on figure 5.



LIST OF SYMBOLS

- NBS - NET BASIN SUPPLY
  - NTS - NET TOTAL SUPPLY
  - D - ASSUMED DIVERSION
  - RPL - BEGINNING-OF-PERIOD LEVEL
  - EPL - END-OF-PERIOD LEVEL
  - H - MEAN LEVEL
  - O - MEAN OUTFLOW
  - IN - ICE OR NEED EFFECT
- SUBSCRIPTS
- S - LAKE SUPERIOR
  - PH - LAKE MICHIGAN-HURON
  - E - LAKE ST. CLAIR
  - O - LAKE ONTARIO
  - SL - LAKE ST. LOUIS

Figure 5(a) - Flow Level Model

Figure 5(b) - Detailed Routing through Lakes Michigan-Huron, St. Clair and Erie

For Lake Superior, the basis of comparison levels and outflows are obtained by routing through the lake the net basin supplies (adjusted to a constant diversion into the lake by way of the Long Lake and Ogoki diversions) in accordance with the present regulation plan, 1949-Rule, which was described earlier.

Because of the nature of the control in the St. Clair and Detroit Rivers, the problem of routing supplies through Lakes Michigan-Huron, St. Clair and Erie is fairly complex. The St. Clair River and Detroit River discharges are dependent not only on Lake Huron levels, but also on the water levels in the lower lake. Therefore, a method of successive approximations as shown on the flow chart is used to route through Lakes Michigan-Huron, St. Clair, and Erie. The procedure is as follows:

1. For any time period, the net total supply to Lake Michigan-Huron is calculated as the routed Lake Superior outflow, plus the Lake Michigan-Huron net basin supply, minus a constant diversion at Chicago.
2. For a first approximation, the beginning-of-period stages (end-of-period stages resulting from the routing in the previous time period) are considered equal to the average levels for the period.
3. A first estimate of the Lake Huron outflow is made with the two-gauge relationship between Harbour Beach and Grosse Pte.
4. A first estimate of the Lake St. Clair total supply is made by adding the Lake Huron calculated outflow to the Lake St. Clair net basin supply.
5. A first estimate of the Lake St. Clair outflow is made from the Grosse Pte.-Cleveland two-gauge relationship.
6. An initial estimate of the Lake Erie total supply is made by adding the Lake St. Clair calculated outflow to the Lake Erie net

basin supply.

7. A first estimate of the Lake Erie outflow is made from the Buffalo single gauge relationship.

8. Having a first estimate of the total supply and outflow from each lake, a first estimate can be made of the change of storage, the end-of-period level, and the mean level for the period.

9. Steps 3-8 are repeated using first estimate mean levels in place of beginning-of-period levels.

10. The process is repeated as often as is required to converge on the solution.

Having the Lake Erie outflows and Lake Ontario net basin supplies, net total supplies for Lake Ontario can be calculated. The basis of comparison for Lake Ontario is then obtained by routing these net total supplies with the current regulation plan 1958-D, which was described earlier.

#### Supply Synthesis

The purpose of synthesizing hydrologic information for water resource projects is to overcome one of the main disadvantages of traditional engineering investigations. That is, if a project is tested only over a single sequence of supplies obtained from recorded data, only a single estimate of its performance is obtained, while no information is gained about its performance over other equally likely patterns of supplies (Chow 1964). For the planner of water resource projects, the generated information provides a tool to determine whether economic benefits are real. The variation in the results thus obtained provides an aid in identifying weaknesses and developing an optimum design.

The basic approach used to synthesize supplies in this study is similar to that developed by the Harvard water resource group, and was described by Megerian and Pentland (1968). The following steps are carried out in the model:

1. Recorded net basin supplies to all four lakes are standardized on a monthly basis:

$$x = \frac{X - \bar{X}}{S}$$

where

x = standardized supply

X = recorded supply

$\bar{X}$  = long-term mean supply

S = standard deviation of supplies

2. The standardized supplies are subjected to a principal component analysis, one analysis of each of the 12 arrays of monthly data. The principal components have the property of linear independence. This means each of the principal components can be extended using serial correlation alone, and no account need be taken of cross-correlations because, by definition, each principal component is independent of all others.

3. All principal components are generated using a standard first order Markov process

$$Z_{t+1} = \bar{Z}_{t+1} + R \frac{S_{t+1}}{S_t} (Z_t - \bar{Z}_t) + t S_{t+1} (1-R^2)^{\frac{1}{2}}$$

where:

$Z_{t+1}$  = generated data for a given month

$\bar{Z}_{t+1}$  = long-term mean

R = serial correlation coefficient

$S_{t+1}$  = standard deviation in month simulated

$S_t$  = standard deviation in antecedent month

$Z_t$  = data in antecedent month

$\bar{Z}_t$  = long-term mean in antecedent month

t = a number selected at random from a normal

distribution with a mean of zero and a variance of one.

4. The simulated principal components are converted to standardized supplies with the inverse linear transform. The inverse transform preserves all the moments of the original standardized data.

5. The standardized supplies are converted to simulated supplies using the long-term mean and standard deviations with the relationship:

$$X = xS + \bar{X}$$

In order to ensure that the synthesized supplies bear a satisfactory resemblance to the recorded supplies, a detailed set of tests has been devised. These tests include:

1. comparison of basic parameters such as monthly means and standard deviations
2. frequency or duration analysis
3. autocorrelation
4. spectral analysis
5. cross-correlation
6. tests for long-term persistence.

#### Supply Forecasting

The ability to forecast is very important to successful regulation. If perfect long-term forecasts were available, reservoirs could be filled in anticipation of droughts, and emptied in anticipation

of floods. There are many approaches which could be taken to forecasting supplies. Some investigators pursue strictly statistical forecasts based on regression theory, or methods based on time series analysis. Of the studies which have been done to date, perhaps the most promising are those which delve into the physical processes themselves. Such an investigation was described by Witherspoon (1967) and will be summarized briefly at this time.

In Witherspoon's hydrologic model, he attempts to keep track of the moisture conditions on the Basin at all times. An estimate of actual regional evaporation is made with an equation developed by Morton (1965).

$$E_R = (1-a) G - E_p$$

where  $E_R$  = actual regional evaporation

$a$  = albedo

$G$  = total insolation

$E_p$  = potential evaporation.

A regional water balance is then used to calculate the change in storage, and available moisture on the basin.

$$P - E_R - R = \Delta S$$

where  $P$  = precipitation

$E_R$  = actual regional evaporation

$R$  = runoff

$\Delta S$  = change in regional moisture storage.

Using the available moisture as the main independent variable, regression equations are developed to make the supply forecast.

THEORETICAL DEVELOPMENTS

Maximization of Economic Benefits

Approach

A useful preliminary stage in developing reservoir operating rules is to define the maximum possible benefits which could be derived from a system, assuming a perfect foreknowledge of supplies. This serves two functions - a) to define an upper limit on benefits which can be compared to order of magnitude costs to get a preliminary estimate of economic feasibility and b) to provide a basis for further development of operational regulation plans. In the Great Lakes studies, a dynamic programming algorithm similar to that described by G.K. Young (1967) was selected for this stage of the work. The single reservoir algorithm suggested by Young was extended to the four lake system with an iterative approach, which was described by Pentland and Eryuzlu (1969). This Section will provide a description of this optimization methodology.

Dynamic Programming

Most dynamic programming problems are of the allocation type, and the objective function usually appears in the form

$$V = \max_i \sum_{i=1}^n V_i (X_i)$$

Subject to the constraint:

$$X_1 + X_2 + \dots + X_n \leq G$$

This means that we wish to allocate G units of a resource to  $X_1$ ,  $X_2$ ,  $X_3$ , ...,  $X_n$  in such a way as to maximize the function  $V_i(X_i)$ . The solution



could be found by examining every possible combination of  $X_1, X_2, \dots, X_n$  but this would be impracticable in most real problems.

The solution begins by presuming that only one user remains to receive an allocation and that any amount of the resource between 0 and G might be available. Define  $f_1(q)$  as the best possible use of the available resource  $q$  by user 1.

$$\begin{aligned} f_1(q) &= \max V_1(X_1) \\ 0 &\leq X_1 \leq q \\ 0 &\leq q \leq G \end{aligned}$$

This is done for all values of  $q$ . Now, consider stage 2 of the process. The total return is simply the sum of returns at each stage. Once the return from stage 2 is fixed, the total return could not possibly be a maximum unless the return from stage 1 is as large as possible from the resource available at the last stage. This is basically Bellman's principle of optimality which states "An optimum policy has the property that whatever the initial stage and initial decision are, the remaining decisions must constitute an optimal policy with regard to the stage resulting from the first decision". Therefore, for a two-stage decision process:

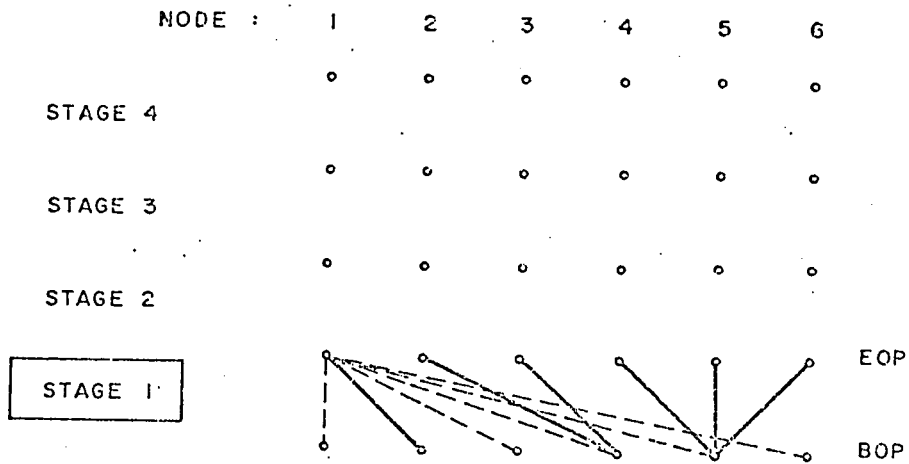
$$\begin{aligned} f_2(q) &= \max V_2 X_2 + f_1(q-X_2) \\ 0 &\leq X_2 \leq q \\ 0 &\leq q \leq G \end{aligned}$$

The general recursive equation for the  $k^{\text{th}}$  stage is therefore:

$$\begin{aligned} f_k(q) &= \max V_k(X_k) + f_{k-1}(q-X_k) \\ 0 &\leq X_k \leq q \\ 0 &\leq q \leq G \end{aligned}$$

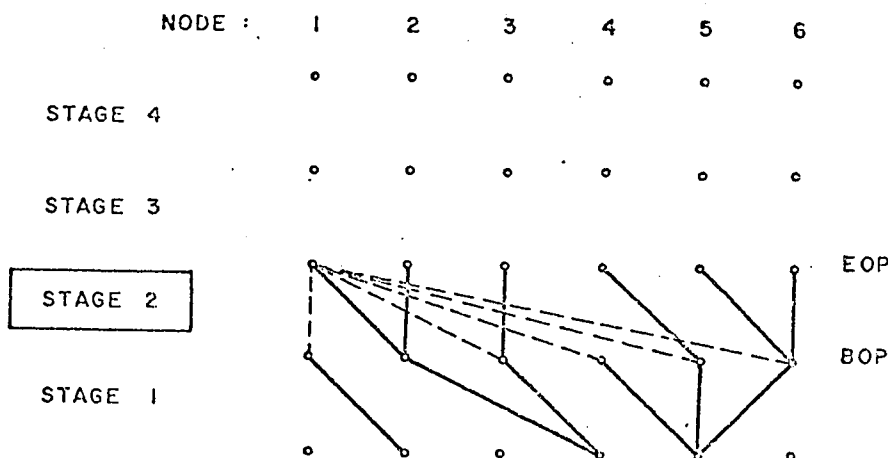
Dynamic Programming Applied to Reservoir Operation

The approach will be explained graphically first, followed by a mathematical description. With a forward-looking algorithm, the first stage (first month) is considered first. At this stage, each possible combination of beginning of period storages (BOP) and end of period storages (EOP) are examined with the objective of determining the "best" route to each EOP. The criterion for the selection is the relative loss associated with the monthly mean storage and mean outflow. The total number of computations required to achieve these results can be demonstrated with the following sketch:

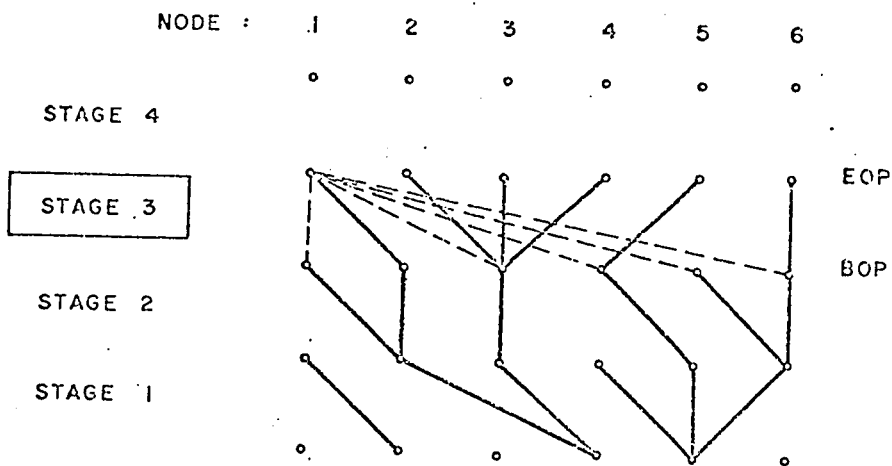


In this example, to find the best BOP for the EOP storage represented by node 1, a comparison between six monthly relative loss values is required. Let us assume that for node 1 the best BOP storage is that of node 2. Therefore, in the diagram this is indicated by the heavy line drawn between the two nodes (1 and 2). The dotted lines indicate the remaining five possible routes to node 1. For simplicity, all six possible routes to node 2 are not drawn. Only one heavy line at the EOP level to node 2 is shown, assuming that as a result of the comparison between the six possible routes to node 2 we found the route from node 4 (BOP) yields the minimum loss. Similarly, let us assume that the best routes for the remaining nodes at the EOP level are as indicated on the diagram. At stage one six best routes must be determined, one to each node, after making six calculations and comparisons at each node. Therefore, at this stage, for  $n$  nodes, a total of  $n^2$  operations are required.

The procedure followed in the second stage together with the results of the first stage is demonstrated in the following diagram:



The six possible routes to node 1 at the EOP level are indicated in the diagram. Of these the heavy line is assumed to be the best route to node 1. Similarly, the best route to the other nodes are determined. In determining each best route at this stage we have  $l$  calculations of the monthly relative loss,  $l$  additions (the loss at stage 2 plus the loss associated with the best route to each EOP node in stage 1) and  $l$  comparisons. Therefore, we have to calculate  $l^2$  outflows and mean storages, determine  $l^2$  monthly relative losses associated with these flows and storages, and perform  $l^2$  additions and comparisons. Now, we may proceed to the third stage as in the following sketch:



The number of calculations and comparisons required at this stage, and at all subsequent stages, is identical to the second stage.

At the last stage, we may choose to end at some predetermined storage level. This can be achieved by doing the traceback from that point instead of from the node giving the minimum loss. This would then act as a constraint on the final stage. Another constraint on the solution might be imposed in the first time period. For a meaningful comparison with historical data, it may be desirable to start at a storage level corresponding to that in the historical data. To achieve this, in the first month, only the BOP storage corresponding to the desired level is considered to be a feasible solution.

G.K. Young (1967) expressed this type of solution in mathematical terms:

Let

$X_i$  = inflow in the  $i^{\text{th}}$  time period

$S_i$  = storage at the start of the  $i^{\text{th}}$  time period

$D_i$  = outflow in the  $i^{\text{th}}$  period

$S_m$  = maximum storage in the reservoir.

A simple water balance equation for any time period is:

$$D_i = S_i - S_{i+1} + X_i$$

The problem is to find values of  $S_i$  ( $i = 2, 3 \dots n+1$ ) which are related to  $X_i$  and  $D_i$  by the water balance equation such that:

$$L = \sum_{i=1}^n \ell(D_i) \text{ is minimized.}$$

This equation expresses the objective function, and the water balance equation forms the constraint on the solution. Therefore, we wish to obtain:

$$Z = \text{Min}_{S_1, S_2, \dots, S_{n+1}} \sum_{i=1}^n \ell(S_i - S_{i+1} + X_i)$$

The values of  $X_i$  ( $i = 1, 2, \dots, n$ ) are given (assumed to be known) and  $S_i - S_{i+1} + X_i$  representing outflow must be positive at all times.

Applying the principle of optimality, we can write:

$$\lambda_k(\phi_k) = \min_{S_k} \ell(S_k - \phi_k + X_k) + \min_{S_2, S_3, \dots, S_{k-1}} \sum_{i=1}^{k-1} \ell(S_i - S_{i+1} + X_i)$$

where  $\phi_k$  is a dummy variable belonging to the same set as  $S_i$ , and  $\lambda_k(\phi_k)$  is the optimal partial return at a given time for a particular storage value. This may be rewritten as:

$$\lambda_k(\phi_k) = \min_{S_k} \ell(S_k - \phi_k + X_k) + \lambda_{k-1}(S_k)$$

which is the standard dynamic programming solution.

#### Extension to a Four Lake System

The single lake algorithm described by Young has been extended to the four lake Great Lakes System in the Great Lakes Levels Board Studies (Pentland and Eryuzlu (1969)). The following description is taken directly from that paper. The method adopted is a combination of optimization and iterative techniques. The algorithm commences with a consideration of Lake Ontario (the further downstream) alone, assuming that the upstream lakes remain in their natural state. The utility functions are all expressed in terms of economic loss, and therefore the objective is to minimize the total loss. Since there are several reservoirs in series, the local supply is only part of the total supply, and the water balance equation becomes:

$$D_{1,i} = D_{2,i} + X_{1,i} + S_{1,i} - S_{1,i+1}$$

Here, the first subscript represents the lake, and the second the time period. Therefore  $D_{1,i}$  and  $D_{2,i}$  represent outflows from the first and

second lakes, i.e. Lakes Ontario and Erie.  $X_{j,i}$  is the local inflow to lake  $j$  in time frame  $i$  and is made up of runoff from the basin, groundwater inflow, and precipitation on the lake, less evaporation from the lake surface.

Mean storage values  $\bar{S}_{j,i}$  for time period  $i$  and lake  $j$  are assumed to be equal to  $\frac{1}{2} (S_{j,i} + S_{j,i+1})$ . If only Lake Ontario is considered regulated, the utility functions which can affect the outcome on Lake Ontario are:

$U_{i,k} (D_{1,i})$  = relative loss of utility  $k$  in month  $i$  due to outflow  $D_{1,i}$  and  $f_{i,m} (\bar{S}_{1,i}, \bar{S}_{2,i}, \bar{S}_{3,i}, \bar{S}_{4,i})$  = relative loss of utility  $m$  in month  $i$  due to mean storages on all four lakes.

The reason all four elevations must be considered in this case even though  $\bar{S}_{2,i}$ ,  $\bar{S}_{3,i}$  and  $\bar{S}_{4,i}$  are not being altered is that the navigation loss functions depend on controlling (or minimum) depth on all four lakes. By varying Lake Ontario levels, any of the other three lakes could be controlling. The total loss which can be affected by Lake Ontario regulation in month  $i$  can be expressed as:

$$L_i = \sum_{k=1}^K u_{i,k} (D_{1,i}) + \sum_{m=1}^M f_{i,m} (\bar{S}_{1,i}, \bar{S}_{2,i}, \bar{S}_{3,i}, \bar{S}_{4,i})$$

Therefore, for the total period under consideration:

$$L = \sum_{i=1}^n \left\{ \sum_{k=1}^K u_{i,k} (D_{1,i}) + \sum_{m=1}^M f_{i,m} (\bar{S}_{1,i}, \bar{S}_{2,i}, \bar{S}_{3,i}, \bar{S}_{4,i}) \right\}$$

To simplify the notation, this equation could be rewritten as:

$$L_i = R_i (D_{1,i}, \bar{S}_{1,i}, \bar{S}_{2,i}, \bar{S}_{3,i}, \bar{S}_{4,i})$$

With the solution constrained by the water balance equation, we wish to determine a set of storages  $S_{1,1}, S_{1,2}, S_{1,3} \dots S_{1, n+1}$  on Lake Ontario such that the total loss is minimized:

$$\hat{T}_n = \text{Min}_{S_{1,1}, S_{1,2}, \dots, S_{1,n+1}} \left\{ \sum_{i=1}^n R_i (D_{1,i}, \bar{S}_{1,i}, \bar{S}_{2,i}, \bar{S}_{3,i}, \bar{S}_{4,i}) \right\}$$

Applying the principle of optimality:

$$\hat{T}_n = \text{Min}_{S_{1,n+1}} \left\{ R_n (D_{1,n}, \bar{S}_{1,n}, \bar{S}_{2,n}, \bar{S}_{3,n}, \bar{S}_{4,n}) + \hat{T}_{n-1} \right\}$$

This concludes the formulation of the initial minimization on Lake Ontario. The solution then proceeds upstream to Lake Erie. The loss function now includes Lake Erie outflow as an additional variable:

$$\hat{T}_n = \text{Min}_{S_{2,n+1}} \left\{ R_n (D_{1,n}, D_{2,n}, \bar{S}_{1,n}, \bar{S}_{2,n}, \bar{S}_{3,n}, \bar{S}_{4,n}) + \hat{T}_{n-1} \right\}$$

The solution is constrained by:

$$D_{2,i} = D_{3,i} + X_{2,i} + S_{2,i} - S_{2,i+1}$$

and  $D_{1,i} = D_{2,i} + X_{1,i} + \ell_{1,i} - \ell_{1,i+1}$

$\ell_{1,i}$  and  $\ell_{1,i+1}$  are the beginning and end of period storages of Lake Ontario resulting from the initial minimization. In other words, any change which is made in the Lake Erie outflow from its natural state is transferred in its entirety to the Lake Ontario outflow. Since Lake Ontario outflows depend on the change in Lake Erie outflows from their natural state, the economics associated with Lake Ontario outflows act as part of the objective in the Lake Erie minimization.

After optimizing the operation of Lake Erie, a second iteration is carried out on Lake Ontario. This is done because of the fact that the Lake Ontario minimization was performed assuming natural inflows to Lake Erie. In other words the set of storages  $\ell_{1,1}, \ell_{1,2}, \dots, \ell_{1,n+1}$  may not correspond to the optimal route after the inflows to Lake Ontario are changed. Therefore, in the second iteration a new set of storages is determined for which lower total loss will result. Any change from the

initial route must necessarily improve the overall result. As was indicated earlier, the minimization is accomplished by a) minimizing the economic loss by the consideration of a single reservoir at a time, and b) minimizing the total economic loss by an iterative approach.

The operation then proceeds to the third reservoir (Lake Michigan-Huron). The objective function is:

$$\hat{T}_n = \text{Min}_{S_{3,n+1}} \{ R_n (D_{1,n}, D_{2,n}, D_{3,n}, \bar{S}_{1,n}, \bar{S}_{2,n}, \bar{S}_{3,n}, \bar{S}_{4,n}) + \hat{T}_{n-1} \}$$

Constrained by:

$$D_{3,i} = D_{4,i} + X_{3,i} + S_{3,i} - S_{3,i+1}$$

$$D_{2,i} = D_{3,i} + X_{2,i} + l_{2,i} - l_{2,i+1}$$

and  $D_{1,i} = D_{2,i} + X_{1,i} + l_{1,i} - l_{1,i+1}$

This is followed by a second iteration on Lake Erie and a third iteration on Lake Ontario. Finally, Lake Superior is considered:

$$\hat{T}_n = \text{Min}_{S_{4,n+1}} \{ R_n (D_{1,n}, D_{2,n}, D_{3,n}, D_{4,n}, \bar{S}_{1,n}, \bar{S}_{2,n}, \bar{S}_{3,n}, \bar{S}_{4,n}) + \hat{T}_{n-1} \}$$

Constrained by:

$$D_{4,i} = X_{4,i} + S_{r,i} - S_{4,i+1}$$

$$D_{3,i} = D_{4,i} + X_{3,i} + l_{3,i} - l_{3,i+1}$$

$$D_{2,i} = D_{3,i} + X_{2,i} + l_{2,i} - l_{2,i+1}$$

and  $D_{1,i} = D_{2,i} + X_{1,i} + l_{1,i} - l_{1,i+1}$

A second iteration on Lake Michigan-Huron, a third iteration on Lake Erie, and a fourth iteration on Lake Ontario conclude the procedure.

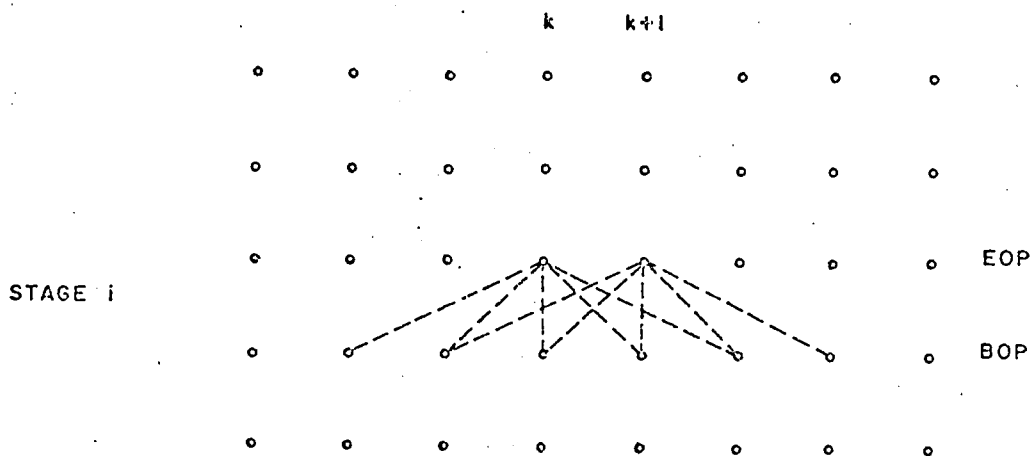
At this stage, we have defined a set of flows and stages for the entire system, along with an associated economic loss. This is not necessarily the absolute optimal solution. In fact, the overall result could

probably still be improved by simply continuing the iterative process. However, it is possible to estimate the proximity of the solution to the absolute optimum. This can be done by evaluating the maximum benefit obtainable for each of the three major interests separately, i.e. navigation, shore-property, and power. The benefits to each of the three major categories are not additive because they are conflicting (competing for the same resource). From experience with the system, a rough estimate of the proportion of the total which might actually be achieved can be made and compared with the results from the Dynamic Programming Algorithm.

#### Computational Efficiency

In a large problem like the one being described, computational problems are usually encountered. On each lake, several hundred (say 400) incremental stages must be considered for each of the 816 time frames under consideration. Computer time is proportional to  $t n^2$  where  $t$  represents the number of time frames and  $n$  represents the number of incremental storages considered. Therefore, in the computer program, considerable care was taken to reduce the number of computations to an absolute minimum. Some of the short cuts used are discussed below:

a) Elaborate experimentation was carried out in the preliminary studies to demonstrate that the lake level from the beginning of the month to the end of the month would not change by more than a given amount. Therefore, from each EOP level, only a limited number of BOP levels had to be considered. This maximum possible change depends on the surface area of the lake, and the variability of the supplies. Schematically the reduction was made as follows:



If  $m$  is the maximum number of increments representing the maximum change in storage in any time period, then only  $2m+1$  BOP levels must be examined for each EOP, instead of all  $\ell$  operations (total number of increments). This results in very large savings since in a single stage we have to make only  $\ell(2m+1)$  calculations instead of  $\ell^2$  calculations.

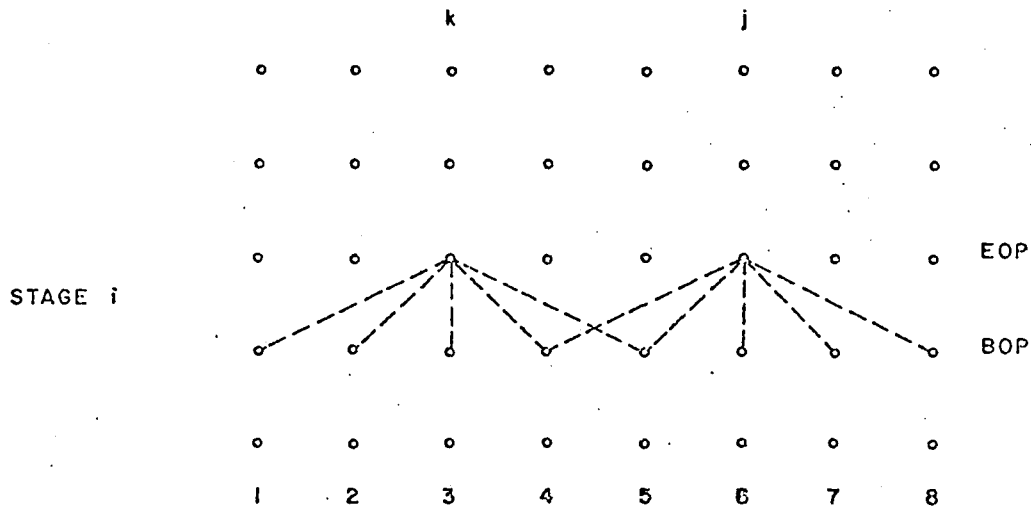
b) During stage  $i$ , the water balance equation (say for Lake Ontario) is:

$$D_{1,i} = D_{2,i} + X_{1,i} - \text{change in storage.}$$

At any particular stage,  $D_{2,i}$  and  $X_{1,i}$  are constant.

$$D_{1,i} = \text{constant} - \Delta S$$

Therefore, within any one stage there can only be as many different outflows as there are different changes in storage. The maximum number of outflows that have to be evaluated throughout stage  $i$  is  $2m+1$ . This can be illustrated schematically:



If  $m=2$ , then there are only  $2m+1 = 5$  possible changes in storage.

For example, in the above diagram:

$$\Delta S(K,1) = \Delta S(J,4)$$

$$\Delta S(K,2) = \Delta S(J,5)$$

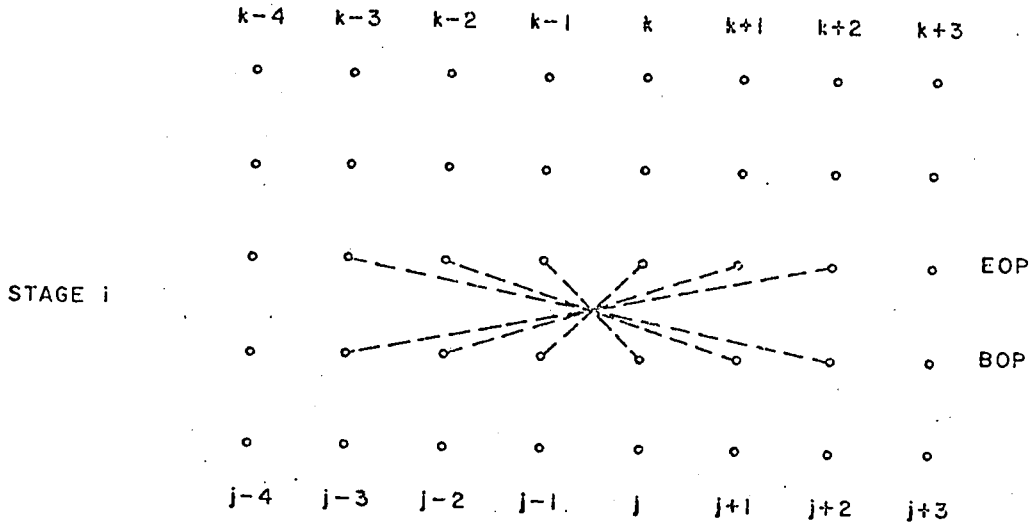
$$\Delta S(K,3) = \Delta S(J,6)$$

etc.

Regardless of the number of nodes, therefore, there can only be  $2m+1$  possible outflows, and the loss functions associated with outflow need only be examined  $2m+1$  times for each stage.

c) Many of the objective functions depend on monthly mean storages. During any stage, the mean elevations on all the lakes except the one which is being optimized is fixed. Examining all possible combinations of beginning and ending storages at stage  $i$ , it can be seen that a great many

are equal to one another.



Considering the mean storages between the two nodes ( $k$ ) and ( $j-1$ ), it can be seen that this mean is equal to that between the nodes ( $k+1$ ) and ( $j-2$ ), ( $k+2$ ) and ( $j-3$ ), etc. By inspection, it can be seen that there are only  $(2l - 1)$  unique mean storages. Therefore, in terms of storage, only  $2l - 1$  loss determinations have to be repeated at each stage instead of  $l^2$ .

Several other time-saving devices have been included in the program. These include limiting the outflows to realistic values, and combining loss curves where possible prior to the optimization process.

## Reservoir Operating Rules

### Overall Approach

As mentioned previously, the second stage of the optimization studies is the maximization of net benefits with realistic supply forecasts, and the cost of regulatory works taken into account. This implies a two-level optimization procedure a) optimization of the physical design, and b) optimization of the operation of the system. In terms of project configuration, there are four basic alternatives to be considered: (1) the present system with the possibility of improved regulation on Lakes Superior and Ontario (2) Lakes Superior, Michigan-Huron and Ontario regulated (3) Lakes Superior, Erie and Ontario regulated and (4) all four lakes regulated. Within each of these alternatives, there are many design variables which might vary, for example the capacity of the connecting channels, and features of the control structures.

This type of two-level optimization problem can be handled by fixing a design, and optimizing the operation. The design can then be changed and the operation reoptimized. If this procedure is repeated often enough, some indication of the shape of the response surface can be determined by comparing costs and benefits for various designs. This chapter is broken-up into two parts, the first dealing with the operational problem, and the second dealing with the design problem.

### Operation

Operation might be defined for the purposes of this report as a time schedule of releases from reservoirs. The establishment of such schedules, which indicate quantities of water to be affected through the actions of the manager at defined points in time, forms the basis of operating rules, or a regulation plan. Operating procedures usually consist

of three kinds of decisions. Storage and release of water must be apportioned 1) among reservoirs, 2) among purposes (interests), and 3) among time periods.

One of the most important factors to be recognized in reservoir operation is that any given decision is likely to have consequences over a considerable period of time. This is particularly true of the Great Lakes. Due to the very large storage volumes in the lakes, and the limited discharge capacities in the outflow rivers, extreme high or low levels tend to persist long after the factors causing them have changed. For example, under present conditions, it takes three and one-half years for one-half of the effect of a regulation decision on Lake Superior to be realized in the outflows of Lake Ontario.

In the dynamic programming algorithm discussed earlier, this time lag factor could be easily handled because perfect forecasts were inherent in the methodology. The problem of developing operational regulation plans is therefore a more difficult task because the concept of decision making under uncertainty must be introduced. Although the dynamic programming solution described earlier relies on deterministic hydrology, stochastic dynamic programming solutions to the reservoir operating problem have been formulated (Butcher 1968). Unfortunately, this type of solution has not yet reached the stage of development where it would be useful in current Great Lakes studies. Nevertheless, it will be described briefly below.

In order to formulate a pure stochastic dynamic programming solution, it is necessary to analyse the inflows and express the relation between them in terms of transition or conditional probabilities. With the conditional probabilities of month to month supplies, the problem can be formulated as follows:

$$f_i(S_i, q_{i+1}) = \text{Max}_d \{R(d) + \sum_{q_i=0}^{q_i=q_i} \max (P(q_i/q_{i+1}) \cdot f_{i-1}(S_i+q_i-d-e_i, q_i))\}$$

where:

$f_i(S_i, q_{i+1})$  = expected return from the optimal operation of a system which has  $i$  time periods to the end of the planning period,

$S_i$  = the volume in storage at the start of the  $i$  time period

$q_i$  = the flow into the reservoir in the  $i^{\text{th}}$  time period

$d$  = release of water during the time period being considered

$R(d)$  = the return obtained consequent on releasing a quantity of water  $d$  in the  $i^{\text{th}}$  time period

$P(q_i/q_{i+1})$  = the transition probabilities connecting the flow in the  $i^{\text{th}}$  time period  $q_i$  with the flow in the  $(i+1)^{\text{th}}$  time period  $q_{i+1}$

$e_i$  = the loss of water in the storage by evaporation during the  $i^{\text{th}}$  time period.

By starting this procedure at an arbitrary future time period, and stepping backwards one period at a time, it is possible, using any objective function, to find optimum releases to be made at any state in the system. The result of this calculation is a matrix of releases  $d$  to be made under all states of the system, i.e. for all values of  $S_i$  and  $q_{i+1}$ .

This formulation provides a policy which is capable of being used both for design studies and for actual operation. In it the state of the system is described by two variables, the flow in the preceding month  $q_{i+1}$  and the quantity of water in storage  $S_i$ .

These values are available in advance of a decision to release so that foreknowledge of any kind is not required as in the case of deterministic dynamic programming. Naturally, the use of this policy, which maximizes the expected value of the return will always be inferior to that which depends on full knowledge of hydrologic events.

A practical alternative to stochastic dynamic programming is an indirect approach sometimes referred to as Monte Carlo Dynamic Programming (MCDP). The solution strategy as described by G.K. Young (1968) is as follows:

1. Estimate the coefficients of a pre-specified synthetic hydrology model from the recorded data and exogenous data.
2. Generate synthetic data.
3. Apply deterministic dynamic programming to the complete synthetic data set to determine optimal storage variables and reservoir releases. Decision variables are the storage volumes and outflow values.
4. Merge the storage and outflow data with the inflow and exogenous data so that each observation consists of values which correspond to the same time frame.
5. Use least squares regression or some other means of inference to obtain decision rules which specify the outflow as a function of inflow values, storage volumes, and other exogenous data.

This is similar to one of the approaches being investigated in current Great Lakes studies. The reason why MCDP has been chosen in preference to stochastic dynamic programming (SDP) is that, even though it is an indirect solution, it is much more efficient in terms of computer time and memory requirements than the direct solution. For example, in a recent study, the storage value was discretized in 150 slices and 150 time frames were considered. A MCDP solution using an IBM 360/65 required 10

minutes while the SDP time requirement was estimated at 280 minutes.

A second approach being investigated in current Great Lakes studies is that of developing so-called probabalistic regulation plans i.e. plans which are designed to fail some predetermined percentage of the time. In earlier development of regulation plans, standard rule curve and limitation methods were used. Regulation plans were usually developed painstakingly over a few critical low and high supply sequences and then tested over a longer period. The result was that the plans would work very well over the extremes of the period of record since in fact perfect forecasts of droughts and floods were inherent in the design of the plans. However, operational benefits proved to be smaller in practice than the planned benefits because extremes could not be accurately forecasted.

During the drought of 1964, the operational Boards responsible for the control of Lakes Superior and Ontario were required to make discretionary deviations from the plans. These deviations were made with a consideration of future supply probabilities. From this experience, a natural evaluation towards probabalistic or stochastic regulation methods developed. In the Great Lakes Levels Board studies, the probabalistic approach is being used extensively, in which recognition is given to the fact that fixed criteria will be violated some small percentage of the time. One of the probabalistic plans studied earlier was described by Clark and Cavadias at the Fort Collins Symposium in 1967. The basic elements of the plan were as follows:

(a) The main aim of the plan was to keep the storage in the lake (Ontario) as high as possible while recognizing that the target elevation would be exceeded a small percentage of the time. This was achieved by selecting the discharge each month in such a way that the level 245.77

(one foot below the maximum specified in the current regulation criteria) for the following June would be exceeded one year in each ten. In other words the plan was in fact "designed to fail" ten percent of the time.

(b) Another feature of the plan was that, during drought years the storage available was subdivided into "seasonal" and "carry-over" storage and the operation was based on rules aimed at increasing the fall and winter outflows for the benefit of the downstream interests.

It would be expected that this type of approach when tested over many sequences of generated supplies would give a realistic estimate of the benefits to be gained from future operation. This type of plan could be extended to a multi-reservoir system in a number of ways. One way might be the route water supplies with a given probability of exceedance through the system for the time period in question. The levels resulting from this routing could then be compared with objective conditions, and the outflow adjusted to offset noted violations. Another method might be to treat the water supply to the lakes as a system rather than dealing with it on an individual lake basis. Dealing with the water supply in this fashion has the advantage that the total supply to the system has less variability than the individual lake quantities and thereby permits more accurate forecasting. If this option were chosen, the objective might be to regulate the outflow from the lower reservoir in such a way that a target storage in the total system is violated some predetermined percentage of the time, and then regulate the other reservoirs so as to balance the storage between all lakes.

In a probabalistic type of plan such as those discussed above, there is no direct link with the economics such as is available in Monte Carlo Dynamic Programming. Therefore, the link must be introduced

separately. This can be done by determining desirable objectives from the output of the deterministic dynamic programming as discussed in Chapter 4, and using these desirable objectives to set target levels in the probabalistic plan. The dynamic programming output is also useful in stipulating realistic outflow limitations.

### Design

There are several types of system components which can be changed from one simulation run to the next. These include design variables such as channel capacity, and features of the control structure, and operating policy parameters such as target levels, outflow limitations, and operational objectives. These components can all be classified in one of the two following categories:

- 1) Physical dimensions of the Civil Engineering works (this category includes present structures, their extensions, and future works)
- 2) The set of rules defining a regulation plan.

As mentioned previously, one approach to optimizing the total design might be to fix the physical dimensions, and optimize the regulation plan. This could be repeated for several physical designs to get some idea about the shape of the response surface. Although current studies have not yet reached this stage of development, one possible method of accomplishing this was described by Pentland, Rosenberg and Cavadias (1968).

(a) The feasible ranges of all parameters which can be changed would be chosen and a decision made regarding the number of intermediate values of the parameters that will be considered.

(b) System designs corresponding to all combinations of parameter values would be identified and numbered in a systematic way. Due to time and cost limitations, it would be impossible to evaluate all the resulting

designs even by digital computer, and therefore a sampling procedure would be used (i.e. each design requires a specific detailed cost estimate of the control works which depends on foundation conditions, hydraulic considerations, etc. This work limits the number of designs which can be considered). A certain number of designs could be chosen at random among all the possible designs and the corresponding benefits and costs could be evaluated. Although the mathematical equation of the net benefit surface in terms of the system parameters is not known, the selection of "n" designs at random permits the following useful probability statement to be made. The probability "P", that at least one of the "n" designs will be in the upper "p%" of the distribution of the benefit function is  $P = 1 - (1-p)^n$ . For example, if  $n = 50$  and  $p = 0.10$ , then  $P = 0.995$ . This statement does not depend on the number of system parameters considered, but on the other hand it does not indicate how close to the optimum value the best of the n designs investigated is.

(c) At the end of step (b), some indication of the shape of the response surface will be available. If all designs give similar results, this would indicate a fairly flat surface, and no further refinement might be necessary. However, if the results varied greatly, it would be advisable to use the best designs determined by the sampling procedure as starting points for a "steepest ascent approach". In view of the many uses, and geographical differences, the desirability of this refinement cannot be evaluated at the present time.

### DISCUSSION AND CONCLUSIONS

Previous sections have dealt with methodology being developed in current Great Lakes regulation studies, with an emphasis on the systems approach. Since the studies are still three years from completion, it is not possible to present numerical results at this time. Nevertheless, a general discussion on the adequacy and shortcomings of the techniques based on preliminary results is presented.

a) Flow-level Model - simulation is perhaps the most powerful of all tools available to the water-resource systems analyst. The flow level model being used in the Great Lakes Levels Board studies would fall into this category. After having included economic evaluation subroutines with the model describing the physical movement of water through the system, it is a simple matter to evaluate the economic impact of any set of decisions (both design and operating rules). It has been found that the model satisfactorily reproduces historical events and would therefore be expected to satisfactorily describe what these events would have been under different design and operating conditions. Projecting the economic functions into the future, and varying management policies in the model therefore provides a convenient method of studying a large number of alternative strategies.

Based on considerable use of the model, it can be concluded that it adequately represents the real-life situation. Future developments will therefore be concentrated on computationally efficiency, since it is expected that the model will be used hundreds of times. The program at present requires about two minutes of computer time (UNIVAC 1108) for a 68-year test. It is expected that this will be reduced substantially as the studies progress.

b) Supply synthesis - Based on experience to date, it is concluded that the synthesized supplies will play a key role in defining the sensitivity of the economics to hydrologic variability. However, there is still a need to further refine the generating model. Although the synthesized supplies themselves exhibit a close statistical resemblance to recorded supplies, the levels resulting from routing these supplies point out a fairly important weakness in the technique. That is, insufficient long term persistence appears to have been introduced into the synthesized data to produce droughts as serious as those which have been recorded with a reasonable frequency. It is expected that this problem can be overcome with a multiple lag model rather than the first order Markov process used.

c) The optimizing technique - While the technique used (the single reservoir dynamic programming algorithm extended to the four-lake system by an iterative technique) is adequate in terms of accuracy, it is extremely expensive to use. Therefore there will be a need later in the studies to re-examine the optimizing problem from an efficiency point of view. It is expected that changes will be made to the economic inputs several times during the course of the studies, and re-optimization may be required. It would also be advantageous to have a more efficient optimization tool for use in sensitivity studies. For these reasons it is probable that alternative mathematical programming techniques will be considered further as the study progresses.

d) Reservoir operating rules - The techniques being used for developing reservoir operating rules from the output from the Dynamic Programming Algorithm were discussed earlier. This work will not be fully completed for some time, and therefore the conclusions reached to

date are only tentative. Nevertheless, it now appears that benefits obtainable in the second stage optimization studies (realistic supply forecasts) will be in the order of one-half to three-quarters of those obtainable in the first stage (perfect supply forecasts). It also appears that benefits in this order of magnitude will be obtained with any of the techniques described i.e. none is obviously superior to any other. Another technique is currently being investigated (that of operating to maximize benefits in the current time period without consideration of the long-term effect of the decisions). This approach shows promise but it is too early to reach any conclusions as to its adequacy.

- 80 -

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