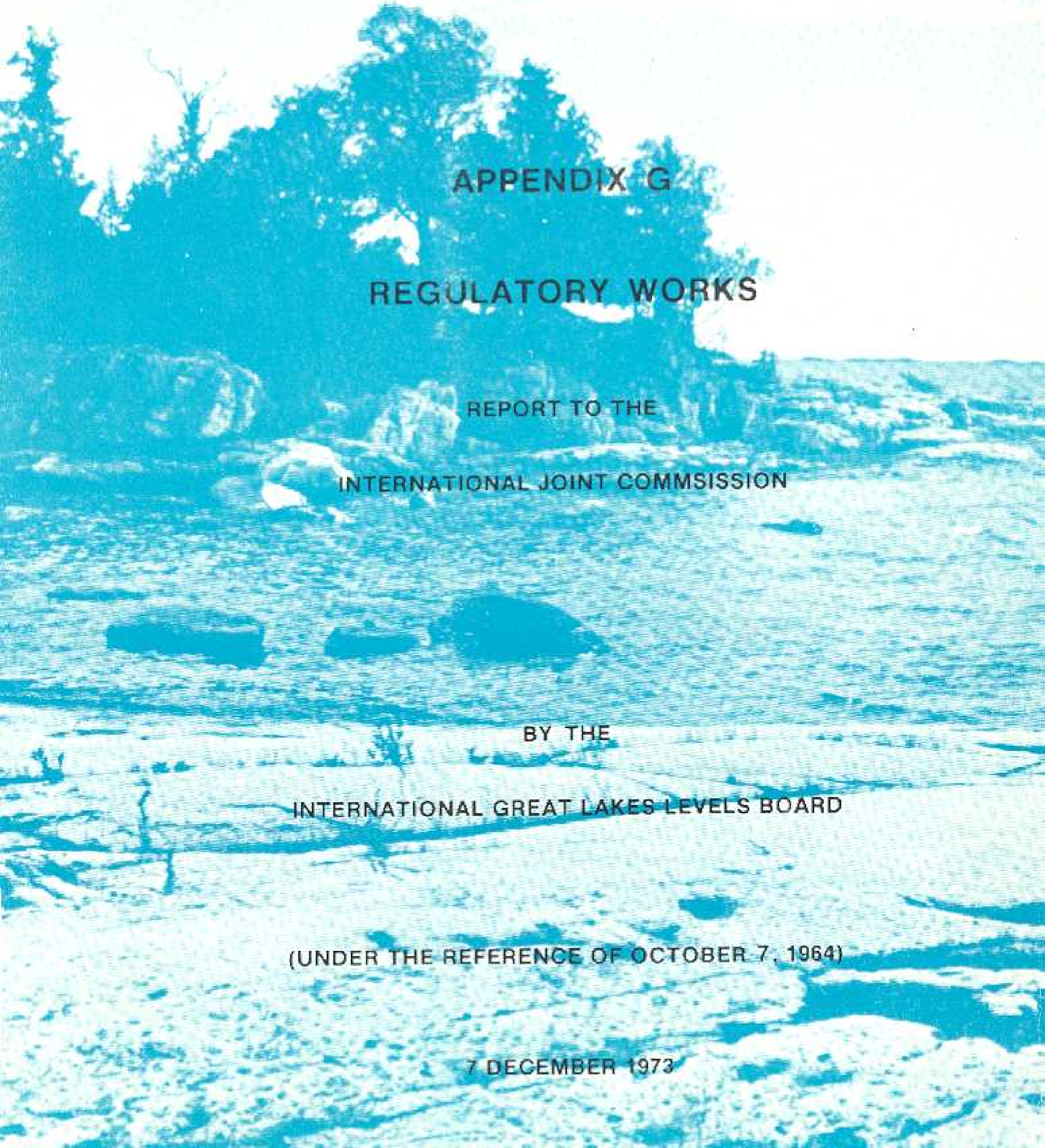
REGULATION OF GREAT LAKES WATER LEVELS



APPENDIX G

REGULATORY WORKS

REPORT TO THE

INTERNATIONAL JOINT COMMISSION

BY THE

INTERNATIONAL GREAT LAKES LEVELS BOARD

(UNDER THE REFERENCE OF OCTOBER 7, 1964)

7 DECEMBER 1973

REGULATION OF GREAT LAKES WATER LEVELS

APPENDIX G

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(UNDER THE REFERENCE OF OCTOBER 7, 1964)

SYNOPSIS

The Regulatory Works Appendix describes the engineering works that would be necessary to accomplish further regulation of the levels and flows of the Great Lakes, as described in the report on Regulation of Great Lakes Water Levels, dated December 7, 1973.

Regulation of any lake generally requires two basic facilities: first, one or more control structures capable of reducing the outflow, especially when low lake levels occur, and second, dredging of its outlet river so that greater flows can be released at times when high lake levels occur. This appendix describes the existing facilities in the outlet rivers of Lake Superior and Lake Ontario and discusses the problems to be faced in providing new facilities, particularly in the outlet rivers of the presently unregulated Lakes Michigan, Huron and Erie. It also describes the site investigations carried out, the design criteria and methods used, and the environmental factors considered in preparing preliminary designs and cost estimates of the works necessary for the various selected regulation plans considered in the Board's study.

For all the selected plans it was found that the existing works in the St. Marys River have adequate capacity and sufficient remaining life so that no major engineering work will be necessary. However, improved regulation of Lake Superior would be possible if the gates of the compensating works at Sault Ste. Marie could be operated, when necessary, to vary the flow during winter. Normally, ice conditions restrict operations during December through April. The appendix describes a series of practical tests conducted at the Sault Ste. Marie works during the four winters 1968-69 to 1971-72. These tests demonstrated the feasibility of winter operations and provided costs and other data. The average annual costs of improved regulation of Lake Superior, as provided by Plan SO-901, is $70,000.

Plans involving the regulation of Lakes Michigan-Huron would require works in the St. Clair-Detroit Rivers system. Due to extensive shoreline development along and the requirements of navigation through this 89-mile system, it would be necessary to undertake considerable dredging and to construct a series of at least nine control structures in order to regulate the outflow of Lakes Michigan-Huron and to maintain the hydraulic profile of the rivers. The average annual costs of such works is shown to be in the $18-21 million range, depending on the plan considered, which far exceeds the estimated economic benefits. Furthermore there would be unacceptable environmental consequences particularly in the Lake St. Clair area.

The Board's three alternative approaches to the regulation of Lake Erie, represented by selected plans SEO-33, SEO-901 and SEO-42P, would require different regulatory works in the Niagara River. It is shown that the least expensive is SEO-901 at an average annual cost of $99,000; that SEO-42P, at $380,000, is the most attractive alternative; and that the average annual cost of Niagara River works for plan SEO-33, $8 million, produces a benefit-cost ratio less than unity.

No designs or cost estimates are given for new or improved works in the St. Lawrence River, the outlet of Lake Ontario, since all the selected plans would use the existing works.

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MAIN REPORT—REGULATION OF GREAT LAKES WATER LEVELS
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APPENDIX A - HYDROLOGY AND HYDRAULICS

A detailed description of the hydrology and hydraulics of the Great Lakes system, including an outline of the "state of knowledge" of the various factors which govern its water supply and affect the response of the system to its supply.

APPENDIX B - LAKE REGULATION

A documentation of the studies related to the regulation of all the Great Lakes and various combinations of them and a presentation of an array of plans for regulating the levels of these combinations.

APPENDIX C - SHORE PROPERTY

A documentation of the methodology developed to estimate in economic terms the effects of changes in water level regimes on erosion and inundation of the shoreline, marine structures and water intakes and sewer outfalls, and of the detailed evaluations of selected regulation plans.

APPENDIX D - FISH, WILDLIFE AND RECREATION

A documentation of the methodology developed to assess the effects on fish, wildlife and recreation of changes in water level and outflow regimes and of the detailed evaluations of the effects of selected regulation plans on these interests.

APPENDIX E - COMMERCIAL NAVIGATION

A documentation of the methodology applied in the assessment of the potential benefit or loss to shipping, using the Great Lakes-St. Lawrence navigation system, as a consequence of changes in lake level regimes and the evaluation of the economic effects on navigation of regime changes that would take place under selected regulation plans.

APPENDIX F - POWER

A documentation of the methodology applied in the assessment of the effects of regulatory hydroelectric power production at installations on the outlet rivers of the Great Lakes and of the detailed evaluation of the economic effects of selected regulation plans on the capacity and energy output of these installations in terms of the costs to the associated power systems.

APPENDIX G - REGULATORY WORKS

A description of the outlet systems of the lakes, problems to be faced in providing new regulatory facilities at the outlets, site investigations carried out, design criteria and methods used, environmental factors considered, and design and cost estimates of engineering works required for selected regulation plans.

APPENDIX G

REGULATORY WORKS

Section 1

INTRODUCTION

1.1 General

Pursuant to the October 7, 1964 Reference to the International Joint Commission by the Governments of the United States and Canada, the Commission established the International Great Lakes Levels Board to carry out the studies necessary ". . . to determine whether measures within the Great Lakes basin can be taken in the public interest to regulate further the levels of the Great Lakes or any of them and their connecting waters so as to reduce the extremes of stage which have been experienced, and . . . for the purpose 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; (d) flood control; (e) agriculture; (f) fish and wildlife; (g) recreation; and (h) other beneficial public purposes."

Regulation of any lake generally requires two basic facilities; first, an increase in the discharge capacity of its outlet river so that, when necessary, more water can be released than under unregulated conditions, in order to reduce high lake levels; second, one or more control structures capable of decreasing the outflow, at other times, in order to raise low lake levels. The channel improvements, new structures and ancillary engineering works considered necessary to accomplish further regulation of the Great Lakes are the subject of this Appendix. Appendix "G" is part of the Final Report of the International Great Lakes Levels Board to the International Joint Commission, dated December 1973.

1.2 Purpose

The purpose of this Appendix is to describe the outlet systems of the lakes, the problems to be faced in providing regulatory facilities therein, the site investigations carried out, the design criteria and methods used, the environmental factors considered, and the designs and cost estimates of the engineering works which would be required to institute the various regulation plans selected in this study.

1.3 Scope

To regulate the Great Lakes System, control facilities would be required in the four outlet rivers concerned. Section 2 of this Appendix deals with the St. Marys River, the outlet of Lake Superior, and is mainly concerned with modifications to the existing control structure at Sault Ste. Marie so that it can be operated more effectively. Lakes Michigan-Huron discharge through the St. Clair River, Lake St. Clair and the Detroit River system, described in Section 3, a waterway having complex hydraulic profile characteristics. A considerable amount of channel dredging and a series of control structures, with associated high capital costs, would be required to institute regulation of these two large lakes. The type of control structures and channel capacity improvements are discussed in Section 4. In the case of the St. Lawrence River, outlet of Lake Ontario, the scope of the regulatory works study was constrained by the fact the the Moses-Saunders Generating Plants, the key feature of the St. Lawrence Seaway and Power Project, is designed as the instrument for regulating Lake Ontario. It has been assumed that any improvement over the present regulation plan must be accomplished using the facilities of this project. The St. Lawrence River is the subject of Section 5. The designs and cost estimates of regulatory facilities required for the various selected regulation plans are presented in Section 6 of this Appendix.

Throughout the course of these studies, a number of reports were prepared by various governmental agencies and private consulting engineering firms from which appropriate material was drawn for the purpose of the preliminary design of regulatory works. Abstracts of these contributory reports are contained in Annex C.

All data which were used during the course of this study, including contributory reports, are filed in a central location in Canada and the United States. These data may be obtained at either of the following agencies:

Canada Centre for Inland Waters
Environment Canada
P. O. Box 5050
BURLINGTON, Ontario
Canada
L7R 4A6

or

Detroit District
U. S. Army Corps of Engineers
150 Michigan Avenue
DETROIT, Michigan 48231
U. S. A.

1.4 Study Organization

The Working Committee, established by the Board, created a Regulatory Works Subcommittee in September 1967 to conduct the necessary engineering studies and to prepare designs and cost estimates of the works which would be required to implement the proposed regulation plans. The terms of reference of the Subcommittee is reproduced as Annex A. The Subcommittee was comprised of senior engineers from the U. S. Army Corps of Engineers and the Canadian Departments of Public Works, Transport and Environment. The members and associates of this Subcommittee are listed in Annex B.

1.5 Prior Studies

The most significant relevant prior study was that conducted by the U. S. Army Corps of Engineers, the findings of which were published in a report dated December 1965, entitled "Water Levels of the Great Lakes, Report on Lake Regulation." Appendix F thereto described the works necessary to implement two alternative proposed plans for the regulation of Lakes Michigan-Huron and one for Lake Erie. No consideration was given in that study to modifications of existing works in the St. Marys or St. Lawrence Rivers. The work accomplished in the Corps' investigation has been of considerable value to the present international study. However, the engineering works proposed in that study for the St. Clair-Detroit Rivers constituted only one possible scheme capable of regulating Lakes Michigan-Huron in accordance with each given plan. In the present study it was possible to develop more sophisticated computerized techniques whereby the number and location of structures, and the layout and extent of dredging, could be optimized.

The Subcommittee has also, wherever appropriate, referred to and drawn upon information given in report of other IJC studies and the results of various independent studies such as the investigation of dredging improvements in the St. Lawrence Ship Channel in the Canadian reach of the river by the Canadian Ministry of Transport.

Section 2

ST. MARYS RIVER SYSTEM

2.1 Description of the System

The St. Marys River, which forms the only outlet from Lake Superior, links Lake Superior at its most easterly point with Lakes Michigan-Huron. The river flows in a generally southeasterly direction over a distance of 61, 63 or 75 miles, depending on the route traversed. A location map of the area is shown on Figure G-1. The following paragraphs describe the river system in more detail.

2.1.1 General

From Whitefish Bay near Gros Cap, Ontario and Pt. Iroquois, Michigan, to its outlet on Lake Huron near DeTour Village, Michigan, the St. Marys River falls approximately 22 feet on the average. At Sault Ste. Marie, the river is divided into an upstream and downstream reach by existing regulatory facilities, including power and navigation facilities and the compensating works, which together control the total outflow from Lake Superior. The average fall of the upper reach, extending from Whitefish Bay to the head of the navigation locks, a distance of some 14 miles, is approximately 0.25 feet. Over the next 1.5 miles, the average fall across the regulatory facilities and rapids is approximately 20 feet. From the foot of the rapids to its outlet on Lake Huron, the average fall of the lower reach is approximately 2 feet. The water surface profiles for maximum, median and minimum flow conditions, for ice-free periods, are shown on Figure G-2.

During the period from 1900 through 1967, the discharge of the St. Marys River, as recorded at Sault Ste. Marie, averaged 74,000 cfs (cubic feet per second) and has ranged from a maximum monthly mean discharge of 127,000 cfs to a minimum monthly mean discharge of 41,000 cfs. However, the latter flow resulted from a labour strike at one of the power plants that reduced the average flow about 20,000 cfs for 2 months.

The International Boundary follows, in general, the median course of the Upper St. Marys River. However, in the Lower St. Marys River, the International Boundary lies in the various channels and designates the Islands of Sugar, Neebish, Lime and Drummond as United States territory and the Islands of St. Marys, Whitefish, St. Joseph, Cockburn and Manitoulin as Canadian territory.

Over the past 118 years, man-made alterations have been made intermittently to the various channels of the St. Marys River for navigation purposes. At the present time, the St. Marys River is capable of passing ships with lengths up to 1,000 feet and a draft up to 25-1/2 feet at L.W.D. (Low Water Datum) between Lake Superior and Lakes Michigan-Huron. In the Upper St. Marys River a navigation channel, having a minimum depth of 28 feet below L.W.D. and a minimum width of 1,200 feet, is maintained. In the Lower St. Marys River a navigation channel, with a minimum depth of 27 feet below

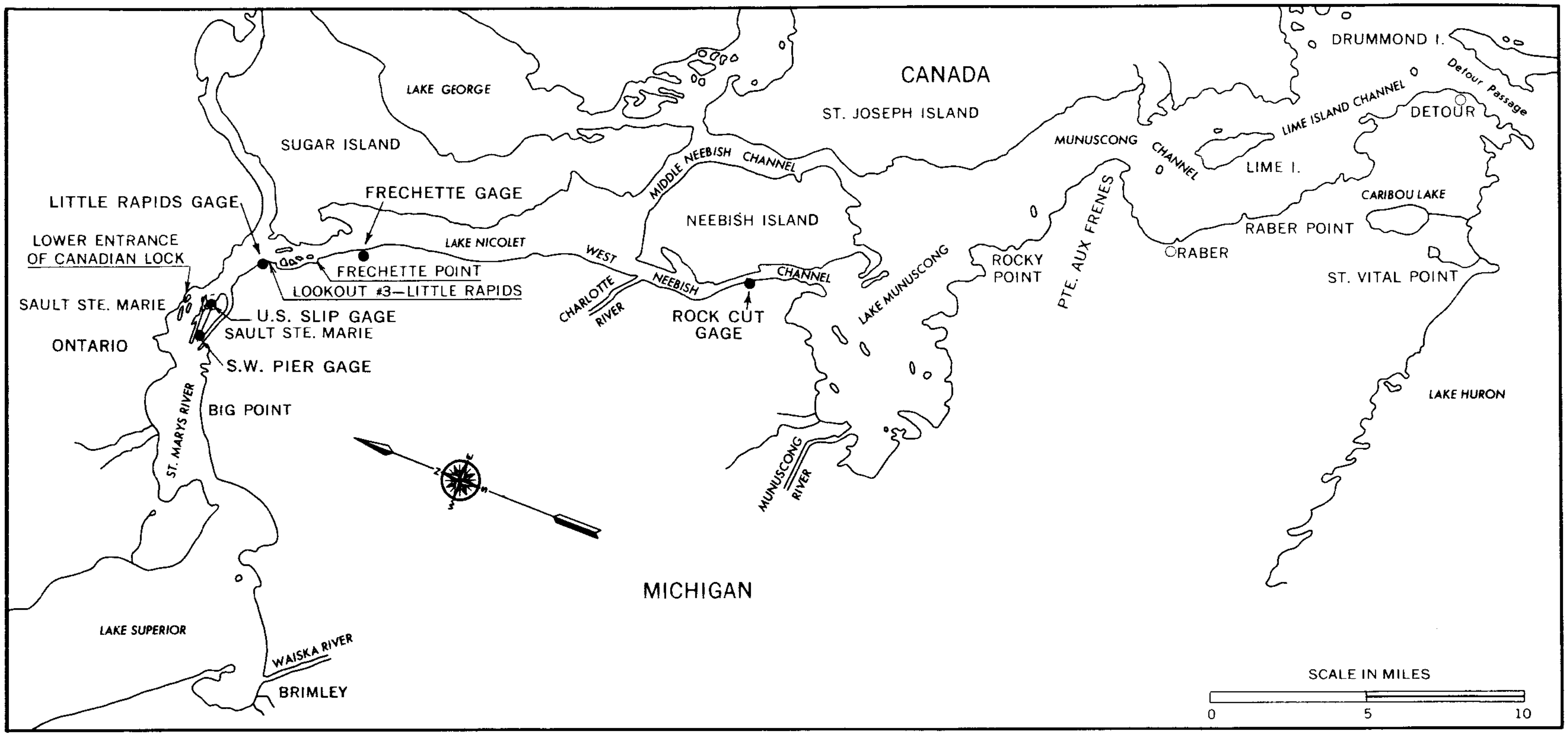


Figure G-1
ST. MARYS RIVER LOCATION MAP

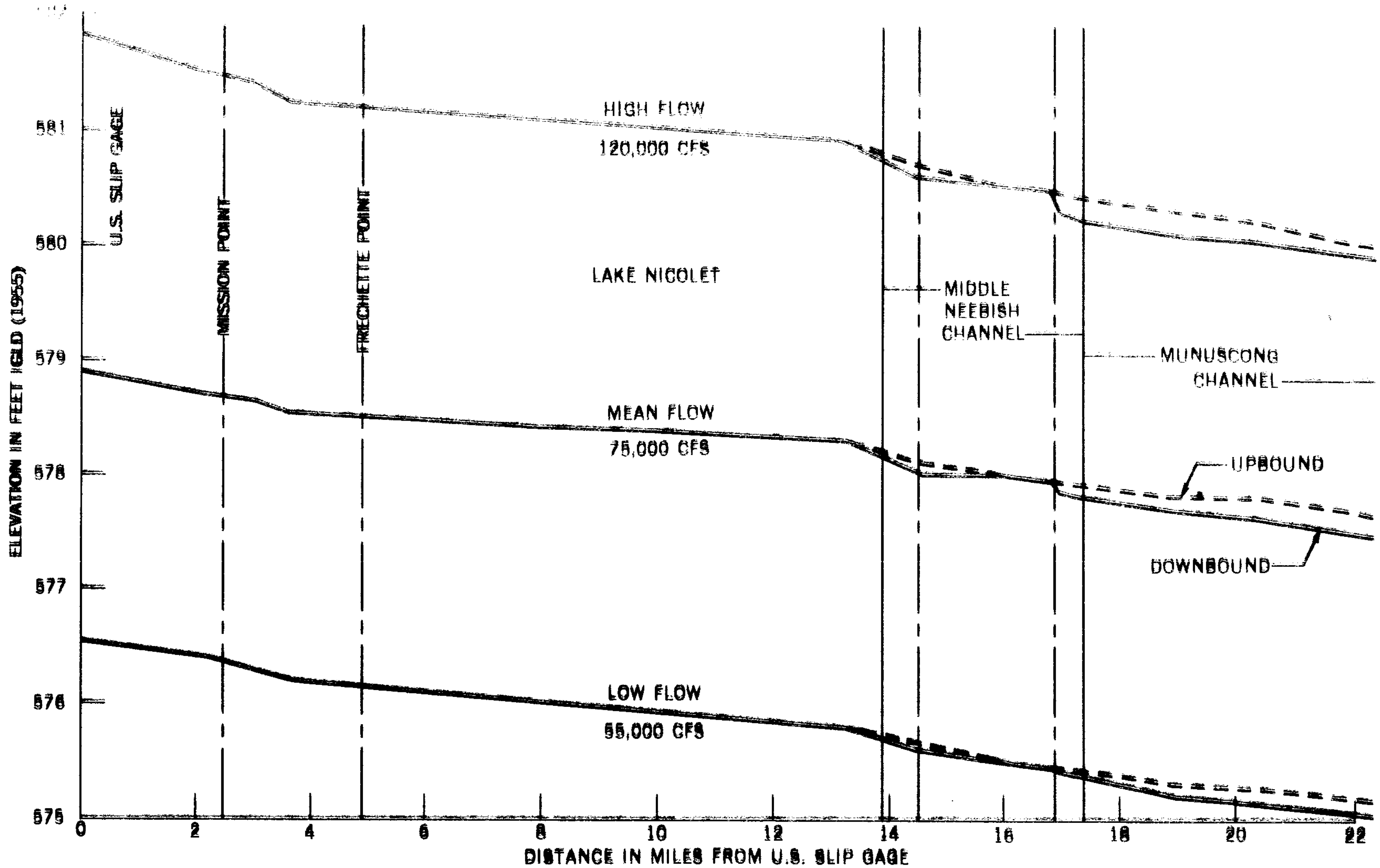


Figure G-2
LOWER ST. MARYS RIVER—WATER SURFACE PROFILES

L.W.D. and minimum widths of 300 feet and 600 feet for one-way and two-way traffic respectively, extends through the Little Rapids reach, south of Sugar Island and through Lake Nicolet to Neebish Island, at which point it becomes divided. Downbound traffic normally uses the west channel while upbound traffic normally uses the east channel. The navigation channel then joins in Lake Munuscong and passes between Michigan's Upper Peninsula and Lime and Drummond Islands where it enters Lake Huron at DeTour Passage. Several dredging projects, for the purposes of widening some of the bends in the existing navigation channels of the Lower St. Marys River, were carried out in 1972 and others are planned for 1973-74. These projects have no appreciable hydraulic effect on the regime of the St. Marys River. The Lake George route has not been dredged this century and consequently has adequate depths for small recreational craft only.

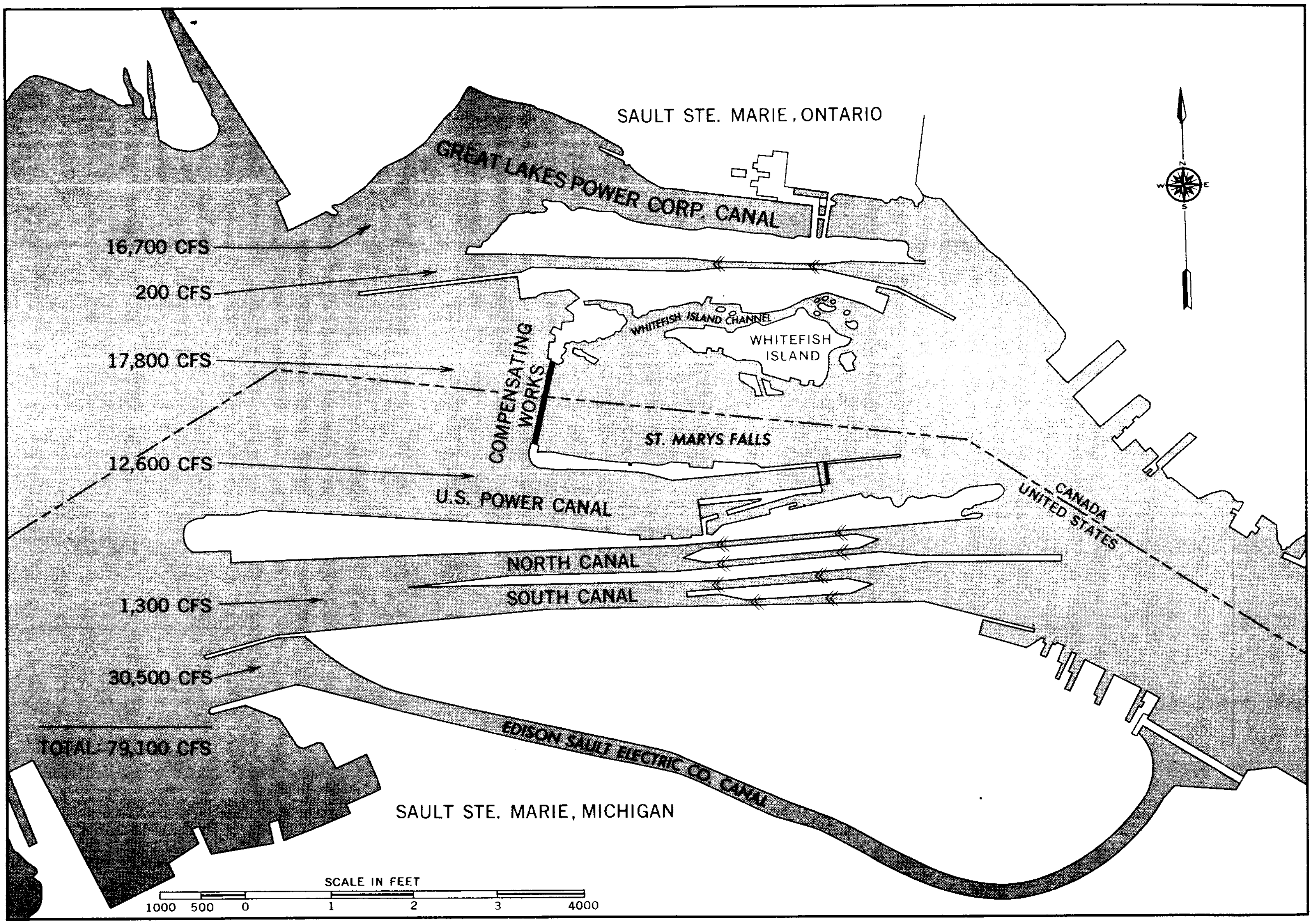
At Sault Ste. Marie, various man-made facilities have been constructed over the years including navigation locks, hydro-electric power plants and regulatory facilities. A location plan of the area is shown on Figure G-3. The approximate distribution of flow through each of these facilities is also illustrated. An aerial photograph of the navigation and compensating works at Sault Ste. Marie is shown on Figure G-4.

2.1.2 Existing Regulatory Structure

As indicated in the previous paragraph and illustrated on Figure G-3, the total outflow of Lake Superior is controlled by power and navigation facilities and the compensating works. Normally, full power flow requirements can be met and the gates of the compensating works are set to flow the remainder of the required Lake Superior outflow as specified by the current plan of regulation, the 1955 Modified Rule of 1949, a description of which may be found in Section 2.1.7. The gates of the compensating works were last rated between 1926 and 1935 utilizing the S.W. Pier gauge, above the U. S. navigation locks, as a reference gauge. Further measurements were taken in 1965 and 1970 as a check, but were not of sufficient detail to establish a new rating of the compensating works.

The compensating works consist of 16 gates, numbered 1 through 16 commencing on the Canadian side. Gates 1 through 8 are owned, operated and maintained by the Great Lakes Power Corporation Limited, based in Sault Ste. Marie, Ontario. Gates 9 through 16 are owned, operated and maintained by the U. S. Army Corps of Engineers. That organization has entered into a contract with the local U. S. utility, the Edison Sault Electric Company, to perform the actual operation and maintenance of the United States gates as required. The layout and a typical section of the compensating works are shown on Figure G-5. An aerial photograph of the compensating works, with all gates in the full-open position, is shown on Figure G-6.

Gates 1 through 4 were constructed in 1902. Their piers consist of brick and concrete with granite nosings. The elevation of the sills is 589.8 feet IGLD (International Great Lakes Levels Datum, 1955). The remaining 12 gates have a sill elevation of 588.8 feet IGLD and have concrete piers. Gates 5 through 16 were completed in 1919. With the completion of


SAULT STE. MARIE, ONTARIO
GREAT LAKES POWER CORP. CANAL
16,700 CFS
200 CFS
WHITEFISH ISLAND CHANNEL
WHITEFISH ISLAND
17,800 CFS
COMPENSATING WORKS
ST. MARYS FALLS
12,600 CFS
U.S. POWER CANAL
CANADA
UNITED STATES
NORTH CANAL
SOUTH CANAL
1,300 CFS
30,500 CFS
TOTAL 79,100 CFS
EDISON SAULT ELECTRIC CO. CANAL
SAULT STE. MARIE, MICHIGAN
SCALE IN FEET
1000 500 0 1 2 3 4000

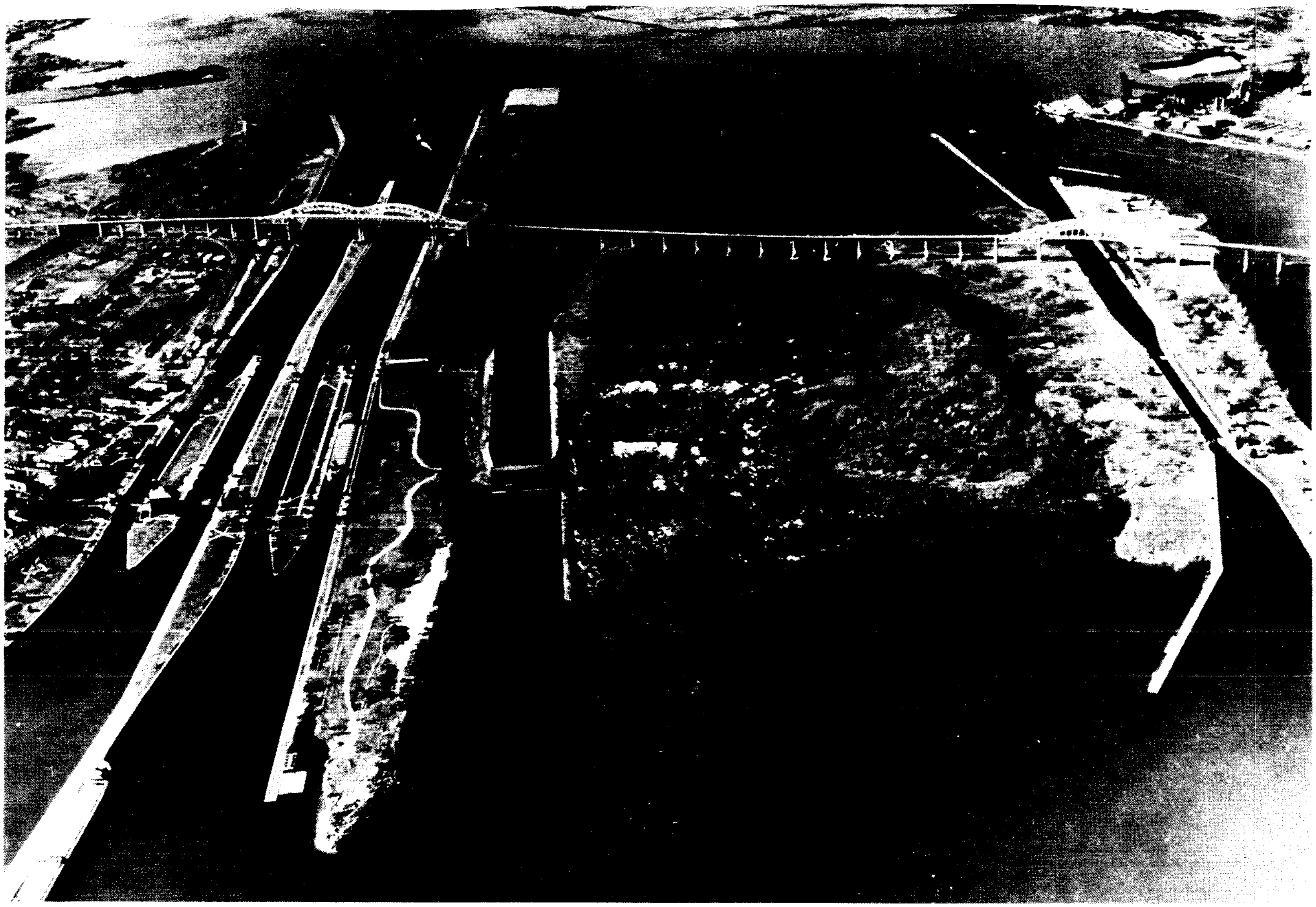


Figure G-4
St. Marys River at Sault Ste. Marie—Aerial Photograph

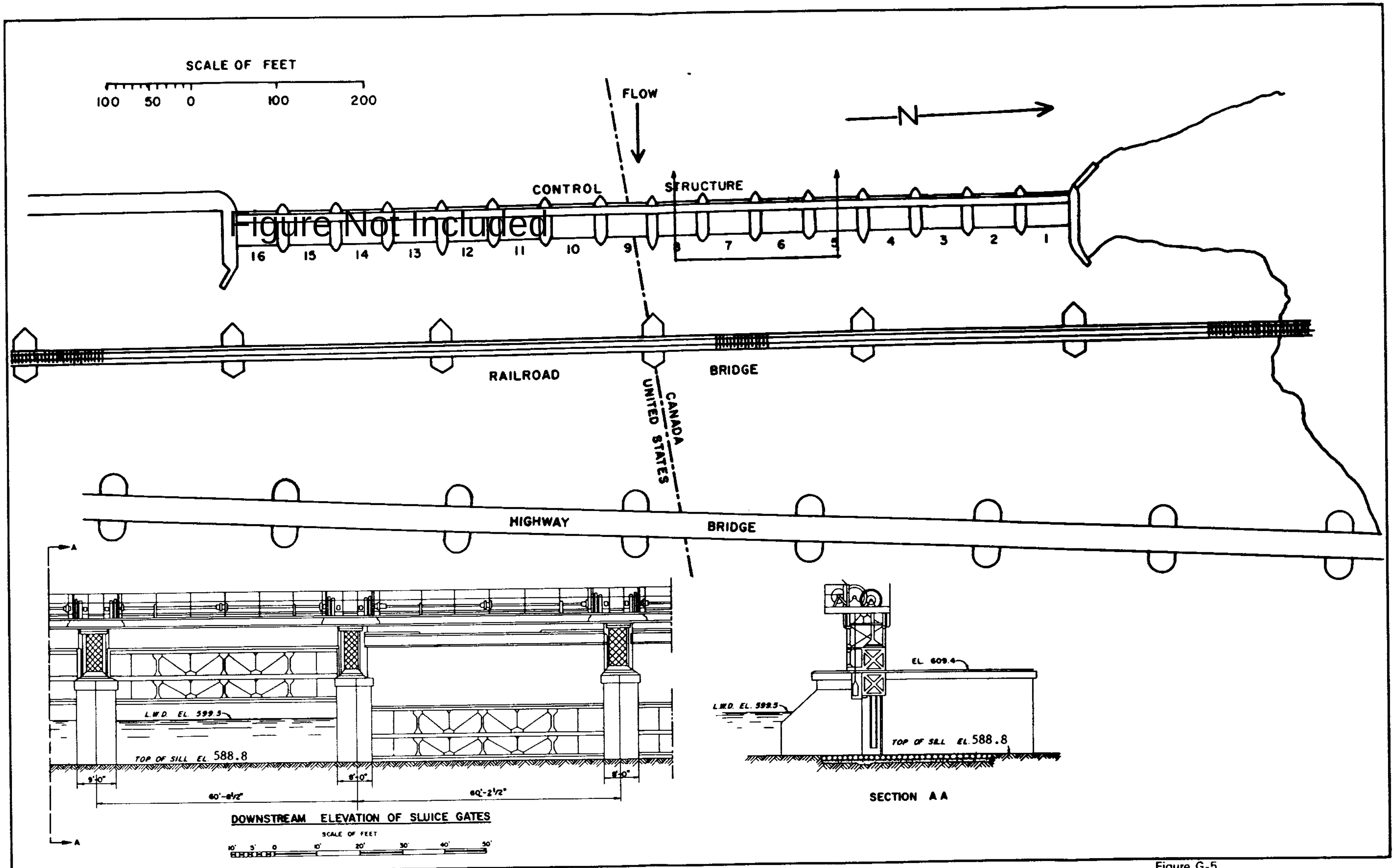


Figure G-5
EXISTING COMPENSATING WORKS AT SAULT STE. MARIE—
PLAN, SECTIONAL AND DOWNSTREAM ELEVATION VIEWS

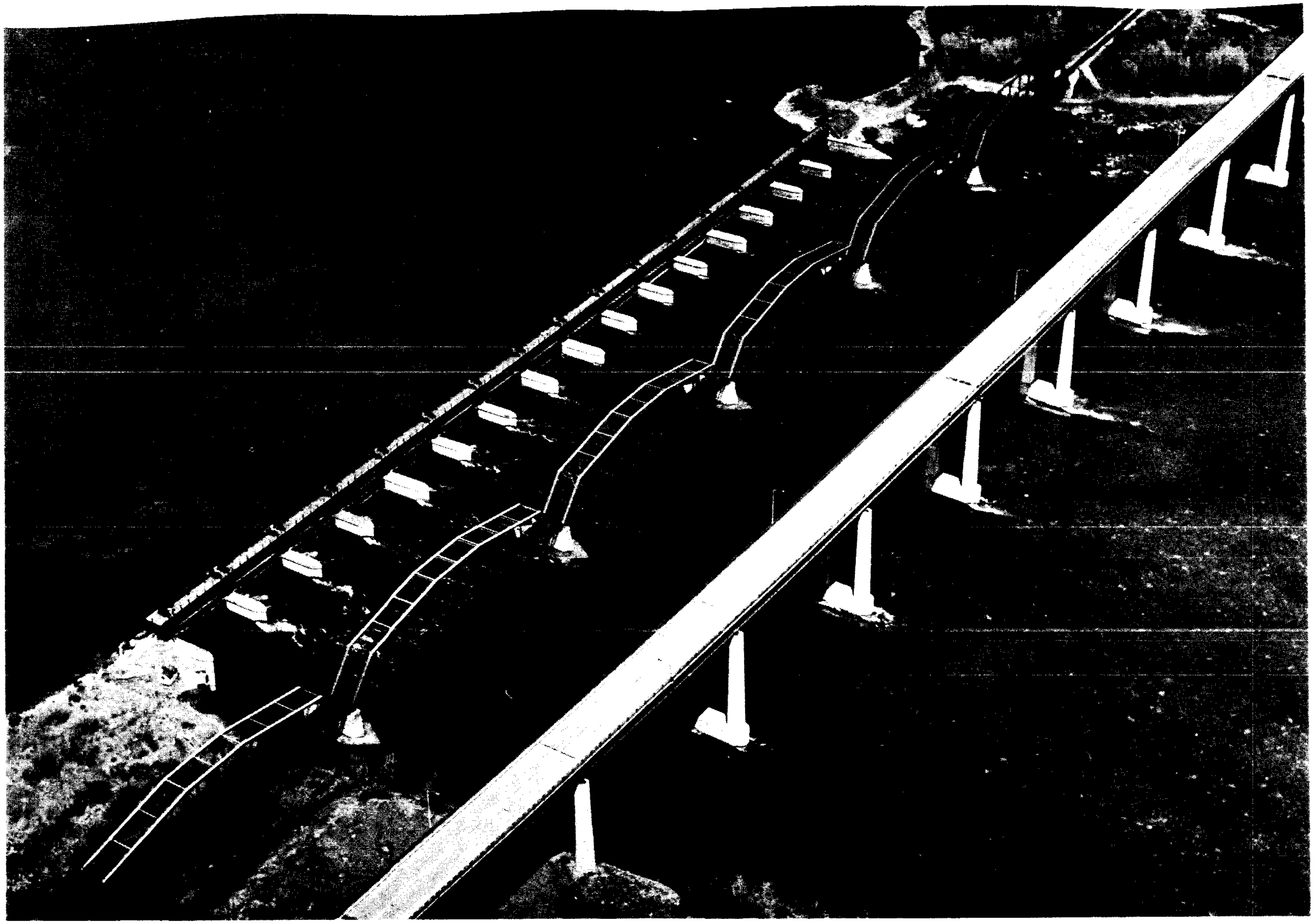


Figure G-6
Compensating Works at Sault Ste. Marie—Aerial Photograph

the south dyke in 1921, the outflow from Lake Superior was under complete control. The distances between centres alternate between 60 feet-8 inches and 60 feet-2 inches. The gates, which are 52 feet-2 inches wide and 14 feet high, lift vertically and are counterweighted. Each gate essentially consists of a stiffened upstream skinplate, supported by horizontal beams framed into vertical end members which bear on their Stoney rollers which in turn transmit the forces into the piers by means of embedded tracks. The gates are manually operated by means of winches at either end. The superstructure consists entirely of steel with a woodplank deck.

Because the gates are unheated, efficient winter operation was not possible in the past. However, tests conducted for this study during the winter periods of 1968-69, 1969-70, 1970-71 and 1971-72 indicated that efficient winter operation is possible and feasible utilizing temporary steam heating facilities. A summary of the test experience is contained in Section 2.3.2. For reasons of safety and reliability, more permanent methods of gate and gain heating, as discussed in Section 2.3.3, were investigated.

The compensating works, in their entirety, appear to be structurally sound, but the Canadian portion is in need of some repair. (See Lake Superior Board Calendar Year 1973 Report.) Provided maintenance and repair to the entire structure is carried out when necessary, it is considered that it should provide 50 more years of useful life.

2.1.3 Power Facilities and Flows

Four hydro-electric power plants are located at Sault Ste. Marie, Ontario and Michigan and all use water from the St. Marys River. Since the average gross head on these plants is approximately 20 feet, all are classed as low head run of the river plants and consequently the physical size of each plant is large in comparison with its kilowatt output. However, these plants are unique in having Lake Superior for a reservoir, one of the largest lakes in the world with a water surface area of 31,700 square miles.

The Great Lakes Power Company plant, located between St. Marys Island and the Canadian mainland, was constructed between 1916 and 1918. Additional units were installed in 1921 and 1931. The plant has 28 generators having a total installed capacity of 21,520 kilowatts at a rated head of 18.5 feet. Power from this plant is utilized by the Ontario Hydro System. Water is diverted through the plant from upstream of the compensating works to a point downstream of the rapids. Water requirements are normally 18,000 cfs.

In May 1970, the Abitibi Pulp and Paper Company, Sault Ste. Marie, Ontario, converted from hydraulic power driven pulpwood grinders to electric driven grinders. Since the hydraulic system required a water flow of approximately 7,000 cfs, diverted from the forebay of the Great Lakes Power Company plant, the total outflow capacity of the regulatory facilities is reduced by about 7,000 cfs at high lake stage. The hydraulic turbines can be operated, but with difficulty, to discharge water. Moreover, due to deterioration of some of the control mechanisms, a capacity of only about 5,000 cfs is now possible.

The United States hydroelectric plant is located in the State of Michigan between the United States navigation locks and the rapids. It consists of two separate structures having a common forebay and convergent tailraces. This plant has a total installed capacity of 18,300 kilowatts, and contains 5 generators. Power from the plant is used to supply the requirements of the United States navigation locks, the City of Sault Ste. Marie, Michigan and surrounding areas. Because of the ever-increasing demands for more electrical energy, this plant, like its counterpart in Ontario, is operated at full capacity. Water is diverted from upstream of the compensating works to the lower end of the rapids. Water requirements are approximately 12,700 cfs.

The Edison Sault hydroelectric power plant, constructed in 1902, is served by a two and one-half mile long canal which diverts water from a point just above the United States navigation locks and delivers it to the plant located about one-half mile below the locks. This plant is about one-quarter of a mile in length, has 78 generators horizontally mounted, and has a total installed capacity of 41,300 kilowatts. Water requirements are approximately 30,000 cfs. Power is used to provide for the needs of the eastern half of the Upper Peninsula of Michigan. The plant is normally operated at full capacity.

2.1.4 Navigation Facilities and Flows

There are five locks at Sault Ste. Marie enabling both commercial and recreational craft to bypass the rapids, a drop of about 20 feet. The maximum size of vessel allowed through the locks is one with a 105-feet beam by 1,000 feet length. The principal dimensions of the locks are shown on Table G-1.

TABLE G-1

PRINCIPAL DIMENSIONS OF NAVIGATION LOCKS AT SAULT STE. MARIE

Lock	Length in Feet between Inner Gates	Width in Feet	Depth of Water in Feet over Sills Referred to L.W.D.
Poe	1200	110	32.0
MacArthur	800	80	31.0
Davis	1350	80	23.1
Sabin	1350	80	23.1
Canadian	900	60	16.8

At Sault Ste. Marie, an additional channel with a minimum depth of 24 feet below L.W.D. and a minimum width of 500 feet serves the Canadian lock and the wharves of Algoma Steel Corporation.

Total water usage for lockage requirements during the historical navigation season, April 1 through mid-December, are in the order of 1,500 cfs, equivalent to an average annual flow of approximately 900 cfs. During the 1969, 1970, 1971, 1972 and 1973 navigation seasons, attempts were made with the cooperation of shipping interests, to extend the normal St. Marys River navigation season beyond the customary December 17 closing date. As a consequence, shipping was extended to January 11, 1970; January 30, 1971; February 1, 1972; and February 8, 1973. However, only a limited number of ships participated in the season extension program.

2.1.5 Bridges, Ferries, Wharves and other Facilities

Two bridges, on railway and one highway, cross the St. Marys River at Sault Ste. Marie. Both are located near the head of the locks and rapids and downstream of the compensating works. The railway bridge is a low level structure with three movable spans to allow the passage of ships. Normally, during the navigation season, all three spans are kept open providing a vertical clearance of 120 feet above high water. East of the railway bridge, a two-lane high level highway bridge, having a vertical clearance of 120 feet above high water, links United States and Canada.

In the vicinity of Sault Ste. Marie, wharves at the Algoma Steel Corporation Limited and Reiss Coal Company provide these companies with access to navigation in the upper river. Immediately downstream of the locks and rapids, numerous wharves serve various industries and the public on both sides of the river. The elevations of many of these wharves are approximately at 582.9 feet IGLD, the maximum allowable level below the locks as specified in the 1914 IJC Orders of Approval. Consequently, they are vulnerable to high water levels especially during spring breakup.

Three ferries operate on the Lower St. Marys River. The Sugar Island ferry, the Neebish Island ferry, and the Drummond Island ferry. The Sugar Island and Drummond Island ferries normally operate year-round while the Neebish Island ferry usually ceases operation about January 1st because of ice conditions, and resumes service in March.

2.1.6 Ice Problems

Ice problems in the St. Marys River are due to three principal causes, summarized as follows:

(1) Ships operating toward the latter part of the navigation season break very young ice into floes of various sizes which, moving downstream, tend to cause ice jams in the more restricted river cross sections. The Little Rapids Channel, beginning about two miles downstream from the U. S. navigation locks, is particularly susceptible to the formation of such jams.

(2) Each winter before the surface ice becomes firmly established or where velocities are too high to allow ice cover formation, a certain amount of anchor or frazil ice forms, especially in places of high water velocity such as in the St. Marys Rapids. This type of ice contributes to ice jams in the river by reducing the river cross section and frequently causes difficulty at the hydroelectric power plants by clogging penstocks and scroll cases. Since ice booms have little or no effect on frazil ice, it is difficult to control at power plants and consequently restrictions in power output during this period are not uncommon.

(3) During the spring breakup, the upper river becomes covered with broken drift ice. This type of ice has not caused ice jams but does make operation of the locks and navigation on the river extremely difficult. Ice booms installed across the power canal entrances, have been successful in preventing drift ice from entering the power plants.

Attempts to extend the navigation season have caused problems with the Sugar Island ferry. Normally this ferry operates all winter in a narrow open water slot with fixed ice bridges above and below. When the navigation season extends into the ice formation period, passage of ships breaks up the ice bridge that has been formed, and, carried by the current, the broken ice fills up the open-water area, stopping ferry operation. Attempts to cope with these problems were made during the 1971-72 extended navigation season experiment with the installation of a high velocity bubbler system in the mainland slip, where most of the difficulty is encountered. This continued in subsequent winters.

2.1.7 Current Plan of Regulation

Subsequent to the completion in August 1921 of the compensating works in the St. Marys River at Sault Ste. Marie, the outflows from Lake Superior have been controlled. The regulation of the lake is in accordance with the Orders of Approval of the International Joint Commission issued May 26-27, 1914, in response to application for authorization of diversions of water around the rapids for the production of hydroelectric power. The Orders provide that the works be so operated as to maintain the lake levels within a specified range and in such a manner as not to interfere with navigation. Further, they provide safeguards against extremely high and low levels on Lake Superior, and high levels on the St. Marys River. The operation of the river control works and the determinations of the outflow are under the direct supervision of the International Lake Superior Board of Control established by the International Joint Commission in accordance with the terms of its Orders.

The present plan of regulation on Lake Superior is the 1955 Modified Rule of 1949 and has been in force since December of 1955. The "rule curve," shown on Figure G-7, was designed to maintain the levels of Lake Superior between elevations 600.5 feet and 602.0 feet (IGLD). The "rule curve" flow is determined on the first of every month from the mean Lake Superior stage of the previous month. Between December 1 and April 30, changes to the plan outflow are made only when successive monthly mean stages of the lake move from an intermediate range, as defined by a range of levels for each winter

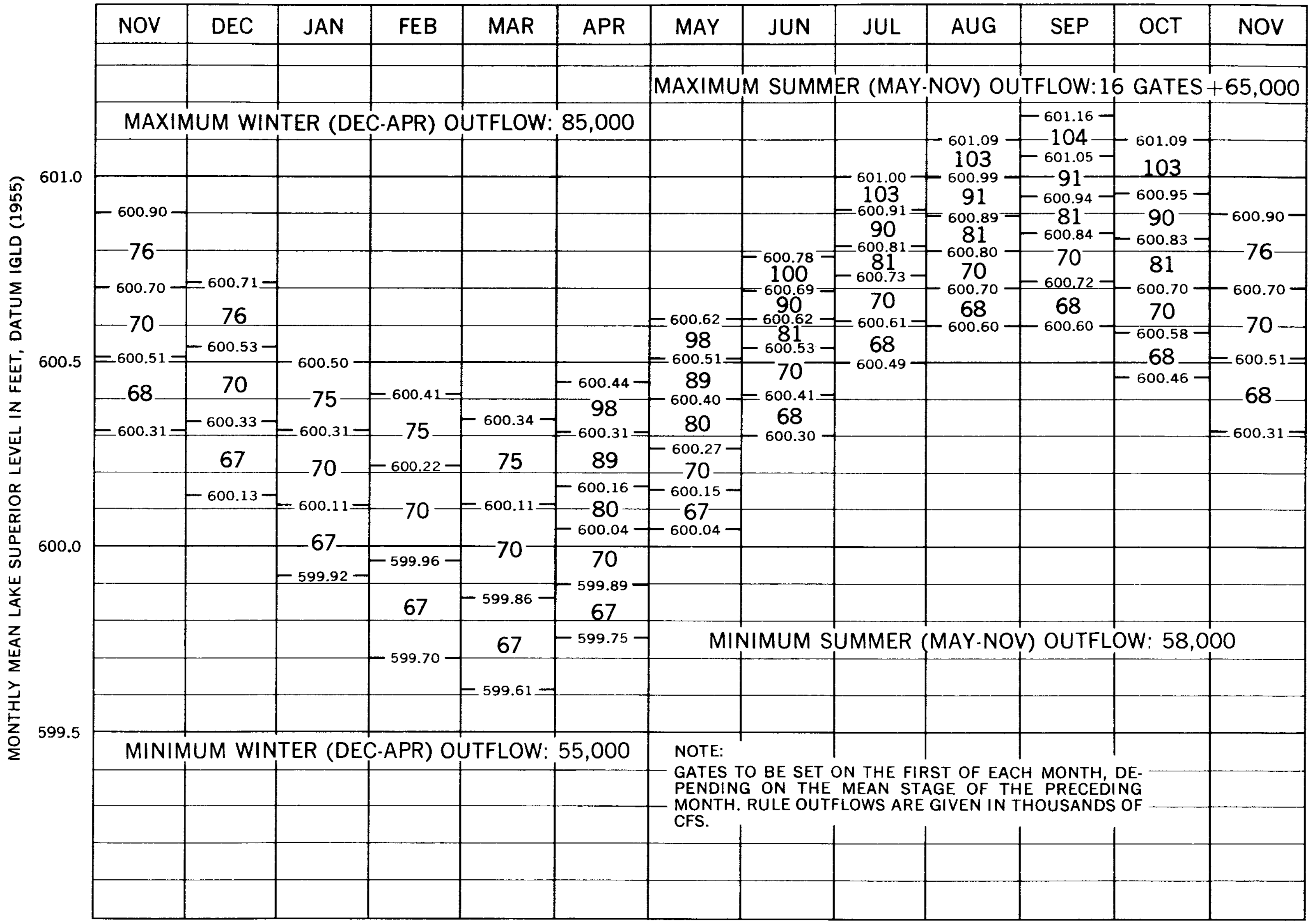


Figure G-7
LAKE SUPERIOR REGULATION PLAN—1955 MODIFIED RULE OF 1949

month, to a maximum or minimum condition or when successive monthly mean stages move from a maximum or minimum condition to the intermediate range. During the winter period, the present plan specifies a maximum discharge of 85,000 cfs and minimum discharge of 55,000 cfs. During the summer period, May 1 through November 30, the maximum outflow is that corresponding to 16 gates fully open, plus about 65,000 cfs through the power and navigation facilities. The minimum outflow during the summer period is 58,000 cfs.

The compensating works were built without specific provision for operating the gates when icing of the mechanisms occurred. Consequently, there did did not exist an efficient means of moving the gates during the winter period even though the existing regulation plan calls for such action as specified above.

2.2 Assumptions

A number of preliminary regulation plans were developed which utilized an increased outflow capacity through the St. Marys River. Preliminary analysis of these regulation plans disclosed that no additional benefits would accrue by increasing the discharge capacity of the St. Marys River. Accordingly, no major capital improvements would be required for the St. Marys River. However, it has been determined that additional benefits do accrue to a regulation plan which employs flow changes during the winter months as a normal procedure.

In determining the requirements for a regulation plan for Lake Superior requiring no major capital improvements, the following assumptions and limitations were used:

(1) A maximum discharge capacity, corresponding to that with all 16 gates of the compensating works fully open, plus 65,000 cfs through the power and navigation facilities, will be available during the life of the project.

(2) Power and navigation flows will continue at the present rate.

(3) The remaining life of the control structure, with proper maintenance, would be about 50 years.

(4) The existing range of water level profiles must be maintained in the river system.

(5) The gates could be operated throughout the winter period.

2.3 Methodology

As indicated in Section 2.2 above, a study of various preliminary Lake Superior regulation plans revealed that it was not feasible to enlarge the channel capacity or alter the existing structure to achieve a higher channel carrying capacity; however, winter operation of the gates was feasible. The following sections discuss: the methodology of regulation plans; the experience gained during the winter gate test programs; and, the design and cost estimates of various alternate gate heating methods which were considered for winter operation of the Lake Superior compensating works.

2.3.1 Plans of Regulation (Superior-Ontario)

Since Lakes Superior and Ontario are already regulated under separate Orders of Approval issued by the IJC, the International Great Lakes Levels Board studied plans for the coordinated regulation of Lakes Superior and Ontario which would provide additional benefits to the system without significant economic loss to any interest on the Great Lakes, their connecting channels and the St. Lawrence River and, at the same time, without involving major capital costs for reconstruction of the existing regulatory works or outlet channel capacities. For this condition, the existing facilities for regulating the outflow from Lake Superior and its discharge channels through the St. Marys River are considered to be adequate. The maximum outflow during the open-water period (May-Nov. incl.) is limited to the discharge capacity of the 16 control gates of the Lake Superior Regulatory Structure, plus a flow of 65,000 cfs through the power and navigation facilities. The maximum monthly outflow during the winter period (December-April incl.) should not be greater than 85,000 cfs to minimize the possibility of flooding caused by ice conditions downstream of the St. Marys River Rapids. It was further assumed that the minimum outflow for all months would be limited to 55,000 cfs. In addition to the above limitations, the change from month to month in the outflow would be limited to a maximum of 30,000 cfs. Although the maximum and minimum flows may not exceed the existing limitations, the pattern of releases may change and as a result could adversely affect the downstream profiles. Therefore, monthly computations of the water surface elevation at the U. S. Slip gauge, representing the upstream extremity of the lower St. Marys River, were computed using the regulated Lake Superior outflows and resulting elevations of Lakes Michigan-Huron. These elevations were computed using the existing open-water stage-fall discharge relationship between the U. S. Slip and Mackinaw City gauges (IGLD, 1955), stated as follows:

$$Q = 1659\ (\text{U.S. Slip} - 567.19)^{1.5}\ (\text{U.S. Slip} - \text{Mackinaw})^{0.4}$$

During the winter months, the regulated flow was increased by 3,000 cfs to account for ice retardation. A graphic representation of the above equation is shown on Figure G-8. The computed U. S. Slip gauge elevation was then compared to basis-of-comparison levels as well as to the maximum allowable elevation below the locks of 582.9 feet as specified in the IJC Orders of Approval.

2.3.2 Winter Flow Tests

During the study it became evident that it was possible to realize economic benefits to the Great Lakes as a system if greater flexibility could be achieved in the regulation of Lake Superior using the existing control facilities. Currently, inflexibility occurs during the five winter months when the outflow remains fixed except for rare instances when a change in gate setting is required to or from intermediate to maximum or minimum outflow. The present maximum winter outflow permitted is a discharge of 85,000 cfs in the St. Marys River. The existing policy for winter setting of the gates is due to the difficulties of moving them when they are frozen in ice; the flow limitation was arbitrarily set at what was considered to be a "safe" maximum as a result of past experience with ice jams at higher flows. The questions posed, therefore, were:

(1) Is this "safe" maximum too conservative? Can it be increased?

(2) If the St. Marys River can carry a higher flow during winter (or during part of it), when, and to what limit?

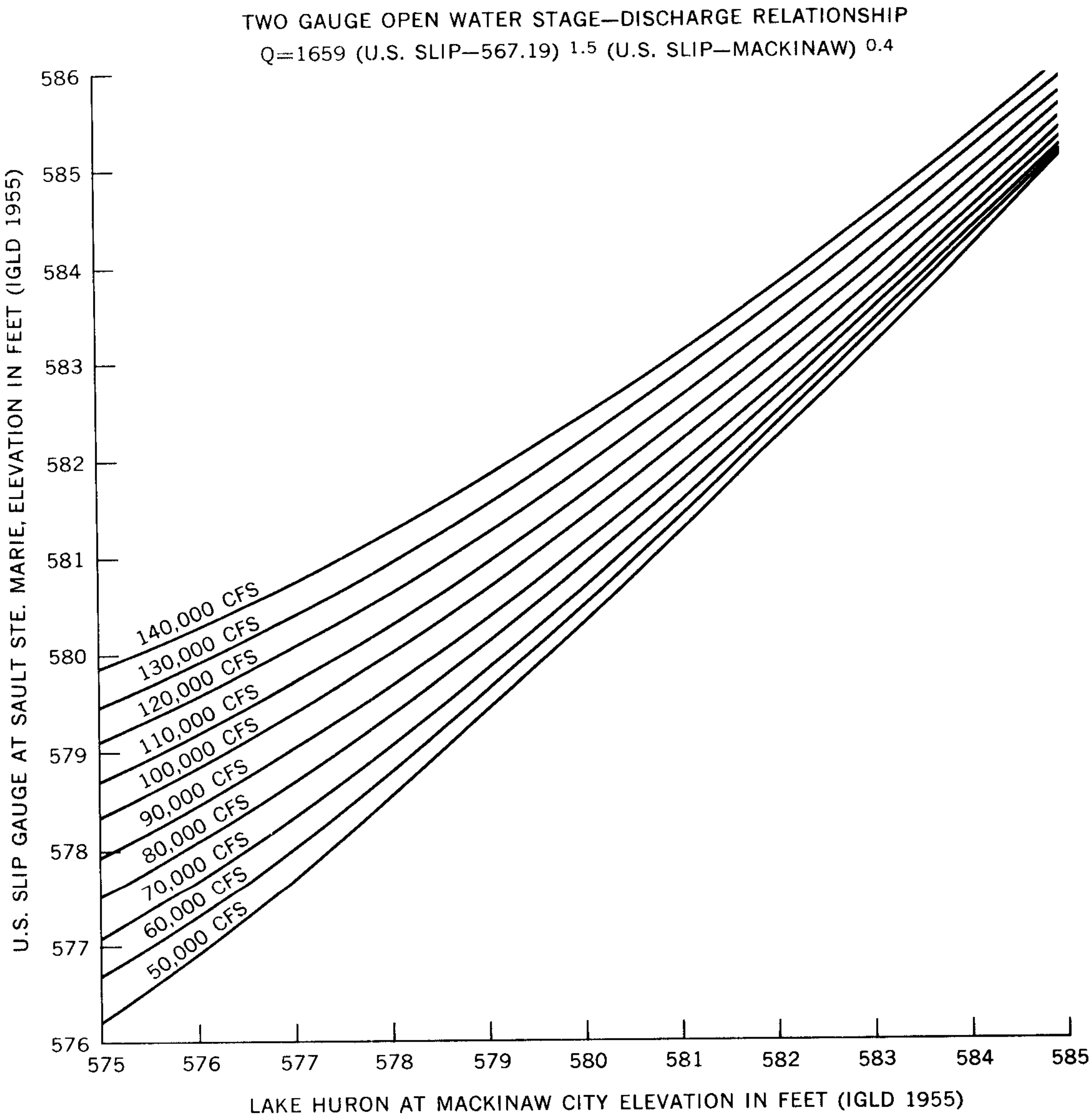


Figure G-8
LOWER ST. MARYS RIVER—TWO GAUGE OPEN WATER STAGE DISCHARGE RELATIONSHIP

(3) Is it practicable to change the gate settings and vary the flow as a normal procedure during winter? If so, by what means and how much would it cost?

To answer these questions, a test program was carried out during the four winters of 1968-69 through 1971-72. The three key features of the program were: the installation of steam heating equipment for de-icing the gates of the compensating works; the monitoring of the ice and hydraulic conditions in the river throughout the winter; and, a waterlevel monitoring system and emergency procedures for quickly closing gates in the event that an ice jam developed. The experiments varied from year to year depending on the prevailing weather and hydraulic conditions, navigation activity in the river and the regulation requirements of the International Lake Superior Board of Control, with whose approval the tests were carried out. The following is a summary:

Site Preparations: Preparatory work for the 1968-69 test program consisted of, among other things: the installation of a gravelled access road between the Canadian lock and the north end of the control structure; the erection of a 24' by 40' prefabricated building to house the steam heating equipment and to provide crew facilities; the erection of a timber stairway at the north end of the control structure to provide safe access to the machinery deck; a thorough overhaul of all gate hoisting equipment and ancillary machinery; and, the installation of steam heating facilities. The steam heating equipment consisted of two oil-fired steam boilers, watertank, pumps and steam lines. An insulated steam line was installed along the downstream side of the machinery deck, between piers 1 and 15, and connected to the control building. At each pier, a header connection was made to the main line, each header having twin valves and boss fittings to which 50-foot flexible steam hoses were attached as the need arose. Machinery-deck lighting was installed and a 25-KVA diesel-powered generator was procured to provide electric power to the site. In the first two years of tests, a pontoon bridge was constructed upstream of the Canadian lock so that heavy equipment could be transported to the site. However, this was eliminated in subsequent years by transporting all equipment by barge prior to freeze-up.

In subsequent years, site preparation consisted primarily of recommissioning of the temporary equipment installed. In addition, minor maintenance was carried out each year on the gate hoisting and gate heating equipment to ensure their satisfactory performance during the test programs.

Hydraulic Monitoring: Existing permanent water level gauges in the Lower St. Marys River were augmented with additional installations at strategic locations so as to permit quick identification of water level anomalies in critical reaches due to ice conditions. The locations of water level gauges in the St. Marys River are shown on Figure G-2. The Little Rapids reach, which discharges about 70 per cent of the flow of the Lower St. Marys River, was selected as the critical reach due to its small cross-sectional area and previous history of ice jamming. Open-water gauge relationships were developed between the various gauges so that any departure from the open-water condition could be identified. Criteria were established to determine critical elevations above and below the Little Rapids reach. An "alert" condition, whereby key personnel would be notified of an impending ice jamming situation, was established. The "alert" condition could develop from:

(1) Jam above Frechette gauge; if the gauge reads 1.1 feet below its normal open water reading

(2) Jam below Frechette gauge; if the gauge reads 0.9 foot above its normal open water reading

During the alert condition, all gauges were carefully observed. If the U. S. Slip gauge indicated a rate of rise of 0.10 foot/hour or more or approached elevation 582.4 feet IGLD, the "action" condition was established whereby gates would be closed so as to provide a water level elevation no greater than 582.9.

An attempt was made to have the Frechette gauge sound an alarm when the "alert" condition arose; however, this proved unsatisfactory. Each year the method of calculating open water levels at the various gauges was improved due to the additional hydraulic data collected. The gauge installations were also improved. At present all gauges are automatic, providing a continuous record and may be interrogated by telephone whenever required.

In addition to water level data, supplementary data were collected during the winter period in order to properly analyze the ice and hydraulic regime of the river. Sets of aerial photographs were obtained at several key times each year to provide data on the extent and nature of the ice cover. This was supplemented by ice measurements taken at several stations along the river detailing the thickness and nature of the cover. Local climatological data, taken at the airports of Sault Ste. Marie, Michigan and Ontario, were also used. Other data included movie film of the operations and still photographs of the control structure.

Test Program: The test program carried out each year with minor variations, consisted essentially of three phases: de-icing test; a test involving an increase in flow; and a simulated emergency test. The main objectives of the de-icing test were: (1) to check the capacity and efficiency of the steam heating equipment; (2) to familiarize the crew with the equipment and to train them to work under winter conditions; (3) to develop work methods and site procedures; (4) to evaluate the adequacy of safety gear and ice-cutting and chiseling equipment; (5) to ensure adequate operation of the gate hoisting equipment; and, (6) if weather conditions were severe enough, to determine the time to open and/or close the gates.

An increase in flow test consisted essentially of opening sufficient gates to flow 95,000 cfs, careful monitoring of water levels and, if weather conditions were severe enough, to determine the time required to open gates under such conditions.

The simulated emergency test was designed to check out the effectiveness of the planned procedures which would have to be followed in the event of an ice jam situation occurring. The ability to close gates quickly would be of little value in itself if an inadequate warning system, together with the inability to muster the crew quickly enough, resulted in no action being taken until it was too late to avert flooding. Accordingly, planning was not revealed beforehand in an attempt to simulate an actual emergency as close as possible. Photographs of the compensating works under winter conditions are shown on Figure G-9.

The 1968-69 test program consisted of a de-icing test on January 7, a simulated emergency test at 2:20 a.m. on January 24, at a flow of 85,000 cfs, and an increased flow test, during which the flow was increased to 95,000 cfs, on February 11. A flow of 95,000 cfs was maintained until April 15 at which time river conditions permitted the opening of all 16 gates to decrease high lake levels. The weather conditions were about normal but certainly not as rigorous as could be expected in some years. It had taken only 45 minutes to muster the crew for the simulated emergency test and a further 65 minutes to raise a full head of steam. The total elapsed time for the operation, which consisted of mustering the crew and closing 2 gates, was 2 hours and 43 minutes.

The 1969-70 test program consisted of a de-icing test on January 15, an increased flow test on February 9-11, during which the flow was increased to 95,000 cfs, and the termination of the increased flow test by a simulated emergency test on March 5. Weather conditions during the winter period were slightly more severe than the previous winter. The extended navigation season, which terminated on January 14, 1970, may have had some effect on the ice and hydraulic regime of the river but it could not be quantified. The simulated emergency test was carried out on March 5 under relatively mild conditions but in the aftermath of a severe freezing rain storm. The overall elapsed time from the sounding of the alarm to the complete closure of 7-1/2 gates was 7 hours - 18 minutes.

The 1970-71 test program consisted of preliminary tests carried out between December 16-18 having the same objectives as the de-icing tests but under less severe conditions. The flow was increased to 95,000 cfs on December 18 and due to ice jamming conditions, resulted in emergency gate closing operations on January 28. Weather conditions were very severe with high snowfall and subzero temperatures pervailing between mid-January and mid-February. The navigation season was extended to January 30, 1971, and had a definite effect on the ice and hydraulic regime of the river, although this could not be quantified. The emergency gate closing operations were carried out on January 28 with temperatures ranging from -10° to 10°F, and with winds gusting to 50 m.p.h. resulting in snow squalls which at times reduced visibility to nearly zero. Not only were levels reaching the critical elevation of 582.9 IGLD below the locks, but, an anomaly of 0.20 foot existed between the U. S. Powerhouse Tailrace and U. S. Slip Gauges, located approximately 2,000 feet apart, which was thought to have been caused by an ice jam between the gauges. There was apprehension that this ice mass might let go at any time and plug the Little Rapids reach. The total elapsed time from when the closure orders were given by the International Lake Superior Board of Control to the final closure of three gates was 11 hours - 7 minutes. The water levels eventually stabilized and, on March 9, one gate was opened to partially restore required outflow.

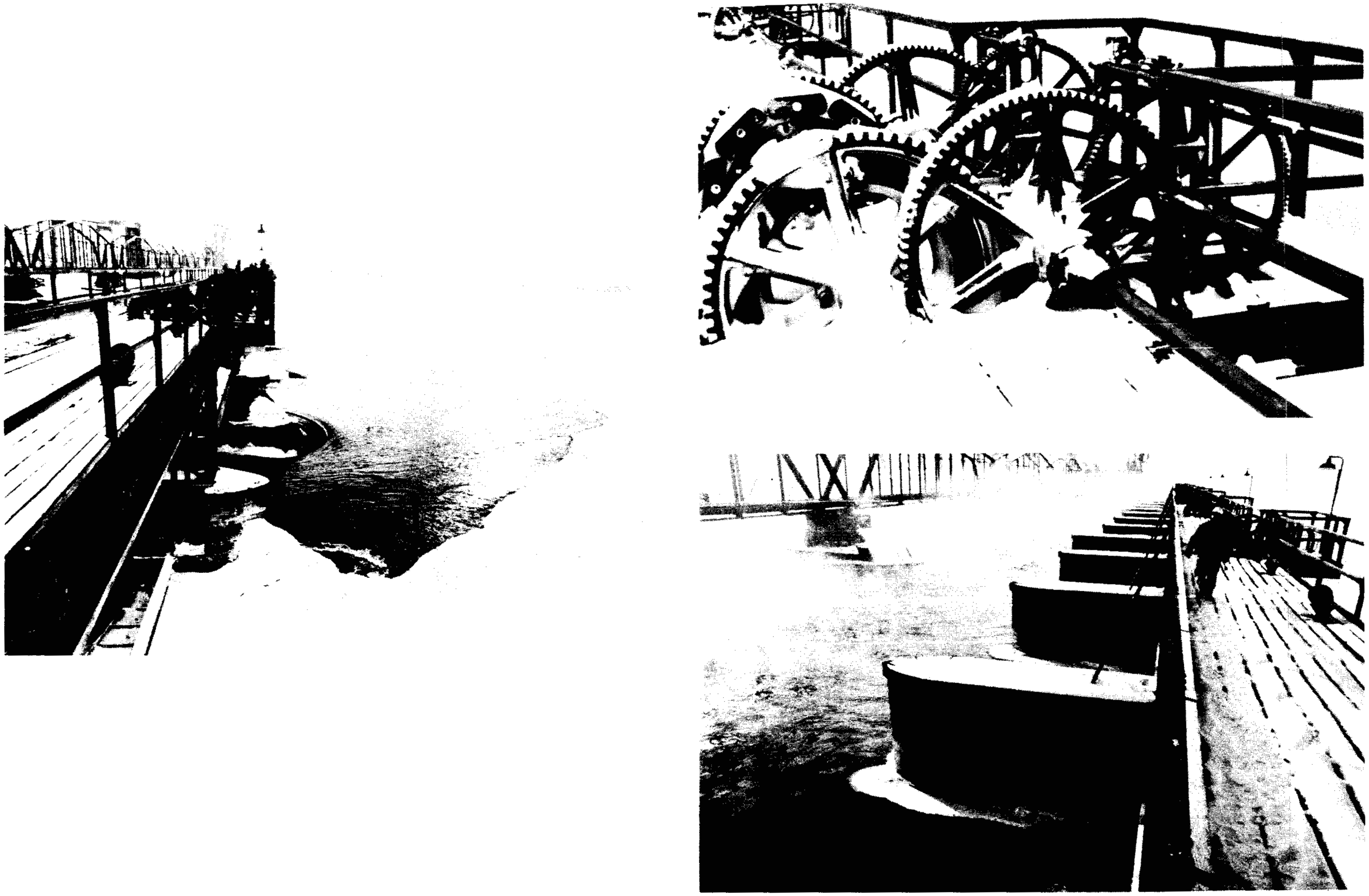


Figure G-9
Photographs of Compensating Works at Sault Ste. Marie Under Winter Conditions.

Due to the extended navigation season, the Board planned to conduct the 1971-72 winter gate test program after the termination of navigation on the river by increasing the flow from the "rule curve" flow of 85,000 cfs to 95,000 cfs. However, under a flow of 81,200 cfs, the level at the U. S. Slip gauge reached elevation 582.7 IGLD and together with an anomaly, as in the previous year, between the U. S. Slip and U. S. Powerhouse Tailrace gauges, the International Lake Superior Board of Control ordered an emergency closure of 2 gates on January 26. The weather conditions were very severe with a temperature of -7°F and winds gusting from 14-30 m.p.h. To make matters worse, approximately 10 feet of snow plugged the access road to the control structure while the machine deck and work area was covered with 2 to 3 feet of snow and ice. The total elapsed time to move the crew to the control structure and close 2 gates was 7 hours - 20 minutes. Although this emergency gate closing operation was not a part of the program, it did provide additional experience. The cause of the anomaly between the U. S. Slip and U. S. Powerhouse Tailrace gauges was determined and is discussed below. Water levels in the lower river did not permit an increase in the flow of 95,000 cfs during any part of the remaining winter period. An aerial photograph, taken on January 31, 1972, Figure G-10, of the Bayfield channel, looking downstream from the U. S. navigation locks illustrates the ice conditions of the river in the reach between the locks and Sugar Island.

Anomaly between U. S. Slip and U. S. Powerhouse Tailrace Gauges: During the 1970-71 and 1971-72 winter periods, an anomaly existed in water surface elevation between the U. S. Slip and U. S. Powerhouse Tailrace gauges, located some 2,000 feet apart. A review of water levels as recorded at these gauges, over the period of record, revealed that no such difference had occurred in the past. During the 1970-71 winter period, the anomaly reached a maximum of 0.80 foot while during the 1971-72 winter period, a maximum difference of 0.60 foot was recorded. A plot of the water surface elevation, as recorded at these gauges during the 1971-72 winter period, is shown on Figure G-11.

Surveys conducted by the U. S. Army Corps of Engineers and the St. Lawrence Seaway Authority between February 10, 1972 and March 22, 1972 revealed that a large mass of compacted ice fragments, with thickness up to more than 25 feet had formed between a point located some 800 feet downstream of the East Centre Pier of the U. S. Canal and the open-water area at the foot of the Rapids. A map of the area showing the location of survey points and a tabulation of the thickness and composition of ice at these survey points is shown on Figure G-12.

A definite explanation of the cause of this mass of slush ice has not yet been explained. However, due to the fact that this ice buildup had not been experienced prior to the "Extended Navigation Season Program," it is postulated that winter navigation was a contributing factor.

Cost Estimates: The cost estimates contained herein are based on the experience gained during the winter gate test program, as summarized above, utilizing steam heating and ancillary equipment. The cost estimates, shown on Table G-2, may be broken into three major categories: capital expenditure, annual maintenance and annual operations.

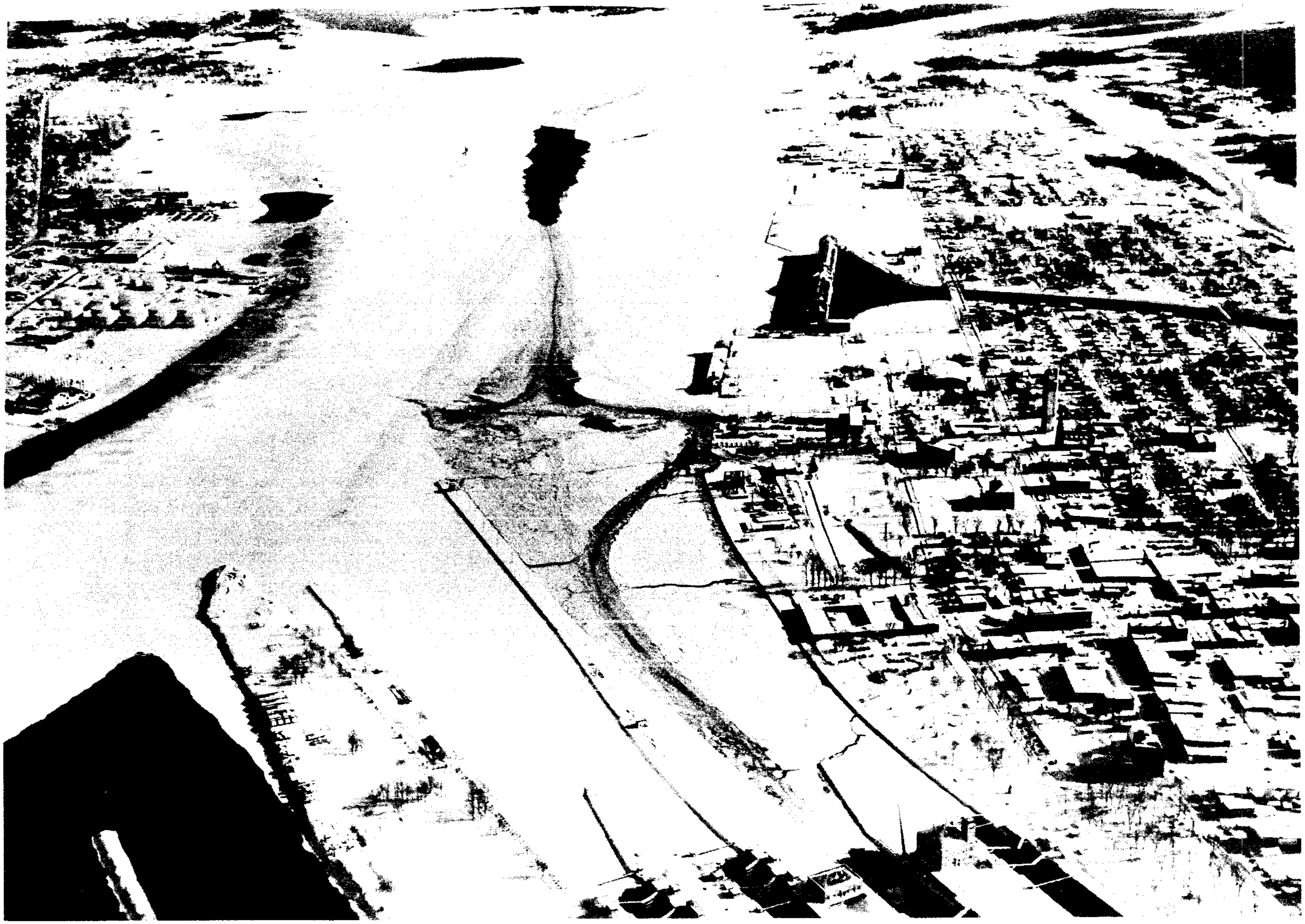


Figure G-10
Aerial Photograph of The Bayfield Channel Below Navigation Locks at Sault Ste. Marie.

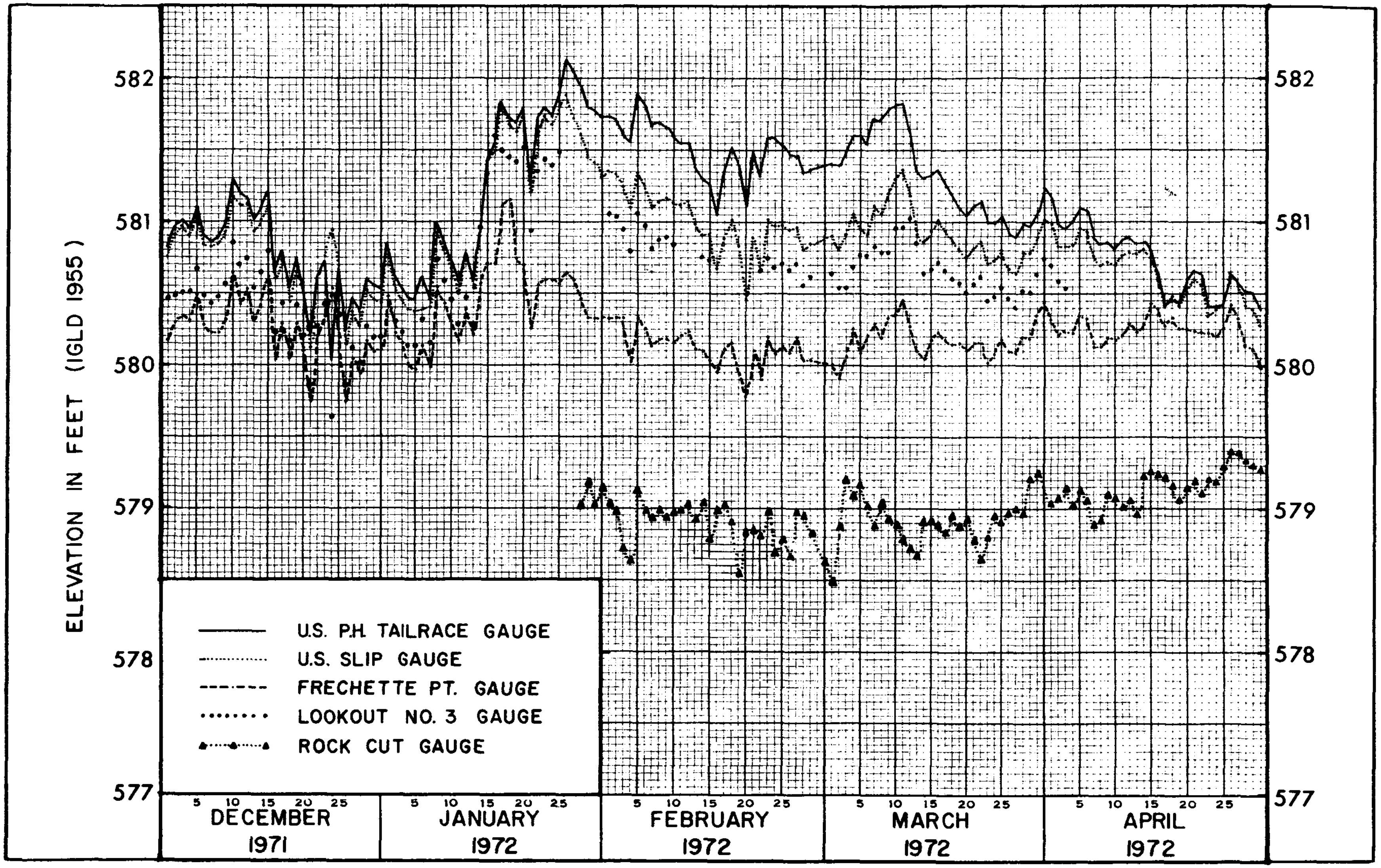
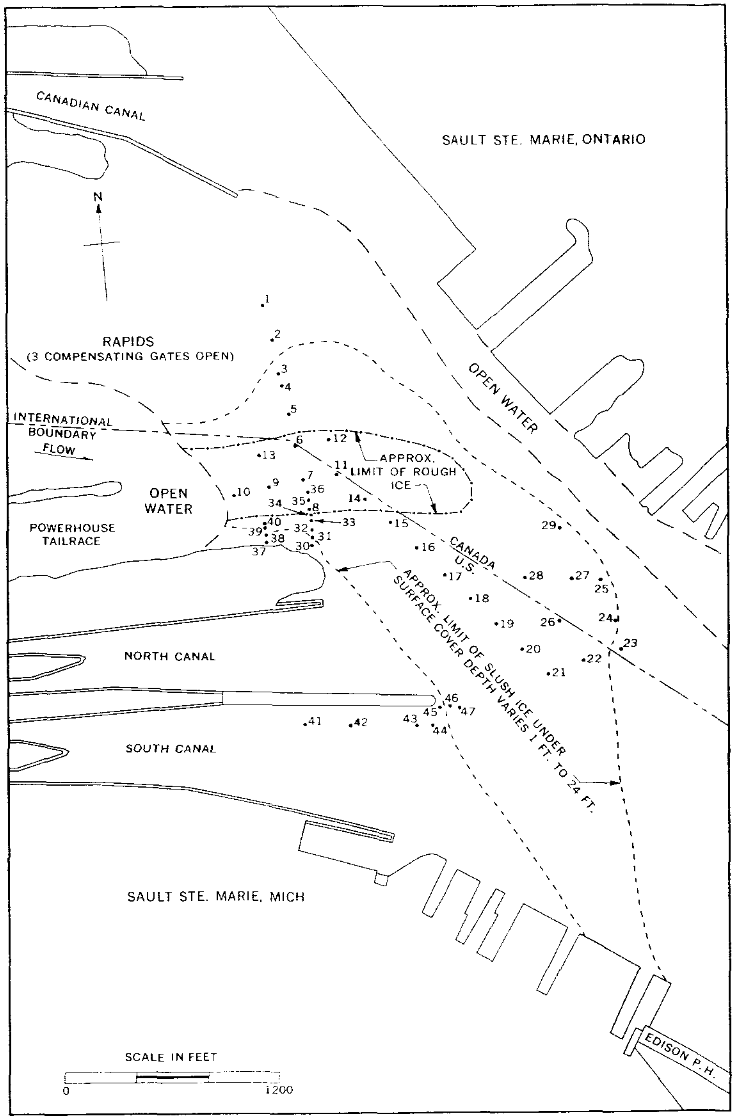


Figure G-11
LOWER ST. MARYS RIVER—HYDROGRAPHS OF DAILY MEAN WATER LEVELS DURING 1971/1972 WINTER PERIOD



ICE CONDITIONS
UPPER BAYFIELD CHANNEL AND BELOW RAPIDS
AT SAULT STE. MARIE

HOLE NO	SNOW ON ICE	SLUSH ON ICE	HARD ICE	SOUNDING	STIFF SLUSH UNDER ICE	REMARKS
1	10"	4"	16"	20'	NIL	MEASUREMENTS TAKEN MARCH 13, 1972
2	10"	10"	14"	21'	NIL	
3	8"	10"	14"	25'	13'	
4	10"	12"	15"	26'	18"	
5	10"	6"	16"	21'	10'-6"	
6	11"	NIL	NIL	25+	24'+	1" CRUST SNOW
7	12"	NIL	NIL	25+	24'+	1½" CRUST SNOW
8	17"	13"	½"	25+	12'	
9	16"	2"	14"	18"	10'	
10	17"	3"	10"	25'+	11'	
11	12"	3"	10"	25'+	24'+	
12	12"	4"	20"	25'+	24'+	
13	14"	5"	11"	25'+	14'	
14	17"	0	13"	25'+	9'-8"	MEASUREMENTS TAKEN MARCH 16, 1972
15	5"	0	17"	25'+	18'	
16	17"	0	17"	25'+	10'-10"	
17	10"	0	19"	25'+	23'-6"	
18	12"	0	16"	25'+	19'	
19	13"	0	14"	25'+	20'	
20	17"	0	16"	25'+	18'	
21	13"	0	17"	25'+	18'-6"	
22	14"	3"	16"	25'+	2'	
23	18"	0	15"	25'+	NIL	
24	12"	0	12"	25'+	1'	
25	14"	0	17"	25'+	6'-6"	
26	12"	0	17"	25'+	13'	
27	16"	11"	10"	25'+	7'-6"	
28	12"	0	10"	25'+	18'	SURVEY EQUIPMENT
29	8"	0	12"	25'+	4'	LIMITED TO 25 FT.
30		0	2½"	9'	0	MEASN'T TAKEN MARCH 8, 1972
31		0	18"	21'	2'	FAST CURRENT
32	13"	0	11"	28.2'	0	FAST CURRENT
33	12"	0	22"	26.9'	9'	FAST CURRENT
34	16"	0	17"	31	7'	FAST CURRENT
35	18"	0	9½"	27.6'	7'-6"	FAST CURRENT
36	12"	0	14"	25'	11'	FAST CURRENT
37		0	8"		0	*USING 25 OZ. SNDG. WT. FASTENED TO 50' BOX TAPE. LEAD PROGRESSES THRU SLUSH SLOWLY WITH DROPS OF APPROX. 0.5 FT. (FEB. 10)
38		0	8"		0	
39		0	7"		5'	
40		0	1"		5'	
41		0	22"		0	AVERAGE 3 TO 4 DROPS TO PENETRATE ONE FOOT OF SLUSH (MEASN'T TAKEN MARCH 22, 1972)
42		0	32"		0	
43		0	32"		0	
44		0	22"		0	
45		0	29"		0	TOP OF ICE 0.2' ABOVE W.S.
46		0	20"		15'-11"	TOP OF ICE 0.6' ABOVE W.S.
47		0	16"		22'-10"	TOP OF ICE 0.8' ABOVE W.S.

Figure G-12
ICE SURVEY OF THE ST. MARYS RIVER BELOW THE RAPIDS AT SAULT STE. MARIE TAKEN DURING 1971/1972 WINTER PERIOD

TABLE G-2

Average Annual Costs of Winter Operations of the Control Structure at Sault Ste. Marie Utilizing Steam Heating Facilities

1.	CAPITAL EXPENDITURE		
(a)	Average annual costs of service building and major equipment with an initial cost of $30,800 and a useful life of 20 years, based on a 7% interest rate.*	$ 2,900	
	Average annual cost of minor equipment with an initial cost of $2,400 and a useful life of five years, based on a 7% interest rate.	800	
			$ 3,700
2.	ANNUAL MAINTENANCE		
(a)	Maintenance of service building and major equipment.	500	
(b)	Maintenance of Minor equipment.	200	
(c)	Routine maintenance of the control structure, heating of building, and other overhead necessary to ensure winter serviceability.	1,000	
(d)	Snow removal and site access.	300	
			2,000
3.	ANNUAL OPERATIONS		
(a)	Average cost of gate operations.	1,300	
(b)	The cost of hydraulic monitoring of the river and emergency stand-by procedures.	17,000	
			18,300
		TOTAL:	$24,000**

*Assuming straight line depreciation and full salvage value on the unexpired portion of the capital asset.

**Expressed in 1971 price levels.

The installations required for steam-heating the gates consisted essentially of a service building, steam boilers, generator, water and oil tanks and other auxiliary equipment, together with a heated room for personnel; insulated steam lines extending along the structure; and other facilities, such as adequate lighting of the machinery deck. Capital expenditure was estimated on the basis of the cost of the service building and major items of equipment having a minimum expected useful life of 20 years (assuming that normal maintenance procedures are followed) and other minor equipment having a useful life of 5 years.

Annual maintenance costs cover the upkeep of the above described installations, heating of the service building, snow removal and site access, and other overhead costs necessary to ensure winter serviceability of the works. These expenses would be required annually whether or not gate movements are carried out in any particular winter.

The annual operating costs comprise labor, materials and fuel expenses, with the major item being the payroll. Based on experience to date, unit costs per gate movement (opening or closing, under routine or emergency conditions) have been developed. Analysis of a typical regulation plan (SO-901) shows that, applied to the period of record, there would have been 14 years out of the 68 years of the study period when no winter gate movements would have been required. For the remaining years, the estimated operating costs, at current prices, would have ranged from $450 to $3,650 depending on the number of gate movements required. The simple average cost for all 68 winters would have been $1,300.

Due to the effects that changing the winter flows can have on the stability of downstream ice conditions, it would be necessary to provide close surveillance of the river throughout the winter. This included ground observations, aerial reconnaissance and photography, and the installation and operation of strategically located water level gauges to detect water profile changes that could signify the onset of ice jamming conditions. Coupled with this was an emergency warning and communications system, and at certain critical periods, personnel standby arrangements, so that immediate gate closing action could be taken to alleviate the effects of any incipient ice jam. These hydraulic monitoring and emergency standby procedures constitute a major part of the cost of winter operations.

Conclusions: Four winter seasons of field tests were insufficient to provide conclusive answers to the basic questions posed. It would require many seasons, under a wide variety of hydrologic and meteorological conditions, to investigate adequately the capability of the river to safely handle a range of higher flows, at different lake stages, under various ice conditions, throughout or during specific periods of winter; there are too many combinations of these parameters which would have to be tested. Furthermore, lengthening the navigation season is a complicating factor, since this involves ice-breaking activity and thus disturbance of the natural ice cover in the river.

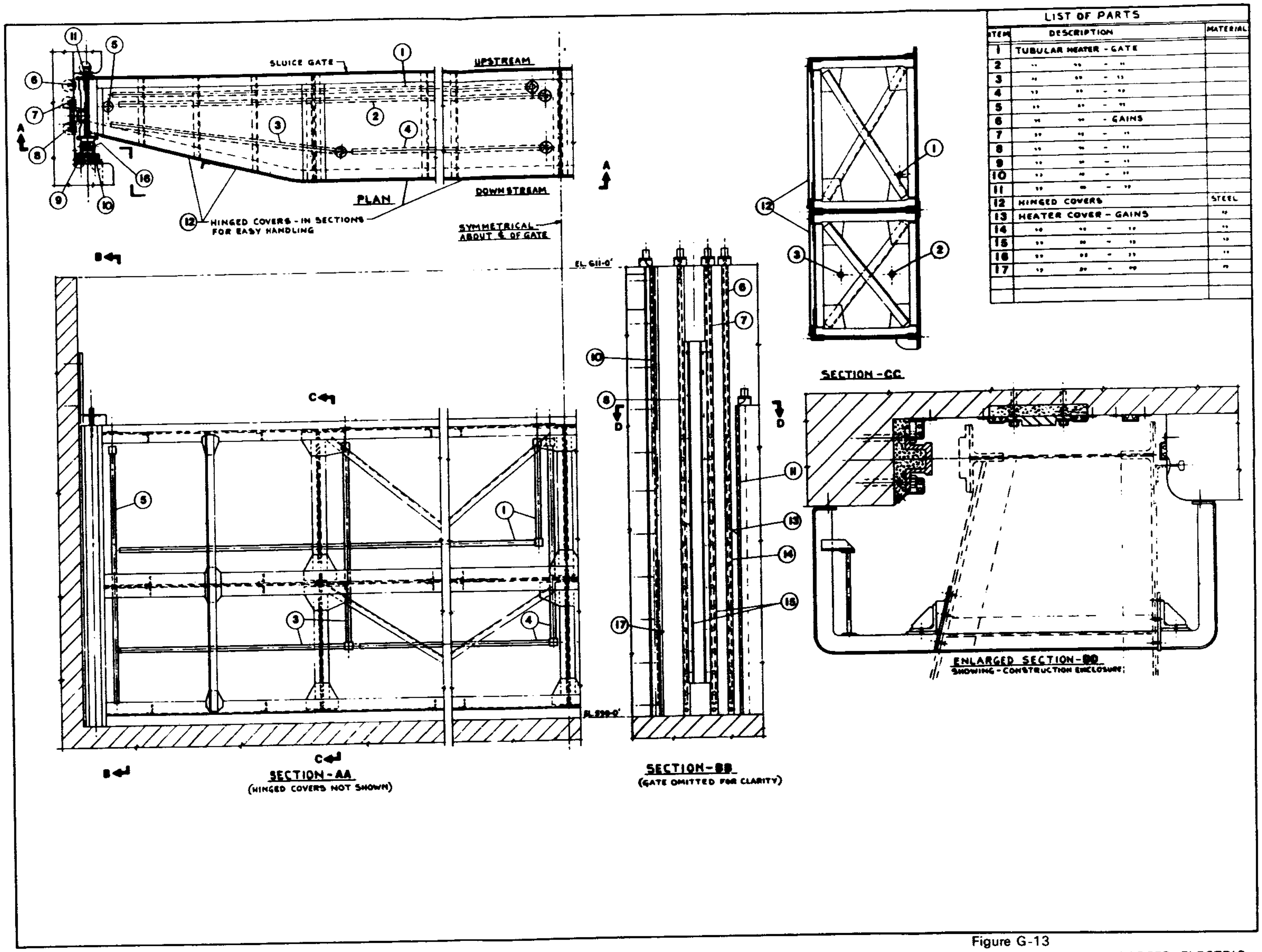
Nevertheless, under conditions which prevailed in these four particular years, it was found that it is definitely possible to change gate settings during the winter, even under quite severe conditions, and that the costs of such operations are reasonable. De-icing and closing gates was easier and quicker than de-icing and opening them. Flows of 95,000 cfs are generally feasible, although it appears desirable not to exceed 85,000 cfs until after stable ice cover conditions have been established. This latter proviso may well be the key to the problem and, if so, then even higher flows may be possible on this basis. Even if higher flows did produce ice jamming, the dangers of resulting flooding could be averted by promptly closing the compensating gates to reduce the flow. This calls for continuous monitoring of ice conditions and water levels in the river, particularly in certain critical reaches, to enable immediate identification of any developing ice jam and prompt action at the control structure. The test program demonstrated the practicability of the monitoring procedures used and their ability to give adequate lead time for responsive action in any emergency and is being continued.

2.3.3 Alternate Methods of Gate and Gain Heating

Although steam was chosen as the most expedient method for the purposes of the winter gate test program, it is only one of a number of alternate methods of heating the gates. These include electrical, air bubbles, radiant and hot air systems. The Board decided that a more permanent method should be examined in the interest of crew safety, working conditions and reliability. The following paragraphs summarize the various methods investigated.

Hot Air Heating for Gates: The hot air system as applied to the existing gates would consist of 2-stage thermostatically controlled blower heating units located in the top compartment of the gate. Ductwork would be provided to carry hot air to the lower compartment of each gate. The downstream side of each gate would be enclosed with hinged steel covers such that the hot air would be confined and circulated within the gate. The hot air system is frequently used for heating of control gates in the "dry" and has been found to be efficient, reliable, safe and economical. However, this system would not be readily adaptable to the compensating works at Sault Ste. Marie for the following reasons: (1) the middle girder of each gate would seriously hamper air circulation, (2) since it is more efficient to melt ice between a metal-ice interface than an ice-water interface, the hot air system would be inefficient in melting ice in the lower compartment, and, (3) since the gains, which cannot be enclosed, would have to be heated, a more efficient system would have to be employed for this purpose.

Electric Tubular Heaters for Gate and Gain Heating: This system is comprised of 2-stage thermostatically controlled heating elements, encased in steel tubes, located at strategic points in the gains and within the gates as shown on Figure G-13. The electric tubular heating system is normally used for "sluice" gates, such as those of the compensating works, for operation under winter conditions and, furthermore, it is considered the only acceptable method of heating the gains. Gain heaters would be of the hairpin type and would be protected from damage, due to ice and debris, by structural members bolted to the pier faces. As with the hot air system, the electric tubular system provides a reliable, safe and economical method of heating the gates of the compensating works. Furthermore, it is readily adaptable



LIST OF PARTS		
ITEM	DESCRIPTION	MATERIAL
1	TUBULAR HEATER - GATE	
2	" " - "	
3	" " - "	
4	" " - "	
5	" " - "	
6	" " - GAINS	
7	" " - "	
8	" " - "	
9	" " - "	
10	" " - "	
11	" " - "	
12	HINGED COVERS	STEEL
13	HEATER COVER - GAINS	"
14	" " - "	"
15	" " - "	"
16	" " - "	"
17	" " - "	"

Figure G-13
LAKE SUPERIOR COMPENSATING WORKS—PROPOSED ELECTRIC TUBULAR HEATING ARRANGEMENT FOR GATE AND GAIN HEATING

to the compensating works and performs equally well in the "dry" and in the "wet".

Air Bubbler System: While the above systems are better suited to ice prevention and ice removal in the gate and in the gains, air-bubbler systems can be readily applied to ice prevention and removal along the face of the gates. The air bubbler system operates by releasing compressed air from nozzles located near the bottom of the river bed. The air from the submerged nozzles breaks into bubbles, mixes with the surrounding water, which is relatively warmer than the surface water, and rises to the surface thus melting ice already formed and/or preventing the formation of new ice. There are a number of gate installations using this system, primarily as a backup unit, but, in general, it has been found to be unsatisfactory. A number of problems ranging from the formation of ice within the piping due to air moisture condensation, the blocking of nozzles due to dust particles in the air to an inadequate differential in water temperature between the bottom and surface waters make this system unreliable. In addition, it would have to be backed up by heating systems to remove ice from the gate and gains.

Radiant Heating System: Gas fired or electrical radiant heaters would be mounted in protective housings above the water level on the upstream and downstream portions of the gates. This system operates by removing ice or preventing ice formation by direct radiation. Radiant heaters have an efficiency of only 50% under ideal conditions and, under windy conditions, the efficiency drops to 25%. This system has high operating costs and is unreliable under severe weather conditions. Furthermore, it cannot be applied to gain heating due to the restrictive space around the gains which seriously hampers air circulation.

Steam Heating System: This system is essentially a modification to the existing temporary facilities used for the winter gate test program augmented by electric tubular gain heaters. The modifications include: (1) horizontal steam lines incorporating nozzles, spaced some 2 to 3 feet apart, running along the upstream and downstream face of each gate and supported from each pier by chain hoists, (2) handrails located around the top girder of each gate, and (3) access to each gate from the bridge deck by means of a caged ladder which would be covered by a hinged plate when not in use. As with the existing system, this system may prove to be unsafe and unreliable under severe winter conditions.

Comparison of Alternate Gate Heating Methods: An analysis of the heating systems, as described above, was carried out assuming that 10 gates and 10 sets of gains would be required for winter operation. A summary of the cost estimates is shown in Table G-3.

Although gate heating by steam is least expensive, the safety hazards to the crew, the unreliability of the system under severe weather conditions and potential labour difficulties far outweigh the added cost of the other systems. For these reasons, gate and gain heating by electric tubular heaters was selected for further design and cost estimates. A more detailed discussion of the design and cost estimates is presented in Section 2.3.4.

TABLE G-3

LAKE SUPERIOR REGULATORY WORKS

SUMMARY OF COST ESTIMATES* FOR ALTERNATE

GATE HEATING METHODS (10 GATES AND 10 PAIRS OF GAINS)

METHOD	ANNUAL COST**
GATE HEATING BY HOT AIR HEATERS GAIN HEATING BY ELECTRIC TUBULAR HEATERS...........	$36,800
GATE AND GAIN HEATING BY ELECTRIC TUBULAR HEATERS....................................	37,600
GATE HEATING BY AIR BUBBLER SYSTEM GAIN HEATING BY ELECTRIC TUBULAR HEATERS...........	39,500
GATE HEATING BY RADIANT HEATERS GAIN HEATING BY ELECTRIC TUBULAR HEATERS...........	42,900
GATE HEATING BY STEAM (MODIFIED SYSTEM) GAIN HEATING BY ELECTRIC TUBULAR HEATERS...........	26,900

*Expressed in 1971 price levels

**Based on an interest rate of 7%

Ancillary Works: In addition to the above described works, a number of other works are required both to service the works and to provide additional safety and reliability in the operation of the gates under winter conditions. The following paragraphs describe these ancillary works in more detail.

1. Energy Supply to the Site. Four alternate methods were examined, namely; electrical power from the Canadian side, electrical power from the U. S. side, natural gas from the Canadian side and diesel generated power at the Compensating Works. Electrical power from the Canadian side offered the lowest capital and annual costs. The annual cost was estimated at $14,500 assuming a 200 KW demand. It should be pointed out that of the annual cost of $14,500, the annual cost of energy was $8,100 based on a Great Lakes Power Company's standard commercial power contract whereby the minimum monthly bill is not less than 75% of the highest monthly bill established over the life of the agreement. Significant savings could be achieved by successful negotiation of a contract whereby either different rate structures for winter and summer months could be established, or, a yearly contract to accommodate some years of lower winter power demand.

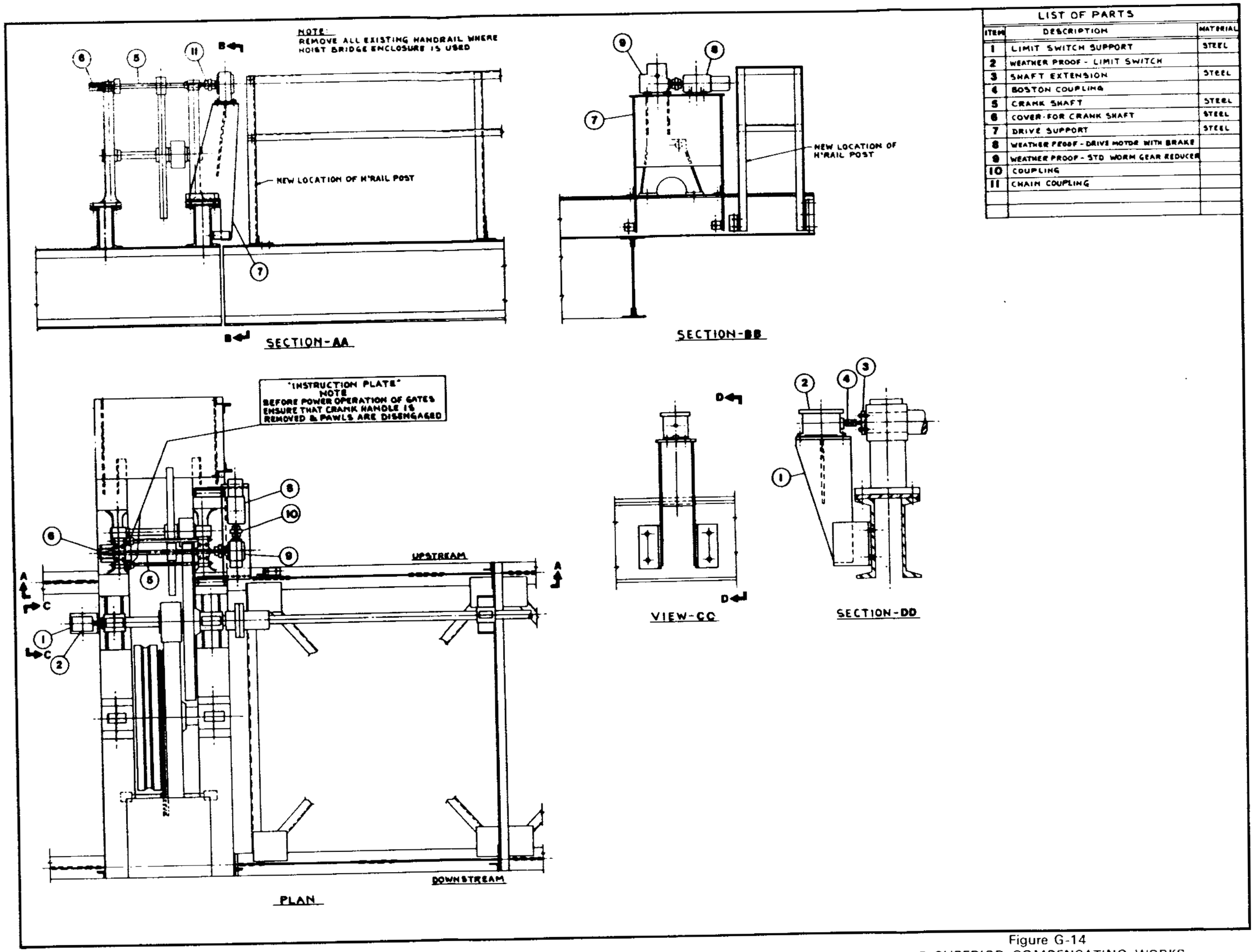
2. Telephone Service to the Site. The annual cost of telephone service to the site is estimated at $1,000 comprising $750 for capital cost of telephone line, $100 for annual maintenance and $150 for annual operations. If the electrical power line and telephone line were combined, the annual cost of telephone service could be reduced by $425.

3. Mechanization of Gate Hoist Machinery. In order to achieve operational efficiency and modern standards of personnel safety, it was considered essential that all 16 gates be mechanized and all exposed open gearing be enclosed. An electric motor drive was selected, incorporating magnetic reversing controllers and the necessary limit switches. Dial type gate opening indicators would be provided for operational convenience. The control gates could be moved manually as a backup to the system. Hinged sheet steel covers would be provided to cover exposed gearing. The proposed modifications are illustrated on Figure G-14. The estimated annual cost of these facilities is $8,100.

4. Enclosure over 10 Gates of Compensating Works. It was considered essential that an enclosure should be placed over at least those gates which would conceivably be utilized during the winter months if not over the entire structure. For purposes of cost estimating, it was assumed the middle 10 gates would be enclosed to provide a continuous weather-proof housing. Two alternate types of material were investigated, a metal-clad prepainted enclosure at an annual cost of $9,125 and an asbestos-clad enclosure at an estimated annual cost of $11,075. The metal-clad enclosure was selected in that it offered more durability at a lower cost. Convenient lighting would be provided inside the enclosure. The proposed enclosure is illustrated on Figure G-15. The low regular profile, the neutral colour (dawn grey), would provide an aesthetically pleasing structure with minimal visual impact.

2.3.4 Alternate Method of Winter Operation

As an alternate measure, the installation of additional gates on the north end (Canadian side) would satisfy the requirements of winter operation.



LIST OF PARTS

ITEM	DESCRIPTION	MATERIAL
1	LIMIT SWITCH SUPPORT	STEEL
2	WEATHER PROOF - LIMIT SWITCH	
3	SHAFT EXTENSION	STEEL
4	BOSTON COUPLING	
5	CRANK SHAFT	STEEL
6	COVER-FOR CRANK SHAFT	STEEL
7	DRIVE SUPPORT	STEEL
8	WEATHER PROOF - DRIVE MOTOR WITH BRAKE	
9	WEATHER PROOF - STD WORM GEAR REDUCER	
10	COUPLING	
11	CHAIN COUPLING	

Figure G-14
LAKE SUPERIOR COMPENSATING WORKS—
PROPOSED MODIFIED DRIVE MACHINERY ARRANGEMENT

LIST OF PARTS

ITEM	DESCRIPTION	MATERIAL
1	HOIST BRIDGE ENCLOSURE	
2	CONTROL & HEATING PANEL	
3	WELDED FRAME	STEEL

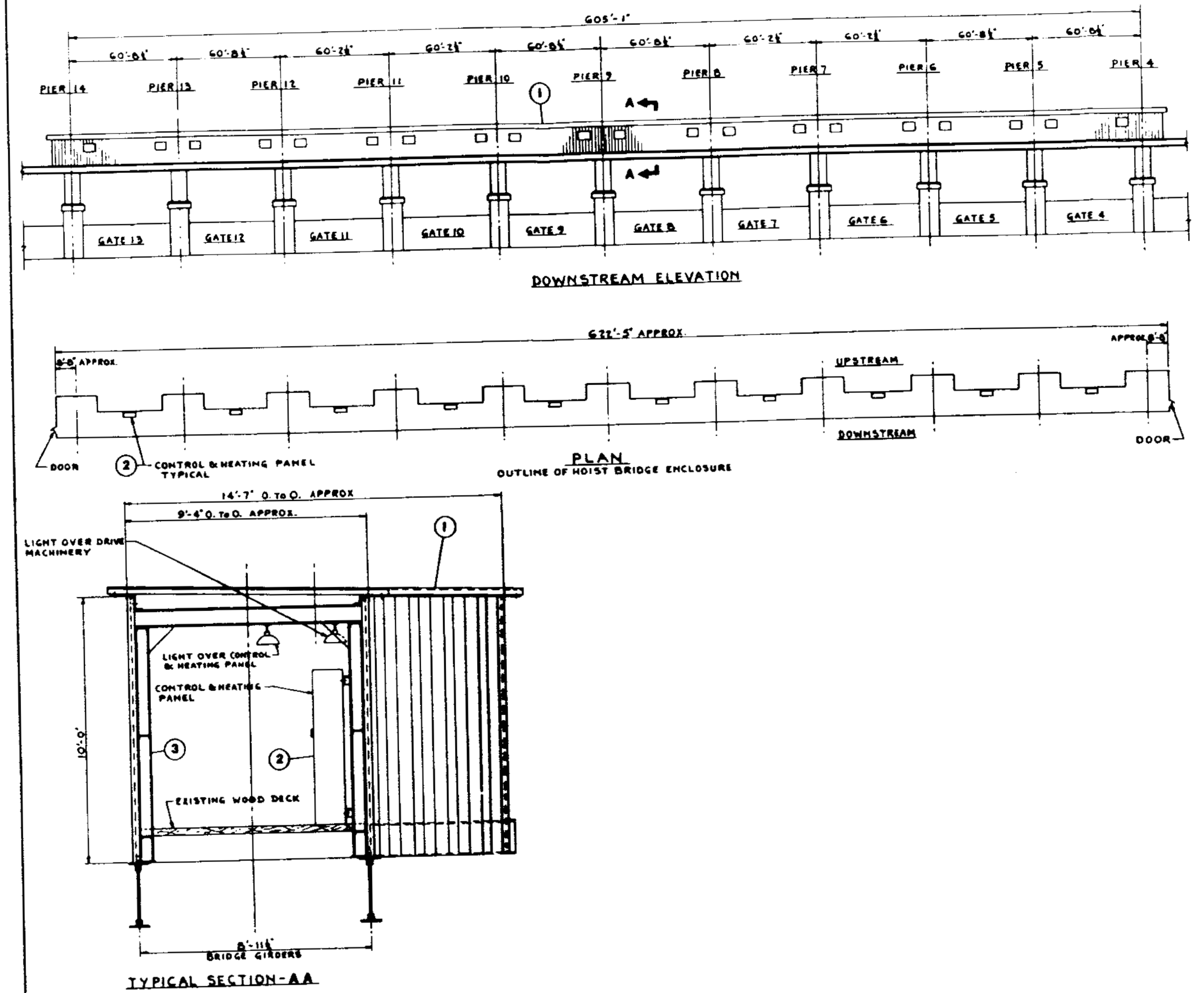


Figure G-15
LAKE SUPERIOR COMPENSATING WORKS—
PROPOSED HOIST BRIDGE ENCLOSURE ARRANGEMENT

It is estimated that three gates, having similar dimensions as existing gates would be required and these could be constructed without necessity for major modifications to the existing approach and exit channels of the structure. These gates, which would be electrically heated and completely mechanized, would be operated to provide the appropriate change in regulated winter flow above a base flow which would be discharged through the existing works. In other words, the gates of the Compensating Works would be set at the onset of the winter period and any required variations in the flow during the period would be obtained by manipulating the new, additional gates. These gates would also compensate for the lost capacity of the control works (noted in Section 2.1.3) during maximum flow requirements. These gates would also increase the capacity of the existing works (such as Abitibi units) especially during periods of maximum flow requirements. It is estimated that these works would require a capital outlay of $3,900,000 (1971 price levels) which when amortized over a 50-year project period, at an interest rate of 7%, is equivalent to an annual cost of $283,000. It is estimated that the annual operation and maintenance costs, principally electrical energy for the heating of the gates, hydraulic monitoring of the river and annual maintenance of the gates would be $30,000. On this basis, the total annual cost of these works are estimated at $313,000. Due to the preliminary nature of this design and the recognized necessity of model studies, estimates of 20% for engineering design, supervision and administration and 30% for contingencies were incorporated into the above cost estimate. Details for the construction of these additional gates were not pursued beyond this point since the capital expenditures far exceeded that of the following recommended method.

2.3.5 Recommended Facilities for Winter Operation

An analysis of Plans SO-802 and SO-901 (see Section 8, Main Report) revealed that the maximum gate setting would be 6 gates open during the winter months corresponding to a flow of 85,000 cfs. If a maximum winter flow of 95,000 cfs were to be specified, a gate setting of 8 gates open would be required. It was therefore decided to provide electrical tubular heaters for 6 gates and electrical tubular heaters for 8 pairs of gains. If in the future, a winter maximum flow of 95,000 cfs was adopted, electrical tubular heaters could be provided for the additional 2 gates since construction methods are relatively simple and inexpensive. However, the installation of the tubular heaters in the gate gains is relatively complex and expensive. Since the tubular heaters have to be installed in the gains with the gate in the fully open position, the construction method selected consists essentially of forming slots in the piers, downstream of the gate, which would serve as receptacles for a bulkhead gate thus eliminating flow of water through the gate. Only one bulkhead gate would be constructed and this would be moved by derrickboat from gate to gate as the installation progressed. Not only would this bulkhead gate be used for installation of the tubular gain heaters, but it could be used subsequently for underwater sill, pier, or gate slot repairs required from time to time as part of the normal maintenance program. All other equipment is in accordance with Section 2.3.3.

Table G-4 summarizes the cost of winter operations based upon the recommended works as discussed above. The estimated annual cost based upon this

TABLE G-4

AVERAGE ANNUAL COSTS* OF WINTER OPERATION
OF THE CONTROL STRUCTURE AT SAULT STE. MARIE

(Recommended Method, Using Electrical Equipment)

		INITIAL CAPITAL COSTS		ANNUAL** COSTS
1.	CAPITAL EXPENDITURE			
(a)	Tubular gate heaters for 6 gates & tubular gain heaters for 8 pairs of gains	$208,000		$15,600
(b)	Structural modifications for 6 gates	54,000		4,050
(c)	Electrical power line through Great Lakes Power Company to the north end of the structure	80,000		6,000
(d)	Telephone line to north end of structure	10,000		750
(e)	Modifications to provide motorized drives for all 16 sets of gate hoist machinery	102,000		7,650
(f)	Hinged sheet steel covers over open gears of 16 sets of gate hoist machinery	5,000		375
(g)	Metal clad enclosure over 10 gates including convenient lighting	115,000		8,625
	Total Capital Cost:	$574,000	Sub-Total:	$43,050
2.	ANNUAL MAINTENANCE			
(a)	Maintenance of heating equipment			300
(b)	Maintenance of motorized drives			200
(c)	Maintenance of power line, sub-station & telephone line			600
(d)	Maintenance of metal housing & lighting equipment			450
(e)	Snow removal and site access			300
			Subtotal:	$ 1,850
3.	ANNUAL OPERATIONS			
(a)	Annual cost of gate heating operations			8,100
(b)	Annual cost of gate moving operations			250
(c)	Annual cost of operation of lighting equipment & telephone			200
(d)	Annual cost of hydraulic monitoring of the river & emergency standby procedures			16,000
			Subtotal:	$24,550
	Total Annual Cost:			$69,450

*Expressed at 1971 price levels

**Based on a useful life of 40 years at an interest rate of 7%

method is $69,450 (1971 price levels). The following paragraphs outline the basis of the elements of these annual costs: capital expenditure, maintenance and operation.

The installations required consist of electric tubular gate heaters for 6 gates, electric tubular heaters for 8 pairs of gains, associated structural modifications for 6 gates, separate electrical power and telephone lines to the north end of the structure, modifications to provide motorized drives for all 16 sets of gate hoist machinery, a metal clad enclosure over 10 gates and hinged sheet steel covers over open gearing of all 16 sets of gate hoist machinery. The estimated capital cost of this equipment is $574,000 which when amortized over a useful life of 40 years at a 7% interest rate, corresponds to an annual cost of $43,050. Although the estimated life of the control structure is 50 years, the actual life of both equipment and the structure is dependent upon upkeep and repair. The 40-year figure for equipment life was only used for amortization purposes.

Annual maintenance costs cover the upkeep of the above described works plus the cost of snow removal and site access. These annual costs, $1,850, would be expended annually, whether or not gate movements are required during any particular winter period.

The annual cost of operations consists of labour costs to move the gates, annual cost of lighting equipment and telephone and two other major items; the cost of gate heating operations, and, the cost of hydraulic monitoring and emergency standby procedures. The cost of gate heating operations (electrical energy) was based on heating of 2 gates continuously over the winter period and heating others as required. This criterion would satisfy regulation Plan SO-901. Due to the effects that changing winter flows can have on the stability of downstream ice conditions, it is essential that close surveillance of the river be maintained over the winter period. This surveillance includes; ground observations, aerial reconnaissance and photography, the installation and operation of strategically located water level gauges capable of detecting changes in water surface profile that could signify the onset of ice jamming conditions, and the operation and maintenance of an emergency warning and communication system, the annual cost of operations is estimated at $24,550.

2.3.6 Costs of Increasing the Storage Capacity of Lake Superior

Alternative SO regulation plans were developed during the Board's Study which would expand the range of stage on Lake Superior by lowering the minimum level up to one foot. Approximate economic evaluations revealed that benefits in the order of $4 million, over and above that of Plan SO-901, could be obtained by such a lowering. It was determined that there would be no adverse effects on navigation provided all channel and harbours on Lake Superior and in the upper St. Marys River were dredged one foot deeper.

Preliminary estimates by the Canadian Department of Public Works revealed that the total capital costs of dredging public and private harbours and slips along the Canadian shoreline of Lake Superior to maintain present depths, if the minimum water level of Lake Superior were lowered one foot, would be $17 million, $9 million for public facilities and $8 million for private facilities.

Similar cost estimates for dredging harbours and navigation channels in United States were carried out by the U. S. Army Corps of Engineers. The total capital costs of such dredging was computed to be $31.5 million. It was assumed in the Corps' Study that costs associated with dredging of recreational and commercial fishing harbours would be minimal.

The total capital cost of dredging harbours and navigation channels situated along and in Lake Superior and the upper St. Marys River, to maintain existing vessel draft under a lowering of the range of Lake Superior by one foot would be $48.5 million. The total annual costs would be $3.8 million, at an interest rate of 7% and a project period of 50 years. It must be pointed out that this is a very preliminary estimate and would have to be refined in order to determine the feasibility or desirability of this improvement in the operation of a regulation plan. The environmental effects of the extensive dredging required must also be examined.

2.4 Data

The following sections list the basic and derived data pertinent to the design and cost estimates of Lake Superior regulatory works.

2.4.1 Basic Data

The following subsections list the basic data which existed or was updated during the course of the study and the collected data which was obtained during the study.

Existing Data:

(1) Recorded Water Levels at U. S. Slip Gauge (1900-1972).

(2) Recorded Water Levels at C.H.S. 012 Gauge (1908-1972).

(3) Recorded Water Levels at Lookout No. 3 Gauge (1900-1972).

(4) Recorded Water Levels of Lake Superior (1900-1972).

(5) Recorded Water Levels of Lakes Michigan-Huron (1900-1972).

(6) Recorded Adjusted Water Levels of Lake Superior (1900-1972).

(7) Recorded Adjusted Water Levels of Lakes Michigan-Huron (1900-1972).

(8) Lake Superior Outflow, 1860-1968, Coordinating Committee on Great Lakes Basic Hydraulic and Hydrologic Data, June 1970.

(9) Lake Superior Outflow, 1968-1972, Noble and Woodard Tables, U. S. Army Corps of Engineers, Monthly Publication.

(10) Rating curves for Lake Superior Regulatory Structure, Report on Discharge of St. Marys River, H.M. Edmands, U. S. Army Corps of Engineers, March, 1931.

(11) Stage-fall discharge equation between U. S. Slip and Mackinaw City Gauges, U. S. Army Corps of Engineers.

(12) Regulated Levels and Flows, Plans SO-801, SO-802, SO-803 and SO-901, (1900-1967).

Collected Data:

(1) Water Levels at Frechette Gauge (1968-1972).

(2) Water Levels at Rock Cut Gauge (1968-1972).

(3) Winter Gate Tests:

(a) Water Level Profiles, St. Marys River.

(b) Aerial Photographs of St. Marys River, Several Sets Each Year.

(c) Ice Thickness and Characteristics.

(d) Climatological Records.

2.4.2 Derived Data

Data derived during the course of the study is listed below:

(1) Critical design Elevations at U. S. Slip Gauge for the regulation plans.

(2) Cost Curves for Winter Gate Operation.

(3) Gauge Relationships in the Lower River for Open Water Conditions.

(4) Report entitled: "Lake Superior Regulatory Structure, Report on Present Condition and Terms of Reference for Feasibility Study of Winter Operation".

(5) Report entitled: "Lake Superior Regulatory Structure, Feasibility Study for Improvements to Lake Superior Regulatory Control Works", Acres Consulting Service, March 1972.

Section 3

ST. CLAIR - DETROIT RIVER SYSTEM

3.1 Description of the System

The St. Clair-Detroit River system forms the outlet for Lakes Michigan-Huron discharging southerly into Lake Erie. The system, about 86 miles long, is characterized by relatively uniform water surface profiles with no rapids or falls. The river bed is composed for the most part of heavy blue clay and, except for the actions of man, is stable.

3.1.1 General

The St. Clair-Detroit River system, shown on Figure G-16, is divided into three distinct parts: The St. Clair River, which has a length of about 38 miles; Lake St. Clair, extending between the mouth of the St. Clair River and the head of the Detroit River, a distance of about 16 miles; and the Detroit River which extends about 32 miles to Lake Erie. The fall in the water level from Lakes Michigan-Huron to Lake St. Clair is about 5 feet and from Lake St. Clair to Lake Erie it is about 3 feet. Figure G-17 shows the water surface profile for the low, mean, and high flow conditions of 152,000 cfs, 186,000 cfs and 210,000 cfs respectively under the present (1962) hydraulic regimen. Average current velocities in the St. Clair River range from 3 to 7 fps (feet per second) depending on the characteristics of a specific reach. Maximum velocities occur in the narrow constriction, 800 feet width, near the Blue Water Bridge. Similarly, depending on the location, average velocities in the Detroit River range from 1.5 to 3 fps.

3.1.2 St. Clair River

The St. Clair River, shown on Figure G-18, can be separated into three reaches. The upper contracted reach, extending downstream from Lake Huron for about 4 miles, is about 800 feet wide at the narrowest point and has mid-channel depths varying from about 30 to 70 feet. The middle reach extends downstream over the next 23 miles, is about one-half mile wide, and has channel depths varying from about 27 to 50 feet. Located in this reach are Stag and Fawn Islands and a middle ground shoal opposite the City of St. Clair, Michigan. The lower reach extends about 11 miles to Lake St. Clair and it is in this reach that the river begins to divide into a number of distributaries which flow across the delta shaped area called the St. Clair Flats. It is in this latter area where major changes in the channels have taken place through private, Canadian and U. S. Government dredging operations.

3.1.3 Lake St. Clair

Lake St. Clair, a shallow embayment in the St. Clair-Detroit River system, occupies a wide, expansive, relatively shallow basin having an average depth of about 10 feet, with low, marshy shores. A location map of Lake St. Clair is shown on Figure G-19. It has a water surface area of about 430 square miles. The drop in level in the 16 miles across the lake from

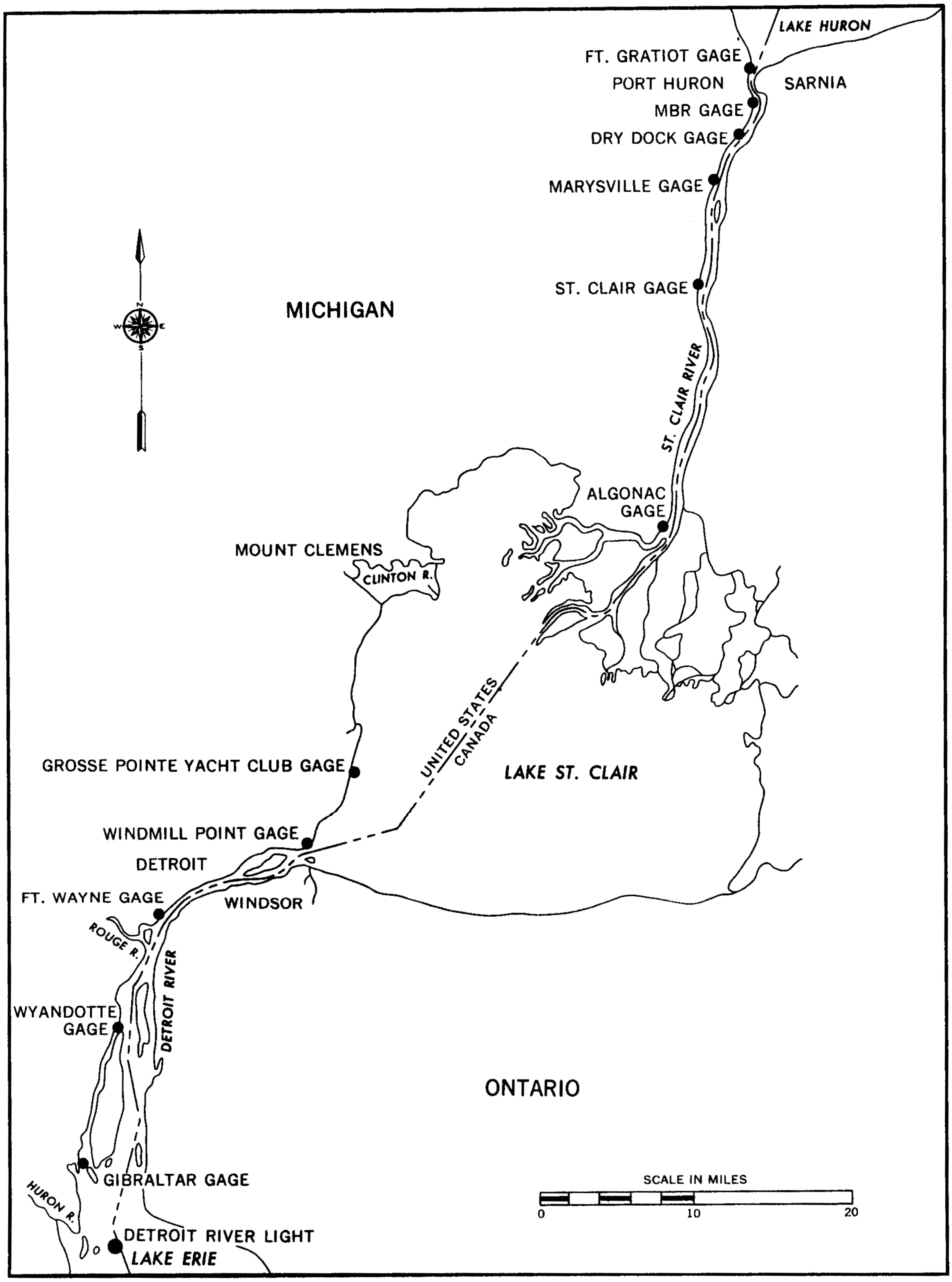
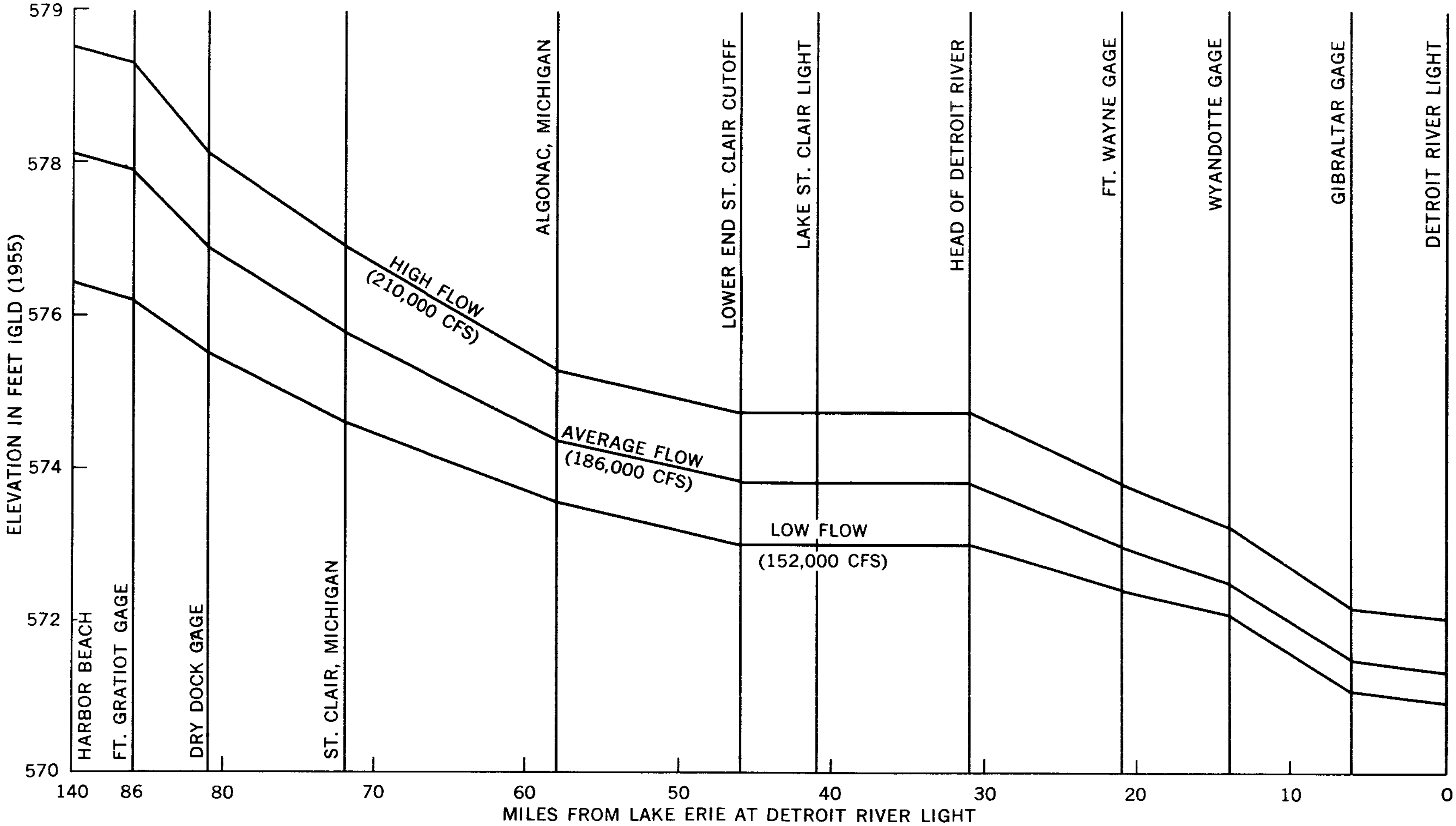


Figure G-16
ST. CLAIR—DETROIT RIVER SYSTEM—LOCATION MAP



RIVER PROFILES FOR HIGH (210,000 CFS), AVERAGE (186,000 CFS) AND LOW (152,000 CFS) FLOWS WITH THE RIVER SYSTEM IN EXISTING (1970) CONDITION.

Figure G-17

ST. CLAIR—DETROIT RIVER SYSTEM—WATER SURFACE PROFILES the St. Clair Flats to the head of the Detroit River is in the order of 0.1 foot. The shallow depth requires a dredged commercial navigation channel throughout its length. Improvements for navigation have provided a navigation channel 27.5 feet deep and 800 feet wide.

3.1.4 Detroit River

Except at its head where Peach Island and Belle Isle are located, the upper 13 miles of the Detroit River is characterized by relatively uniform cross sections, having a width of about one-half mile and channel depths varying from 27 to 50 feet. In the lower 19 miles, from the head of Fighting Island to Lake Erie, the river broadens and is characterized by many islands and shoals created by extensive outcroppings of limestone. The improved main navigation channels through the lower river are located between the west side of Fighting Island and the east side of Grosse Ile. In the lower 7 miles, starting from a point east of Grosse Ile, to Lake Erie, downbound traffic is via the Livingstone Channel and upbound is via the Amherstburg Channel. Except for the work of man, the natural channels in the St. Clair and Detroit Rivers have remained virtually unchaged due to the stability of the heavy blue clay which constitutes their bed. A location plan of the Detroit River is shown on Figure G-20.

3.1.5 Navigation Channels

A minimum 25-foot navigation channel was constructed throughout the St. Clair-Detroit River system in 1932-36. A minimum 27-foot, deep-draft channel exists throughout the entire length of the system as the result of a deepening program initiated in 1957 and completed in 1962. In the upper St. Clair River, an unmaintained channel with a depth of 21 feet is available on the east side of Stag Island. It is at times used by upbound vessels. In the St. Clair Flats area, vessels use the St. Clair Cutoff Canal, which was constructed in 1962 to eliminate the hazard of navigation encountered in the sharply curved Southeast Bend Channel and to reduce the vessel passage time through a shortened route. The Southeast Bend Channel, although no longer maintained, has at the present time (1972) good available depths (25 feet below LWD). In the lower Detroit River, from Bar Point, Ontario, to Ballards Reef, two channels are provided: the Amherstburg Channel is for upbound traffic and the Livingstone Channel is for downbound traffic. Another deep-draft navigation channel in the Detroit River, called the Trenton Channel, extends from the main ship channel north of Grosse Ile, Michigan, approximately nine miles from its point of origin. The upper 5.5 miles has a minimum depth of 27 feet and the remainder 21 feet. Navigation channels for the St. Clair River, Lake St. Clair and the Detroit River are shown on Figures G-18, G-19, and G-20, respectively.

3.1.6 Recreational Navigation

Numerous recreational boating harbours, maintained by the various levels of government and by private individuals and clubs, are located throughout the Great Lakes system. In particular, the St. Clair River, Lake St. Clair and Detroit River waterway is heavily used for recreational boating purposes and, as such, forms a major consideration in the design of regulatory works.

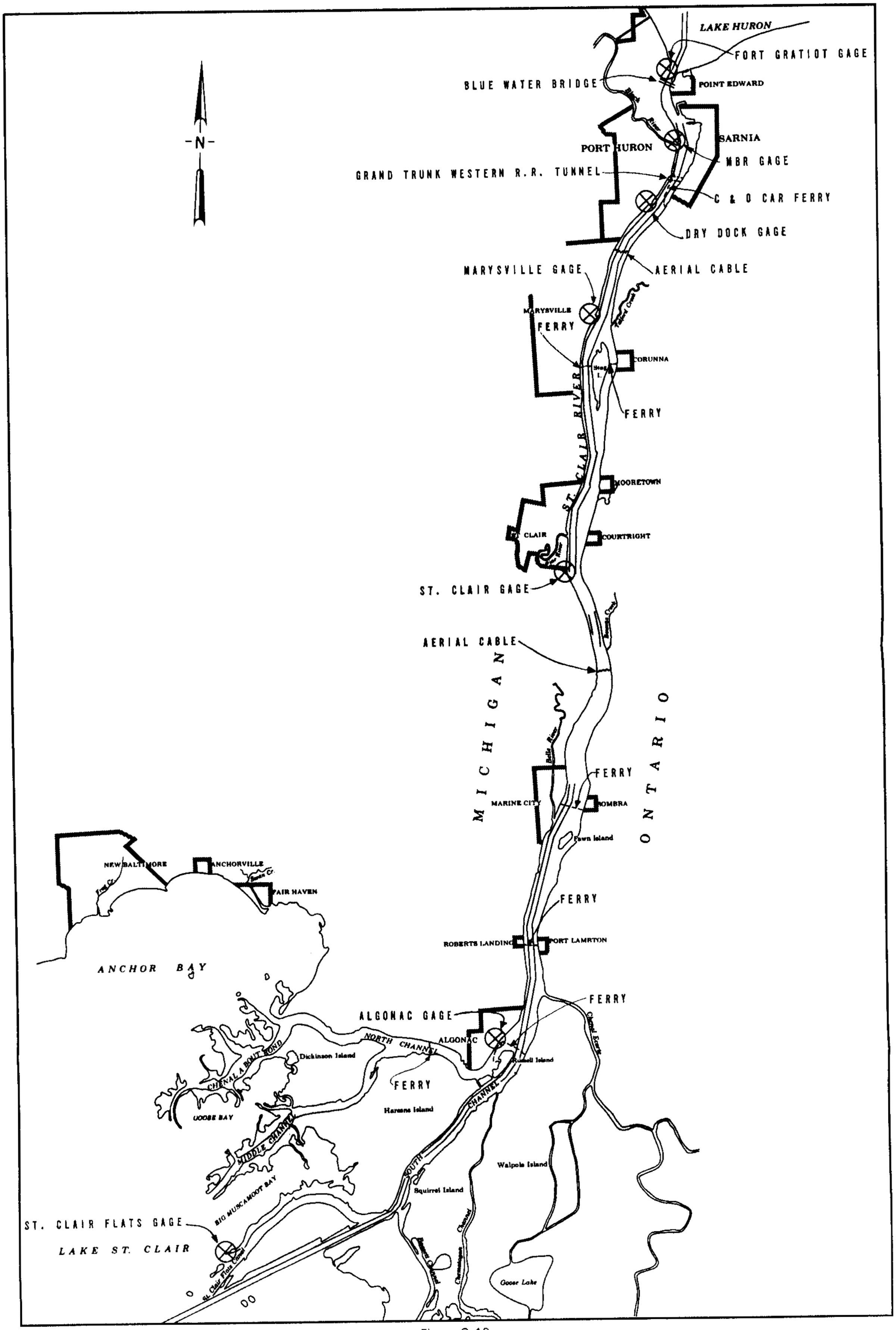


Figure G-18
ST. CLAIR RIVER—LOCATION MAP

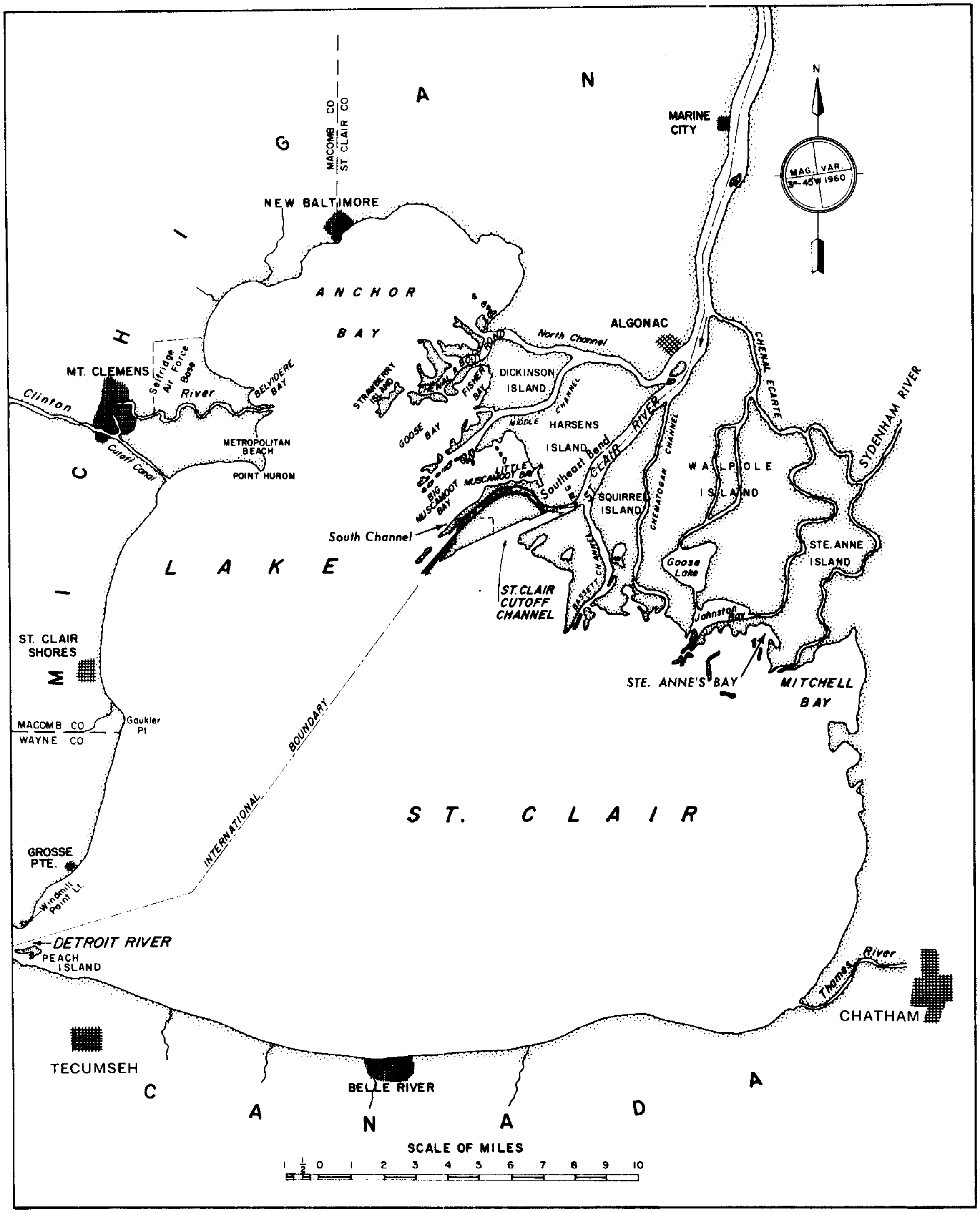


Figure G-19
LAKE ST. CLAIR—LOCATION MAP

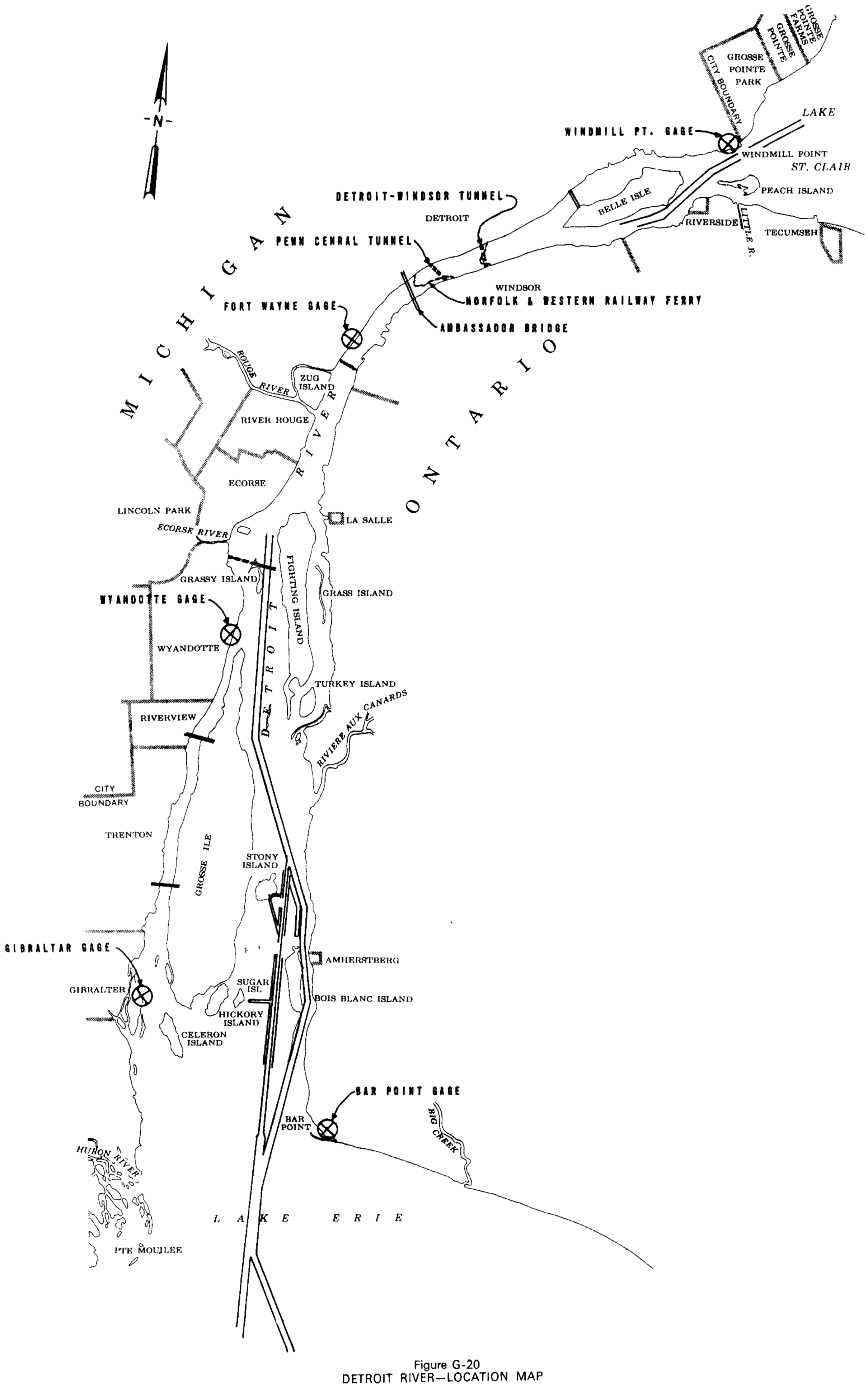


Figure G-20
DETROIT RIVER—LOCATION MAP

3.1.7 Existing Compensating Works

Compensating dikes have been constructed on the lower Detroit River to partially offset the lowering of water levels due to past authorized navigational improvements in 1912, 1936, and 1962. However, similar compensation in the St. Clair River, resulting from the 25 and 27-foot navigation projects has not been made.

On the lower Detroit River, compensating dikes have been constructed along sections of both sides of the Livingstone Channel and the westerly side of the Amherstburg Channel from near the lower end of Bois Blanc Island to the channel junction as shown on Figures G-20. In addition, the Sugar Island compensating dike has been constructed across a portion of the river from the Livingstone Channel to near Sugar Island at the south end of Grosse Ile as part of the 25-foot project in 1936. Compensation for the 27-foot project, completed in 1962, consisted of added width and length to existing dikes. Studies of the U. S. Army Corps of Engineers indicate that sufficient compensation has not been provided in the Detroit River to offset the lowering effect due to the 27-foot navigation project.

3.1.8 Pollution and Environmental Consideration

In conjunction with providing regulatory works and dredging associated with Lakes Michigan-Huron regulation plans, factors affecting pollution and other environmental factors were considered. An abstract of a report dealing with this subject is contained in Annex C to this Appendix. For example, tests performed by U. S. and Canadian agencies indicated that the bottom materials of the St. Clair-Detroit River system contain mercury and other pollutants at unacceptable levels. To prevent further distribution of the polluted bottom materials throughout the dredging activities, it is proposed that the dredged materials be confined by dikes which would generally be located on upland sites. In another case, during preliminary design stages, coordination with other interests revealed that structure location at the junction of the main North and Subsidiary Middle Channels would have an adverse effect on the environment of Anchor Bay in Lake St. Clair. As a result of these considerations, the designs of the regulatory structures included provision for unimpeded flow around the end of the structures and flushing flows through the gated portions.

3.1.9 Bridges, Ferries, and Other Facilities

The St. Clair-Detroit River system is intensively developed with domestic, commercial and industrial facilities. Those facilities, in direct connection with changes in the St. Clair-Detroit River system, include bridges, tunnels and docks for automobile and railway car ferries, commercial vessels and recreational craft. A list of the major bridges, tunnels and ferries and their locations is presented in Table G-5. The location of each of these facilities is further illustrated on Figures G-18 and G-20.

TABLE G-5

ST. CLAIR - DETROIT RIVER SYSTEM
HIGHWAY BRIDGES, TUNNELS AND FERRIES

Name	Location	Type	Clearance, Ft. Max.	Min.
1. HIGHWAY BRIDGES				
Blue Water	Port Huron - Sarnia	Cantilever	155	135
Ambassador	Detroit - Windsor	Suspension	156	133
Wayne County Highway	Grosse Ile (Trenton Ch.)	Swing	-	-
Grosse Ile Bridge, (Private)	Grosse Ile (Trenton Ch.)	Swing	-	-
2. TUNNELS				
Railway				
Grand Trunk Western	Port Huron - Sarnia			
Penn Central (New York Central)	Detroit - Windsor			
Automobile				
Detroit - Windsor Tunnel	Detroit - Windsor			
3. FERRIES				
Railway				
Chesapeake & Ohio	Port Huron - Sarnia			
Norfolk & Western	Detroit - Windsor			
Automobile	Corruna - Stag Island			
Automobile	Marine City - Sombra			
Automobile	Roberts Landing - Port Lambton			
Automobile	Algonac - Russell Island			
Automobile	Algonac - Walpole Island			
Automobile	Algonac - Harsens Island			

3.1.10 Ice Problems

Lakes Michigan-Huron, as with all of the Great Lakes, does not freeze over with a permanent ice cover during the winter, primarily due to the influence of wind action and of the heat stored in the water. Ice which forms on the surface is able to persist as coherent sheets only in protected areas. The ice which forms in exposed central parts of the lakes is continually broken up and moved about by the action of the wind. Most of the ice eventually decays on the lake surface with the advent of warmer weather, but part of it finds its way into the St. Clair River at the outlet of Lakes Michigan-Huron. It is normal to find ice jams in the St. Clair River periodically through the winter season. The supply of ice delivered to the river and the consequent degree of jamming are highly variable, being the result of such inherently variable climatic factors as winds, temperatures and snowfall. During the times when jams are present, the outflow of water from the lakes may be reduced due to the obstructing effect of the ice in the outlet rivers. Ice jamming is an important factor in the natural winter regime of the St. Clair-Detroit Rivers Systems. Jams form in places where the capacity of the river to carry away floating ice is less than that necessary to remove the output of ice being delivered from upstream. Lake St. Clair, and the lower St. Clair River channels discharging into it, normally freeze over early in the winter. Ice carried down to this point from Lake Huron forms heavy jams in these regions. It has been estimated that the average flow in the St. Clair River during the 3-month period, January through March, is approximately 25,000 cfs less than that which would occur under open-water conditions. This value applies to the channels as they existed prior to the 27-foot navigation improvement which was completed in 1962.

The Detroit River is shielded from heavy ice runs by the intact cover on Lake St. Clair, and ice jams seldom occur at the outlet from Lake St. Clair. The river itself is frequently frozen over in its lower reaches. The average January through March flow retardation by ice in the Detroit River is estimated to be 4,000 cfs. This value applies to the channels as they existed prior to the 27-foot navigation improvement.

The effect of ice on the flow resistance of the river system between Lake Huron and Lake Erie is reflected in the water levels. During severe ice jams, water levels in the St. Clair River may rise as much as 3 feet, whereas the level in Lake St. Clair (below the jams) may drop by as much as 1.5 feet.

With the dredging of the various channels of the St. Clair-Detroit River system over the years, the depth and cross-sectional area of critical reaches have been increased and, as such, have decreased the amount of ice retardation relative to natural conditions. In this context, the existing ice regime would be altered by the regulatory works which would be required for a SMHO or SMHEO regulation plan, as discussed in Section 3.3.15.

3.2 Assumptions

The following assumptions have been made with respect to the design of regulatory works and channel excavations for the St. Clair-Detroit River system.

(1) Commercial navigation locks would not be tolerable to commercial navigation in the St. Clair-Detroit River system.

(2) There should be minimum restriction of recreational boat traffic.

(3) The existing range of water level profiles as adjusted to the pre-1933 channel regime, should be maintained in the St. Clair-Detroit River system.

(4) The general flow and current pattern in Lake St. Clair should be maintained.

(5) The existing ice regimen should be maintained in the system.

(6) Adverse environmental impact of all proposed works should be minimized.

(7) The structures should be operable all year.

3.3 Methodology

Because no hydraulic control exists in the St. Clair-Detroit River system, the flow is sub-critical with the outflow from Lakes Michigan-Huron being a function of the levels of Lakes Michigan-Huron, St. Clair and Erie. A change of one foot on the level of Lake Erie is reduced by the backwater effects in the system to a change of only 0.25 foot on Lakes Michigan-Huron. The regulation of the outflow from a lake or reservoir will, by definition, require a change from the flow that would have occurred under natural channel conditions. Artificial releases with construction of regulatory works would affect the water surface profiles in the connecting channels, in this case the St. Clair-Detroit River. Due to the discharge capacity characteristics of the natural channels, regulated or artificial releases which are greater than the natural flow would result in higher water surface profiles. For this condition, the capacity of the channels must be increased in order to maintain the same profile that would occur for natural conditions. In contrast, artificial releases which would be less than natural would result in lower natural water surface profiles. This condition requires that the flow be retarded in order to raise the profile to the natural conditions. The development of channel designs which are necessary to satisfy these changes are based on the selection of the most unfavourable or extreme changes that could be experienced under regulated conditions. The methodology was developed with a view to providing the necessary channel capacity increases and retardations in order to maintain the natural profiles.

One of the major constraints in the channel design and cost determination was the necessity of providing cost curves for use in evaluating regulation plans that were in the developmental stage. Stated in another way, the cost curves for channel design were developed independently of the yet to be developed regulation plans. As a result, the following procedures were followed:

1. Due to the unavailability of regulated levels and flows, extreme design conditions were determined from recorded prototype conditions.

2. For regulation plans requiring both channel capacity increases and decreases, regulatory structures were required to reduce or retard the flow to a degree that returned the channel capacity increase to zero or to the 1933 natural channel capacity. From this condition, the regulatory strue tures were further required to reduce or retard the flow by an additional amount as specified by the channel capacity decrease. Thus the total amount of channel capacity decrease includes the enlargements in the channels provided for the channel capacity increase. Since the combination of conditions for channel capacity increases and decreases are innumerable, the structures required for channel designs were limited to two conditions of channel increases, no increase and an assumed maximum. Both of these conditions were combined with various channel capacity decreases. Thus, the product of the structural design activities consisted of two cost curves: (1) structure costs versus a range of channel capacity decreases for no channel capacity increases and, (2) for similar channel capacity decreases incorporating the enlarged channel determined for the maximum channel capacity increase.

The structure costs for preliminary regulation plans were determined from these two curves, with the intermediate values of channel capacity increases being obtained by interpolating between the two curves, no increase and maximum increase. The costs for channel capacity increases were determined from a relationship between channel capacity increase and costs, as illustrated in Section 3.3.12. For selected regulation plans, which provide the required regulated levels and flows, the channel designs and costs are keyed to the hydraulic conditions created by the plans. The methodology developed here is applicable to both the final and preliminary applications.

Another constraint that affected the methodology was the requirement to maintain the St. Clair-Detroit River water surface profiles to the hydraulic regimen which existed prior to 1933, before the start of the 25-foot navigation project.

3.3.1 Objectives

The principal objectives in channel design were to determine the required changes in the channels, the optimum location of regulatory works and the costs thereof. These objectives were achieved on the basis of assumptions discussed in Section 3.2.

3.3.2 Outline of Procedures

The procedures followed in reaching the objectives for channel design are outlined below:

1. Determination of design conditions, extremes of levels and flows

2. Determination of 1933 channel condition water surface profiles for the design conditions

3. Development of mathematical model of St. Clair-Detroit River system for present (1962) channel conditions

4. Application of mathematical model in the design of channel enlargements and location of regulatory structures

5. Conceptual design and cost estimates of regulatory structures

6. Determination of cost estimates for channel excavations

7. Development of cost curves

8. Determination of costs for preliminary regulation plans

9. Channel design and cost estimates for selected regulation plans

The details of the procedures outlined above are discussed in the following subsections.

3.3.3 Determination of Design Conditions of Regulated Levels and Flows

In the development of a regulation plan there exists two extreme conditions of flow that have the maximum deviation from natural conditions. These are:

1. The maximum channel increase, the flow regulated minus flow natural, is the maximum positive difference or:

   Max Channel Increase = Qregulated-Qnatural = Qmax (+)

2. The maximum channel decrease, the flow regulated minus the flow natural, is the maximum negative difference or:

   Max Channel Decrease = Qregulated-Qnatural = Qmax (-)

Regulatory works designed to satisfy these two extreme conditions would also satisfy all intermediate conditions.

Design Conditions for Preliminary Regulation Plans: In order to provide channel design costs for optimizing the benefits and costs of preliminary regulation plans, cost curves were required for a range of channel changes. Since no regulated levels and flows were available for obtaining the extreme design conditions, recorded monthly water levels for the period, 1900-1967, were substituted for regulated monthly levels of Lakes Michigan-Huron and Lake Erie. The assumption was made that the regulated conditions most difficult to satisfy hydraulically would be abnormal, i.e. other than average. Also, in the selection of the recorded levels, it was realized that the most extreme levels might not recur since the objective of the 1964 water levels Reference to the IJC is to study the feasibility of reducing the extremes in stage which have been experienced. Since the extreme levels would not be appropriate design parameters, a stage-duration curve for Lakes Michigan-Huron levels, April through November, was derived and employed. This diagram is shown on Figure G-21. The water levels near the 10 and 90 percent frequencies were identified by dates of occurrence. Recorded Lake Erie water levels, corresponding to the dates of the selected Lakes Michigan-Huron levels, were extracted from the historical data. Abnormal slopes between the two lakes were selected from these data for use in determining the channel capacity increase and decrease design conditions. These design elevations, shown on Figure G-22, occurred at the 8 and 93 percent frequency exceedence of the Lakes Michigan-Huron stage duration curve. Using the dates on which these selected levels occurred, the corresponding recorded levels of Lake St. Clair were obtained.

The next step was the determination of the flow that would occur under 1933 channel conditions for each river under the two design conditions. These flows were determined from the design levels given, utilizing the stage-fall discharge equations derived from the monthly mean flows for the 1924-1933 period as determined in this study. The results, summarized on Figure G-22, show the profile condition for discharging increased flows. The corresponding St. Clair River flow is 148,000 cfs for the natural (1933 channel) regimen. The condition for retarding flow corresponds to a St. Clair River flow of 220,000 cfs. Employing the information contained on Figure G-22, the water surface profile for the two design conditions and the results are shown on Figure G-23. This difference in flow between the St. Clair and Detroit River profiles has been computed to be about 4,000 to 5,000 cfs, depending on the supply conditions, as determined by the water balance for Lake St. Clair. In order to obtain hydraulic compatability for design in this river system, 4,000 cfs was added to the St. Clair River flow for the low flow condition of 148,000 cfs resulting in a Detroit River flow of 152,000 cfs. Application of 5,000 cfs to the high-flow condition for the St. Clair River results in a comparable flow of 225,000 cfs for the Detroit River. Using these design flows for the Detroit River, together with the derived equations, Lake Erie water surface elevations of 570.62 and 572.38 were computed for the low- and high-flow conditions, respectively. Intermediate water surface elevations between Lake Erie and Lake St. Clair were also computed to define the water surface profile under 1933 channel conditions. A summary of the data derived for the Detroit River design conditions is shown on Figure G-23. The locations of the gauges utilized for the St. Clair-Detroit River system are shown on Figure G-16.

3.3.4 Determination of Channel Design Limits

The next requirement was to establish some limits for the channel capacity increases and decreases that could be expected under regulated conditions. Utilizing the costs obtained in the U. S. Army Corps of Engineers' report entitled "Water Levels of the Great Lakes" dated December 1965 and using as a guide the evaluation of preliminary regulation plans, the upper limit of channel capacity decrease required for regulation would be 32,000 cfs, for both the St. Clair and Detroit Rivers, while the upper limit of channel capacity increase would be 32,000 cfs for the St. Clair River and 26,000 cfs for the Detroit River. These assumptions are applicable only to the determination of the preliminary regulation plan channel design-cost relationships and not to channel designs for a selected regulation plan involving these two rivers. The increments of flow changes used in the channel design, when applied to the flows determined for the 1933 channel conditions, represent the regulated flow. For example, if the increment of flow of

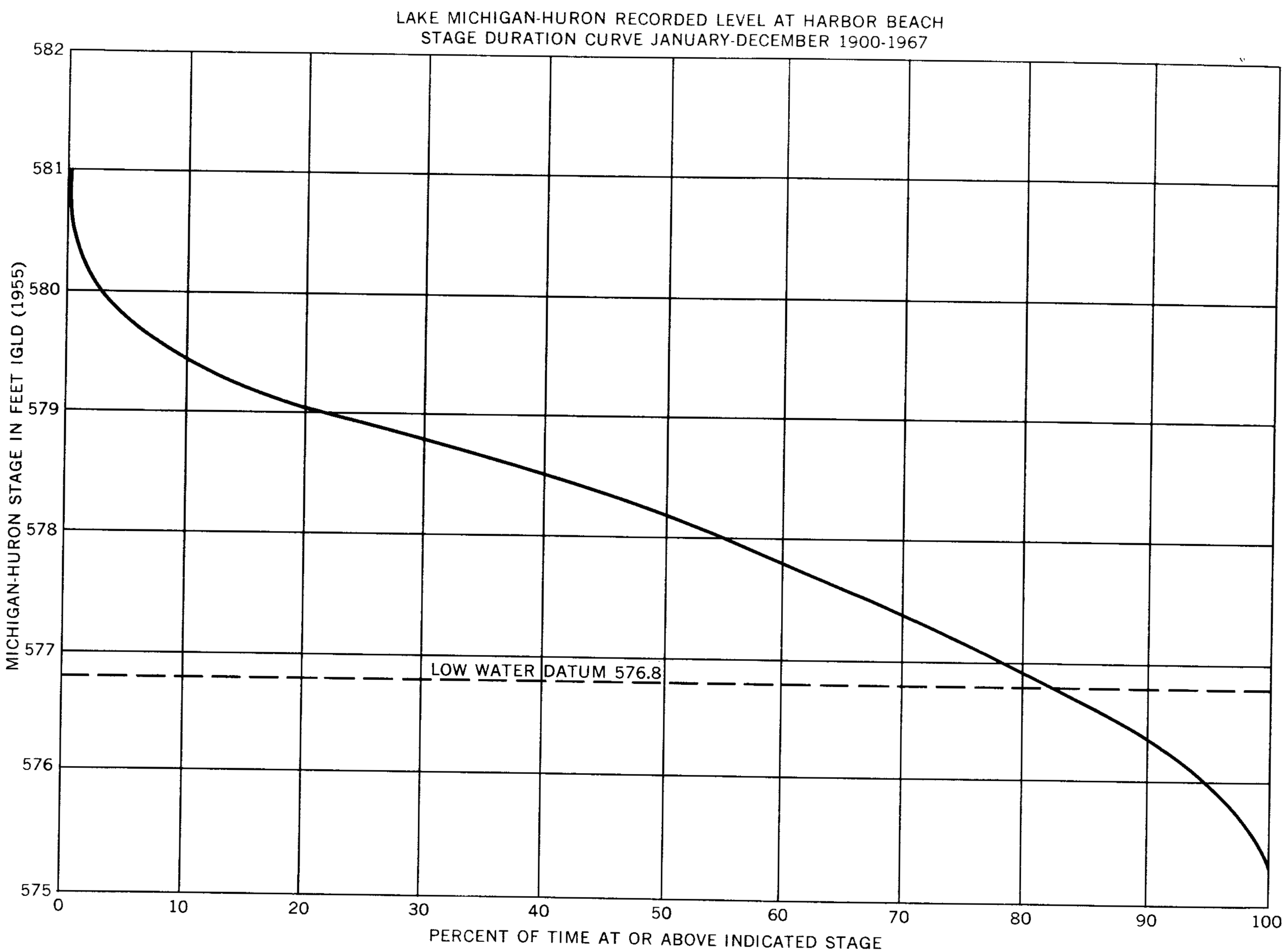


Figure G-21
LAKES MICHIGAN—HURON RECORDED LEVELS—STAGE DURATION DIAGRAM

FIGURE G-22
ST. CLAIR-DETROIT RIVER SYSTEM - PRELIMINARY DESIGN
LEVELS AND FLOWS

DESIGN STAGES FOR HIGH & LOW PROFILES

DESIGN CONDITION	LAKES MICH-HUR STAGE (HARBOR BEACH)	% OF TIME L. MICH-HUR STAGE EXCEEDED	DATE OF OCCURRENCE	L. ERIE STAGE CLEVELAND	FALL BETWEEN L. MICH-HUR & LAKE ERIE	L. ST. CLAIR STAGE (GPYC)	ST. CLAIR RIVER FLOW CFS
CHANNEL INCREASE (LOW)	576.49	93%	APRIL 1959	570.14	6.35	572.72	148,000
CHANNEL DECREASE (HIGH)	580.70	8%	SEP 1952	571.95	8.81	575.41	220,000

ST. CLAIR RIVER - DESIGN STAGES AND INTERM RIVER PROFILES

DESIGN CONDITION	ST. CLAIR RIVER WATER SURFACE PROFILE IN FEET									
GAUGE	HARBOR BEACH	POINT EDWARD	MOUTH OF BLACK RIVER	DRY DOCK	MARYS-VILLE	ST. CLAIR	ROBERTS LANDING	ALGONAC	GROSSE POINTE YACHT CLUB	ST. CLAIR RIVER FLOW (CFS)
CHANNEL INCREASE (LOW)	576.49	575.91	575.80	575.49	575.20	574.56	573.71	573.58	572.72	148,000
CHANNEL DECREASE (HIGH)	580.76	579.74	579.60	579.17	578.75	577.90	576.54	576.27	575.41	220,000

DETROIT RIVER - DESIGN STAGES AND INTERM RIVER PROFILES

DESIGN CONDITION	DETROIT RIVER FLOW (CFS)	LAKE ST. CLAIR	WINDMILL POINT	FORT WAYNE	CLEVELAND (LAKE ERIE)
CHANNEL INCREASE (LOW)	152,000	572.72	572.52	571.98	570.62
CHANNEL DECREASE (HIGH)	225,000	575.41	575.23	574.35	572.38

FIGURE G-23: ST. CLAIR-DETROIT RIVER SYSTEM - PRELIMINARY DESIGN PROFILES

20,000 cfs is selected as the channel capacity increase required under regulation, the regulated flow would be the natural 1933 channel condition flow of 148,000 cfs plus the 20,000 cfs change, or 168,000 cfs regulated flow. Conversely, the increment of flow for the channel capacity decrease is subtracted from the natural flow computed for the 1933 channel condition in order to determine the regulated flow.

3.3.5 Preliminary Channel Design Procedure

To facilitate the computation of channel designs for the development of cost curves for the preliminary channel design, the curves for the channel capacity decreases vs. costs were developed in combination with two channel capacity increases, namely, no channel capacity increase and the maximum assumed channel capacity increase relative to 1933 outlet conditions.

3.3.6 Development of Mathematical Models for Channel Designs

The computation for determining the effects of channel changes, water surface profiles, and design of compensating structures and channels, involves the standard, step-by-step backwater process. Manual trial and error computations are tedius and time consuming. Therefore, this process was computerized into what is referred to as a "mathematical model". The mathematical models used in this study consist of the computerization of the standard backwater computations utilizing Manning's equation. These models have been designed for specific rivers, incorporating features for balancing the river flow around islands, changing roughness coefficients for various river reaches and computing variable coefficients for sudden transitions in flows and eddies. An abstract of a report, describing in detail the processes involved for development of mathematical models for river systems, is contained in Annex C of this Appendix.

3.3.7 Use of Mathematical Models

One of the assumptions for regulatory works design was the requirement to maintain the range in water surface profiles of the St. Clair-Detroit River system to 1933 outlet conditions. Since the two models were designed to reflect existing (1962) conditions, they contain within themselves the uncompensated channel increases for the 25- and 27-foot navigation improvements. The amount of this built-in channel capacity increase was determined by operating the model with the 1933 channel capacity increase condition profile and computing the discharge that would duplicate this design profile. The model results indicated that the St. Clair River under present conditions is carrying about 11,000 cfs more flow than the 1933 channel conditions and the Detroit River is carrying about 6,000 cfs more. These "built-in" increases in the channel capacities were deducted from the results obtained from the model (1962 conditions) in channel design in order to adjust to 1933 channel conditions.

3.3.8 St. Clair River Mathematical Model

The St. Clair River mathematical model was developed in two parts, Lake Huron (Fort Gratiot Gauge) to Algonac at the confluence of the North and

South Channels, and from Algonac to Lake St. Clair. In the lower portion, the model divides into the North, Middle and South Channels. The South Channel in turn is divided by the Southeast Bend and the St. Clair Cut-off Channels. Other minor channels, such as the Chenal Ecarte and Basset Channel, were represented in the model as direct losses based on the percent of flow distribution. The model was divided into two parts due to the storage limitation of the computer used. The model was calibrated using discharge measurements taken in 1968, with corresponding water levels. Flow distributions and cross sectional areas of the channels were determined from hydrographic surveys taken in 1954 and 1970. The results of the calibration are summarized on Figure G-24.

3.3.9 Detroit River Mathematical Model

The Detroit River model, extending from Lake St. Clair (Windmill Point Gauge) to the outlet at Lake Erie, (Bar Point, Ontario) is a computer program divided into six parts (phases). The subdivision of the model was due in large measure to the limited capacity of the computer used. The model was calibrated using discharge measurements taken in 1967, corresponding water levels, flow distribution and cross sectional areas of the channels determined from hydrographic surveys taken in 1966. The results of the calibration are shown on Figure G-25.

3.3.10 Application of Mathematical Model in Channel Design, Channel Capacity Increase

Since the most economical location for dredging would be in those channels in which the structures would probably be located, some judgment was initially applied as to their location. Utilizing the water levels for Lakes Michigan-Huron, St. Clair and Erie, shown on Figure G-22, under the assumed most difficult profile for discharging additional flow, an increment of channel increase was applied to the design condition of flow in both rivers. Amounts of material were removed from the cross sections and the resulting profiles were computed for both rivers. Keeping the flow constant and increasing the amount of material removed, a relationship between the elevations of Lakes Michigan-Huron and material to be excavated on the St. Clair River was developed. Then, by changing the flow and repeating this process, a series of curves were determined as shown on Figure G-26. The intersection of the curves with the design elevation for Lakes Michigan-Huron delineated those computed profiles which satisfied the Lakes Michigan-Huron design elevation. Similar computations were performed for the Detroit River and the series of curves are shown on Figure G-27.

For the maximum channel capacity increase condition to provide for an increase in flow of 32,000 cfs, approximately 28.7 million cubic yards of bottom material is required to be dredged from seven reaches in the St. Clair River down to a depth of 36.0 feet below LWD. This estimate is based on 1962 channel conditions. Approximately 36.8 million cubic yards of bed material from present conditions is required to be dredged from four reaches in the Detroit River to 32.0 feet below LWD to provide for a maximum channel capacity increase of 26,000. The locations of these reaches in the St. Clair and Detroit Rivers are shown on Figures G-28a, G-28b and G-29, respectively.

ST. CLAIR RIVER MATHEMATICAL MODEL

1968 CALIBRATION

AVERAGE DISCHARGE = 192,600 CFS

a. UPPER ST. CLAIR RIVER

Reach	Flow (CFS)	Percent of Total Flow	Manning's Roughness
Ft. Gratiot	192,600	100	
			0.0252
Mouth of Black River	192,600	100	
			0.0234
Dry Dock	192,600	100	
			0.0237
Marysville	192,600	100	
			0.0235
St. Clair	192,600	100	
			0.0233
Robert's Landing	192,600	100	
			0.0265
Algonac	192,600	100	

b. LOWER ST. CLAIR RIVER

Reach	Flow (CFS)	Percent of Total Flow	Manning's Roughness
North Channel - Dick. Isl. to Alg.	102,100	53	0.0213
North Channel	63,600	33	0.0150
Middle Channel	38,500	20	0.0175
South Channel	80,900	42	0.0209
Southeast Bend	34,700	18	0.0170
St. Clair Cutoff	38,500	20	0.0275
Basset Channel	7,700	4	Loss
Chematogan Channel	-	Less than 1%	Loss
Chenal Ecarte	9,600	5	Loss

FIGURE G-24

ST. CLAIR RIVER MATHEMATICAL MODEL - RESULT OF CALIBRATION

DETROIT RIVER MATHEMATICAL MODEL

1967 CALIBRATION

AVERAGE DISCHARGE = 186,000 CFS

REACH	FLOW (CFS)	PERCENT OF TOTAL FLOW	MANNING'S ROUGHNESS
Detroit River	186,000	100	0.0340
West of Peach I.	142,000	77	0.0240
East of Peach I.	43,200	23	0.0240
West of Belle I.	57,000	31	0.0240
East of Belle I.	129,000	69	0.0240
West of head of Fighting I.	143,500	77	0.0230
East of head of Fighting I.	42,500	23	0.0265
Fighting I. Channel opposite Grassy I.	94,500	51	0.0281
West of Grassy I.	49,000	26	0.0265
Trenton	39,200	21	0.0245
Fighting I. Channel opposite Grosse Isle	104,200	56	0.0230
West of Turkey I.	22,000	12	0.0265
East of Turkey I.	20,500	11	0.0265
Amherstburg, Lime Kiln Reach	88,000	47	0.0272
Livingstone, Upper Diked Reach	48,600	26	0.0258
Stony Island	10,200	6	0.0284
Amherstburg, Hackett Reach	67,300	36	0.0273
Bois Blanc Dike	7,200	4	0.0315
Livingstone, Lower Diked Reach	40,100	22	0.0305
Sugar Island Dike	9,100	5	0.0470
Sugar Island	23,100	12	0.0366
West of Celeron I.	27,900	15	0.0267
East of Celeron I.	11,300	6	0.0375

FIGURE G-25
DETROIT RIVER MATHEMATICAL MODEL - RESULTS OF CALIBRATION

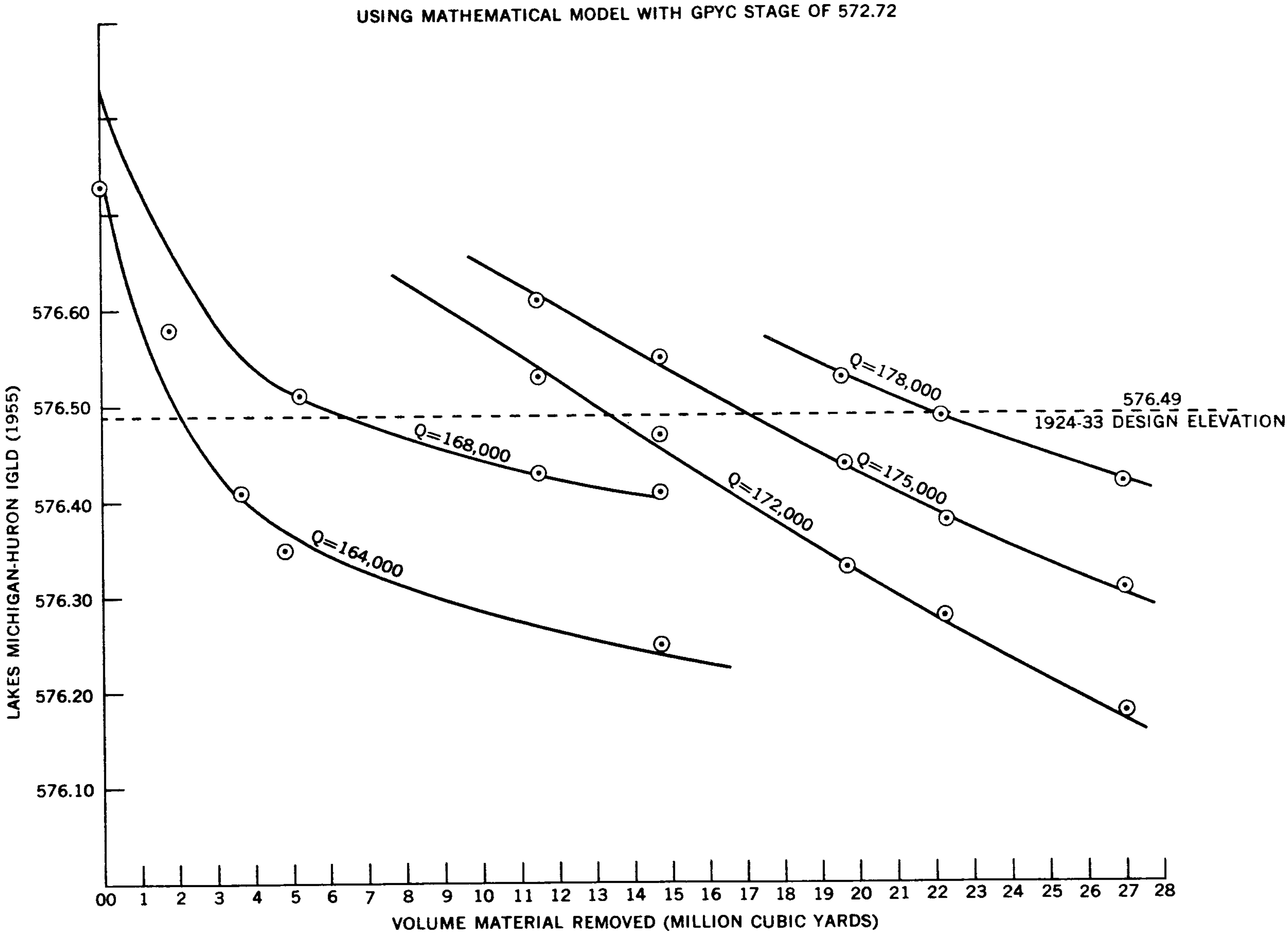


Figure G-26

ST. CLAIR RIVER CHANNEL CAPACITY INCREASE— RELATIONSHIP BETWEEN VOLUME OF EXCAVATION, FLOW AND LAKES MICHIGAN—HURON LEVELS.

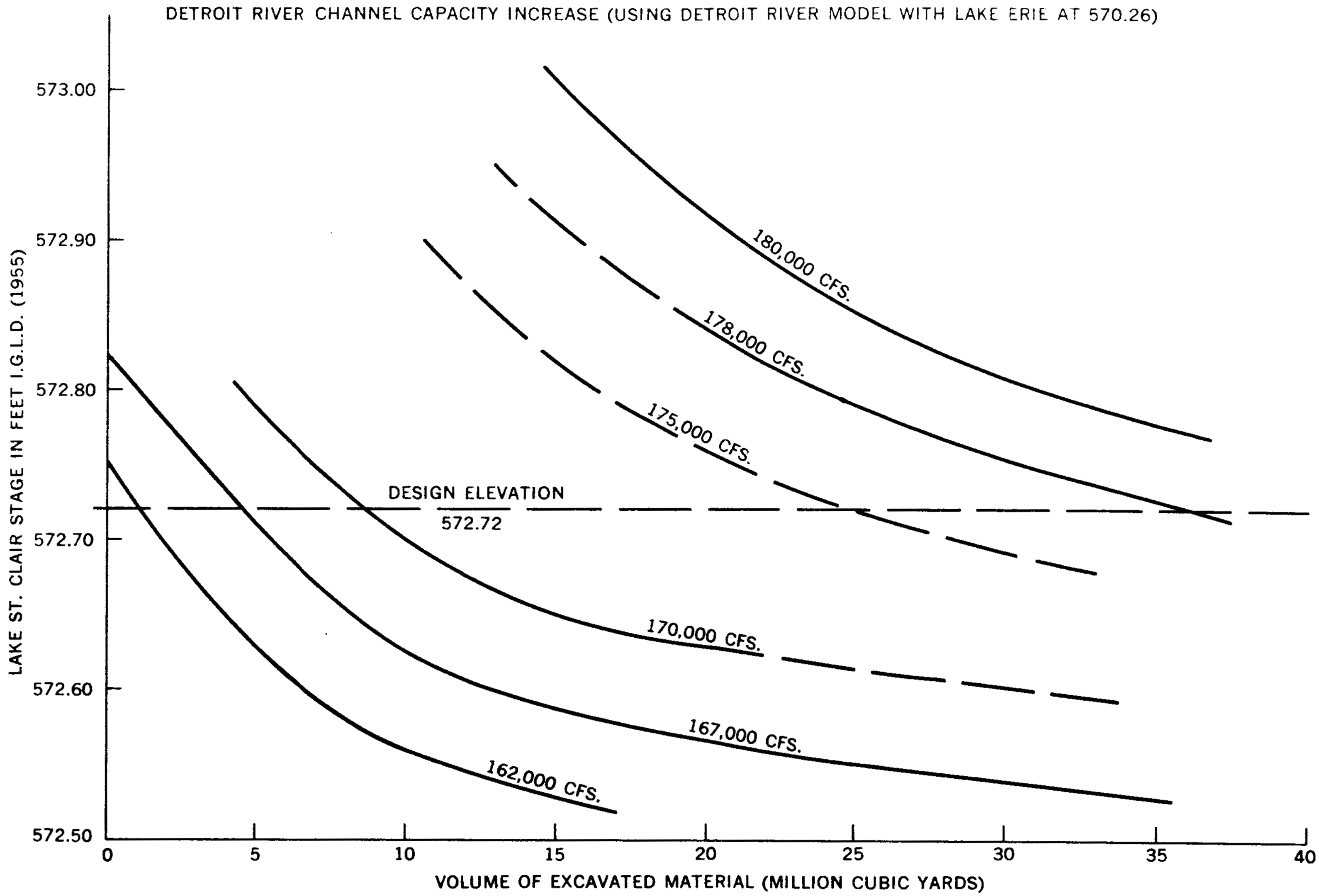


Figure G-27
DETROIT RIVER CHANNEL CAPACITY INCREASE—RELATIONSHIP BETWEEN VOLUME OF EXCAVATION, FLOW AND LAKE ST. CLAIR LEVELS

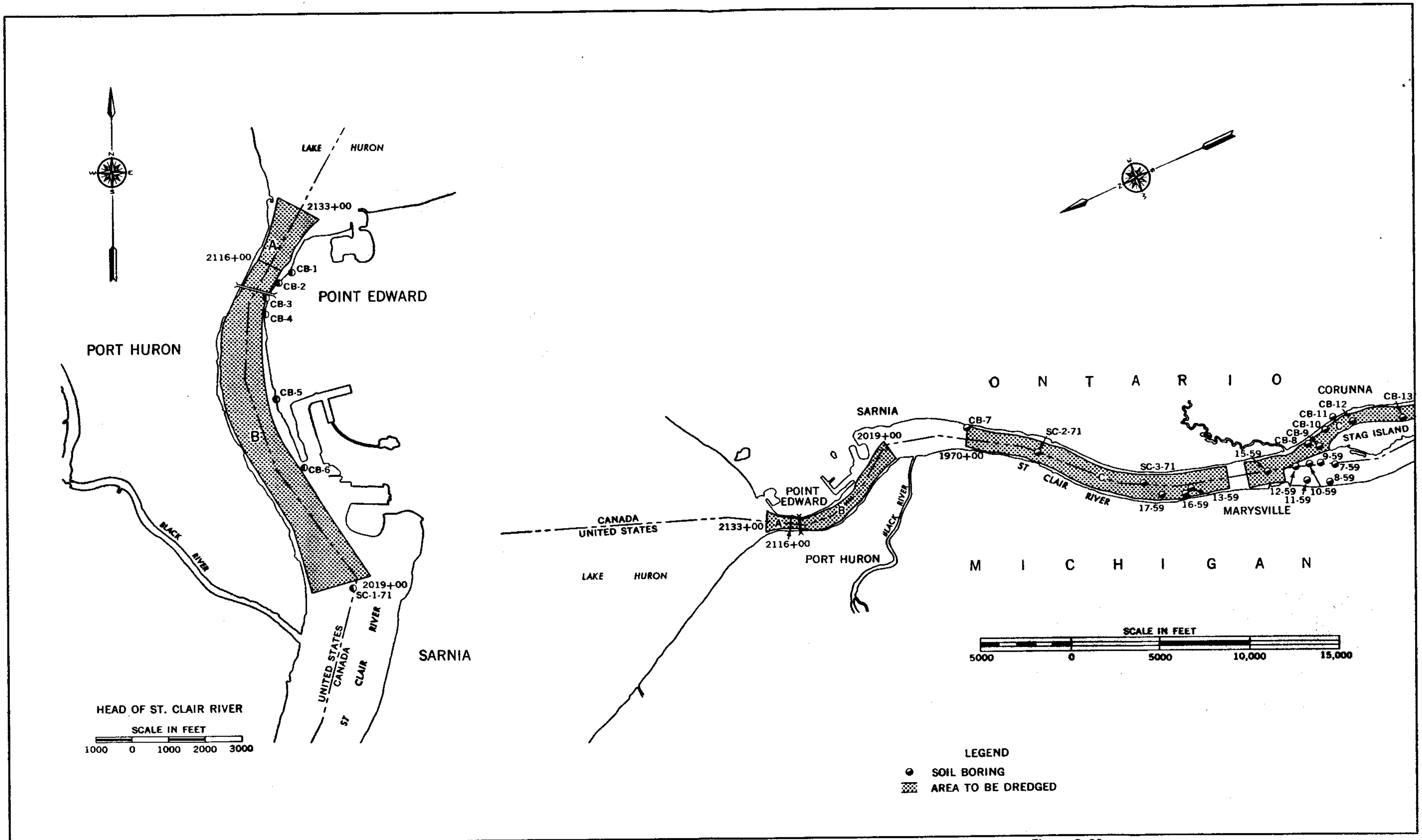


Figure G-28a
LOCATION OF PROPOSED DREDGING IN ST. CLAIR RIVER—MAXIMUM CHANNEL CAPACITY INCREASE

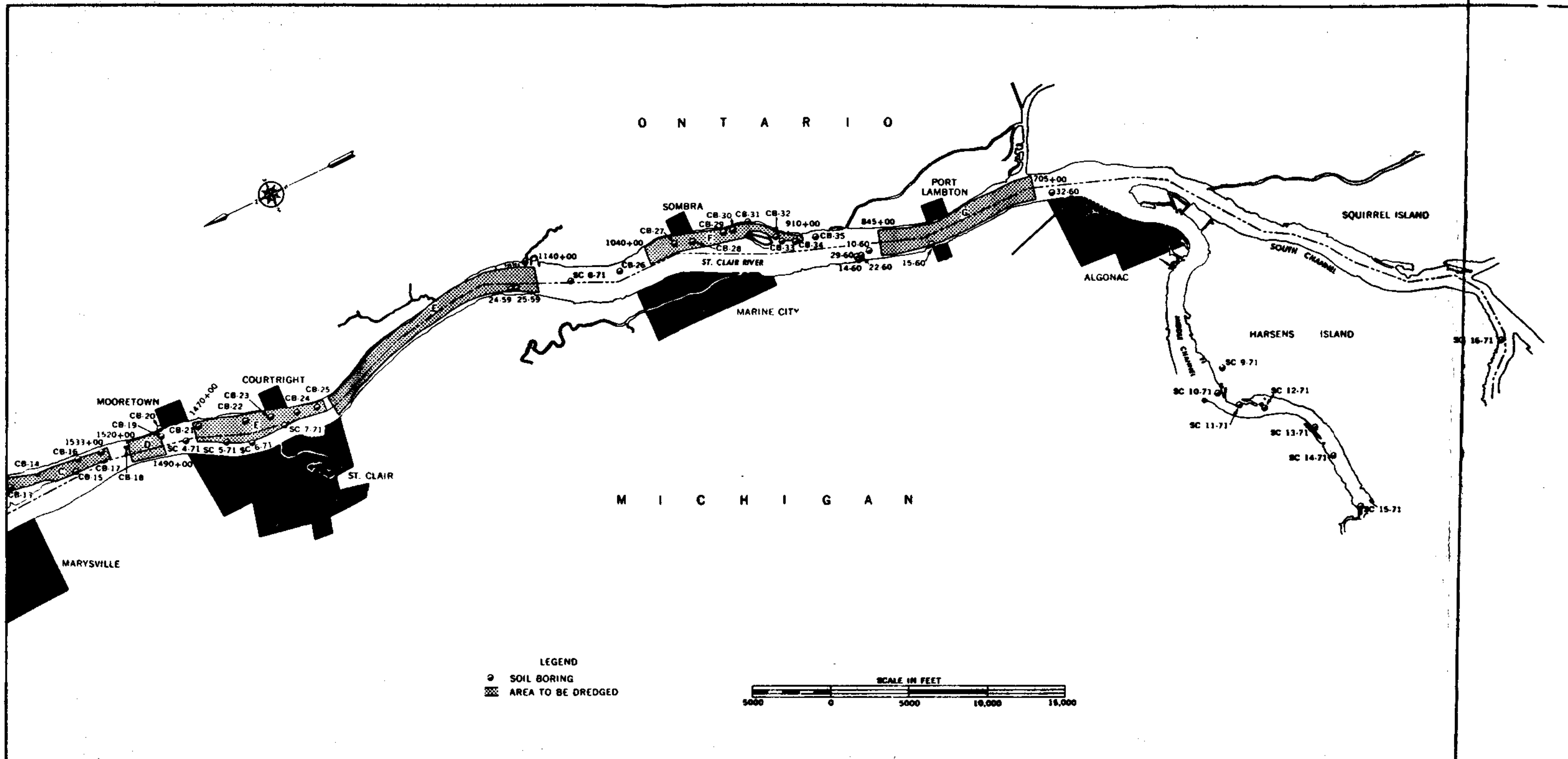


Figure G-28b
LOCATION OF PROPOSED DREDGING IN ST. CLAIR RIVER—MAXIMUM CHANNEL CAPACITY INCREASE

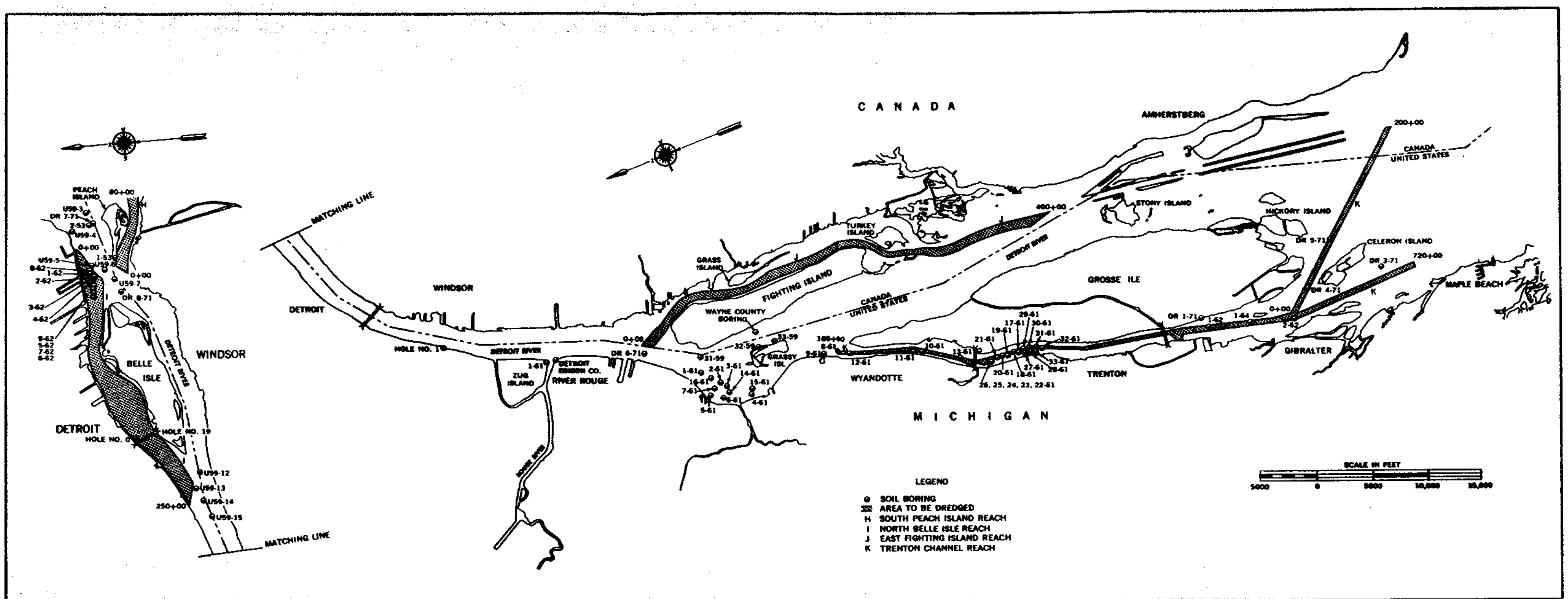


Figure G-29
LOCATION OF PROPOSED DREDGING IN DETROIT RIVER— MAXIMUM CHANNEL CAPACITY INCREASE

The entire widths of the designated reaches would not necessarily be dredged since the dredging was limited to the amount of change required in the cross section in order to obtain the required channel increase. The dredged areas, coded alphabetically on the aforementioned figures, are tabulated on Figure G-30a and G-30b, showing the volume and type of material.

3.3.11 Channelization Requirements

Dredging requirements, disposal and related problems in the St. Clair-Detroit River system were investigated. The principal features examined included the following:

1. The methods of dredging and dredged-material disposal with the emphasis on the environment

2. Cost estimates for the required dredging

3. Cost estimates for the required utility relocation

An abstract of a report on this subject is contained in Annex C to this Appendix. Figure G-31 shows the location of possible disposal areas for the material to be dredged from the St. Clair River. Figure G-32 shows the proposed locations for the Detroit River disposal areas. Not all of the dredged material will require placement in the disposal areas, since some of the material, after removal of the polluted overburden, could be utilized for the construction of training dikes associated with the regulating structures. The quantities and distribution of dredged material to the disposal areas and for use in the training dikes is shown on Figure G-33 for the St. Clair River and Figure G-34 for the Detroit River.

The cost estimates, based on unit costs developed for the St. Clair-Detroit Rivers, are detailed on Figures G-35 and G-36, respectively. The total cost of dredging 28.7 million cubic yards and utility relocations for the St. Clair River would approximate $176 million including an allowance of 25% for contingencies and further engineering, design and administration. The cost estimates for the Detroit River for dredging 36.8 million cubic yards and utility relocations is approximately $375 million, which also includes a similar allowance of 25% for contingencies and further engineering design and administration. The above costs are based on 1971 price levels.

With respect to channel dredging, two main factors that require further study in order to minimize any potentially undesirable effects on the environment are:

1. Design and operation of the dredging procedures to avoid unnecessary redistribution of contaminated sediments

2. Disposal and/or treatment of dredged material to render it non-noxious to its adjacent environment

Figure G-30A

SUMMARY OF QUANTITIES AND TYPES OF MATERIALS TO BE DREDGED FROM THE ST. CLAIR RIVER - MAXIMUM CHANNEL CAPACITY INCREASE

Dredged Area	Location	Quantity (Bank Measure)	Type of Material*	Boreholes
A	C.S.2133+00 - C.S.2116+00	319,000 yd^3	Dense sandy gravel, Some boulders	CB-3, CB-5
B	C.S.2116+00 - C.S.2019+00	2,157,100 yd^3	Dense Sand, Gravel, Silt, Clay	CB-6, CB-7, SC1-71
C	C.S.1970+00 - C.S.1533+00	7,701,000 yd^3	Silty Clay, Sand Gravel	CB-7 thru CB-17 except CB-11; SC2-71, SC3-71, 10-59, 12-59, 13-59, 15-59, 16-59, 17-59
D	C.S.1520+00 - C.S.1490+00	279,900 yd^3	Clay, Silty Clay	CB-18, CB-19, CB-21, SC4-71
E	C.S.1470+00 - C.S.1140+00	10,825,500 yd^3	Clay, Silty Clay, Sand, Gravel	CB-21 thru CB-25: SC4-71 thru SC8-71, 24-59, 25-59
F	C.S.1040+00 - C.S. 910+00	5,362,500 yd^3	Silty Clay, Sand, Gravel	CB-26 thru CB-35
G	C.S. 845+00 - C.S. 705+00	2,091,200 yd^3	Soft Clay	10-60, 15-60, 32-60

*In order of decreasing proportion

FIGURE G-30B

SUMMARY OF QUANTITIES AND TYPES OF MATERIALS TO BE DREDGED FROM THE DETROIT RIVER - MAXIMUM CHANNEL CAPACITY INCREASE

Dredged Area	Location	Quantity (Bank Measure)	Type of Material*	Boreholes
H	South Peach Island Reach	4,370,000 yd^3	Sand, Clay, Gravel, Silt	DR8-71, U59-7
I	North Belle Isle Reach	16,079,000 yd^3	Sand, Sandy Gravel, Clay, Silt	No. 0, No. 19 (City of Detroit, 1920) DR8-71, U59-5, U59-6, U59-12, U59-13, U59-14, U59-15, 1-62 through 9-62
J	East Fighting Island Reach	7,390,000 yd^3	Sandy Clay, Silt, Gravel, Limestone	DR6-71, 31-59 through 34-59, Wayne County Boring
K	Trenton Channel Reach	8,490,000 yd^3	Limestone, Broken Limestone, Sand, Clay, Gravel, Silt	1-38, 2-38, 3-38, 6-38, 7-38, 8-61, 10-61, 11-61, 12-61, 13-61, 17-61 through 33-61, DR3-71, DR4-71, DR5-71, 1-62, 2-62, 1-64

*In order of decreasing proportion

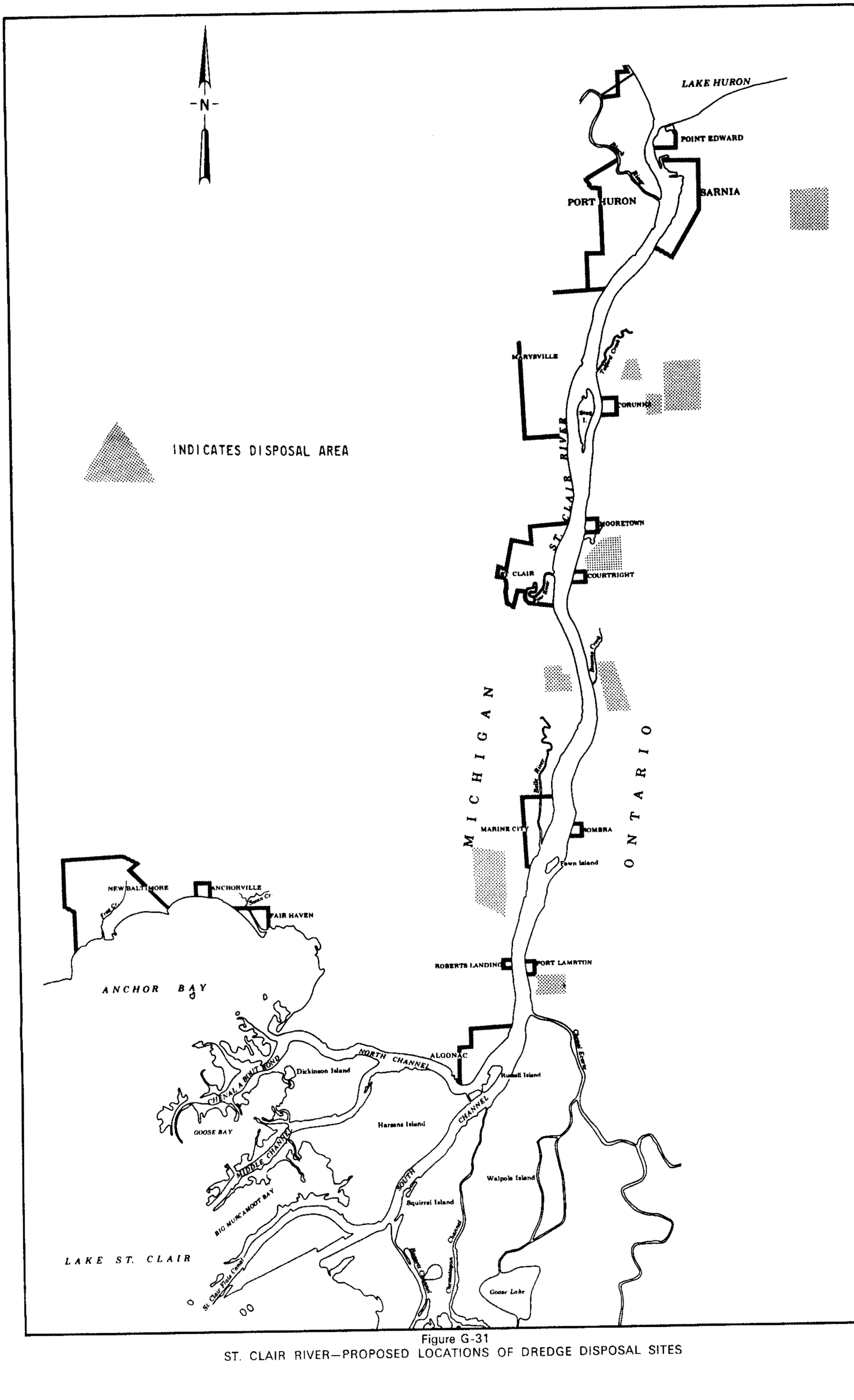


Figure G-31
ST. CLAIR RIVER—PROPOSED LOCATIONS OF DREDGE DISPOSAL SITES

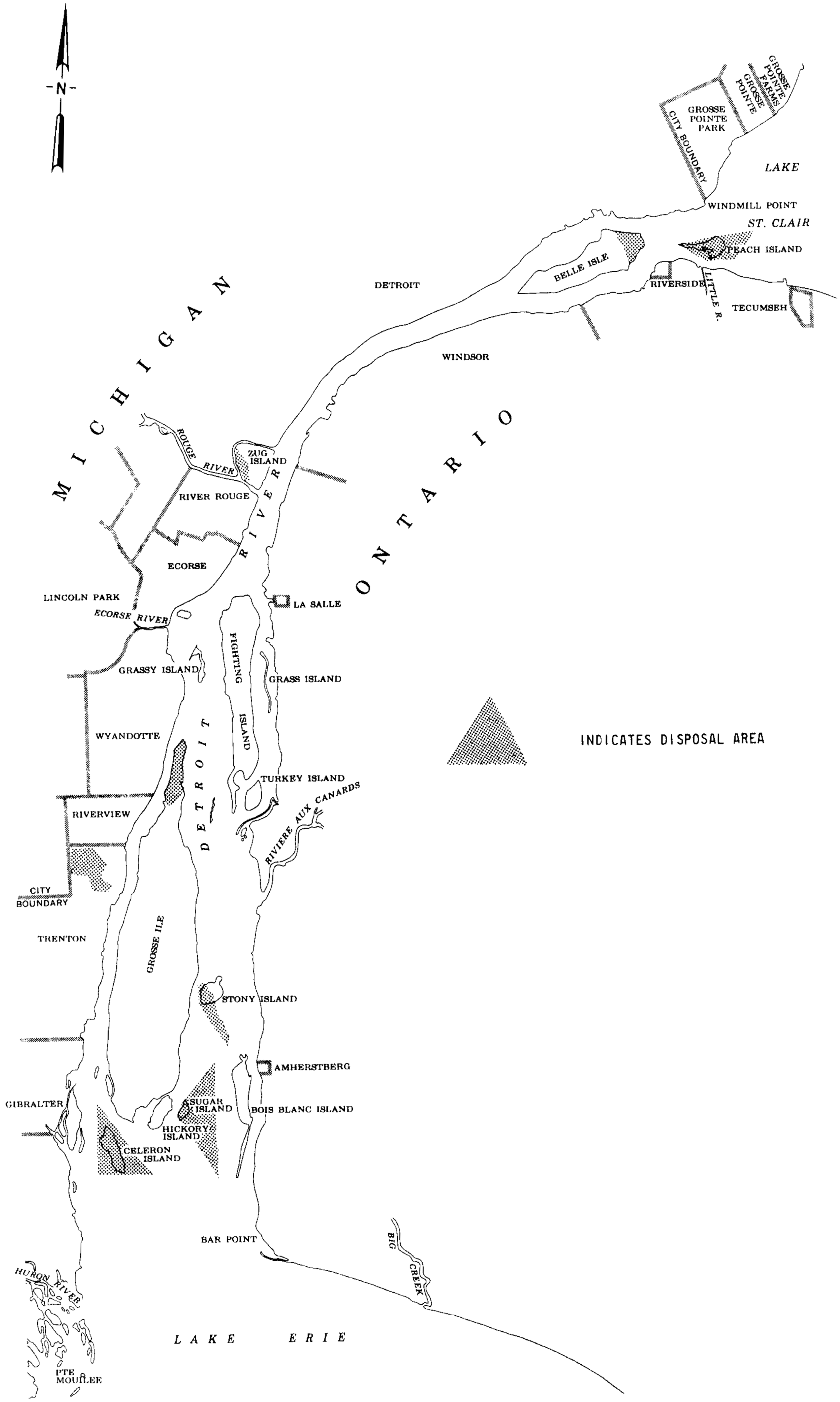


Figure G-32
DETROIT RIVER—PROPOSED LOCATIONS OF DREDGE DISPOSAL SITES

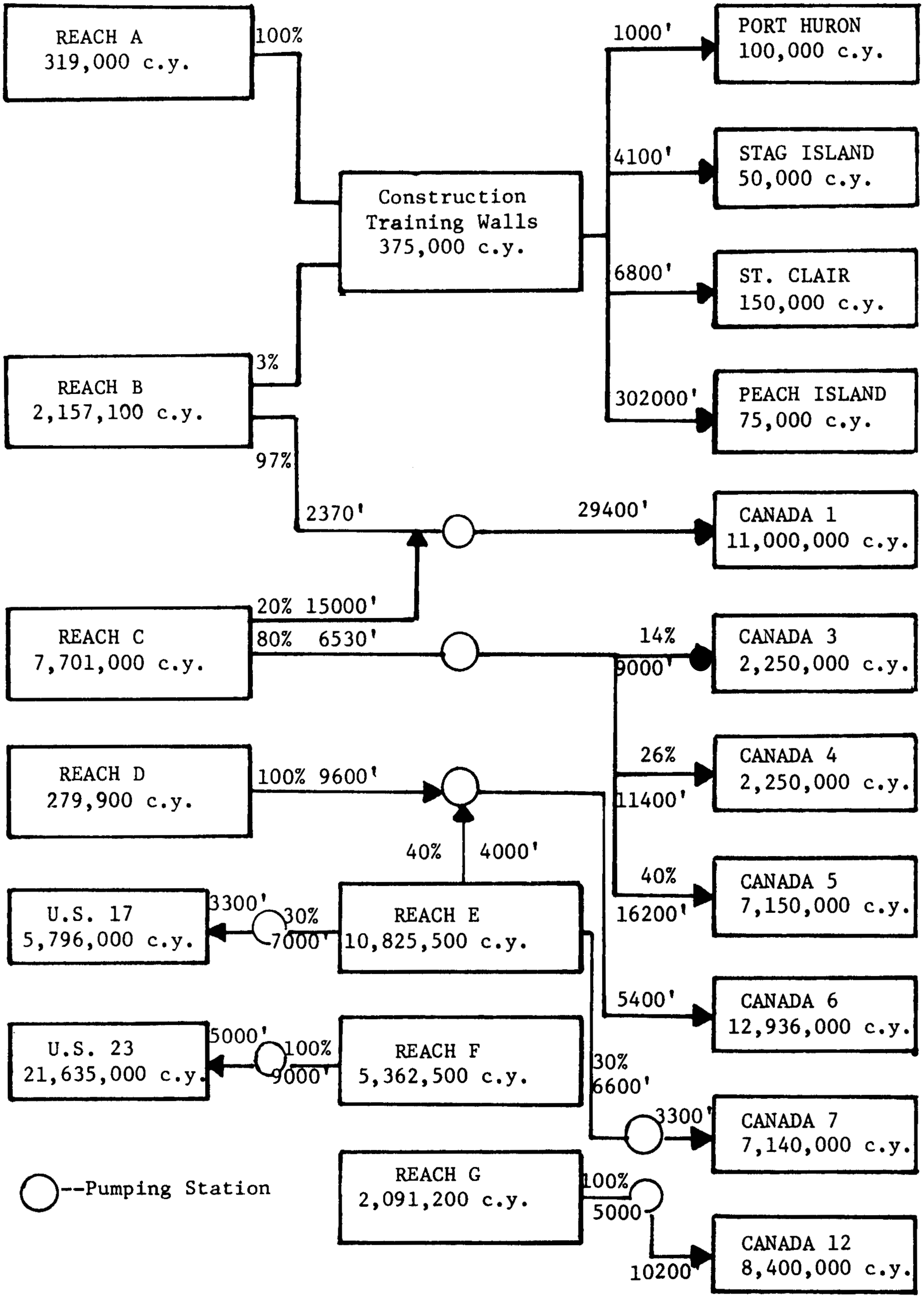


FIGURE G-33
ST. CLAIR RIVER - PROPOSED ALLOCATION OF DREDGE SPOIL
MAXIMUM CHANNEL CAPACITY INCREASE

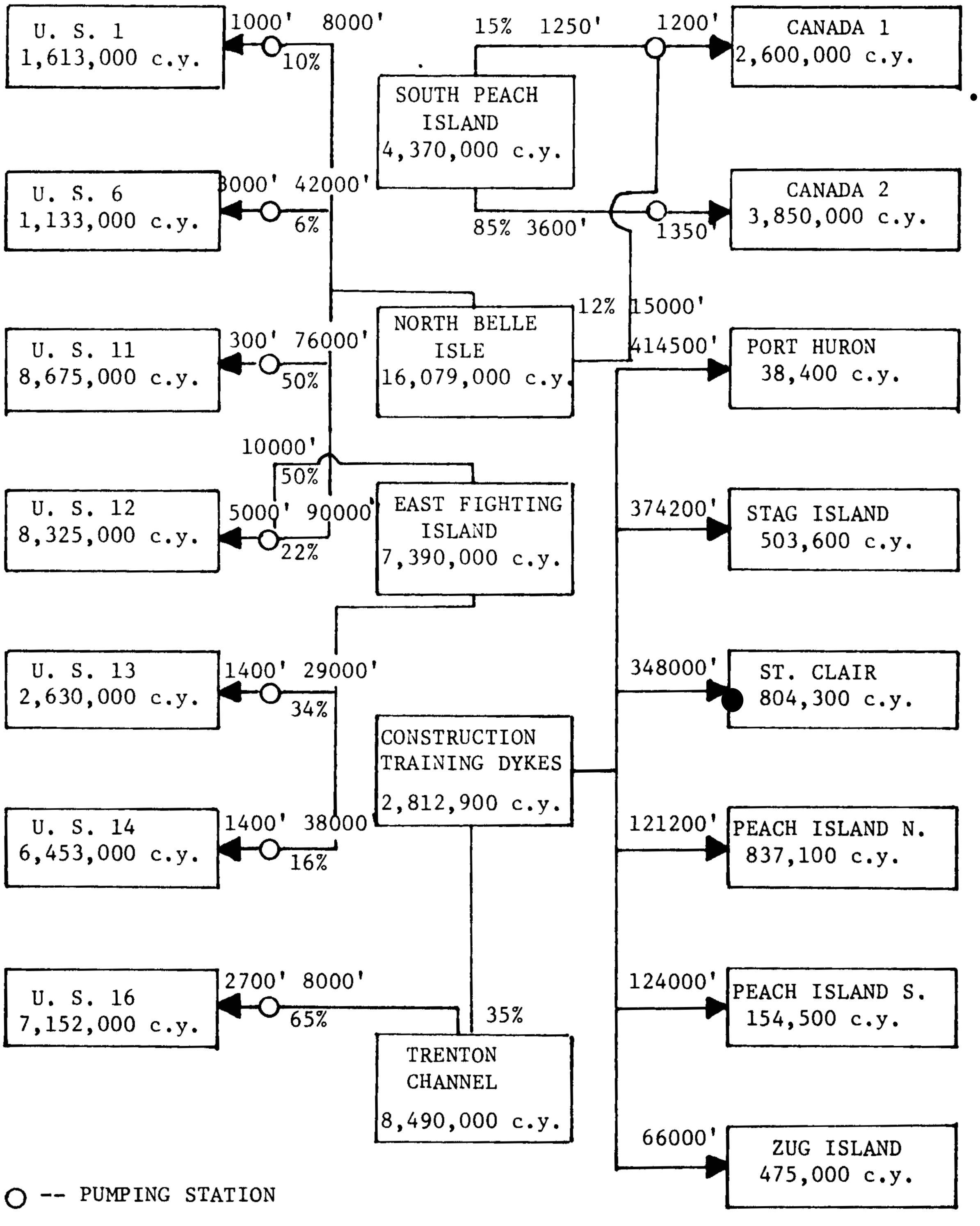


FIGURE G-34
DETROIT RIVER - PROPOSED ALLOCATION OF DREDGE SPOIL
MAXIMUM CHANNEL CAPACITY INCREASE

	ITEM	QUANTITY	UNIT	UNIT PRICE	ESTIMATED COST
1.	Dredging				
	Area A	75,000	yd^3	$ 4.00	300,000
		244,000	yd^3	$ 2.50	610,000
	Area B	2,157,100	yd^3	$ 3.50	7,549,850
	Area C	1,540,200	yd^3	$ 3.50	5,390,700
		6,160,800	yd^3	$ 3.00	18,482,400
	Area D	279,900	yd^3	$ 3.00	839,700
	Area E	10,825,500	yd^3	$ 3.00	32,476,500
	Area F	5,362,500	yd^3	$ 3.00	16,087,500
	Area G	2,091,200	yd^3	$ 3.00	6,273,600
2.	Earth Dykes				
	Canada 1	627,000	yd^3	$ 3.00	1,881,000
	Canada 3	272,000	yd^3	$ 3.00	816,000
	Canada 4	283,000	yd^3	$ 3.00	849,000
	Canada 5	200,000	yd^3	$ 3.00	600,000
	Canada 6	653,000	yd^3	$ 3.00	1,959,000
	Canada 7	584,000	yd^3	$ 3.00	1,752,000
	Canada 12	544,000	yd^3	$ 3.00	1,632,000
	U.S. 17	224,000	yd^3	$ 3.00	672,000
	U.S. 23	400,000	yd^3	$ 3.00	1,200,000
3.	Striping	4,858	Acre	$1,500.00	7,287,000
4.	Clearing	4,858	Acre	$1,000.00	4,858,000
5.	Seeding & Mulching	4,858	Acre	$1,000.00	4,858,000
6.	Trees & Shrubs	4,858	Acre	$1,000.00	4,858,000
7.	Pumpout Facilities				
	Pipelines & Trestles	31,400	L.F.	$ 30.00	942,000
	Oil Skimmers	9	Each	$5,000.00	45,000
	Weirs	18	Each	$7,500.00	135,000
8.	Mooring Facilities (Pile Clusters)	18	Each	$25,000.00	450,000
9.	Real Estate	4858	Acre	$ 1,000.00	4,858,000
10.	Utilities				
	Intakes	19	Each	$77,000.00	1,463,000
	Outfalls	10	Each	$35,000.00	350,000
	Pipe Crossings	1	Each	122,000.00	122,000
	Cable Crossings	5	Each	100,000.00	500,000
	Steel Sheet Piling	3500	L.F.	$ 3,000.00	10,500,000
				Sub-Total	$140,597,250
				25% Contingency	$ 35,149,313
				TOTAL	$175,746,563

Figure G-35
ST. CLAIR RIVER
ESTIMATED COST OF DREDGING
MAXIMUM CHANNEL CAPACITY INCREASE

	ITEM	QUANTITY	UNIT	UNIT PRICE	ESTIMATED COST
1.	Dredging				
	S. Peach Island	4,370,000	yd^3	$ 2.50	10,925,000
	N. Belle Island	1,930,000	yd^3	$ 3.00	5,790,000
		1,607,900	yd^3	$ 3.00	4,823,700
		8,039,500	yd^3	$ 3.50	28,138,250
		964,740	yd^3	$ 3.50	3,376,590
		3,536,860	yd^3	$ 4.00	14,147,440
	E. Fighting Island	5,490,000	yd^3	$ 3.00	16,470,000
		1,900,000	yd^3	$ 30.00	57,000,000
	Trenton Channel	4,490,000	yd^3	$ 3.00	13,470,000
		4,000,000	yd^3	$ 30.00	120,000,000
2.	Earth Dykes				
	Canada 1	197,200	yd^3	$ 6.00	1,183,200
	Canada 2	437,000	yd^3	$ 6.00	2,622,000
	U.S. 1	154,000	yd^3	$ 3.00	462,000
	U.S. 6	154,000	yd^3	$ 3.00	462,000
	U.S. 11	1,640,000	yd^3	$ 3.00	4,920,000
	U.S. 12		yd^3	$ 3.00	
	U.S. 13	270,000	yd^3	$ 6.00	1,620,000
	U.S./Canada 14	198,000	yd^3	$ 6.00	1,188,000
	U.S. 16	570,000	yd^3	$ 6.00	3,420,000
3.	Stripping	623	Acre	$ 1,500.00	934,500
4.	Clearing	623	Acre	$ 1,000.00	623,000
5.	Seeding & Mulching	1,552	Acre	$ 1,000.00	1,552,000
6.	Trees & Shrubs	1,552	Acre	$ 1,000.00	1,552,000
7.	Pumpout Facilities				
	Pipeline & Trestles	7,000	L.F.	$ 30.00	210,000
	Oil Skimmers	9	Each	$ 5,000.00	45,000
	Weirs	18	Each	$ 7,500.00	135,000
8.	Mooring Facility	18	Each	$ 50,000.00	900,000
9.	Real Estate	623	Acre	$ 2,000.00	1,246,000
10.	Utilities				
	Pipe Crossing	8	Each	$122,000.00	976,000
	Cable Crossing	10	Each	$100,000.00	1,000,000
	Intakes	5	Each	$ 77,000.00	385,000
	Outfalls		Each	$ 35,000.00	
				Sub-Total	$299,576,680
				25% Contingency	$74,894,170
				TOTAL	$374,470,850

FIGURE G-36
DETROIT RIVER
ESTIMATED COSTS OF DREDGING
MAXIMUM CHANNEL CAPACITY INCREASE

3.3.12 Channel Capacity Increase Cost Curves

Utilizing the costs as presented above, the volumes of material to be dredged for the various channel capacity increases, shown on Figures G-26 and G-27, were converted to costs in dollars. The conversion into channel capacity increase cost curves for the St. Clair and Detroit Rivers is shown on Figures G-37 and G-38, respectively. These cost curves, based on the result of recorded abnormal lake levels, were designed for determining the costs of Lakes Michigan-Huron regulation plans. These curves, applicable to the 1933 regimen conditions, contain the "built-in" channel capacity increase resulting from the 25- and 27-foot navigational improvements.

3.3.13 Application of Mathematical Model in Channel Design, Channel Capacity Decrease

Initial tests using the mathematical models were conducted for the purpose of determining the most effective locations of structures. The structure locations tested included those determined in the U. S. Army Corps of Engineers report "Water Levels of the Great Lakes, Appendix "F", dated December 1965. However, in the evaluation of the retarding effect of each structure in an accumulative downstream order, the Fawn Island structure was eliminated due to the relatively small hydraulic effect exerted by this structure. The proposed locations for the St. Clair River Control structures are shown on Figure G-39. These are:

1. Port Huron
2. Stag Island
3. St. Clair
4. North and Middle Channels

The proposed locations for the Detroit River structures are shown on Figure G-39. These are:

1. Head of Detroit River, East and West of Peach Island
2. Belle Isle
3. Zug Island
4. East Fighting Island (Grassy Island)
5. Trenton Channel

3.3.14 Hydraulic Design Characteristics

Following the determination of the optimum location of the structures, the St. Clair and Detroit River models were operated for determining the hydraulic characteristics of the structures required for selected amounts of channel capacity decreases, with no channel capacity increase and with the maximum channel capacity increase. For purposes of the design of the structures and their costs, the results of the maximum channel capacity decrease in combination with the maximum channel capacity increase were extracted from the mathematical model output. For conditions with all gates closed, the data provided were as follows:

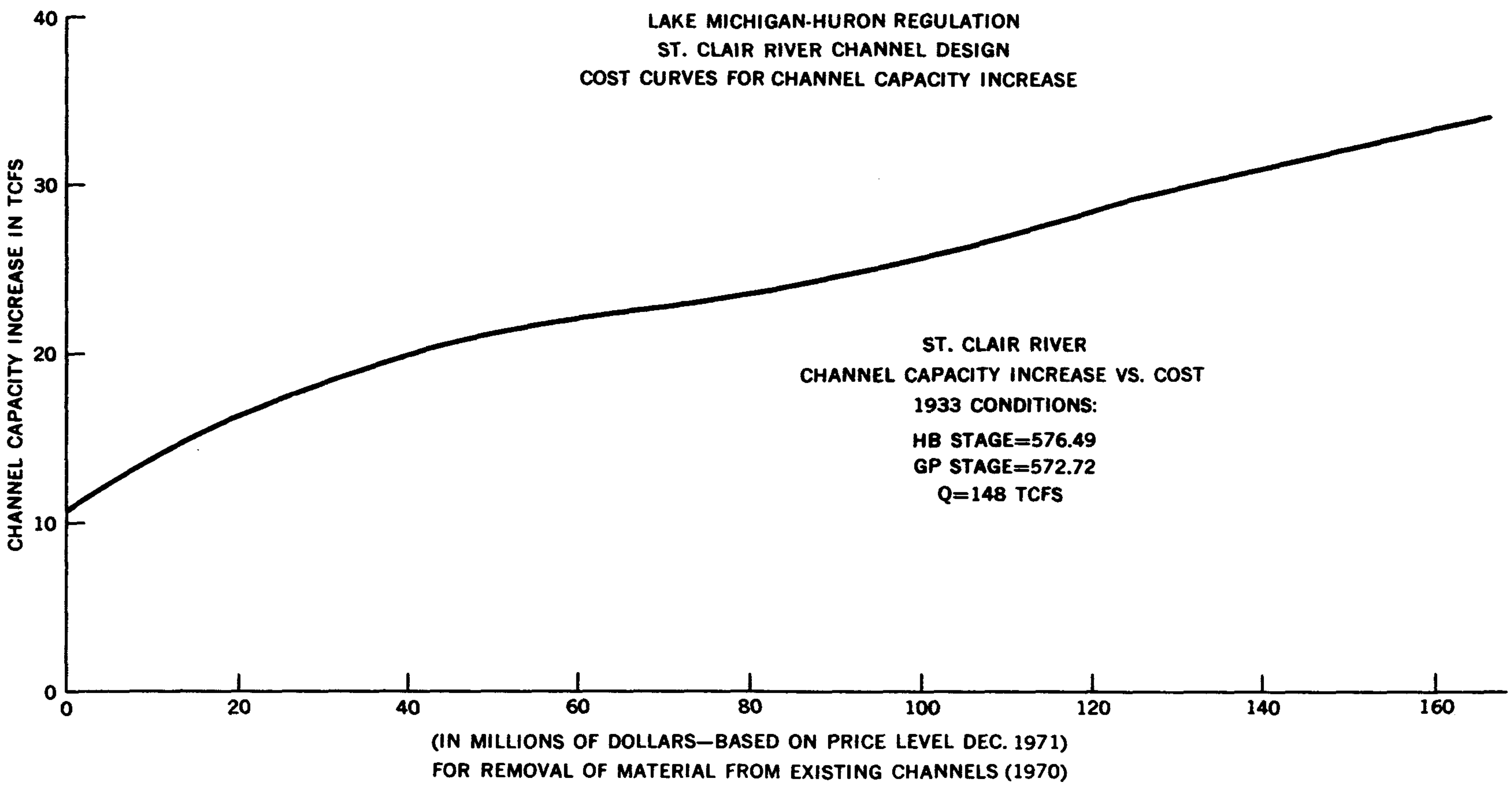


Figure G-37
ST. CLAIR RIVER—CHANNEL CAPACITY INCREASE COST RELATIONSHIP

DETROIT RIVER CHANNEL DESIGN
COST CURVES FOR CHANNEL CAPACITY INCREASES

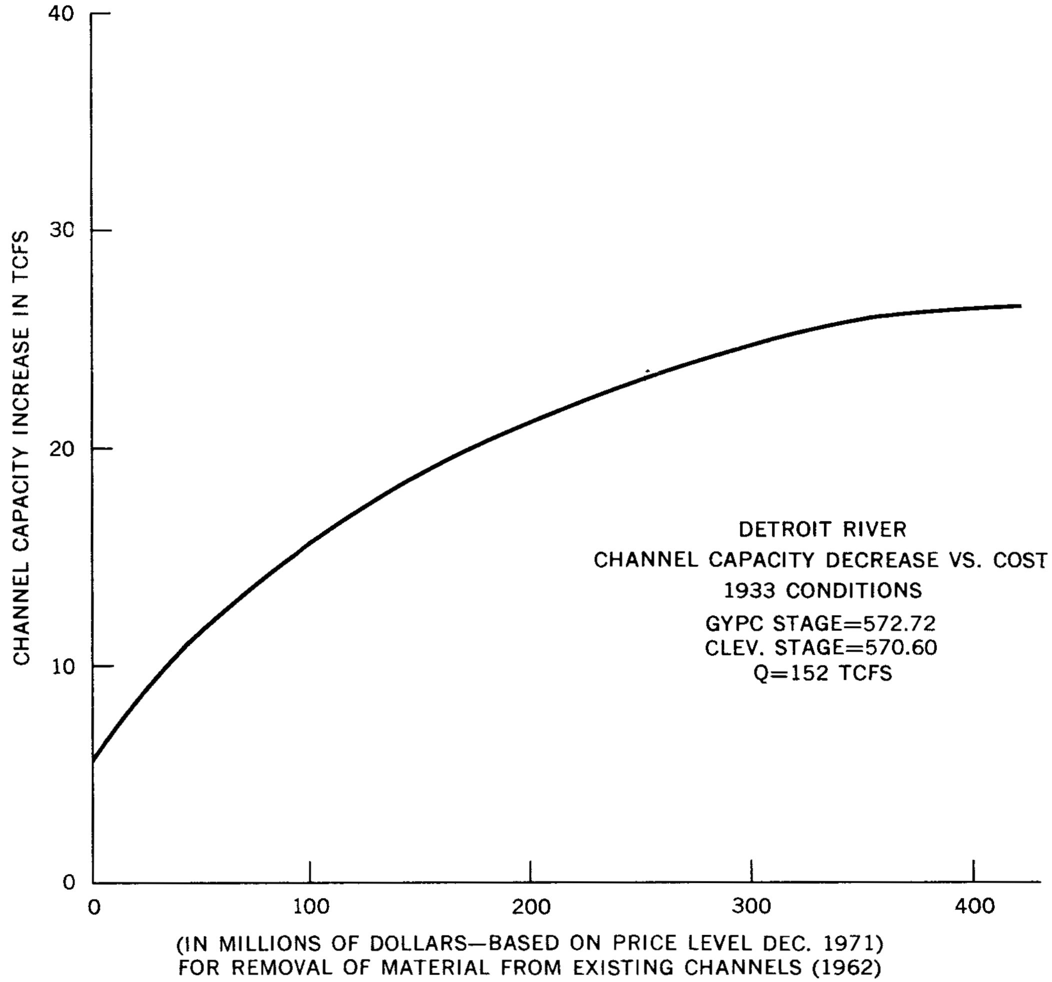


Figure G-38
ST. CLAIR RIVER—CHANNEL CAPACITY INCREASE COST RELATIONSHIP

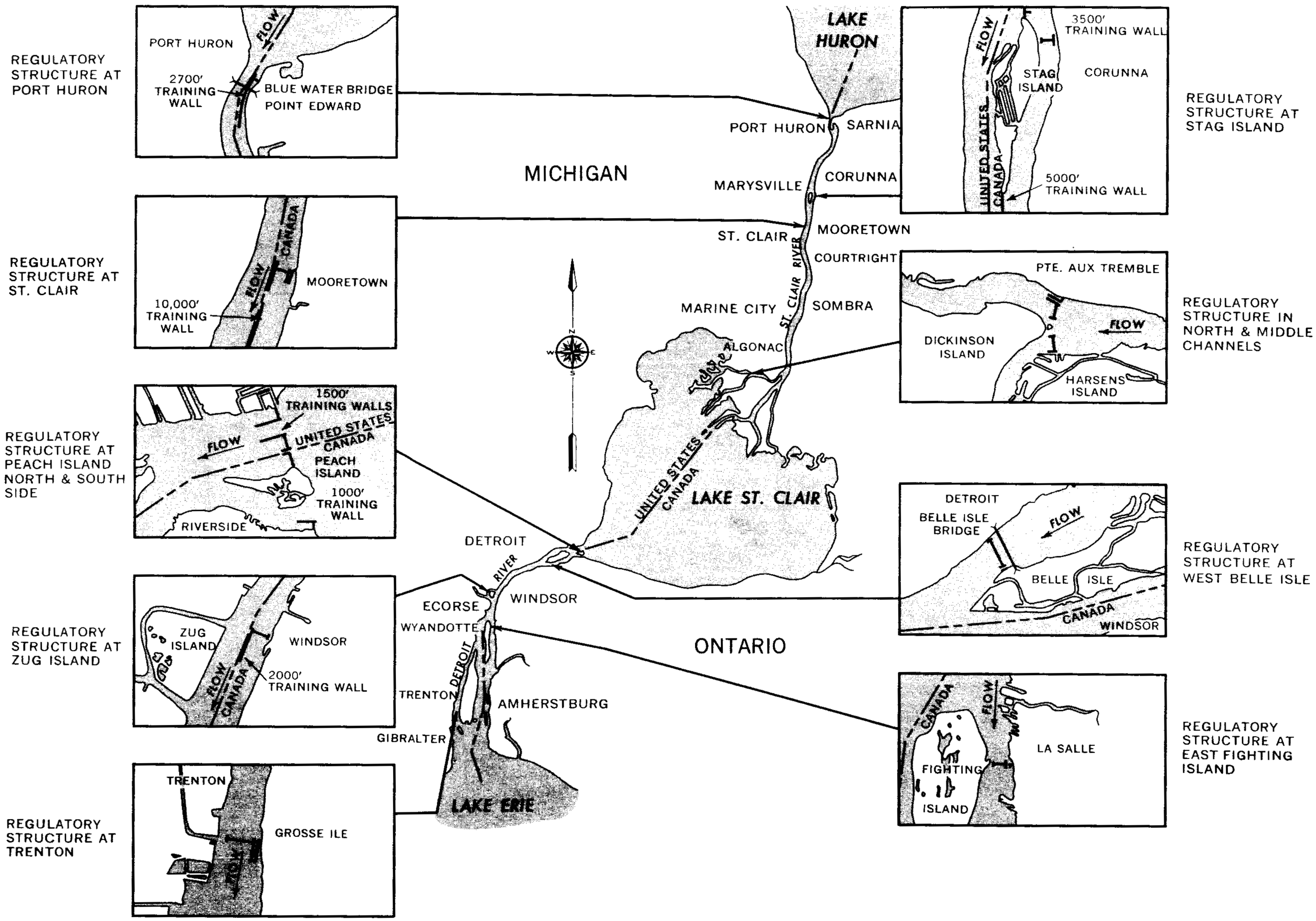


Figure G-39
LOCATION OF PROPOSED REGULATORY STRUCTURES—ST. CLAIR & DETROIT RIVERS

1. Location of proposed control structure
2. Flow around end of structure (ungated portion of channel)
3. Water surface elevation downstream of structure
4. Water surface elevation upstream of structure
5. Downstream velocity at end of structure
6. Upstream velocity at end of structure
7. Velocity at the end of structure
8. Length of ungated structure
9. Channel depth at structure below LWD
10. Sill elevation
11. Total length of training wall

The data obtained for this maximum design condition for channel capacity decrease for the St. Clair and Detroit Rivers are shown on Figures G-40 and G-41, respectively. Examination of the data indicates that the maximum head to be maintained is 1.3 feet at the St. Clair structure location. Also of importance is that substantial amounts of flow would be allowed to pass around the ends of the structure under this severe assumed regulated condition. These flows are in addition to the 2,000 cfs allowed to pass through the structure, with all gates closed, for flushing purposes. The comparisons between the resulting design flows around the end of the structures with flows that would occur under natural channel conditions for the same total river flows are shown on Figures G-42 and G-43. It should be reiterated in this section that the design condition described would, under test regulation plans, occur with a frequency of less than 1%. It should also be remembered that this maximum in design condition may be diminished in degree dependent on the channel requirement for regulation.

3.3.15 Conceptual Designs and Cost Estimates of Regulatory Structures

The necessary engineering studies to prepare structure designs and cost estimates for regulatory structures in the St. Clair-Detroit River system were provided by the firm of Acres Consulting Services Limited, Niagara Falls, Ontario. Terms of reference for the study and concepts were provided in a document entitled "Regulatory Structures on the St. Clair-Detroit Rivers, Development of Conceptual Design, Terms of Reference", an abstract of which is contained in Annex C of this Appendix.

The study was presented in two volumes. Volume I, entitled "Conceptual Design and Estimates", contains the layouts, conceptual designs and cost estimates for the regulatory structures and their associated works, such as the small boat passage, shore protection works, etc., at each of the nine sites. Volume II, entitled "Criteria and Selection of Elements" presents factors and criteria that govern the design of the parts or elements of the structures. Various concepts of the elements of the structures are described, and background environmental considerations and construction methods that relate to the concepts are referred to. Abstracts of the two volumes are provided in Annex C of this Appendix.

Each site and structure shown on Figure G-39 is described in a separate chapter of the consultants report (Volume I) which comprises a summary of the

FIGURE G-40

ST. CLAIR RIVER - HYDRAULIC DESIGN DATA FOR PROPOSED REGULATORY STRUCTURES - MAXIMUM CHANNEL CAPACITY DECREASE

LOCATION OF PROPOSED CONTROL STRUCTURE	GATED STRUCTURE COMPLETELY CLOSED[1]						LENGTH OF UNGATED STRUCTURE[3]	DEPTH AT STRUCTURE (FT. BELOW L. W. D.)	SILL ELEVATION (FT. IGLD)	TOTAL LENGTH OF TRAINING WALL (S) FEET
	FLOW AROUND END OF STRUCTURE (CFS)	ELEV. ACROSS STRUCT.		MEAN VELOCITY PASSING STRUCTURE						
		DOWNSTREAM ELEVATION (FT.)	UPSTREAM ELEVATION (FT.)	DOWNSTREAM VELOCITY (FT. SEC.)	VEL. AT STRUCTURE (FT. SEC.)	UPSTREAM VELOCITY (FT. SEC.)				
Port Huron	182,200[2]	580.17	580.66	4.3	7.4	5.0	800	36.0	540.2	2,700
Stag Island	42,900	578.68	579.35	1.6	6.8	1.6	150	36.0	538.7	8,500
St. Clair	182,200[2]	577.27	578.57	3.1	6.3	3.0	1,100	36.0	538.2	10,000
North Channel	29,200	575.78	576.40	0.7	6.8	0.6	100	Natural	Natural	None
Middle Channel	30,300	575.74	576.39	1.4	7.0	1.3	100	Natural	Natural	None

NOTES:

1. Maximum design conditions; Lake Michigan-Huron = 580.76, G.P.Y.C. = 575.41, Flow = 182,200 CFS.
2. Except 2,000 CFS through control structure for continuous flushing purposes.
3. Length of opening to be provided to pass flow around structure as indicated in Column 2.

FIGURE G-41

DETROIT RIVER - HYDRAULIC DESIGN DATA FOR PROPOSED REGULATORY STRUCTURES - MAXIMUM CHANNEL CAPACITY DECREASE

LOCATION OF PROPOSED CONTROL STRUCTURE	GATED STRUCTURE COMPLETELY CLOSED[1]						LENGTH OF UNGATED STRUCTURE[3]	DEPTH AT STRUCTURE (FT. BELOW L. W. D.)	SILL ELEVATION (FT. IGLD)	TOTAL LENGTH OF TRAINING WALL (S) FEET
	FLOW AROUND END OF STRUCTURE (CFS)	ELEV. ACROSS STRUCT.		MEAN VELOCITY PASSING STRUCTURE						
		DOWNSTREAM ELEVATION (FT.)	UPSTREAM ELEVATION (FT.)	DOWNSTREAM VELOCITY (FT. SEC.)	VEL. AT STRUCTURE (FT. SEC.)	UPSTREAM VELOCITY (FT. SEC.)				
Trenton Channel	36,400	572.50	573.36	1.2	9.7	2.0	300	32.0	536.8	None
E. Fighting Isl. (Grass Island)	19,000	573.47	574.01	0.5	8.1	0.5	65	32.0	538.4	None
Zug Island	190,000[2]	574.08	574.25	2.0	3.9	2.2	1300	Natural	Natural	2,000
Belle Isle	15,900	574.61	575.01	0.4	6.9	0.4	65	32.0	539.4	None
Head of Detroit River (N. Side Peach Island)	136,300	575.04	575.37	1.8	5.4	1.5	800	Natural	Natural	3,000
Head of Detroit River (S. Side Peach Island)	58,700	575.03	575.39	2.4	4.1	1.1	400	Natural	Natural	1,000

NOTES:

1. Maximum design conditions: Lake Erie Stage = 572.38, Lake St. Clair Stage = 575.41, Flow = 190,000 CFS.
2. Except 2,000 CFS through control structure for flushing purposes.
3. Length of opening to be provided to pass flow around structure as indicated in Column 2.

FIGURE G-42

ST. CLAIR RIVER

COMPARISONS BETWEEN NATURAL AND REGULATED FLOW DISTRIBUTIONS UNDER MAXIMUM CHANNEL CAPACITY DECREASE CONDITIONS

DESIGN FLOW = 182,000 cfs

LOCATION OF PROPOSED CONTROL STRUCTURE	NATURAL CHANNEL FLOW (CFS)	FLOW AROUND END OF STRUCTURE(CFS) (GATES CLOSED)	CHANGE IN FLOW (CFS)
PORT HURON	182,200	182,200[1]	- - -
STAG ISLAND	72,900	42,900	-30,000
ST. CLAIR	182,200	182,200[1]	- - -
NORTH CHANNEL	60,100	29,200	-30,900
MIDDLE CHANNEL	36,400	30,000	- 6,100

NOTE:
1. EXCEPT 2,000 C.F.S. THROUGH CONTROL STRUCTURE FOR CONTINUOUS FLUSHING PURPOSES.

FIGURE G-43

DETROIT RIVER

COMPARISONS BETWEEN NATURAL AND REGULATED FLOW DISTRIBUTIONS UNDER MAXIMUM CHANNEL CAPACITY DECREASE CONDITIONS

DESIGN FLOW = 190,000 cfs

LOCATION OF PROPOSED CONTROL STRUCTURE	NATURAL CHANNEL FLOW (CFS)	FLOW AROUND END OF STRUCTURE(CFS) (GATES CLOSED)	CHANGE IN FLOW (CFS)
TRENTON CHANNEL	39,900	36,400	-3,500
GRASS ISLAND	43,700	19,000	-24,700
ZUG ISLAND	190,000	190,000[1]	----
BELLE ISLE	13,500	15,900	+2,400
N. SIDE PEACH ISLAND	146,300	136,300	-10,000
S. SIDE PEACH ISLAND	43,700	58,700	+15,000

NOTE:
1. EXCEPT 2,000 C.F.S. THROUGH CONTROL STRUCTURE FOR CONTINUOUS FLUSHING PURPOSES.

principal features of the hydraulics and geology, the layout of the structures and the detail to which they have been taken to permit the conceptual estimates which were prepared. Cost estimates were given for the fully developed structure, and in successive reductions to 50% of the basic length determined for the maximum design condition. Durations and sequences of construction for each structure were estimated.

Standardization of the design of the elements of similar functions of the nine sites were taken as an important factor in reducing costs of construction and maintenance. The format will provide the most convenient base for future work and cost estimating.

The structures were tentatively located within certain limits, but latitude was given in selecting the exact location of the structures. Following an aerial inspection and a site visit, together with study of aerial photographs, structures at East Fighting Island, Zug Island and Port Huron were relocated within the given tolerance.

The criteria relating to the proposed structures in the St. Clair and Detroit Rivers were provided in the terms of reference together with associated data. Geotechnical information on which the concepts are based is an interpretation of the results of exploratory work conducted in the rivers by the U. S. Army Corps of Engineers and by consultants for Environment Canada.

As the regulatory works involve nine structures that have a similar function, standardization of the elements common to the sites, methods of construction and means of maintenance were taken as primary requirements. The common principal elements, such as the gate, concrete support structure, small boat passage and training wall were studied separately. After refinement, they were adapted to each other and to the various sites. These selected principal elements are discussed in the following paragraphs. A singular feature of the sites is the low hydraulic head across the structures which does not exceed 16 inches.

The Effects of the Proposed Structures on the Ice Regime: The structures which are proposed for regulation of flows in these rivers will have little effect on ice movements when all gates are open. When gates are closed, however, the ability of the river surface to convey ice pieces through various reaches will be altered. The changes may affect the basic mechanisms of ice jam formation and may result in ice jams forming at sites different from those where they occur under present conditions. If the gates of the proposed structures are closed, they will shelter some reaches of the river and permit a stationary ice cover to freeze in place, to the benefit of some properties along the shore. However, operation of the gates during the ice season may cause some less favourable results by triggering ice runs at places and at times when they would not normally occur. For the purpose of the present investigation, it is assumed that these effects will be minor. Final design of a system of structures such as these requires an evaluation of the effects of the ice regime based on an extensive field reconnaissance.

Gate Designs: Gate operating conditions are unique in that the maximum hydraulic load is less than 1.5 feet, whereas the water depth in the channels is in the order of 40 feet at all sites. A specified ice pressure of 40 kips per lineal foot of gate was assumed. This is equivalent to a differential pressure of 16 feet acting over the total height of 40 feet, i.e., over ten times greater than the maximum hydraulic load. It is obvious that considerable savings in gate costs are possible if the ice loads can be avoided. One way to achieve this is by a design in which the gate or its top section can be arranged to submerge if excessive ice loads are applied, allowing ice to pass over the gate. Further economies can be effected if the gate system does not require piers to be massive to withstand the ice loads. These considerations lead to the concept of a simple gate design; a gate requiring minimum maintenance and one which can be readily removed and reinstalled without the necessity for cofferdams or the use of stop logs.

With a number of gates at each site, it is considered that adequate flow control can be provided with only two gate positions fully open and closed. With no intermediate positions being necessary, the required complexity of the mechanisms is greatly reduced. Since the structures are in parallel with ungated channels, it is not necessary to provide tight-fitting seals between or at the bottom of the gates. A gap of up to 4 to 9 inches around individual gates is therefore considered acceptable. This gap would provide flow through the structure at all times, thus keeping the channel free of debris. In summary, the following considerations significantly affect the selection of gate type:

(1) The gate should submerge under excessive ice loads
(2) There should be no piers between adjacent gates
(3) Dewatering should not be required for maintenance
(4) Partial gate openings are not required
(5) Watertight seals are not required

Gate Concepts: Figure G-44 shows five types of gates considered which meet all or some of the above criteria. The five types of gates were as follows:

(1) Sectional Gate
(2) Tainter Gate
(3) Fishbelly Gate
(4) Radial Gate (Submersible)
(5) Double Hinged Gate (Buoyant Flap Gate)

Due to the requirement for piers between adjacent gates and need for stop logs for maintenance, the Sectional, Tainter, and Fishbelly gates did not satisfy all the criteria since: (1) the visible piers and superstructure would be esthetically unattractive, and (2) the piers would have to be massive, hence costly, to resist the heavy ice loadings that would be imposed. The Submersible Radial and Buoyant Flap Gates do not require piers which overcomes this objectionable feature. However, the Submersible Radial Gate has the disadvantages of requiring cofferdaming for maintenance and removal, and a foundation as deep as the depth of water above sill elevation, about 45 feet.

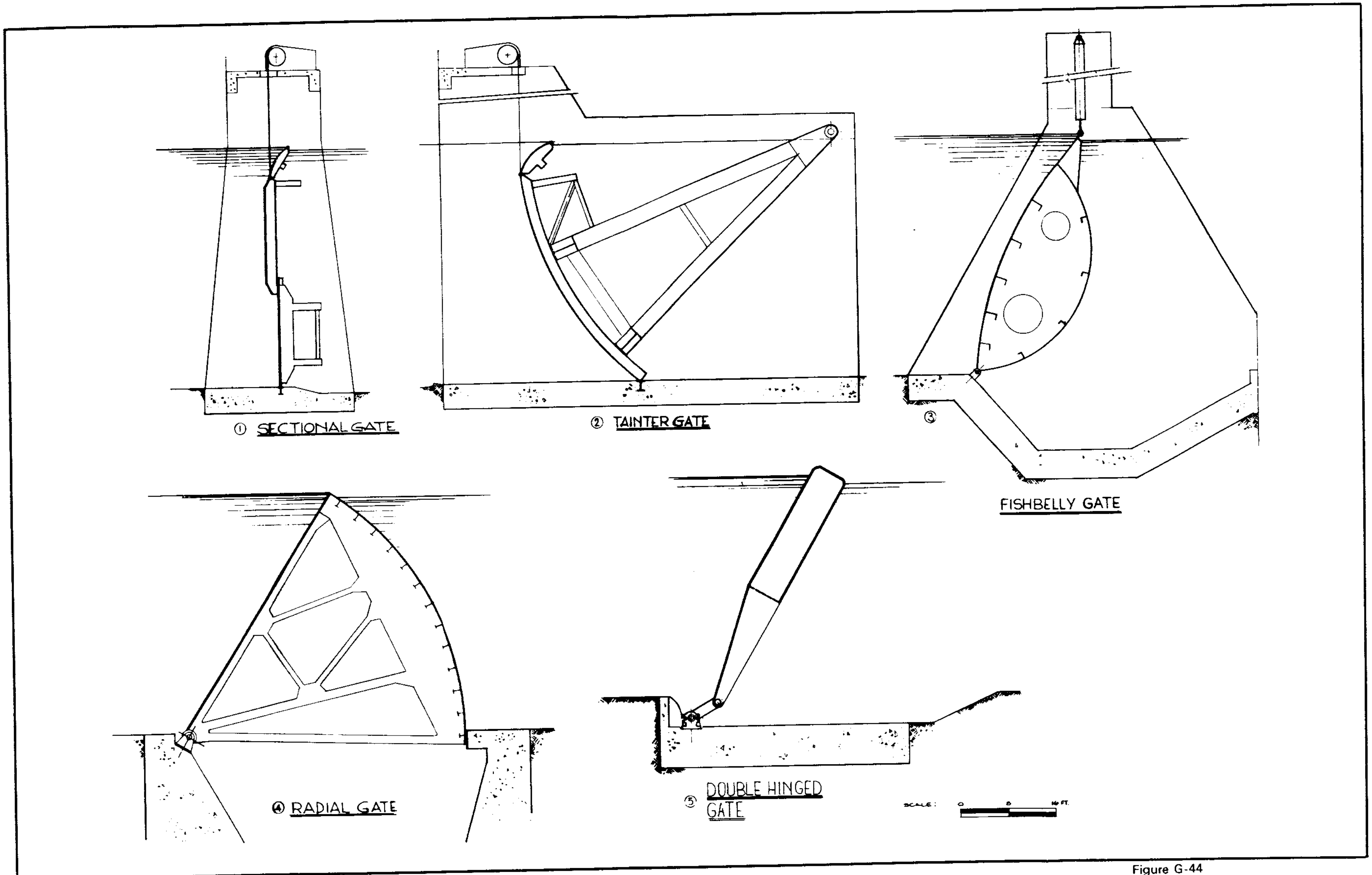


Figure G-44
ST. CLAIR & DETROIT RIVERS—GATE CONCEPTS INVESTIGATED FOR PROPOSED REGULATORY STRUCTURES

Buoyant Flap Gate - Bases of Selection: This gate is similar in principle to the Fishbelly and Submersible Radial Gates in that it is hinged at the riverbed level. The difference is that it is raised by the action of air-filled buoyant tanks alone, whereas the Fishbelly and Submersible Radial Gates require mechanical lifting. This principle is feasible only if the hydraulic loading is very small, as it is at all sites in the St. Clair and Detroit Rivers. No piers are required for the operation of the gate. Pipes run along the sill to carry air to fill the buoyancy tanks. A very simple concrete support structure of minimum depth is required, as the loads transmitted from the gate to the foundation are small. A double hinged gate was proposed in consideration of seismic forces.

There are many advantages of a double hinged type of gate. Its operating principles are independent of any support structure other than a sill. Consequently, the only ice load encountered is the impact of an ice floe before it passes over the gate. This design has the effect of lightening the forces on the gate support structure. The absence of any protruding parts from the river surface is esthetically more desirable than gates requiring large concrete structures. There is no necessity for either cofferdams or the installation of stoplogs for dewatering, as the gate can be easily removed in its entirety for maintenance or for any other purpose. When the gate-bearing anchor belts are released, the gate can be floated to the surface for removal from the site. It is possible for the foundation structure to be built with the gate attached and the ensemble installed utilizing the "float-in" principle, thereby minimizing construction and installation costs. After considering all the factors, the Buoyant Flap Gate was preferred for the service required and was, therefore, selected for refinement to the detail of conceptual engineering design and cost estimate.

Gate Support Structures: In conjunction with the study on gate concepts, similar studies were undertaken for the selection of the most adequate type of support structure. This analysis included rectangular piers, tapered piers, and pierless slab foundation. It follows from the selection of the Buoyant Flap Gate that the pierless slab foundation would satisfy the design requirements. The support structure, by elimination of the pier requirement would become a slab foundation designed only to resist the forces transmitted to it from the gate hinges and the pressures from the soil beneath.

Except for the Port Huron site, which has a granular bed, and the Trenton site, which is on rock, the bearing capacity of the soil is generally not of a high order. In order to keep imposed loading on the soil within permissible limits, it is necessary to have a large spread foundation as shown on Figure G-45. This illustrates a reinforced concrete sill, cellular in construction, with dimensions 50 by 60 feet and 10 feet deep. A 6-foot high wall on the upstream face would accommodate control piping and afford protection to the gate from bed movement. The stability of the sill was examined and found adequate against overturning, slipping and sliding.

The concept of a large cellular concrete sill would meet the foundation requirements without the necessity of deep pile foundations. First cost estimates indicated that it is cheaper to install than a smaller cellular

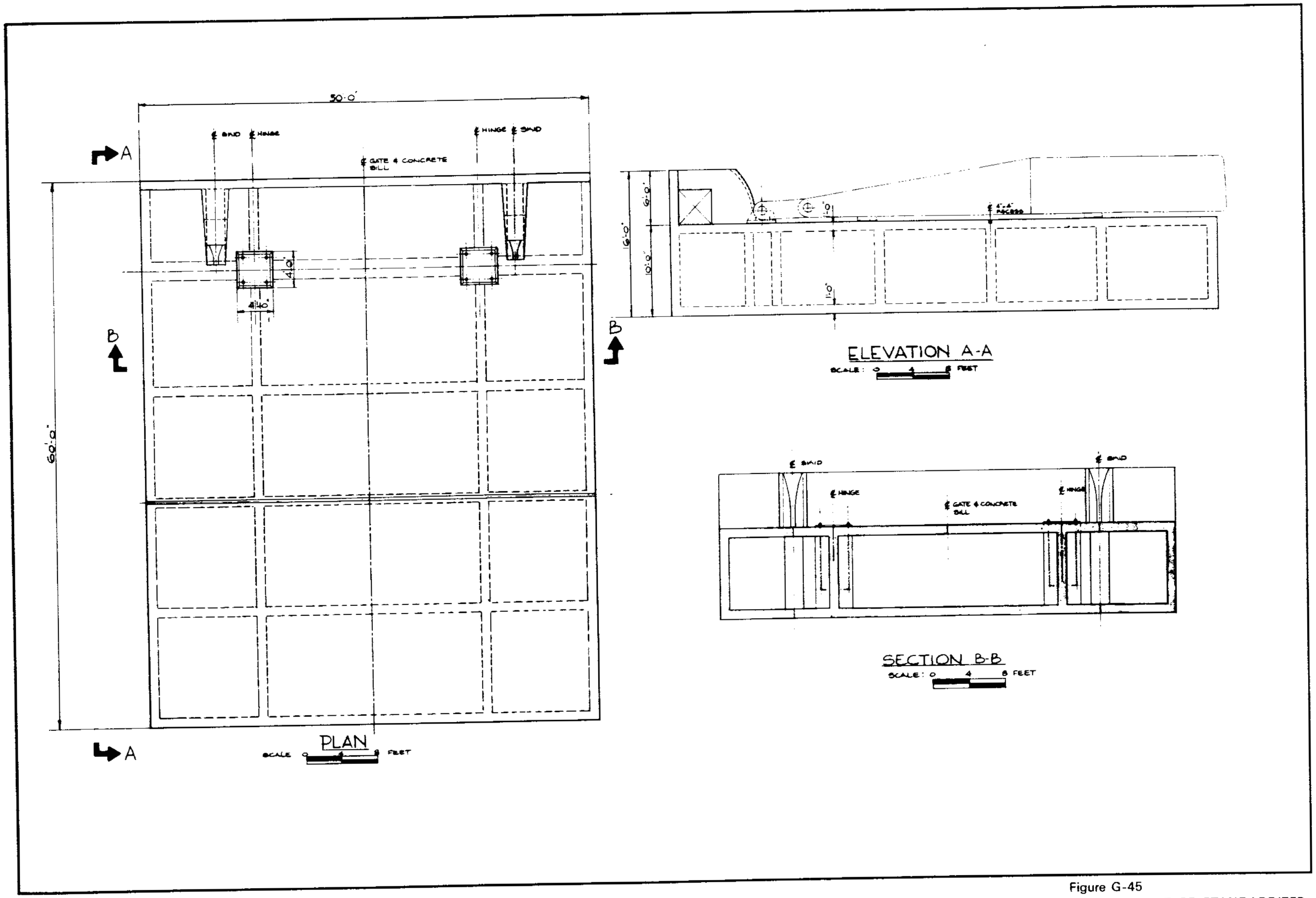


Figure G-45
ST. CLAIR & DETROIT RIVERS—LAYOUT OF STANDARDIZED CONCRETE SILLS FOR PROPOSED REGULATORY STRUCTURES

concrete sill with deep pile foundations, and, therefore, the large cellular concrete sill foundation was accordingly used in the estimates of structure costs.

Construction and Placement of Gates and Sills: The proposed gate sills are reinforced concrete boxes, which can be constructed in one or more locations alongside or in the river, launched and then towed to the various sites for sinking, thus enabling mass production methods to be employed in their manufacture. The selected method for construction is the floating platform or "piggy back" technique. In this technique, two initial units are constructed in a commercial drydock or in an excavation on the river bank and floated to the site. The two units are launched by ballasting the lower unit until the upper unit becomes buoyant and can be floated off, whereupon the lower unit is raised by pumping out the buoyancy tanks. The process of casting another unit may then be repeated. This technique has been used successfully for constructing concrete crib units in major ports throughout the world. The cells are interconnected to form four approximately equal sections to provide control during sinking. The gates may be added at the casting yard or at the steel fabricating yard before sinking of the sill to avoid the need for underwater assembly of gates.

Operation of Control Gates: The gates at each location can be maintained either fully opened or fully closed, and their positions can be kept relatively constant over extended periods of time. Operation of each gate is effected pneumatically from a local compressed air center through a directly connected air line. Air is supplied to the line via a common header, and pressure is maintained from a compressor-supplied storage tank. Tandem operating solenoid valves are used to either pump air into the line, thereby raising the gate, or to vent air from the line, thereby lowering it.

A centralized remote control station is proposed in order that all the structures can be operated as one system. Manual control is provided only for maintenance operations.

Training Walls: Training walls would be required at five of the nine sites where regulatory structures are indicated. The total length would be 27,200 feet; the longest would be 10,000 feet at St. Clair and the shortest would be 1,000 feet at Peach Island (south) location. The purpose of the training wall is to effect a change in the geometry of the river which will, in conjunction with the gated structure, produce and maintain a specified head loss due to the gated structure. The head loss of about 1.5 feet is small in relation to the total height of the training wall.

An evaluation of the function of training walls in this project led to an analysis of three types, namely, rock fill, flexible diaphragms and solid diaphragm. These are described as follows:

(1) Rock Fill

The primary considerations of the design on the section of rock-fill embankment are seepage, due to differential head existing across embankments, and stability with respect to the generally low strength of the clay foundation. In addition to seepage and stability considerations, the known sources of borrow materials and associated costs are parameters that are related to the selection of the section. The embankment section chosen is of rock fill where seepage effects due to low heads are negligible; where head differential is significant, a zone of low permeability is required within the rock-fill shell. This study explored the potentials of structures providing only partial cutoff, where seepage is kept to tolerable limits and which utilizes known available sources of rock fill and sand and gravel.

(2) Flexible Diaphragm

The flexible training wall, a rather unique concept, in principle, consists of an impervious membrane or diaphragm anchored to the bed of the river and supported by a float. The diaphragm acts as an impervious curtain, separating bodies of water having a head differential of less than 1.5 feet. The water flows down the navigation channel side of the training wall at all times, with a maximum average velocity not exceeding 7.5 fps. The water on the other side is static or free flowing, depending on control requirements. The upstream end of the training wall is sealed to the regulatory structure abutment.

The problem of eliminating ice bondage to the float was studied using low bond stress coatings and methods for preventing ice formation. The former method was discarded since known coatings such as Teflon are expensive to bake on and are susceptible to damage by scratching from ice floes. These preliminary studies indicated that the three schemes that offered the best prospects of eliminating the ice bondage to the float were:

(a) Heating the affected half of the flotation collar

(b) Utilizing the water velocity head to take relatively warm water from the channel to the stationary side of the float

(c) An air bubbler system

Using Pyrotenax cables as conductors, the installation of heating elements is feasible and the installation and operating costs are not too high. However, the operating voltage is high and as a result the systems are readily damaged. For this reason, the method was not developed.

Though the water transference method appears to offer the best solution to the ice formation prevention, it was recognized that it is a novel method, and untried. Therefore, the air bubbler system was used in computing the cost of a flexible diaphragm training wall due to the lack of confirmatory tests.

(3) Solid Diaphragm

The variety of suitable types of solid diaphragms that were considered were:

(a) Structural steel sheets and piles

(b) Precast concrete bulkheads

(c) Precast concrete slabs spanning horizontally between solid concrete anchors

(d) A sheet pile wall with the top 7 feet consisting of a Fabridam (an inflatable dam)

(e) Sheet pile cells

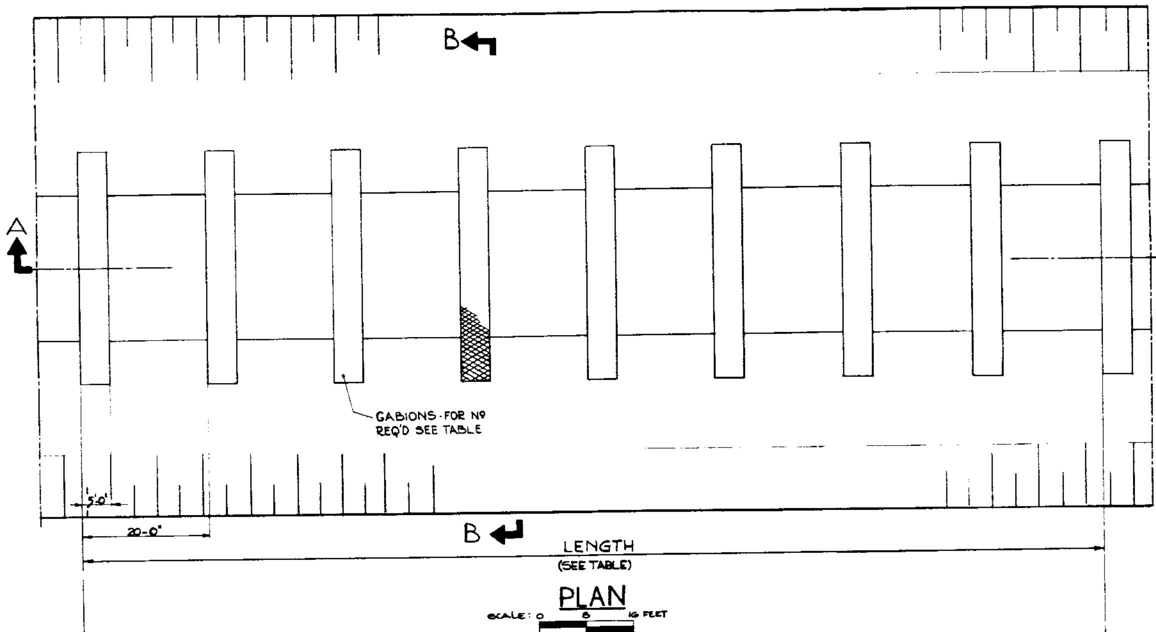
All five types were analyzed and first costed with the last three offering the best solutions.

For the purposes of this report and cost estimate, a rock-fill training wall was the adopted concept of those discussed above. Preliminary comparison of the costs of the rock fill and flexible membrane training walls indicated the possibility of saving up to $20,000,000 in using the concept of the flexible membrane. Should benefit-cost ratios of regulation be found to justify or reject regulation marginally, using costs based on rock-fill wall, the flexible membrane alternative should not be overlooked.

Small Boat Passage: The requirements for the design of the navigation facilities consisted of providing a passage for small recreational boats at all structures during the open water season to minimize the passage of small boats into the main navigation channels thereby preventing congestion and promoting safety. With the exception of the Port Huron and South Peach Island structures, where natural portions of the channel are available for small boats, each facility is to have the capability of handling fifty boats per hour (including sail boats) in either direction, having a maximum draft of 8 feet, an average length of 25 feet and a maximum length of 40 feet. These two latter criteria were set for conventional lock design purposes.

Since the loss of head across each structure is quite small (the maximum being 1.30 feet across the St. Clair Structure), the possibility of an ungated boat passage was included in the study. The advantages of such a passage are: (1) minimal maintenance and attendance, and (2) the avoidance of delay imposed by conventional locking procedures. The basic problem is to dissipate the energy head while at the same time maintaining velocities in the boat passage which do not create difficulties for small craft. Two types of gateless boat passage were considered, one with side roughness elements and the other with bottom roughness elements. In addition, a suitable gated structure was studied in detail. A boat passage employing vertical bottom roughness elements was used for the establishment of cost estimates. Details of this plan are shown on Figure G-46.

Estimates of the cost of conceptual designs shows that the cost of construction of a gateless navigation passage is less than, or competitive with, the cost of a gated lock at all sites other than St. Clair. At this site, the great length of the navigation passage makes it more expensive than the gated alternative. However, when both maintenance and attendance costs are considered and the added convenience of an ungated passage is taken into



STRUCTURE LOCATION	LENGTH OF PASSAGE	NUMBER OF GABIONS	TOTAL HEAD DIFFERENTIAL
STAG ISLAND	575 FT	30	0.67 FT
ST. CLAIR	1,120	57	1.30
NORTH CHANNEL	535	28	0.62
MIDDLE CHANNEL	560	29	0.65
PEACH ISLAND	285	15	0.33
BELLE ISLE	345	18	0.40
ZUG ISLAND	145	8	0.17
FIGHTING ISLAND	465	24	0.54
TRENTON CHANNEL	740	38	0.86

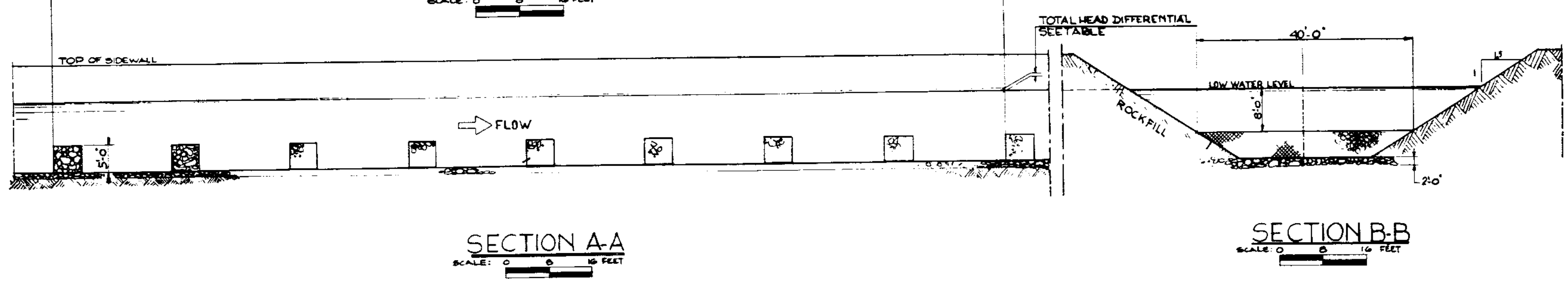


Figure G-46
ST. CLAIR & DETROIT RIVERS—LAYOUT OF PROPOSED TYPICAL SMALL BOAT PASSAGE

account, the ungated passage was adopted and used for cost estimates in this study.

Prior to final design, a physical model would be necessary to verify the operation of the passage and to indicate economies that might be made in the cost of the passage.

Development of Recreational Areas: Some structures lend themselves to development of recreational areas. Consequently, adjacent marinas are anticipated and provided for in the cost estimates. An artist's conception of a regulatory structure in operation with small boat passage and adjacent marina is shown on Figure G-47. Plans for the nine control structures and ancillary works are detailed on Figures G-48 through G-57.

Environmental Considerations During Construction: The proposed construction of the control structures and dikes would have some effect on the environment during construction. Construction work will be active only in relatively small areas at any one time. Prefabricated sections of the work will be built on land, at possibly two separate sites along the two rivers, thus minimizing the land requirements and environmental disturbance during construction. The proposed method of construction does not require the closing off of any major section of the river at any time. In addition, when the structures are in operation, water levels will be maintained within the range of natural variation throughout the system which is a desirable feature.

3.3.16 Development of Cost Curves for Channel Capacity Decreases

Utilizing the unit costs provided by the report on "Conceptual Design and Estimates" discussed previously, costs for the combination of structures required for the range of channel capacity decreases were determined for the St. Clair and Detroit Rivers. Two channel decrease cost curves were derived; one, in combination with no channel capacity increase and the second, in combination with the enlarged channels resulting from the maximum channel capacity increase requirements. The two curves for the St. Clair and Detroit Rivers, are shown on Figures G-58 and G-59, respectively. These cost curves show that for the maximum condition of channel decrease in combination with the maximum channel increase the total first cost of the structures, based on 1971 price levels, is 69.4 million dollars for the St. Clair River and 82.7 million dollars for the Detroit River. Since the channel design is based on 1933 channel conditions, the cost curves for no channel decrease required structures in both rivers to account for the "built-in" channel increase resulting from the 25- and 27-foot navigation improvements. Therefore, it has been determined that in order to return the existing channel conditions to the 1933 river regimen using these concepts, two structures would be required in the St. Clair River; one at the Port Huron site, and the other at the Stag Island site, at a first cost of 29.8 million dollars. Similarly, for the Detroit River, one structure located at the Trenton Channel site would be required at a first cost of 9.8 million dollars. The steps shown in the channel cost curves are a result of the range in retardation available at each structure site, depending on the number of gates required.

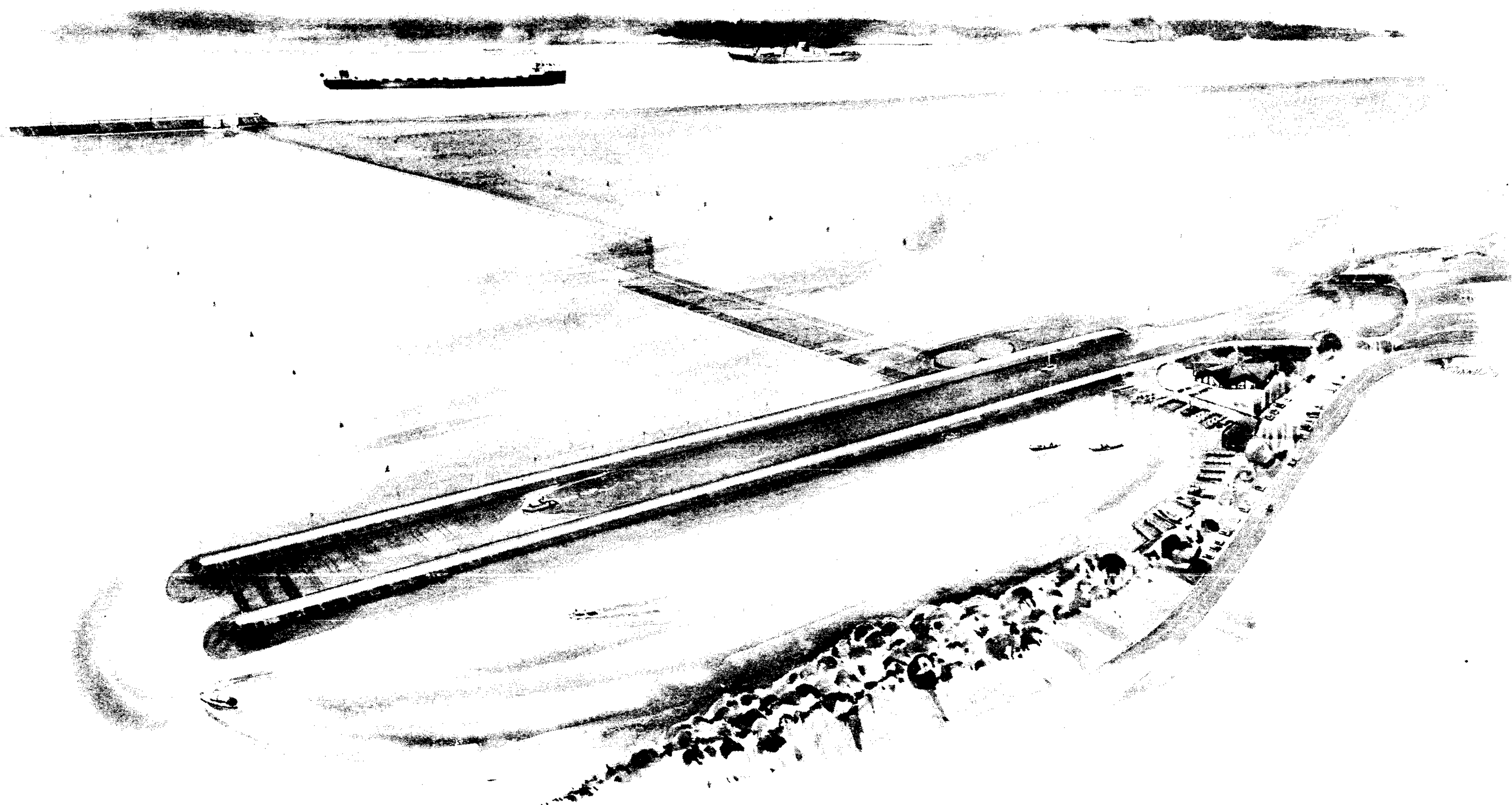


Figure G-47

ST. CLAIR & DETROIT RIVERS—CONCEPTUAL SKETCH OF PROPOSED TYPICAL REGULATORY STRUCTURE

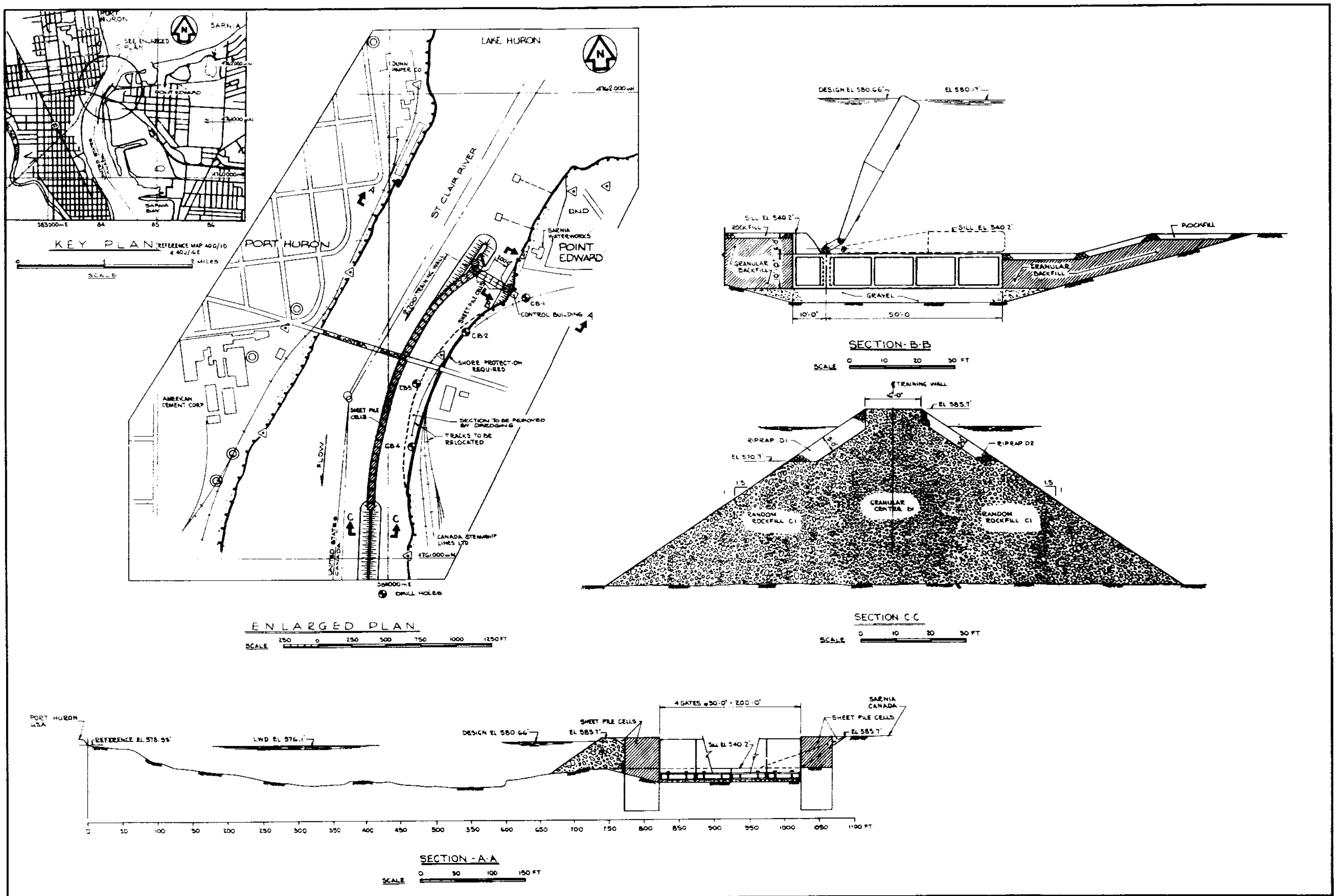


Figure G-48
ST. CLAIR RIVER—PROPOSED REGULATORY STRUCTURE AT PORT HURON

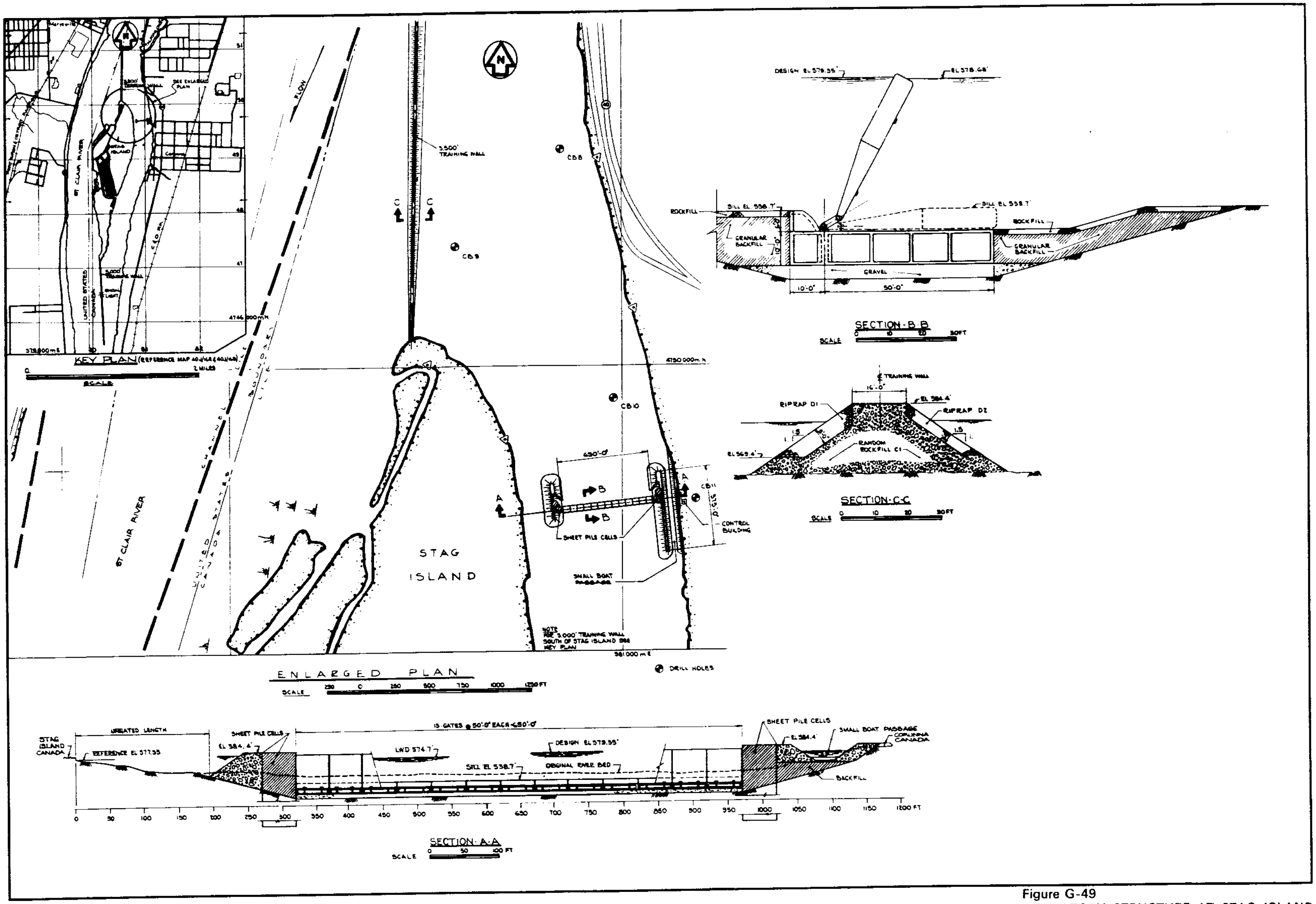


Figure G-49
ST. CLAIR RIVER—PROPOSED REGULATORY STRUCTURE AT STAG ISLAND

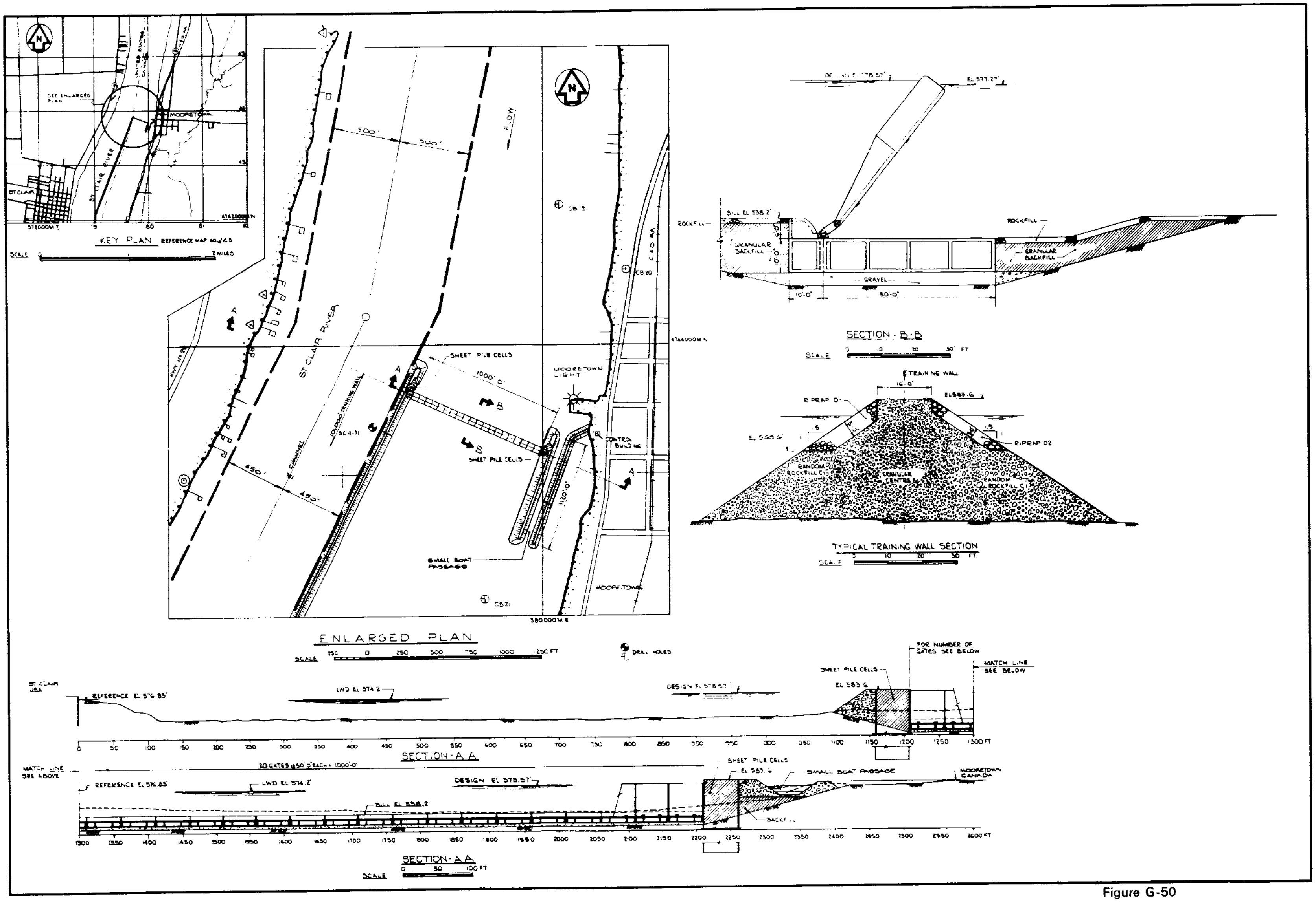


Figure G-50
ST. CLAIR RIVER—PROPOSED
REGULATORY STRUCTURE AT ST. CLAIR, MICHIGAN

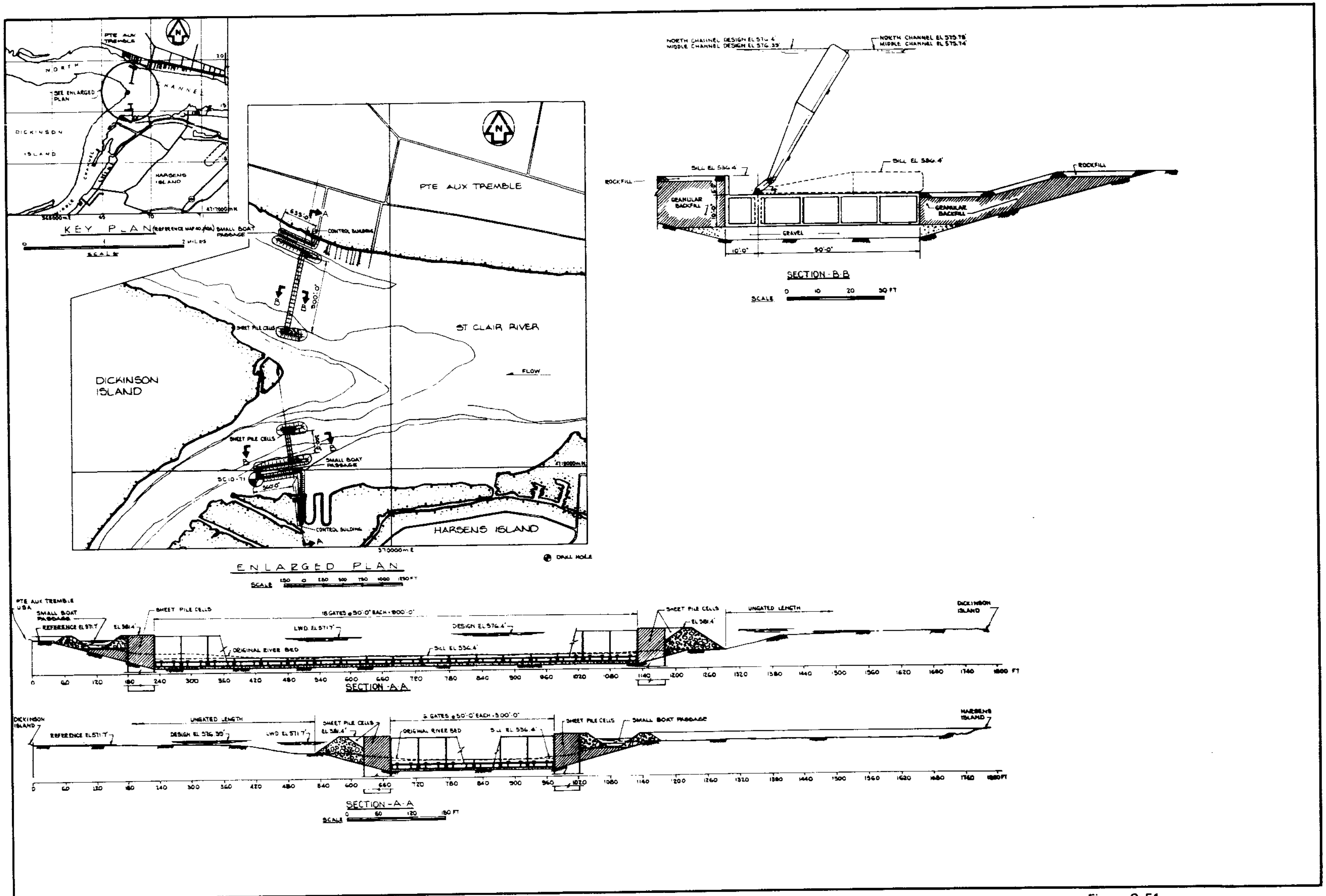


Figure G-51

ST. CLAIR RIVER—PROPOSED REGULATORY STRUCTURE AT HEAD OF NORTH AND MIDDLE CHANNELS

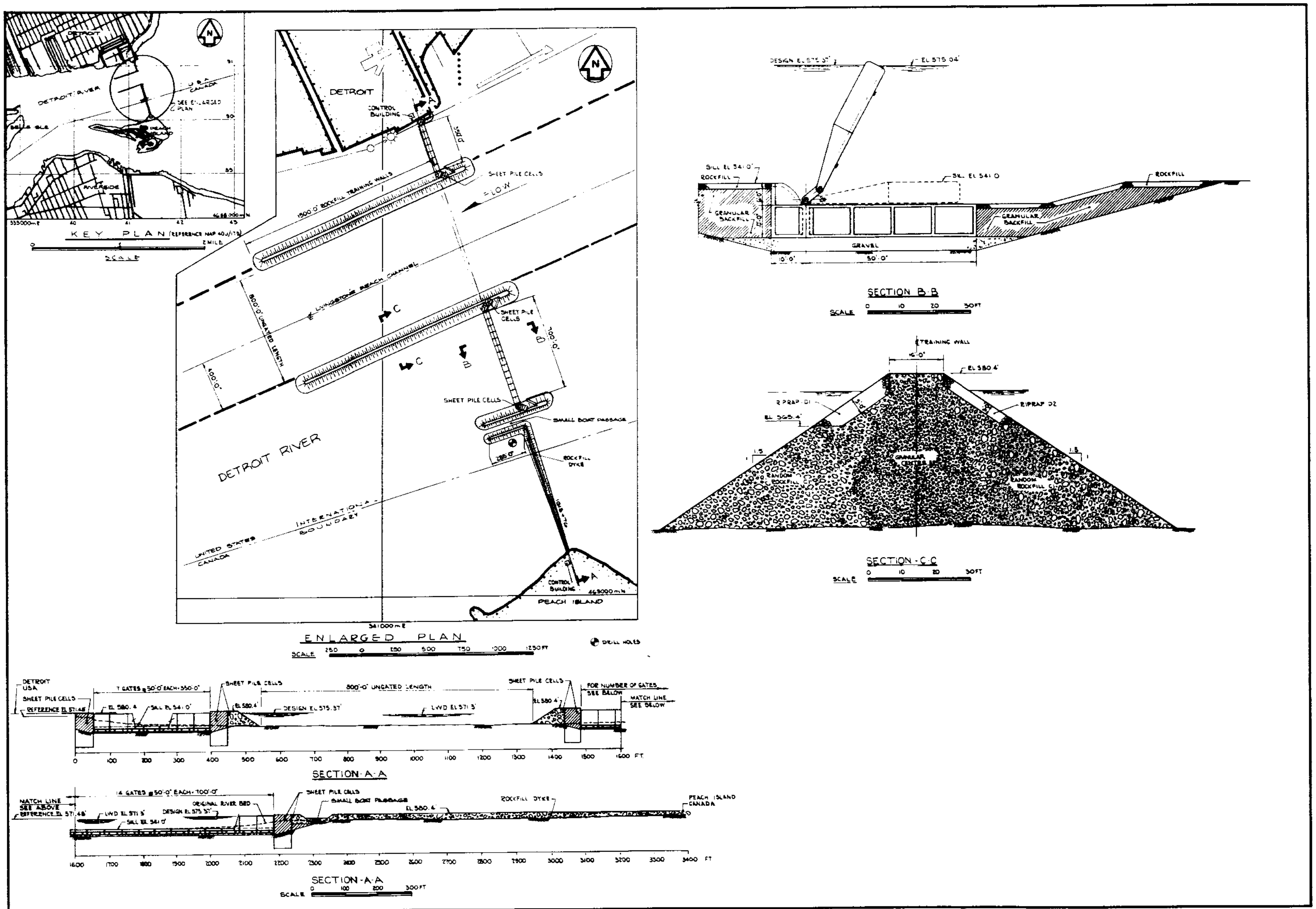


Figure G-52
DETROIT RIVER—PROPOSED REGULATORY STRUCTURE AT PEACH ISLAND (NORTH SIDE)

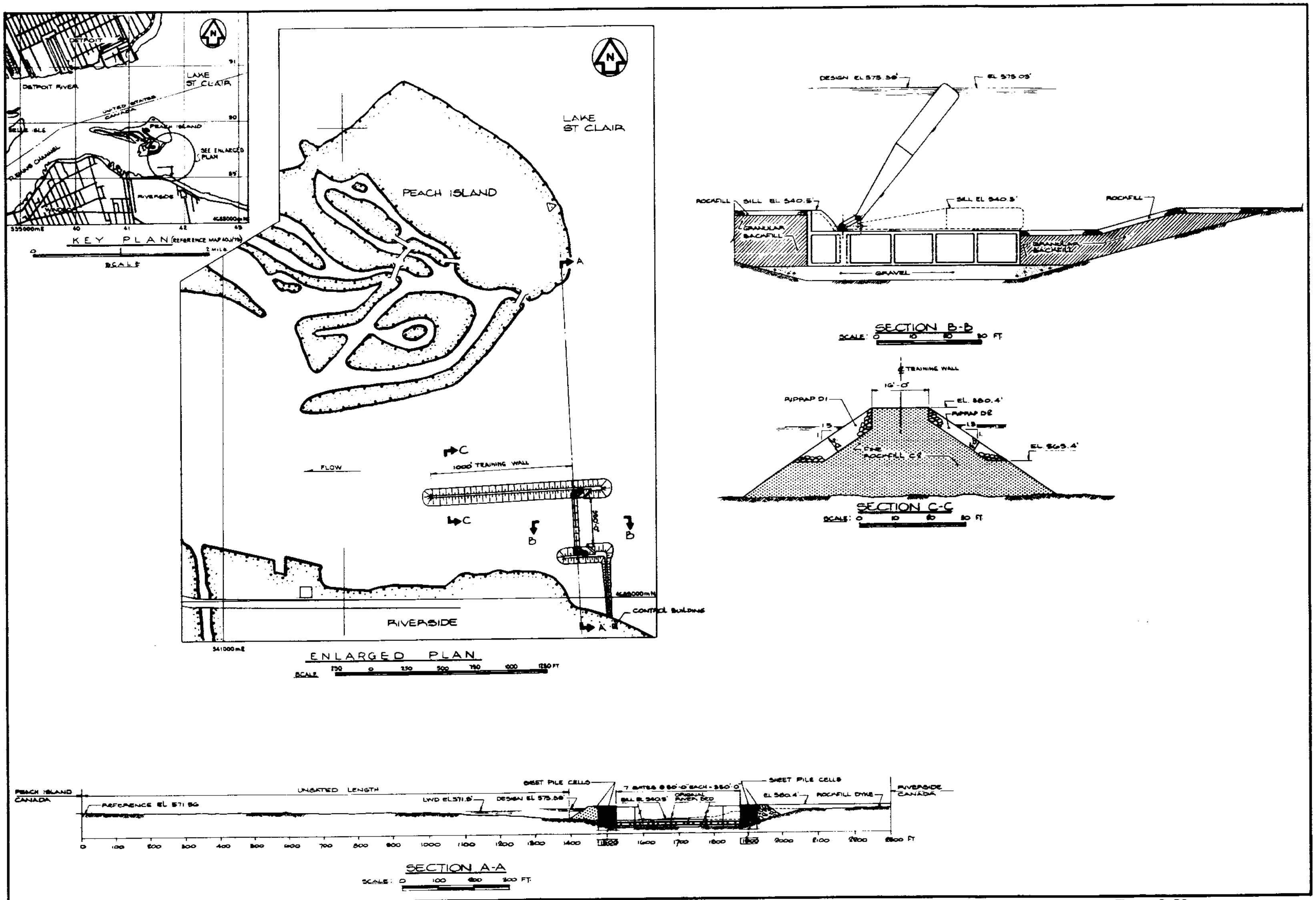


Figure G-53
DETROIT RIVER—PROPOSED REGULATORY STRUCTURE AT PEACH ISLAND (SOUTH SIDE)

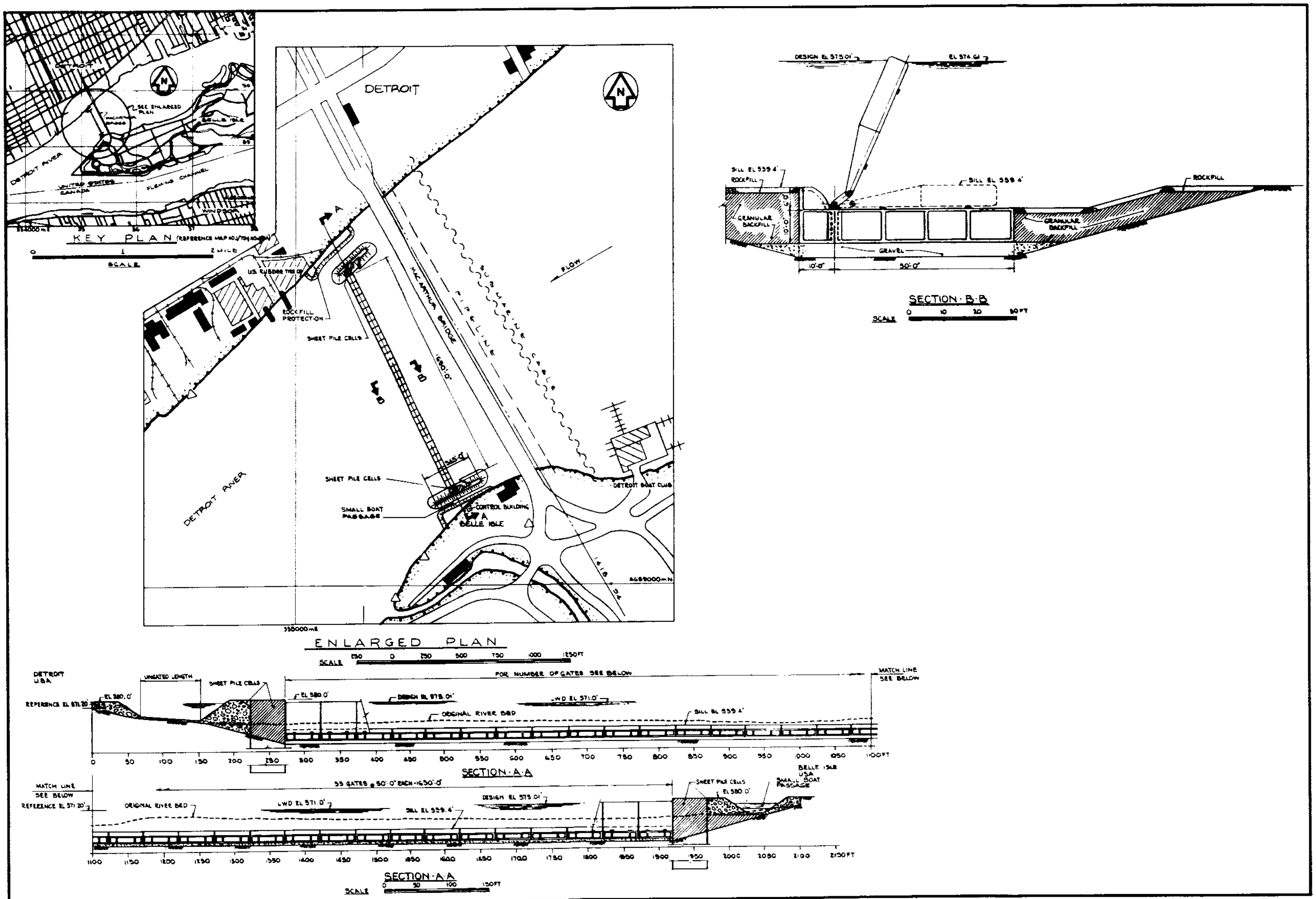


Figure G-54
DETROIT RIVER—PROPOSED REGULATORY STRUCTURE AT WEST BELLE ISLE

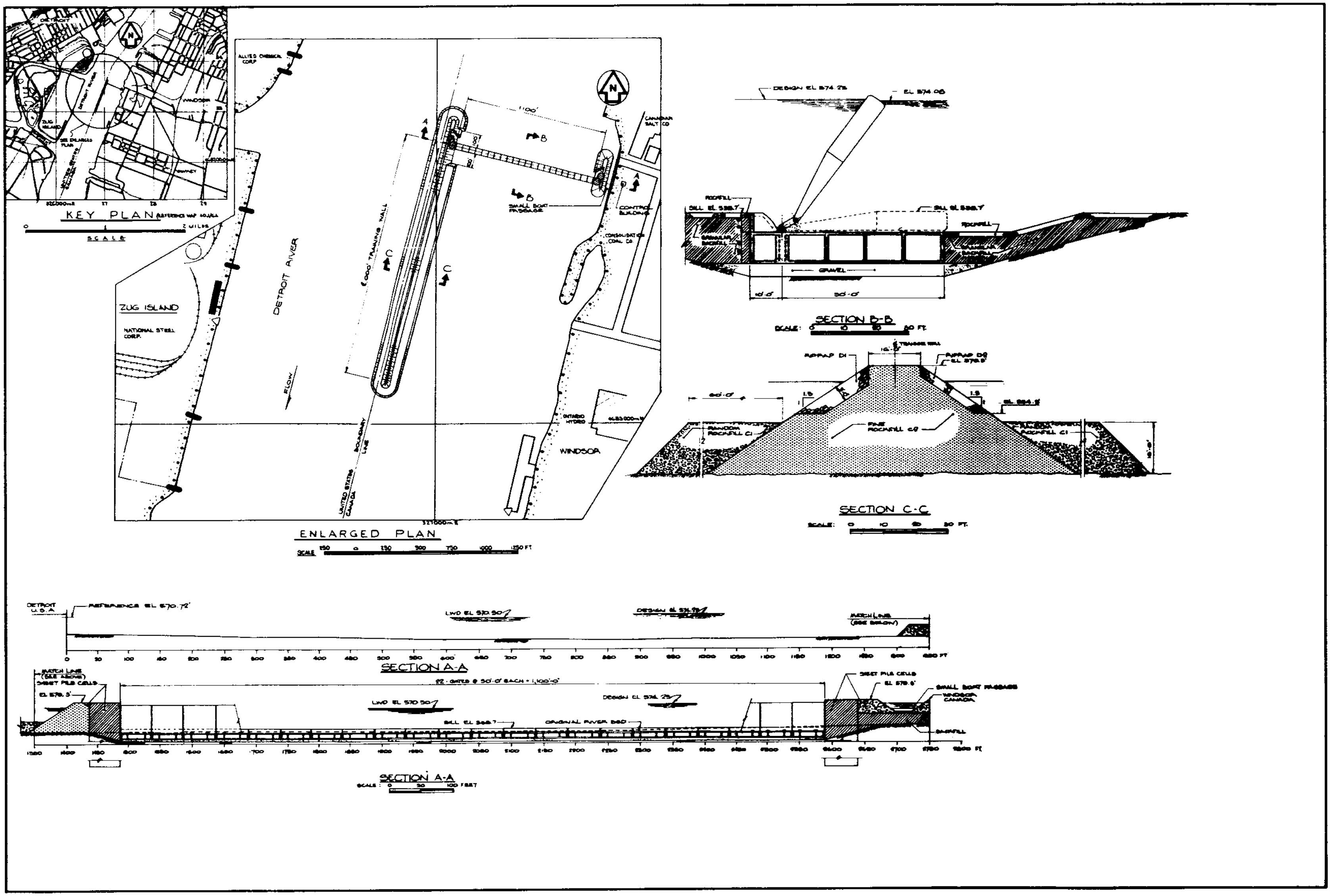


Figure G-55
DETROIT RIVER—PROPOSED REGULATORY STRUCTURE AT ZUG ISLAND

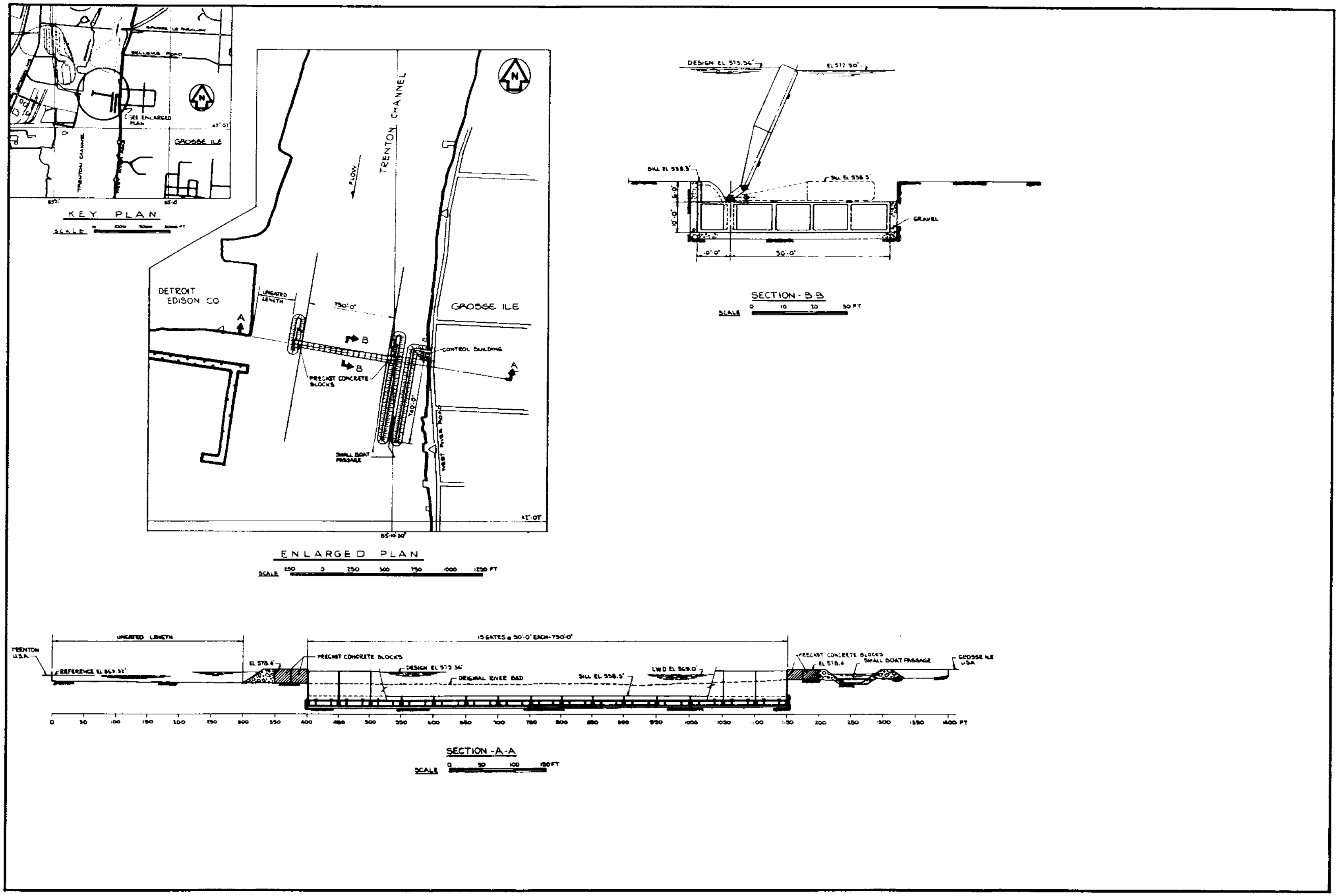


Figure G-56
DETROIT RIVER—PROPOSED
REGULATORY STRUCTURE IN TRENTON CHANNEL

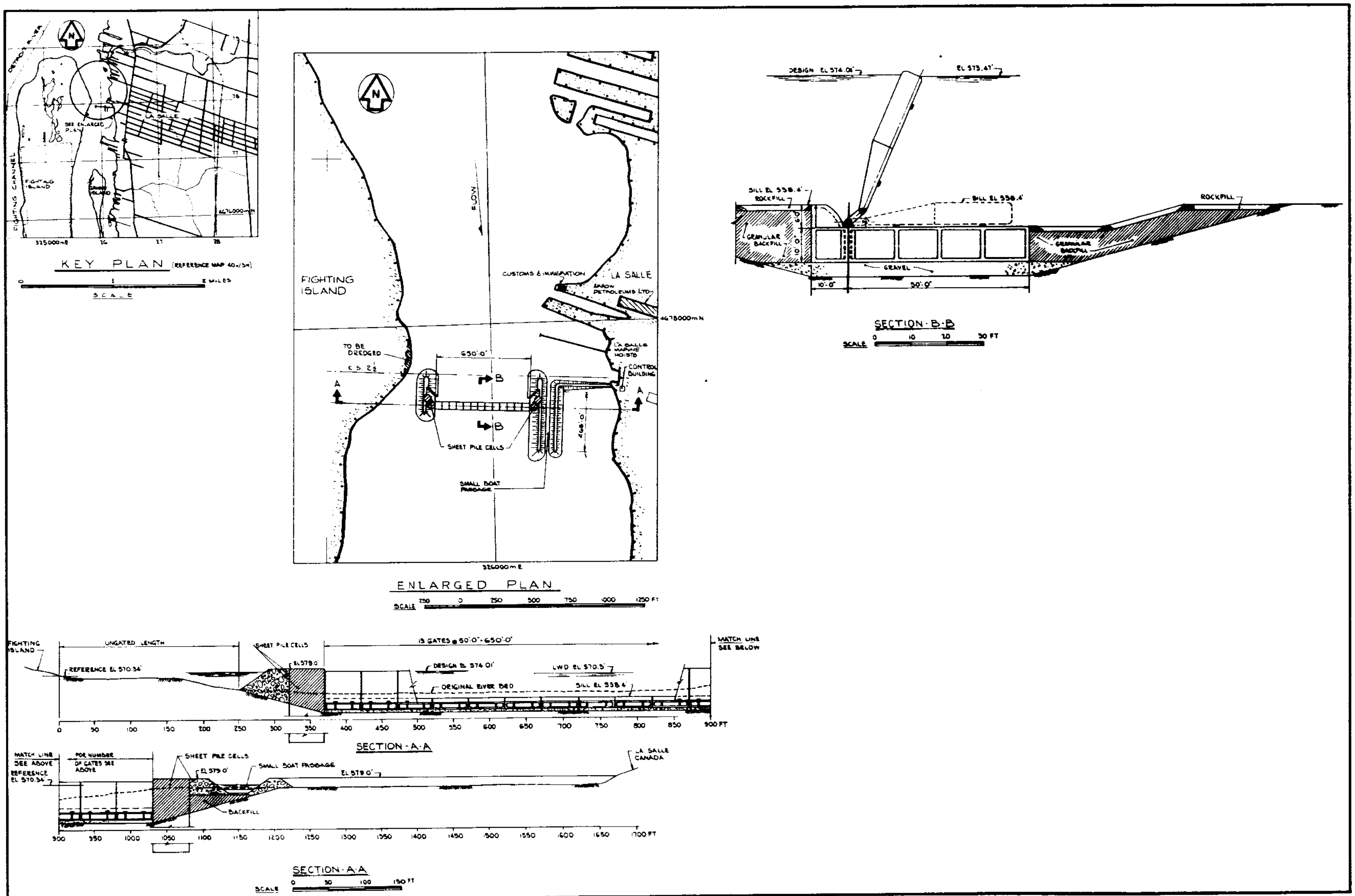


Figure G-57
DETROIT RIVER—PROPOSED REGULATORY STRUCTURE AT EAST FIGHTING ISLAND (GRASSY ISLAND)

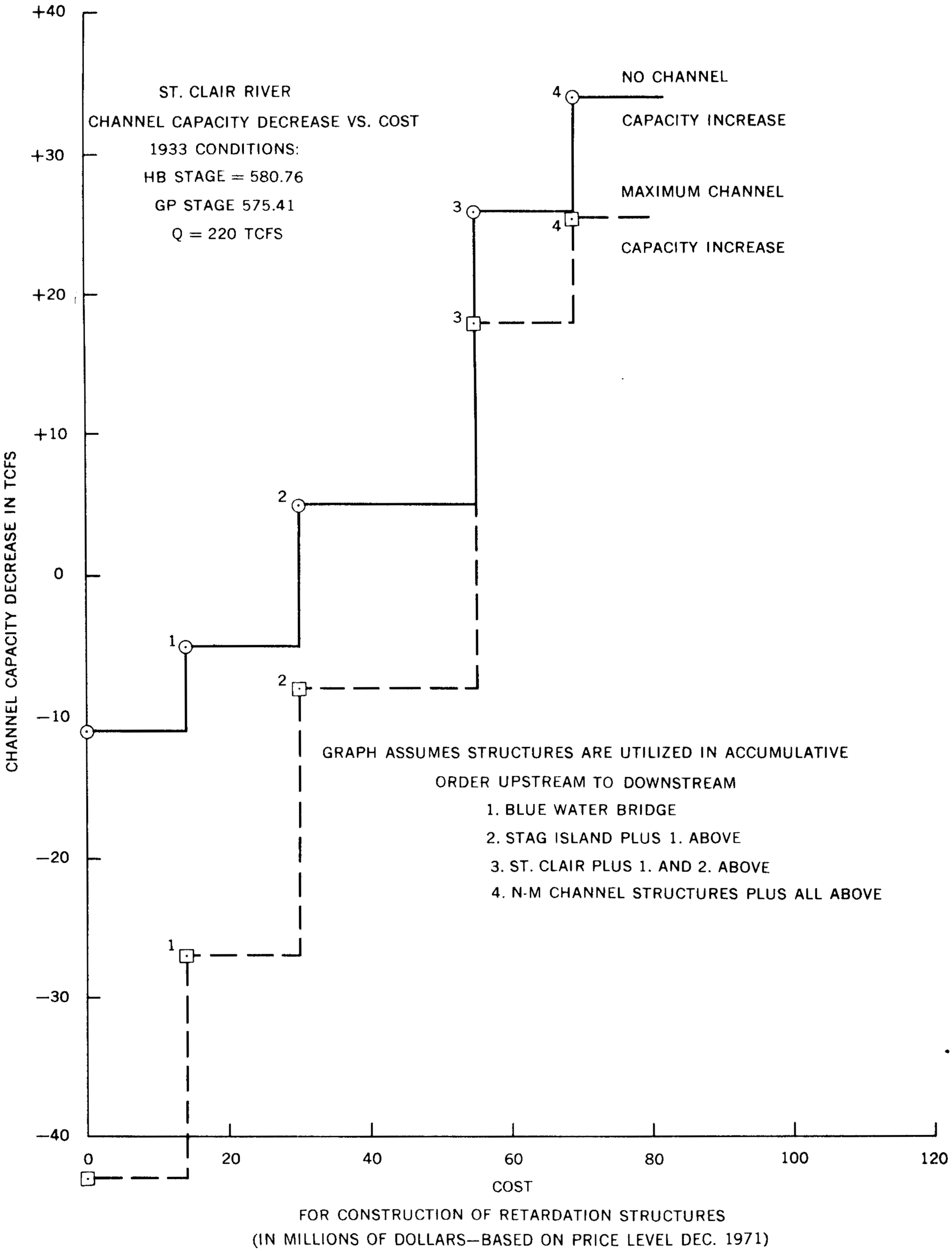


Figure G-58
ST. CLAIR RIVER—RELATIONSHIP BETWEEN COST AND CHANNEL CAPACITY DECREASE FOR RANGE OF CHANNEL CAPACITY INCREASES

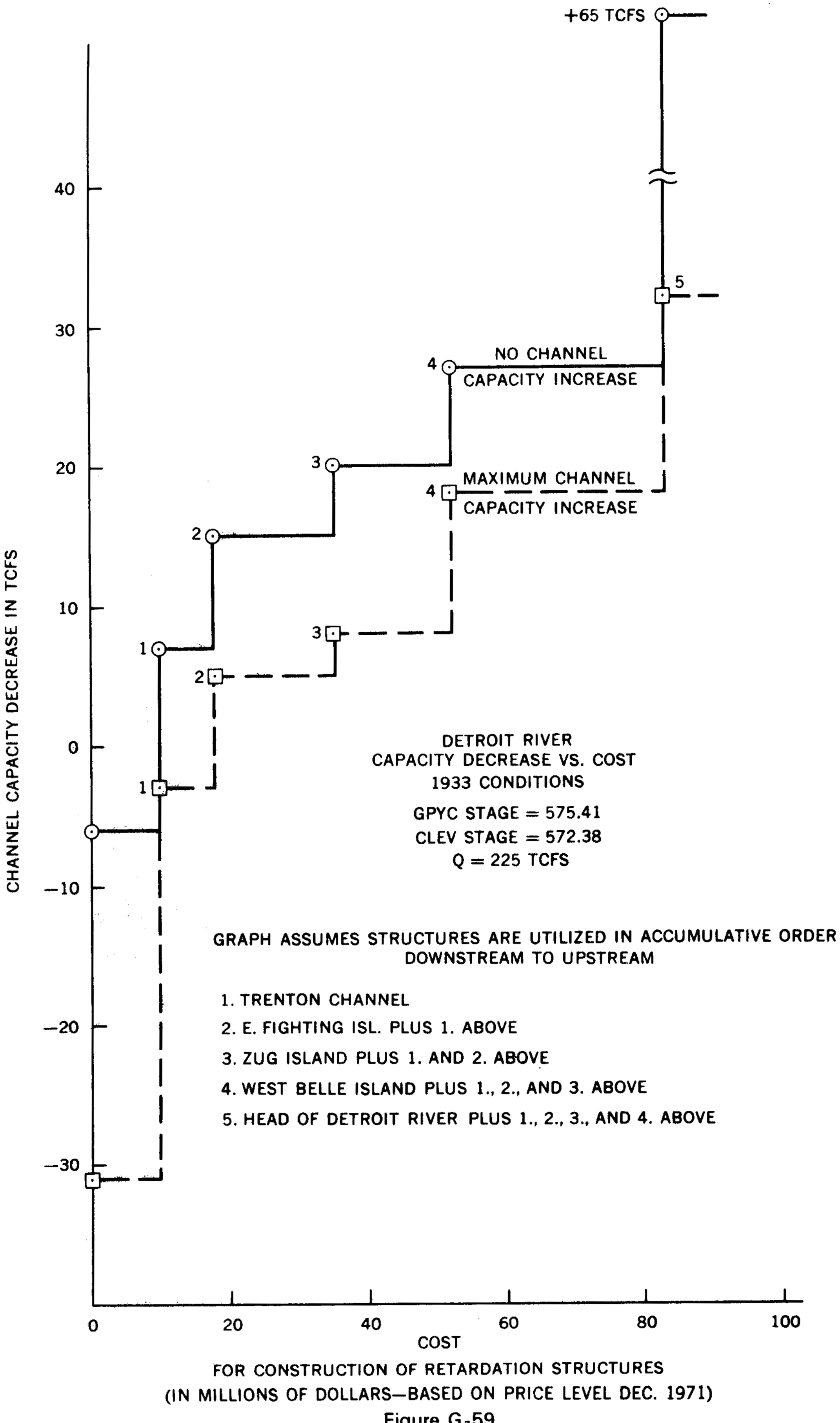


Figure G-59
DETROIT RIVER—RELATIONSHIP BETWEEN COST AND CHANNEL CAPACITY DECREASE FOR RANGE OF CHANNEL CAPACITY INCREASES

3.3.17 Use of Cost Curves

The initial step in determining the cost of a specific regulation plan involved the selection of the extreme condition of channel capacity increase and decrease required according to the procedures previously described. The retarding capacity of the structures is determined by adding the channel capacity increase (dredging) to the amount of channel capacity decrease. This is necessary because an additional amount of channel increase is incurred which must be added to the amount determined for the channel decrease required by the regulation plan.

Proceeding to the channel capacity decrease versus cost relationships for the St. Clair and Detroit Rivers, as shown on Figures G-58 and G-59, respectively, the cost and number of structures are obtained by interpolating between the cost curves for no channel increase and the maximum channel increase.

The cost involved for the required dredging (channel capacity increase) is obtained from the channel capacity increase versus cost curves for the St. Clair and Detroit Rivers on Figures G-37 and G-38, respectively. The two costs added together comprise the total first costs, based on 1971 price levels, for the regulatory structures and the dredging required for a selected regulation plan.

3.4 Channel Design and Cost Estimates for Selected Regulation Plans

The methodology, applied for determining the channel design and cost estimates for selected regulation plans and requiring detailed cost analysis, was in principle similar to the methodology previously described for application to development of preliminary regulation plans. The difference was that for pre liminary plans, the cost curves were based on a given set of extreme conditions of recorded lake levels and computed flows, whereas, in the analysis of the selected regulation plans, the design conditions were based on regulated levels and flows. This latter procedure was necessary in order to assure that the extremes in levels and flow changes required by regulation were reflected in the regulatory works design and cost estimates.

3.4.1 Lake St. Clair Critical Design Elevation

The methodology involves the determination of Lake St. Clair regulated levels which are not provided by the regulation plans. This is in contrast to the methodology used in the development of the cost curves for which recorded Lake St. Clair levels were available as a substitute for profile design elevations.

The solution to the problem of determining regulated Lake St. Clair design elevations involved the computation of the level which would maintain the St. Clair and Detroit Rivers in hydraulic balance for the critical regulated levels for Lakes Michigan-Huron and Erie. The required design elevations for Lake St. Clair and the respective balanced flows for the St. Clair and Detroit Rivers were determined by the simultaneous solution of the stage, fall discharge equation for both rivers. These equations for 1933 channel conditions are as follows:

$$Q\ St.\ Clair = 57.909\ (HB-540.21)^{2.0}\ (HB-GP)^{.5}$$

$$Q\ Detroit = 177.29\ (GP-548.41)^{2.0}\ (GP-Clev)^{.5}$$

Where : HB (Harbor Beach) = Lakes Michigan-Huron level
GP (Grosse Pointe) = Lake St. Clair level
Clev (Cleveland) = Lake Erie level

The regulated levels and flows together with the design requirements and associated costs for selected regulation plans involving Lakes Michigan-Huron are discussed under Evaluation of Regulation Plans, Section 6.

3.5 Basic Data

The following is a list of the principal basic data utilized in this study:

(1) Soil Boring Data:

(a) Report by William Trow and Associates (Hamilton) Limited entitled: "Preliminary Subsurface Investigations, Proposed Flow Control Structures, St. Clair River, Ontario", 1971

(b) Reports by Chicago District, U. S. Army Corps of Engineers entitled: "Subsurface Investigations, Proposed Regulatory Structures Between Lake Huron, Lake St. Clair and Lake Erie" and a supplemental report entitled, "Preliminary Subsurface Investigation, Proposed Regulatory Structures for Detroit River", 1971

(c) Numerous boring logs taken by the U. S. Army Corps of Engineers for previous projects

(2) Hydrographic Data:

(a) Hydrographic surveys undertaken by the U. S. Army Corps of Engineers in the St. Clair River in 1954 and 1970 for obtaining cross sections

(b) Hydrographic surveys undertaken by the U. S. Army Corps of Engineers in the Lower Detroit River in 1966 for obtaining cross sections

(c) Various Lake Survey Center, NOAA, U. S. Department of Commerce Charts and field sheets for additional cross sections

(d) St. Clair and Detroit Rivers discharge measurements and water levels for existing channel conditions taken in 1968 by the U. S. Army Corps of Engineers. Discharge measurements were also taken in the lower St. Clair River for obtaining the distribution of flow in the lower reach of the St. Clair River

(3) Aerial Photographs

(a) Black and white aerial photographs of the St. Clair River, scale: 1/30,000

(b) Color aerial photographs of the Detroit River, Scale: 1 in. = 2,500 feet. Photos available from the U. S. Army Corps of Engineers

Section 4

NIAGARA RIVER SYSTEM

4.1 Description of the System

The Niagara River, about 36 miles in length, links Lake Erie at Buffalo, New York, and Lake Ontario at Niagara-on-the-Lake, Ontario. The average fall over its course is 326 feet about half of which is concentrated at Niagara Falls, located approximately 22 miles below the head of the river. Over the period 1900-1967, the monthly mean Niagara River discharge has varied from 251,000 cfs to 116,000 cfs and has averaged approximately 194,000 cfs. A portion of the Lake Erie outflow is also carried by two artificial channels, the Welland and Black Rock Canals, a more detailed discussion on which is contained in Section 4.1.4.

4.1.1 General

An outstanding physical characteristic of the Niagara River is the rapid change in the water surface profile between various points on the river system. The Niagara River may be considered to consist of three major reaches: the Upper Niagara River; the Niagara Cascades and Falls; and, the Lower Niagara River extending from the foot of the falls at the Maid-of-the-Mist Pool to Lake Ontario. The following paragraphs describe the river system in more detail. A location map of the Niagara River and surrounding area is shown on Figure G-60.

Upper Niagara River: The Upper Niagara River, extending from Lake Erie below Buffalo Harbor to the Cascades and Niagara Falls, is of primary interest in this study since regulatory works must be located in the upper portion of this reach to fulfill the overall objectives of regulating Lake Erie. An aerial photograph of the reach, extending from the International Railroad Bridge to the head of the river, is shown on Figure G-61. From Lake Erie to Strawberry Island, a distance of approximately 5 miles, the channel width varies from 9,000 feet at it funnel-shaped entrance to 1,500 feet at Squaw Island below the Peace Bridge (Highway). The normal fall over this upper 5 mile portion is 6.1 feet. In the upper 2 miles of the river, the maximum depth is approximately 20 feet with velocities as high as 12 fps in the vicinity of the Peace Bridge. This part of the river is paralleled by the Black Rock Canal. Below Squaw Island the river widens to approximately 2,000 feet and becomes more placid with velocities in the order of 4 to 5 feet per second. A navigation channel, with a maintained depth of 21 feet below L.W.D., enters the river at this point and, together with the Black Rock Canal, provide for the passage of larger vessels between Lake Erie and Tonawanda, New York. The Upper Niagara River is suitable for recreational boating with the exception of the downstream portion of Chippawa Grass-Island Pool. (Downstream of Navy Island, boating is discouraged due to the increased velocities and the danger of being swept over Niagara Falls.)

At Strawberry and Grand Islands, the river is divided into two channels, the Canadian, or Chippawa Channel and the American, or Tonawanda Channel.

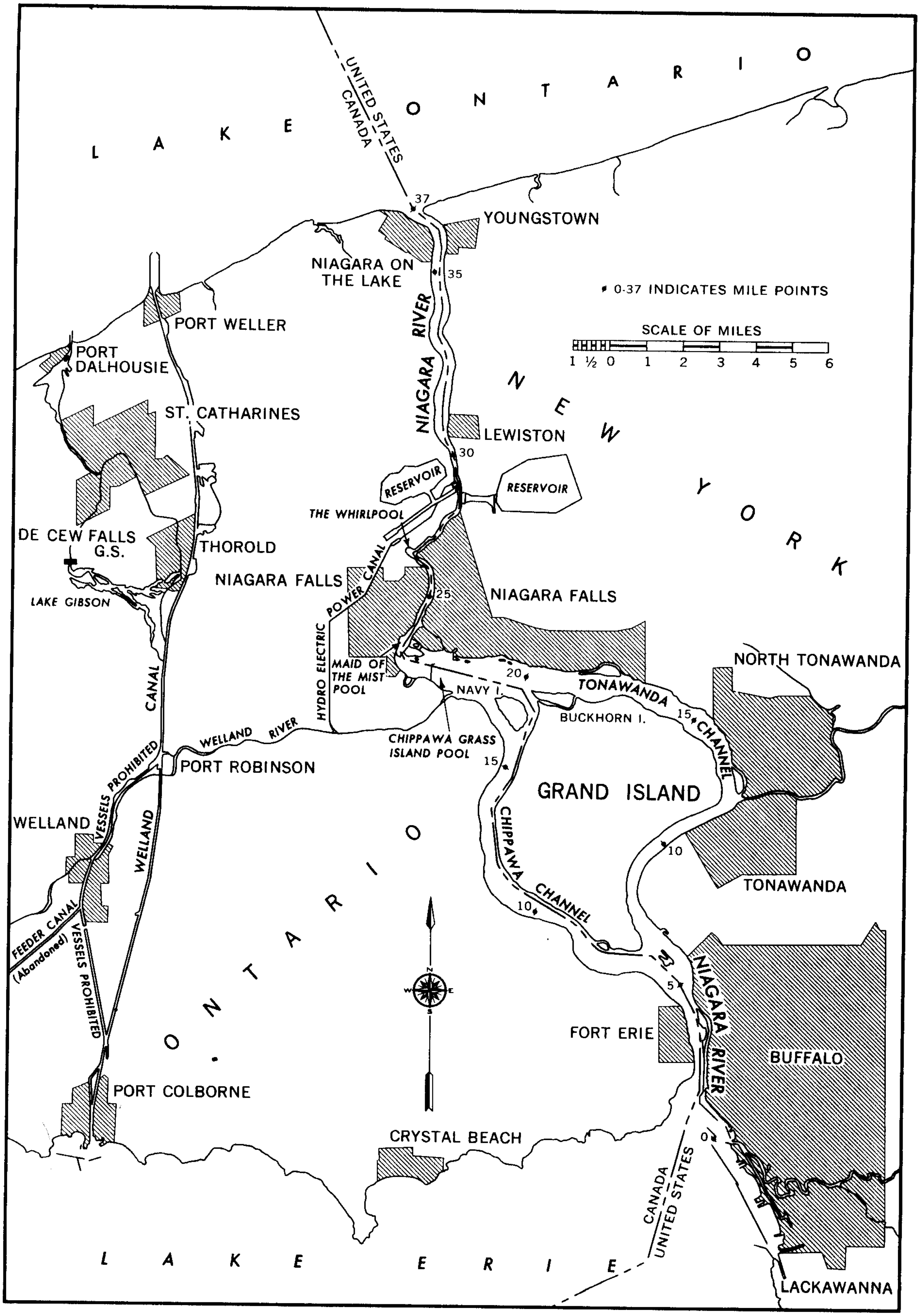
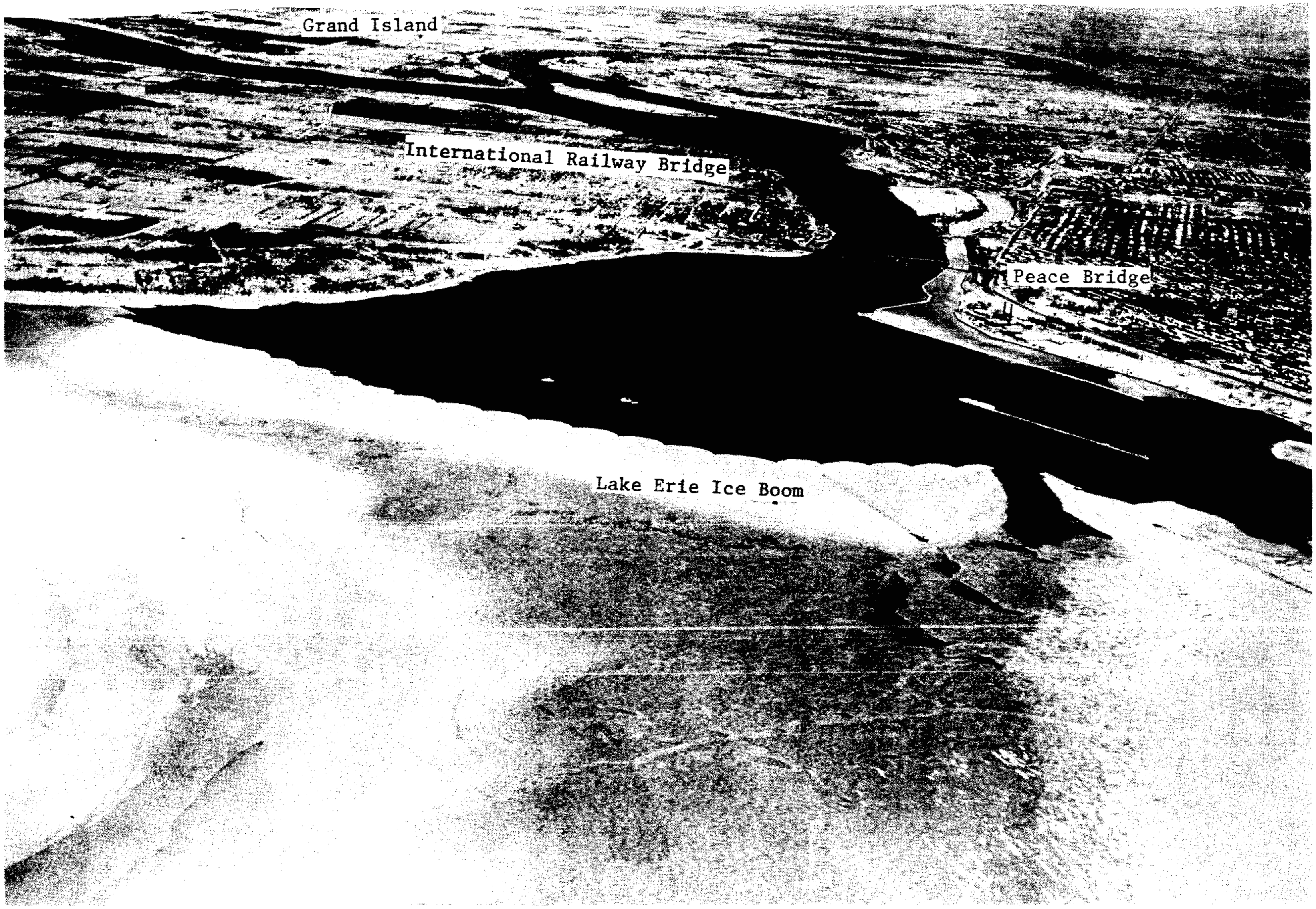


Figure G-60
NIAGARA RIVER—LOCATION MAP



(COURTESY OF THE POWER AUTHORITY OF THE STATE OF NEW YORK) JANUARY 24, 1966

Figure G-61

Aerial Photograph of Upper Niagara River Looking Downstream from Lake Erie

The Chippawa Channel is approximately 11 miles in length and varies from 2,000 to 4,000 feet in width. Velocities range from 2 to 3 fps. The Chippawa Channel carried approximately 57% of the total river flow. The Tonawanda Channel is approximately 15 miles long and varies from 1,500 to 2,000 feet in width above Tonawanda Island. Downstream thereof, the channel varies from 1,500 to 4,000 feet in width. Velocities range from 2 to 3 fps. The Islands of Navy and Tonawanda are located in the Chippawa and Tonawanda Channels, respectively.

At the foot of Grand Island, the channels unite to form the 3-mile-long Chippawa-Grass Island Pool extending to a partial control structure extending from the Canadian shoreline, which is located approximately 4,500 feet upstream of Niagara Falls. The normal fall from Lake Erie to the upstream end of Chippawa Grass Island Pool is 9.1 feet. The normal fall across the Pool is about 0.4 foot.

Niagara Cascades and Falls: Below the control structure, the river falls 50 feet through the cascades and is divided into two channels by Goat Island, to the crest of Niagara Falls. The Canadian or Horseshoe Falls, so named because the crest retains the shape of a somewhat distorted horseshoe, are about 1,200 feet wide across or about 2,500 feet around the crest. For the most part the water falls sheer into the Maid-of-the-Mist Pool, a drop of about 170 feet. There are small accumulations of talus at the flanks.

The American Falls has a crest length of 810 feet in the main section. The Bridal Veil Falls add a further 60 feet to the crest length. The water plunges vertically with distances ranging from 70 to 110 feet. At the foot of the falls, the talus slopes down to the pool water line, about 400 feet from the face of the fall, and ranges in height from 60 to 100 feet at the face of the American Falls.

Lower Niagara River: The Niagara Gorge, which begins at the Horseshoe Falls, extends for seven miles downstream thereof to the foot of the escarpment at Queenston, Ontario. The upper 2-1/4 miles of this reach of the river is known as the Maid-of-the-Mist Pool. The Pool has a normal fall of approximately 5 feet over its course and is navigable for practically the entire distance. The Maid-of-the-Mist Pool is terminated by the Whirlpool Rapids which extend downstream for a distance of approximately 1 mile. The water surface profile drops about 50 feet over their course and velocities reach as high as 30 fps. The Whirlpool, a basin about 1,700 feet long and 1,200 feet wide with depths up to 125 feet, marks the location where the river makes a near right-angled turn. As a result water coming from the rapids forces a rotation in the Whirlpool before it is discharged through its narrow outlet. Below the Whirlpool, there are another set of rapids which drop approximately 40 feet. From Queenston, where the river emerges from the Gorge, the water is discharged to Lake Ontario at Niagara-on-the-Lake, Ontario. The remaining section of the river is about 2,000 feet wide and is navigable over its course.

4.1.2 Existing Regulatory Works

To fulfill the objectives of the 1950 Niagara Diversion Treaty, a control structure was constructed in lower end of the Chippawa-Grass Island

Pool approximately 4,500 feet upstream of the Horseshoe Falls. The structure, which consisted of 13 100-foot gates, was constructed between 1954 and 1957 in a section where the river was approximately 3,800 feet wide. The total length of the structure was 1,500 feet.

Due to the expansion of power facilities, which were put into operation in December 1961, the control structure was found to provide inadequate control. As a result, 5 additional 100-foot gates were constructed between 1961 and 1963. A man-made island, now called Tower Island, was placed during construction and has been permanently retained at the end of the structure. It extends about 150 feet out from the gated structure.

The control structure, in combination with the power diversions, is operated by the power entities so that a flow of not less than 100,000 cfs is maintained over the Falls during the daylight hours of the tourist season and a flow of not less than 50,000 cfs at other times. The International Niagara Board of Control's Directive, dated June 30, 1955, required the power companies to operate the control structure such that the daily mean stage of the Chippawa-Grass Island Pool did not vary more than 0.50 foot, nor monthly mean stage more than 0.30 foot from the pre-1953 hydraulic regime.

The Board of Control's Operating Procedures were revised, effective March 1, 1973, according to procedures defined in a new directive to the Power Entities dated February 27, 1973. In essence, the new operating procedures are designed to maintain the levels of Chippawa-Grass Island Pool as near as may be practicable to its long-term mean natural level of 561.00 feet (IGLD) as recorded at Material Dock gauge. The implications of this new procedure on the methodology is detailed in Section 4.4.3.

4.1.3 Power Facilities and Flows

Data on the existing power developments are summarized on Table G-6. All power diversions are made in compliance with the 1950 Niagara Diversion Treaty so that the criteria as outlined in paragraph 4.1.2 above are met. A brief description of the plants and the corresponding diversions follows. A detailed discussion on these is given in Appendix F, Power.

United States Power Plants: The Robert Moses Plant, which is operated by the Power Authority of the State of New York (PASNY), diverts water from Chippawa-Grass Island Pool above the control structure via two covered conduits to the power plant forebay. From the forebay, water may be either discharged through the main plant to the Niagara River downstream of the Whirlpool or pumped to a storage reservoir and later used by the main plant and/or the Lewiston Pumping-Generating Plant for peaking operations. Operation of the Robert Moses plant began in 1961. In recent years, the monthly mean diversion has ranged from 41,900 cfs to 87,500 cfs. This plant has diverted up to 105,000 cfs.

Canadian Power Plants: There are a total of six plants operating on the Canadian side. They are: (1) Sir Adam Beck Plant (consisting of Unit 1 and Unit 2); (2) Pumping/Generating Plant; (3) Ontario Plant; (4) Toronto Plant (deactivated in April 1974); (5) Rankine Plant; and (6) DeCew Falls

TABLE G-6

EXISTING HYDRO-ELECTRIC POWER DEVELOPMENT
NIAGARA RIVER

Power Plant	Owner[1]	Source of Water Supply	No. of Units	Net Head Used for Power	Installed Capacity	Diversion Capacity
				(Feet)	(kw)	(cfs)
Robert Moses Niagara	PASNY	Niagara River	13	300	1,950,000)	105,000
Lewiston Pump Generating	PASNY	Niagara River	12	85	240,000)	
Sir Adam Beck No. 1	HEPCO	Niagara River	10	294	403,900)	70,000
Sir Adam Beck No. 2	HEPCO	Niagara River	16	292	1,223,000)	
Pump Generating	HEPCO	Niagara River	6	60-85	176,700)	
Ontario Power	HEPCO	Niagara River	12	205	101,500	10,700[2]
Toronto Power (deactivated April 1974)	HEPCO	Niagara River	7	134	64,800	15,300[2]
Rankine	CNPC	Niagara River	11	126	94,700	10,600[2]
DeCew Falls (2 plants)	HEPCO	Welland Canal	8	266 & 283	147,100	6,400

[1] PASNY - Power Authority of the State of New York
HEPCO - Hydro-Electric Power Commission of Ontario
CNPC - Canadian Niagara Power Company

[2] Because of the greater efficiency and head of the Sir Adam Beck plants, these plants normally do not develop power unless excess water is available.

Plants. All these plants are owned and operated by Ontario Hydro with the exception of the Rankine Plant which is owned and operated by Canadian Niagara Power Company.

The Sir Adam Beck Plants divert water from Chippawa-Grass Island Pool above the control structure to a common forebay. Water is carried to the forebay by a set of twin tunnels and an open canal which is located, in part, in the Welland River. The flow in the Welland River is reversed over this short distance. The Beck Plants also have pumped storage facilities and operate them in a way similar to that of Robert Moses Pumping-Generating Station. In recent years, the monthly mean diversion has ranged from 38,000 cfs to 63,400 cfs.

The Ontario, Toronto and Rankine Plants, which are also called the Cascades Plants, divert water from below the control structure to the Maid-of-the-Mist Pool. The Ontario Plant diverts water from the River at Dufferin Island by means of a gathering weir and conveys it to the power plant, located below the falls, via closed conduits. The Toronto and Rankine Plants also divert water from the river by means of a gathering weir extending from the plants themselves, then discharge it through turbines located in a wheel pit under the plant. Water is then conveyed by tailrace tunnels to outlets under the Falls where it is discharged into the Maid-of-the-Mist Pool. The use of the Cascades Plants depends upon the availability of water surplus for diversions to the high head plants at Queenston. Their operation is therefore sporadic and irregular. During 1971, the maximum diversions at the Toronto, Ontario and Rankine Plants were 2,500 cfs, 8,300 cfs and 9,100 cfs, respectively. The Toronto Plant was deactivated in April 1974 due to the deteriorated condition of the plant.

The DeCew Falls Power Plants divert water from Lake Erie through part of Welland Canal and discharge it through Twelvemile Creek to Lake Ontario. During the past 20-year period, the diversion has approximated 5,900 cfs (for power generation only).

4.1.4 Navigation Facilities and Flows

Through traffic between Lake Erie and Lake Ontario utilizes the Welland Canal. The Black Rock Canal parallels the upper reach of the Niagara River from Buffalo Harbor to the downstream portion of Squaw Island at which point the natural channel has been deepened, extending to Tonawanda, New York. The New York State Barge Canal extends from Tonawanda to the Hudson River with an extension to Lake Ontario at Oswego. The following subsections summarize the existing navigation facilities and flows in detail.

Welland Canal: The Welland Canal, with a minimum depth of 27 feet, connects Lake Erie at Port Colborne, Ontario, approximately 18 miles west of the head of the Niagara River, with Lake Ontario at Port Weller, Ontario, 9 miles west of the mouth of the river. The canal is approximately 27 miles long and overcomes a difference in level of about 326 feet by a series of 7 lift locks and 1 guard lock. Ships 730 feet or less in overall length and 80 feet or less in width may transit the canal. The operation of the canal requires a flow varying from 100 to 2,100 cfs. Over the past 20-year period, the flow has averaged 1,200 cfs. Together with the water for the DeCew Falls Power Plant, the average diversion through the Welland Canal over the past 20-year period has averaged 7,100 cfs. For purposes of this study, the mean monthly diversion has been approximated at 7,000 cfs.

Black Rock Canal: The Black Rock Canal has a depth of about 21 feet. It provides an alternate route around the constricted and shallow reach at the head of Niagara River. Extending from Buffalo Harbor to the river above Strawberry Island, the canal is separated from the river by a series of stone and concrete walls and by Squaw Island. The Black Rock Lock, which has a lift of about 5 feet, is located near the lower end of the canal. Operation of the lock requires a flow usage of about 10 cfs.

The navigation channel rejoins the river below Squaw Island where the river widens and becomes placid. A navigation channel with a minimum depth of 21 feet below L.W.D. is maintained between the southern tip of Squaw Island and Tonawanda, New York. From Tonawanda to Niagara Falls, New York, opposite the Southern tip of Grand Island, a navigation channel with a minimum depth of 12 feet below L.W.D. is maintained.

New York State Barge Canal: The New York State Barge Canal has a depth of about 12 feet. It extends eastward from Tonawanda, New York, linking the Niagara River with the Hudson River near Albany, New York. Near Syracuse, New York, an extension runs northward into Lake Ontario at Oswego, New York. The diversion from the river averages 800 cfs on an annual basis with a minimum of 1,100 cfs being diverted during the navigation season. This diversion is considered relatively small, and, since water is withdrawn downstream of the constricted portion of the river near the Peace Bridge, the effect on the levels of Lake Erie is considered to be negligible.

4.1.5 Bridges, Ferries, Docks and Other Facilities

Two bridges linking the Province of Ontario and the State of New York are located at the Upper Niagara River. The Peace Bridge (Highway) crosses the river and the Black Rock Canal near its inlet at Lake Erie. The International Railroad Bridge crosses the river and the canal about 1.5 miles downstream of the Peace Bridge. The North and South Grand Island Highway Bridges traverse the Tonawanda Channel at Kenmore and Niagara Falls, New York. Currently, there are no ferries in operation on the Upper Niagara River.

Docks for recreational crafts are located at many points along the upper Niagara River with a particularly high concentration along Grand Island. There are commercial docks for bulk commodities along the United States shoreline between the lower end of Black Rock Canal and North Tonawanda. The City of Niagara Falls, New York, has a municipal dock at the downstream end of the 12-foot navigation channel.

There are several municipal and industrial water intakes and outfalls in the upper river. Some of these have structures extending above the water surface. The Buffalo sewage treatment plant is located on the upper end of Squaw Island between the Black Rock Canal and the river.

4.1.6 Ice Problems

During winter, thin ice sheets may form in shallow areas of the river near shore. But the principal problem arises from the breakup of the ice field in Lake Erie, resulting in the subsequent passage of broken ice down the river. Lake ice may be broken up by either wind and/or wave action during the winter months or spring thaws. The latter case presents more difficulties. Ice floes with thicknesses up to 20 feet have been observed in the upper river. The Power Entities, PASNY and HEPCO, employ icebreakers in the vicinity of their intakes as well as clear an ice passage around the end of the control structure. Frazil and anchor ice conditions occur periodically, causing reductions in the cross-sectional area of the river channel and power intake openings, thus reducing flows available for power generation. However, frazil and anchor ice problems are considered secondary to those caused by the breakup of the Lake Erie ice field.

In 1964, with the approval of The International Joint Commission, the Power Entities installed a floating timber ice boom in Lake Erie, near the head of the Niagara River. The boom is fastened at intervals to anchors in the rock bottom. It is normally placed in December and removed in April. Its purposes are to facilitate early formation of an ice cover at the outlet of Lake Erie and to retain ice floes that may be created by mid-winter breakup of the ice cover. Under strong winds, the boom is designed to submerge therby allowing some of the ice cover to pass. When the pressure is released, the boom emerges to prevent continuing passage of ice. It has been generally successful in preventing ice jamming in the Niagara River. Normally, by the time the ice boom is removed, air temperatures are high enough to cause a rapid deterioration of the ice cover. As a result, the threat of a serious ice jam in the river and at the power intakes is substantially reduced.

4.1.7 Short Period Water Levels Fluctuations

Of the five Great Lakes, Lake Erie is the shallowest with an average depth of 62 feet. The prevailing wind over the Lake Erie basin is southwesterly which coincides with the longitudinal axis of the lake causing significant wind and wave set-up. Of more significance, however, is the oscillation of the lake surface produced by changes in wind and/or barometric pressure commonly referred to as a seiche. Wind produced seiches follow cessation or shift in wind direction after a period of relatively steady wind in one direction. A rise in water surface elevation, due to seiche, of 5 feet above the prestorm level can be expected annually at Buffalo. The maximum recorded seiche at the Buffalo gauge, based on a 15-minute instantaneous stage, was 8 feet and has been utilized in the design of regulatory works herein.

4.2 Assumptions

Assumptions made in studies of Lake Erie regulatory works are:

(1) The level of Chippawa-Grass Island Pool will be maintained in accordance with the operating procedures, directed by the International Niagara

Board of Control, as detailed in its Order of June 30, 1955. Revised operating procedures were instituted after the Regulatory Works studies had been completed. (See Section 4.1.2.)

(2) Flow diversions through the Welland Canal will not change.

(3) The ice boom will be kept in operation.

(4) Existing lake stage-river discharge relationships will be maintained through any construction period.

(5) Due to alterations to the existing hydraulic regime which would be required for regulation of Lake Erie, retardation of Niagara River flows, which has been caused in the past by ice and weed conditions, would be minimal.

No assumptions of post-project stage-discharge relationships in other reaches of the upper Niagara were made; rather, it was decided that stages resulting from project implementation, and other cause-effect relationships would be quantitatively determined, their impacts on navigation and other interests evaluated, and such impacts then reported. The effects of changed relationships could then be modified by plan reformulation or could simply be weighed against projected plan benefits.

4.3 Methodology - Total Regulation

The following is a brief description of the methodology employed. A more detailed account of the mathematical models developed and their applications may be found in a report entitled, "Development, Calibration and Application of Mathematical Models of the Connecting Channels of the Great Lakes," an abstract of which is contained in Annex C of this Appendix.

4.3.1 Steady-State Mathematical Model

A steady-state mathematical model was developed for the Upper Niagara River extending from Chippawa-Grass Island Pool to the head of the river at Buffalo, New York. Essentially the model is a computer program which performs backwater computations under steady-state flow conditions. Fundamental open channel flow equations were applied using the standard step method of back-water computations. Cross sections, taken at hydraulically strategic locations, were obtained from the most recent hydrographic surveys of the river. The mathematical model was calibrated using flows and levels obtained by joint measurement programs conducted by Water Survey of Canada, Environment Canada, and Lake Survey District, U. S. Army Corps of Engineers, during 1967, 1968 and 1969.

Objectives: The primary objective of the mathematical model was to determine the nature and extent of channel excavation required to meet the hydraulic requirements of any given regulation plan involving the regulation of Lake Erie. The model was also used to determine the length and location of the flow control structure and the associated shore protection works. For any given scheme, the resulting water surface elevations and average discharge velocities, below the structure, were constrained, within tolerable limits, so that they would not exceed those which would occur under natural conditions.

Parameters and Constraints: As indicated above, the mathematical model is essentially a computer program which performs back-water computations employing the standard step-method and standard parameters. Head losses between sections were computed using Manning's equation. In addition, variable coefficients were used to compensate for head losses due to expansion and contraction of the river channel, and for sudden changes in width and depth. All velocity heads were adjusted by a kinetic energy coefficient to account for using the mean velocity at each section.

The existing river profile was classified as a mild (M_1) profile with no one section or reach of the river having 100 percent of the hydraulic control. To ensure that the backwater computations were carried out in the direction of control, critical depth was checked at each section.

Due to the presence of islands in the Upper Niagara River, such as Grand Island, Tonawanda Island and Navy Island, an iterative technique was developed to determine the division of flow in each of the channels.

Development and Calibration: Cross sections of the river channel extending from the Slaters Point gauge, located at the head of Chippawa-Grass Pool, to Lake Erie were incorporated into the mathematical model. At each section, basic data such as cross-sectional area, top width, L.W.D. elevation and the distance from it to the adjacent upstream section were extracted and used as input to the model. The locations of the water level gauges and cross sections used for providing data for the model are shown on Figure G-62. The majority of discharge measurements were taken in the late spring, early summer and late fall periods which are assumed to be generally free of ice and weed retardation. Measurements taken during the mid-summer months were used only as a qualitative check on the model.

Flow measurements were grouped into periods, the duration of which depended on the consistency of flow. Manning's 'n' was computed for each reach using the mathematical model. In some reaches, particularly those from Frenchman's Creek at Grand Island to Buffalo, it was found that Manning's 'n' varied significantly with level and flow, attributed to weed growth during the summer and fall months. Figure G-63 illustrates results of the calibration at each of the gauges. Commensurate with the determination of Manning's 'n', the values of the variable coefficient used to define head losses due to contraction and expansion were analyzed.

Since the model was calibrated over a narrow range of flow (200,000 - 235,000 cfs), it was necessary to verify its performance over a wider range. This was done by computing the profile from recorded monthly flows. Tests made for a random sampling of data indicated that the model satisfied the natural conditions, within an acceptable tolerance, over a wide range of flow conditions. Based on these findings, a stage-discharge relationship was developed for the Buffalo gauge. A comparison of the relationship with the pre-1953 stage-discharge equation is illustrated on Figure G-64. Water surface profiles for high, medium and low flow conditions are shown on

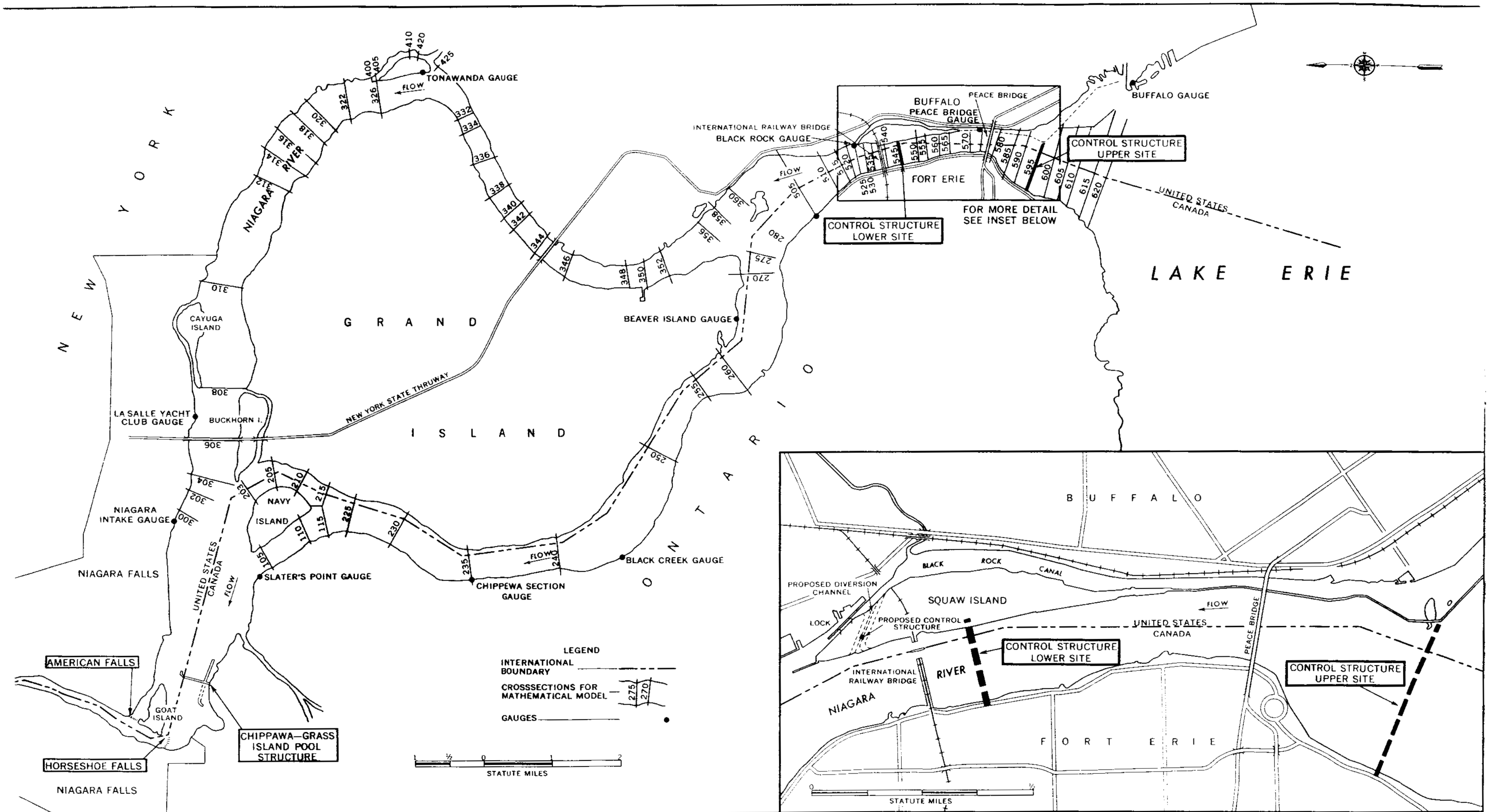


Figure G-62

UPPER NIAGARA RIVER—LOCATION OF WATER LEVEL GAUGES, ALTERNATIVE CONTROL STRUCTURES AND CROSS SECTIONS FOR MATHEMATICAL MODEL

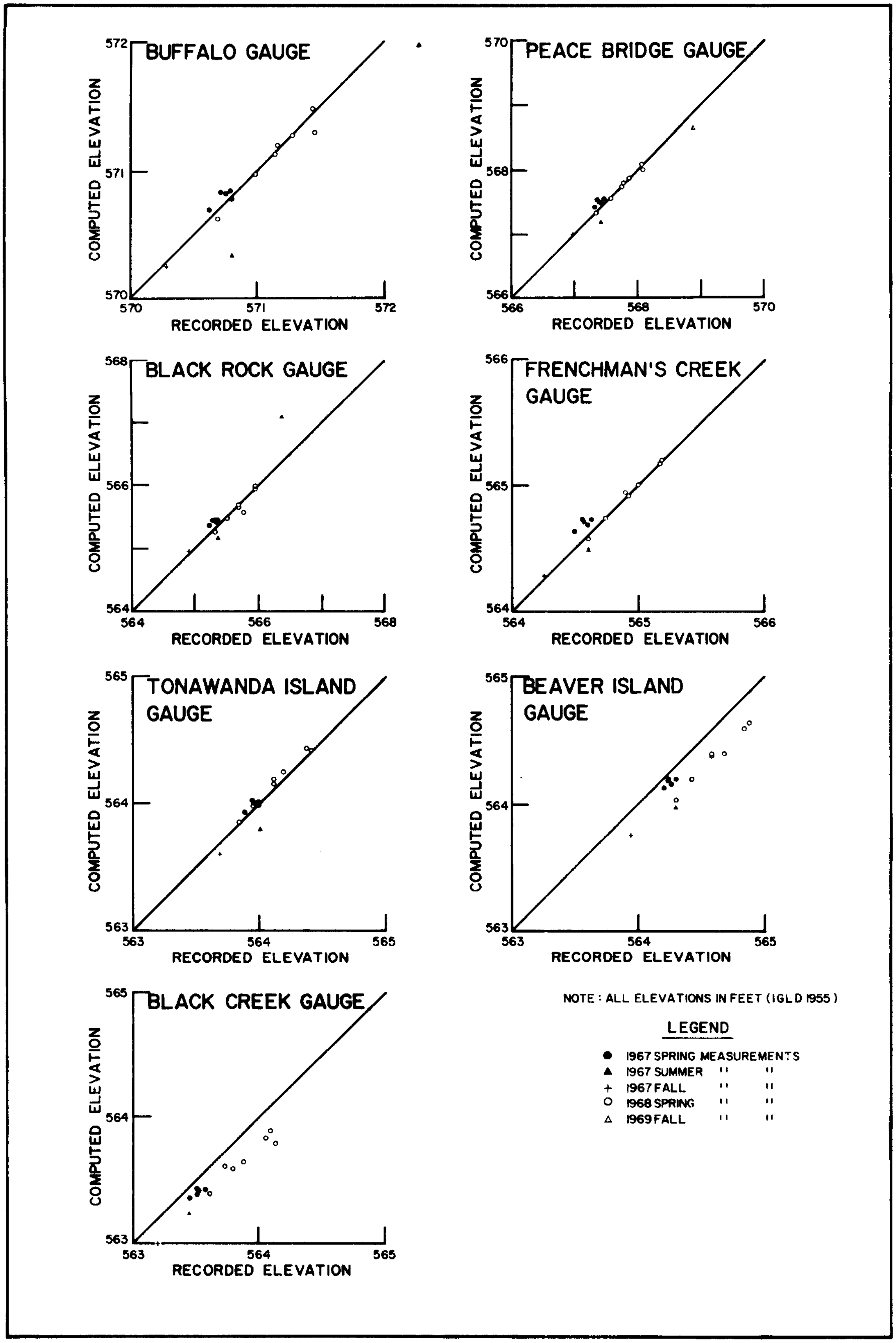


Figure G-63
UPPER NIAGARA RIVER MATHEMATICAL MODEL—RESULT OF CALIBRATION

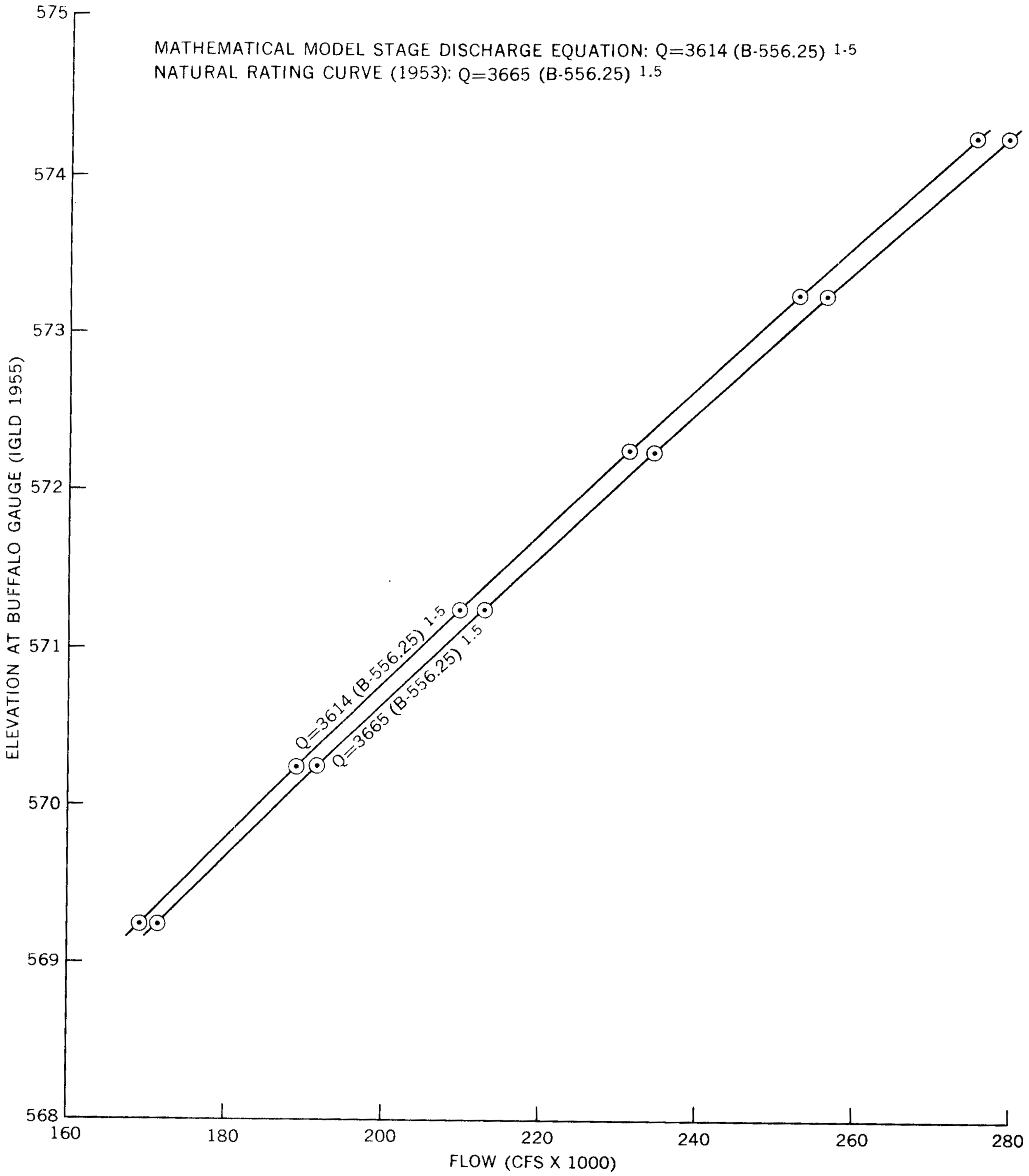


Figure G-64
UPPER NIAGARA RIVER— BETWEEN MATHEMATICAL MODEL
AND BUFFALO RATING CURVE (1953 OUTLET CONDITIONS)

Figure G-65. Since the hydraulic requirements of any given regulation plan are met by regulatory works as determined by the mathematical model, the stage-discharge relationship developed from the model was used to evaluate the degree of channel capacity increase and decrease requirements.

Application: The mathematical model was used to select the location of the flow control structure. Two alternate sites for the location of the control structure were investigated; namely, the "upper site" located some 3,600 feet upstream (south) of the Peace Highway Bridge, and the "lower site" located some 1,000 feet upstream (south) of the International Railway Bridge. A map of the area, illustrating the relative location of each site, is shown on Figure G-62.

The model was applied in a similar manner for both sites. Backwater computations were initiated at Chippawa-Grass Island Pool using the level determined from the Slater's Point stage discharge rating stated as follows:

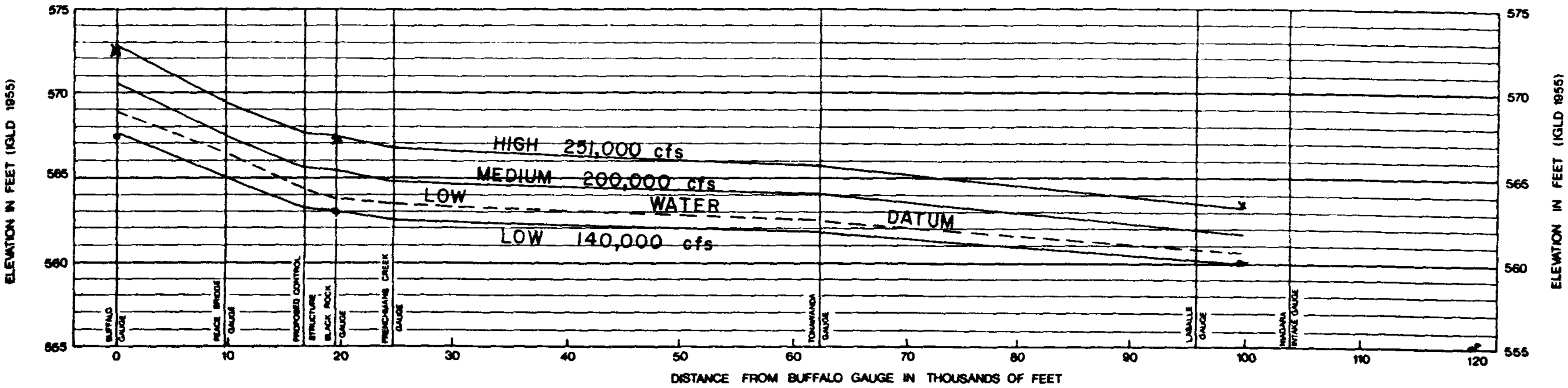
Slater's Point = 560.58 + 0.301 (Niagara River Flow - 160,000)/10,000

At those sections where channel improvements were deemed necessary, the cross-sectional area was increased by the appropriate amount. Computations were terminated at the proposed control structure location which would be a hydraulic control. Head loss through this control was calculated, as outlined in Section 4.3.3, and backwater computations were continued upstream to the Buffalo gauge at Lake Erie.

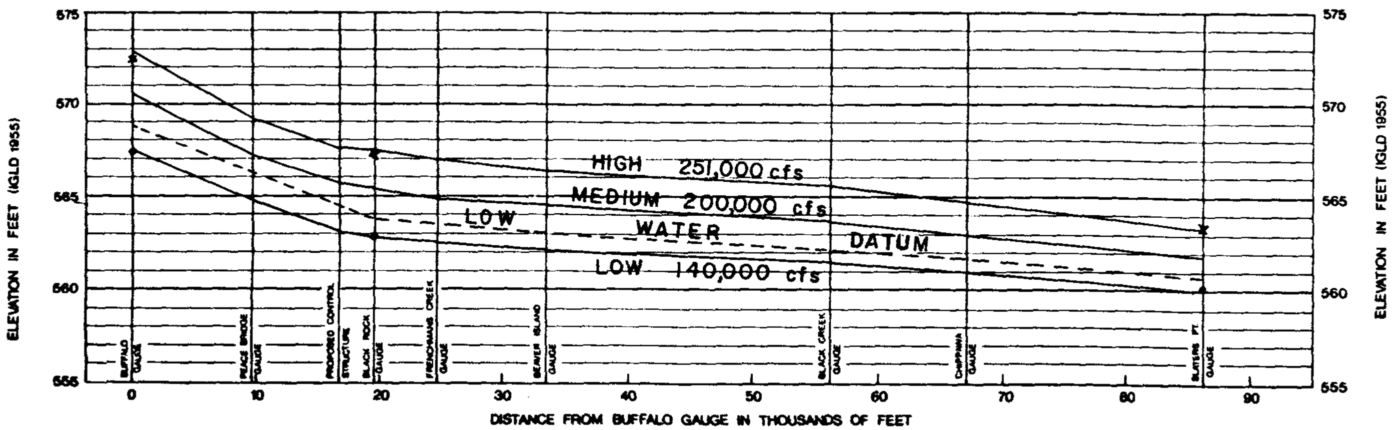
Because plans of regulation were initially not available, a range of hydraulic conditions which would likely encompass those of the selected plans were simulated and used for design purposes. Because the size and location of the control structure and the extent of channel improvements are interrelated, optimization studies were carried out to determine the minimum cost of all regulatory works. The design and cost estimates of regulatory works are presented in Section 4.3.3.

4.3.2 Unsteady State Mathematical Model

A very limited unsteady state mathematical model was developed for this study. Its purpose was to demonstrate whether or not Lake Erie seiches would produce the "tidal bore" buildup phenomenon which has been observed in other areas having topographical configurations somewhat similar to those of the semi-enclosed basin that would be created by construction of a regulatory structure in the Upper Niagara River. The objective was to determine whether or not a higher crest elevation would be required to prevent overtopping of the structure if this phenomenon were to occur. The model utilized two separate but complementary techniques that produced similar conclusions. It demonstrated that the river would rise along with the lake, but with the discharge anywhere in the anticipated range, the resulting friction gradient would produce a river stage somewhat lower than the lake stage. Additional calculations to explore the possibility of resonant oscillations in the reach above the regulatory structures indicated the period of free oscillation to be such that lake wind tides would not likely generate resonant oscillation.



TONAWANDA CHANNEL



CHIPPAWA CHANNEL

WATER SURFACE PROFILES COMPUTED VIA MATHEMATICAL MODEL

x HIGHEST RECORDED MEAN MONTHLY LEVEL

• LOWEST RECORED MEAN MONTHLY LEVEL

A more comprehensive unsteady state model would be required in order to evaluate the effects of short period fluctuations in the Upper Niagara River, such as within-the-day power variations. The main purpose of such a model would be to determine the resultant stages and velocities along the river system, and, if need be, to determine the nature and extent of additional regulatory facilities.

4.3.3 Design and Cost Estimates

As indicated in Section 4.3.1, two alternate sites for the location of the control structure were investigated. In order that a valid comparison of cost between the two sites could be made, common design criteria were utilized throughout. The design and cost estimates were prepared to: (1) determine the better site location, (2) provide a set of cost curves to be used as input during the formulation of regulation plans, and (3) to form a basis for the evaluation of selected regulation plans (as presented in Section 6). The following is a summary of the studies carried out:

Control Structures: The following paragraphs, unless otherwise noted, are generalized for both sites investigated in light of the common design criteria utilized.

1. Topographic and Geotechnical Characteristics of the Upper Site. At the upper site, the control structure would be situated on the natural rock ledge which protrudes into Lake Erie. This rock ledge provides virtually full hydraulic control of the Niagara River discharge. This site is located at the funnel-shaped entrance of the river where its width is approximately 3,660 feet, bounded by the Canadian shoreline and Bird Island Pier. The elevation of the river bed varies from 548 feet in mid-channel to 565 feet near the Canadian shore and to 566 feet near Bird Island Pier. Very little overburden is evident in this shallow reach of the river. Rock outcroppings are in evidence towards Bird Island Pier under low water conditions. A cross-sectional view of the site is shown on Figure G-66.

The description of the bedrock that follows is based upon visual examination of one drill hole located at Bird Island Pier; however, drill holes taken for other projects in the area further substantiate this evidence. In mid-channel the bedrock is exposed at elevation 552 feet. Towards the Canadian shoreline, the rock is overlain by 4 to 5 feet of sand and gravel while towards the U. S. shoreline (Bird Island Pier) the rock is exposed with outcroppings protruding during low flow conditions. The bedrock is dense, crystalline limestone with chert and calcite nodules that frequently display a marbled appearance manifested in flow-like lines. The rock is characterized throughout with stylolitic partings, a few of which are open. The partings are generally sealed due to recrystallization along bedding planes of irregular amounts of insoluble matter. With the exception of more frequent core partings near the surface, the bedrock is considered competent throughout as a medium on which the structure can be constructed.

2. Topographic and Geotechnical Characteristics of the Lower Site. At the lower site, the control structure would be located in one of the deepest and narrowest sections of the Upper Niagara River. The width of the

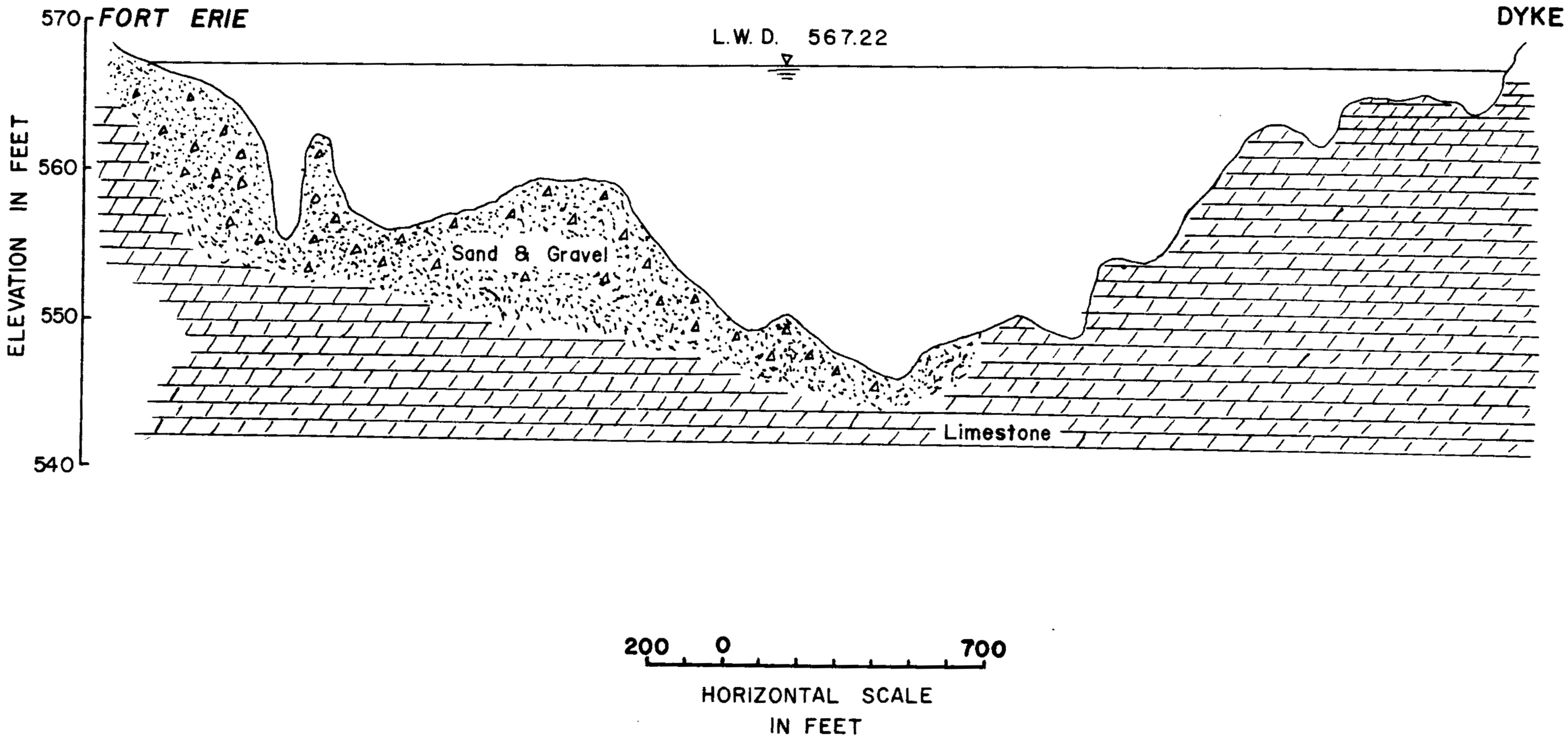


Figure G-66
UPPER NIAGARA RIVER—HYDROGRAPHY AND GEOLOGY OF UPPER SITE—ALTERNATE CONTROL STRUCTURE

section extending between the Canadian shore and Squaw Island is approximately 1,880 feet. The river bed, which is distinctly V-shaped, ranges in elevation from 525 feet in mid-channel to 585 on the Canadian shore and 565 on Squaw Island. A rock-fill dyke running along Squaw Island protects the sanitary land fill from erosion. A cross-sectional view of this site is shown on Figure G-67.

The description of the bedrock that follows is based upon the visual examination of several drill holes, four of which were taken along or near the axis of the control structure, which was carried out by William Trow and Associates (Hamilton) Limited. The surficial bedrock of the river channel at elevation 525 feet is anhydrite/gypsum, dolomite/and shale of the Saline formation. Towards the Canadian shore, the surficial bedrock, varying in elevation from 530 to 545 feet, is anhydrite/gypsum, dolomite/limestone and shale of the Bass Island formation, while on the United States shore, the bedrock is generally dolomite of the Akron formation. The surficial bedrock is slightly weathered exhibiting stress relief jointing and is water bearing for a depth of approximately 10 feet. While the bedrock is generally competent, there is a possibility of loss of support in case of any internal erosion of the anhydrite/gypsum cavities. It is believed that low pressure grouting up to about 30 feet below the structure will consolidate the foundation adequately.

3. Hydraulic Design. In view of the fact that the design stages and flows had not been formulated prior to these studies, the structure had to be designed and cost estimates made for a number of combinations of channel capacity increase and decrease conditions. It was generally accepted early in the study that complete control of Niagara River flow is not required in view of the flow requirements as set forth in the Niagara Diversion Treaty of 1950. As a result, considerable cost savings were achieved. A minimum design flow of 125,000 cfs at a Lake Erie stage of 568.7 feet was selected such that: (1) all conceivable regulation plans would be satisfied, and (2) within-the-day power variations, although not specifically considered in the design stages, could be achieved. It was later shown that insignificant savings would be achieved by increasing this minimum flow. The maximum design flow was taken as 280,000 cfs at a Lake Erie stage of 573 feet, representing a channel capacity increase of 30,000 cfs. However, in view of the consideration that a regulation plan may require a greater channel capacity increase and that within-the-day power variations may be evaluated, a maximum design flow of 320,000 cfs at the above stage was also considered.

Due to relatively high tailwater elevations at either site, submerged uncontrolled flow conditions were assumed. The hydraulic length of structure under this flow condition is defined as follows:

$$L = Q/(C_s h_s \sqrt{2gH})$$

Where: L = Hydraulic length of structure in feet

Q = Niagara River flow in cfs

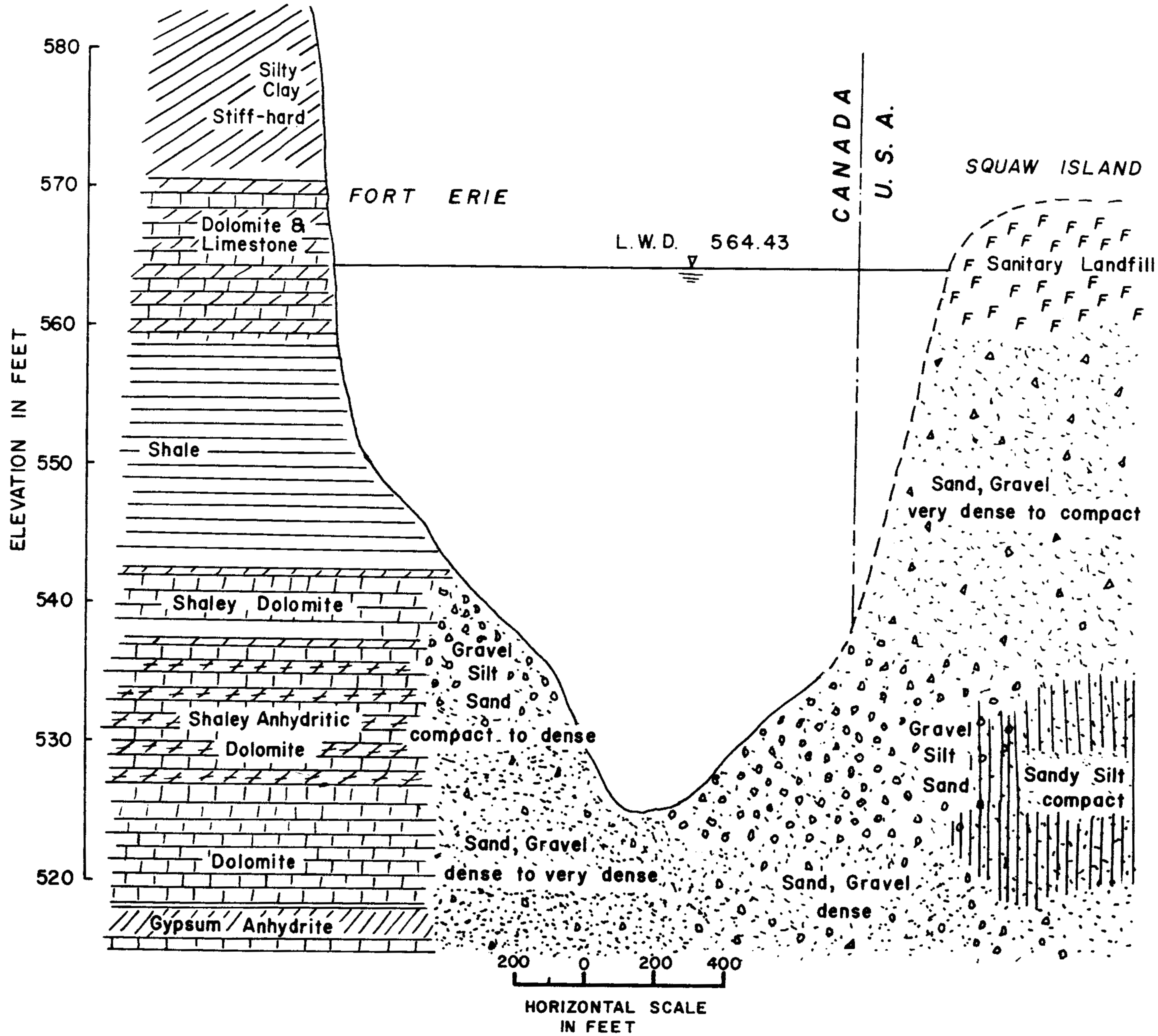


Figure G-67
UPPER NIAGARA RIVER—HYDROGRAPHY AND GEOLOGY OF LOWER SITE—ALTERNATE CONTROL STRUCTURE

C_s = Submerged discharge coefficients taken as 0.90

h_s = Tailwater depth referred to the crest in feet

g = Acceleration due to gravity taken as 32.2 feet per second per second

H = Differential head across structure in feet

The above formula was programmed into the Niagara River mathematical model and a hydraulic length was calculated for each flow condition. On examination of the above formula, it is apparent that the size of structure, the sill elevation and the location of dredging (affects tailwater elevation in some instances) will affect the channel capacity increase and hence the required volume of channel excavation. This is the subject of the paragraph entitled "Optimization Studies" hereunder.

The crest profile was determine based on a study of several profiles for similar structures tested and built by the United States Bureau of Reclamation. The geometry of the crest was based on monographs employing dimensionless parameters which are based on the differential head across the structure. In view of the low Froude Number (less than 2.0), the concrete sill block was extended to provide a stilling basin length, downstream of the gate sill, equal to 2.5 times the operating head over the crest. This was considered adequate for the dissipation of the hydraulic jump and may be decreased subject to further model experiments.

In view of the fact that maintenance of a gate may have to be carried out under high flow, seiche and/or ice floe conditions, one additional gate was provided for this purpose.

4. Gate Type. The following considerations were taken into account when selection of the gate type was made: (1) the gate must be capable of passing ice up to 20 feet in thickness, (2) the normal operating head is in the order of 5 feet, (3) at times, under seiche conditions, the operating head may increase to 15 feet, and the gates must respond such that opening and/or closing can be carried out quickly to avert emergency situations. In view of these criteria, submersible tainter gates were selected. The maximum possible span would be selected such that minimum obstruction to ice would be achieved. Based on a limiting design force of 3,000 kips per trunnion, the maximum possible spans of 100 feet, for the upper site, and 75 feet for the lower site were selected. In view of their submersible operation, an additional skin plate was provided to act as a spillway during the passage of ice. Synchronized electrical motor hoists, which utilize Galle chains for lifting, were provided at each end of the tainter gate. Each gate would be provided with a position transmitter with a mating receiver indicator at the central control point. Side seal heaters would be provided to permit winter operation of the gates.

Stoplog recesses are provided upstream and downstream of each gate to enable dewatering of the gate area for repairs and maintenance. A stoplog crane is provided on the deck over the upstream stoplog recesses while downstream stoplogs would be installed by floating plant. Upstream stoplogs would be stored in the body of the dam, accessible by the stoplog crane, while downstream stoplogs would be stored on shore. Details of the tainter gate are shown on Figure G-68.

5. Structural Design. The control structure is a series of pier buttresses supporting the tainter gates and extending to the bottom of the sill. The width of pier, based on a literature survey of existing practice was taken as 15 feet for each site. A stability analysis of a monolith com prising one pier and half a bay on either side was carried out assuming: (an uplift force on the base corresponding to 100 percent head at normal lak level, (2) an ice loading of 10 kips per lineal foot acting over a four-foo section below the maximum headwater level, and (3) an earthquake with a hor zontal intensity equal to 0.1g (zone 1). The loading conditions examined were: (1) Stoplogs in place, gate area dewatered, maximum headwater level, minimum tailwater level and full uplift, (2) Condition 1 plus ice, and (3) Condition 1 plus earthquake. Results of the stability analysis are summari on Table G-7. At either site a 20-foot thick block of concrete was require for the base of the structure to provide an adequate safety factor against flotation.

6. Layout. The layout of the control structure is basically the same at either site. The dimensions of course are dependent on the site lo tion and regulation plan. Referring to the generalized layout as shown on Figure G-68, the control structure consists of the following factors: (1) a two-storey concrete control building, with a floor area of 5,000 sq. ft., located on the Canadian shoreline and constructed on the platform used for construction of the control structure; (2) a rock-fill dyke, extending from the Canadian shoreline to the abutment of the control structure, which is overlain by a gravelled road surface; (3) the control structure which consi of gated and ungated sections resting on a foundation consisting of a 20-foot-thick block of concrete; (4) the piers and abutments, with both upstre and downstream stoplog recesses, which are 15 feet in thickness and extend from elevation 585.0 feet and 584.0 feet to elevation 502 feet and 485 feet at the upper and lower sites, respectively; wingwalls, splayed at 30° to th flow, with a top width varying from 15 feet to 5 feet are provided upstream of the abutments; (5) a bridge deck formed by reinforced concrete girders placed such that a gap of five feet, covered with a removable steel deck, is available for the placing of stoplogs; (6) a movable gantry crane is pro vided for the placing of upstream stoplogs which are stored in the body of the dam; and (7) a rock-fill dyke extends between the easterly abutment and the United States shoreline.

The gated and ungated sections of the control structure were loca in the deepest part of the channel to minimize local dredging for training the flow through the control structure. The ungated section consists of a even multiple of a bay length such that, if necessary in the future, additional gates may be added with ease.

7. Construction. As stated in paragraph 4.2, the existing lake stage-river discharge relationships will be maintained through any constru

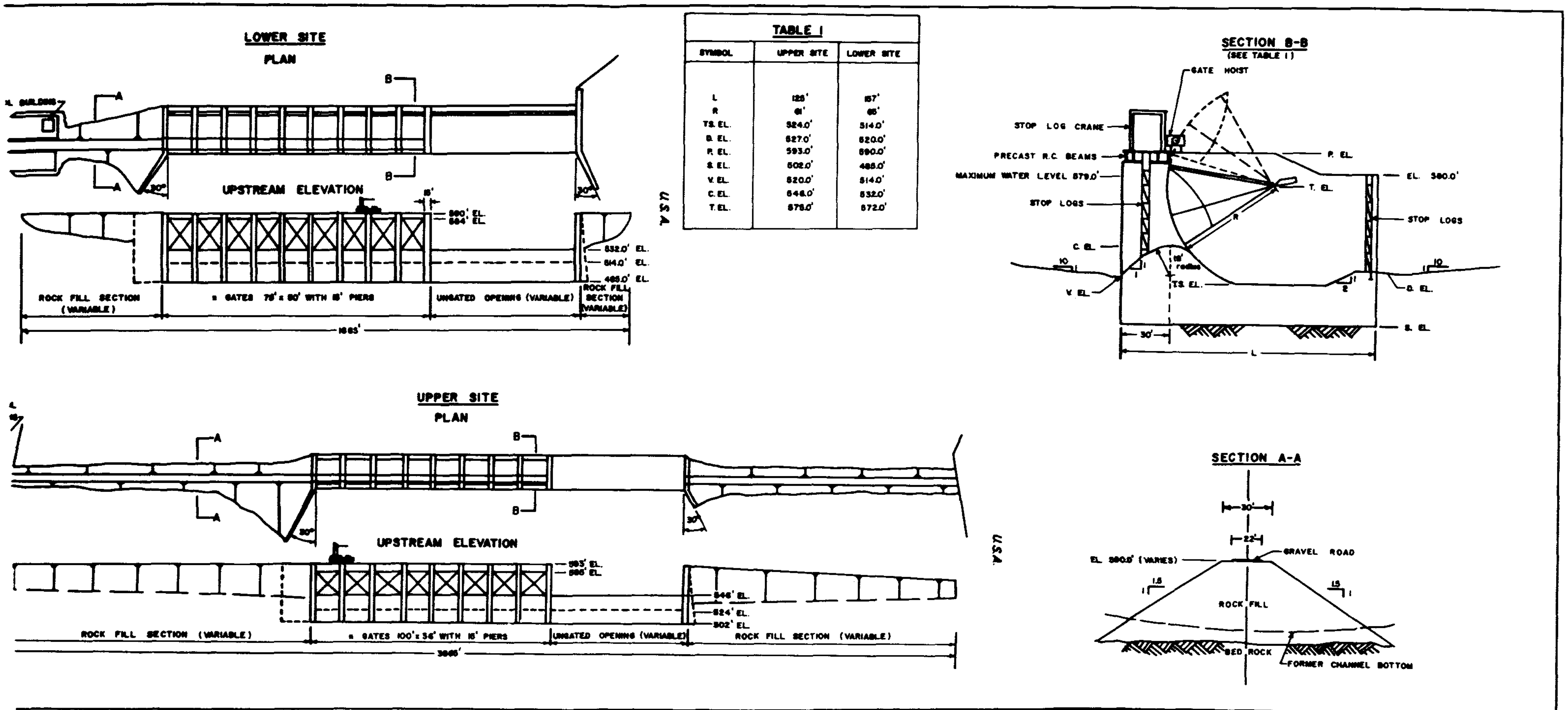


TABLE I

SYMBOL	UPPER SITE	LOWER SITE
L	125'	157'
R	61'	65'
TS. EL.	524.0'	514.0'
B. EL.	527.0'	520.0'
P. EL.	593.0'	590.0'
S. EL.	502.0'	485.0'
V. EL.	520.0'	514.0'
C. EL.	546.0'	532.0'
T. EL.	575.0'	572.0'

Figure G-68
UPPER NIAGARA RIVER—GENERALIZED LAYOUT AND TYPICAL SECTIONS OF PROPOSED CONTROL STRUCTURE

tion period. In essence, this means that any part of the river is cut off due to the construction of the controls structure must be compensated for by channel excavation. It is estimated that construction can be carried out in three and four annual stages at the lower and upper sites respectively, provided the quantity of excavation does not exceed five million cubic yards. Cellular cofferdams, made up of 3/8 inch thick steel plates, and filled with available overburden are proposed. Pertinent dimensions and quantities are given in Table G-8. A construction platform, 200 feet by 200 feet, consisting of rock fill is proposed along the Canadian shoreline. At the lower site, access may also be possible from Squaw Island. A portable concrete batching and mixing plant can be installed on the platform. Aggregates can be either obtained from nearby quarries or from the dredged material which would be crushed as required.

8. Unit Costs. Unit costs for plant, labor and materials were obtained from past projects carried out in the area by Ontario Hydro and the U. S. Army Corps of Engineers. All unit costs are expressed in 1971 price levels. An estimate of 20 percent was applied as an allowance for contingencies to obtain the total direct costs of the works. Indirect costs which include allowances for detailed investigations, model experiments, engineering design, and construction supervision and administration were estimated at 15 percent of the total direct costs. Based on a cost estimate of particular layouts, unit component costs were developed as shown on Table G-9.

9. Cost Estimates. Based on the above unit component costs, cost curves were developed for each site as shown on Figure G-69. The cost estimates from these curves later serve as input into optimization studies as explained below. It should be pointed out that various sill elevations, particularly for the upper site, were investigated but, by and large, geotechnical considerations were found to prevail.

Channel Improvements: As indicated in paragraph 4.3.1, the determination of the amount and location of channel improvements was carried out using the Niagara River mathematical model. Basically, there are two major areas for channel excavation, namely: (1) the constricted reach below Peace Bridge, and (2) the entrance and exit channels of the control structure. The following paragraphs summarize the studies that were carried out.

1. Nature and Extent. A series of ten channel excavation plans were developed for each site ranging in volume from two to fifteen million cubic yards, which for all practical purposes, were considered to be entirely rock excavation. The location of the excavation was optimized for each of these plans utilizing the mathematical model. Based on the results of these plans, a width of 800 feet was selected as the most optimum. This width was modified in the approach and exit channels of the control structure.

Each of the ten basic excavation plans were combined with varying sizes of the structure at each site, because, as indicated earlier, the size of the control structure and volume of excavation conflict in terms of channel capacity increase. Graphs were prepared for each site and each design flow, an example of which is shown on Figure G-70.

Table G-7
DESIGN OF NIAGARA RIVER REGULATORY WORKS
RESULTS OF STABILITY ANALYSIS

State	Loading Condition	Factor of Safety Upper Site	Lower Site	Acceptable
Flotation $\frac{W}{u}$	1, 2, 3	1.28	1.24	1.1
Sliding $\frac{H}{W}$	1	0.3	0.29	0.6
	2	0.55	0.51	0.75
	3	0.85	0.82	0.90
Shear Friction $\frac{f\ A + c\ A^*}{H}$	1	10.0	10.0	5.0
	2	5.3	5.9	4.0
	3	3.96	3.55	3.75
Overturning $\frac{Mr^{**}}{Mo}$	1	1.32	1.26	1.20
	2	1.22	1.20	1.10
	3	1.30	1.26	1.10
Bearing Maximum (ksf)		12.50	14.70	20.00

*f (coeff. of friction) = 0.6, c(shear) = 20 psi. (2.88 ksf)
**The resultant falls within the middle third in all cases.

Table G-8
DESIGN OF NIAGARA RIVER REGULATORY WORKS
DETAILS OF CELLULAR COFFERDAMS

	Upper Site	Lower Site
Height (feet)	35	55
Diameter (feet)	40	63
Spacing (feet)	46	69
Steel (Tons/Unit)	72.5	170
Fill (cu yd/Unit)	1900	5000
Number of Units Required	30	21

Table G-9
DESIGN OF NIAGARA RIVER REGULATORY WORKS
UNIT COMPONENT COSTS OF STRUCTURE

	Upper Site	Lower Site
Rock fill ($/linear foot)	610	1,090
Ungated Control Structures ($/linear foot)*	17,500	25,100
Gated Control Structure ($/Bay)**	3,550,000	4,280,000

*Includes abutments required at either side
**Bay, centre to centre between piers, equals 115 feet for the upper site and 90 feet for the lower site.

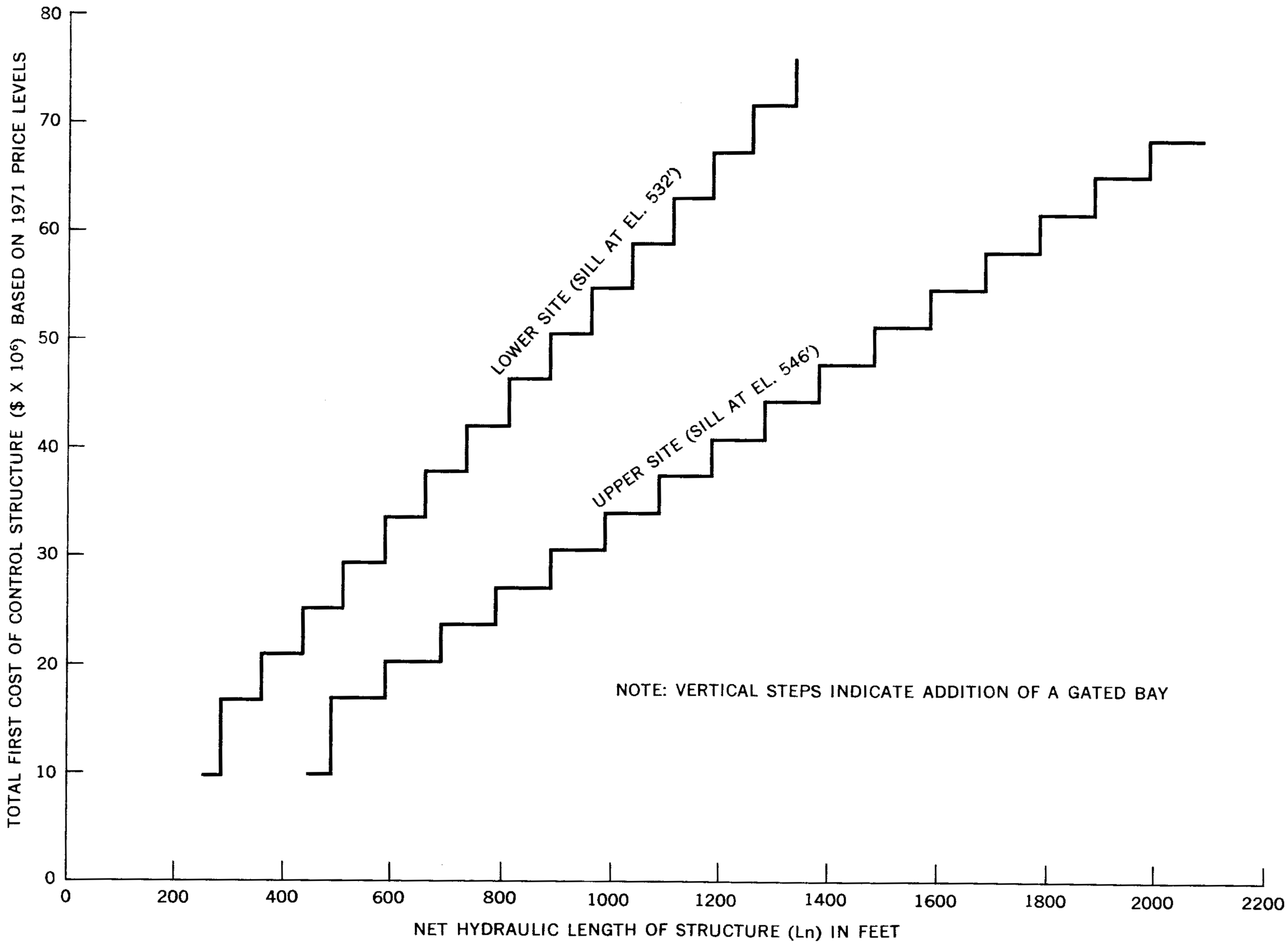


Figure G-69
UPPER NIAGARA RIVER—RELATIONSHIPS BETWEEN COSTS AND NET HYDRAULIC LENGTH OF ALTERNATE CONTROL STRUCTURES

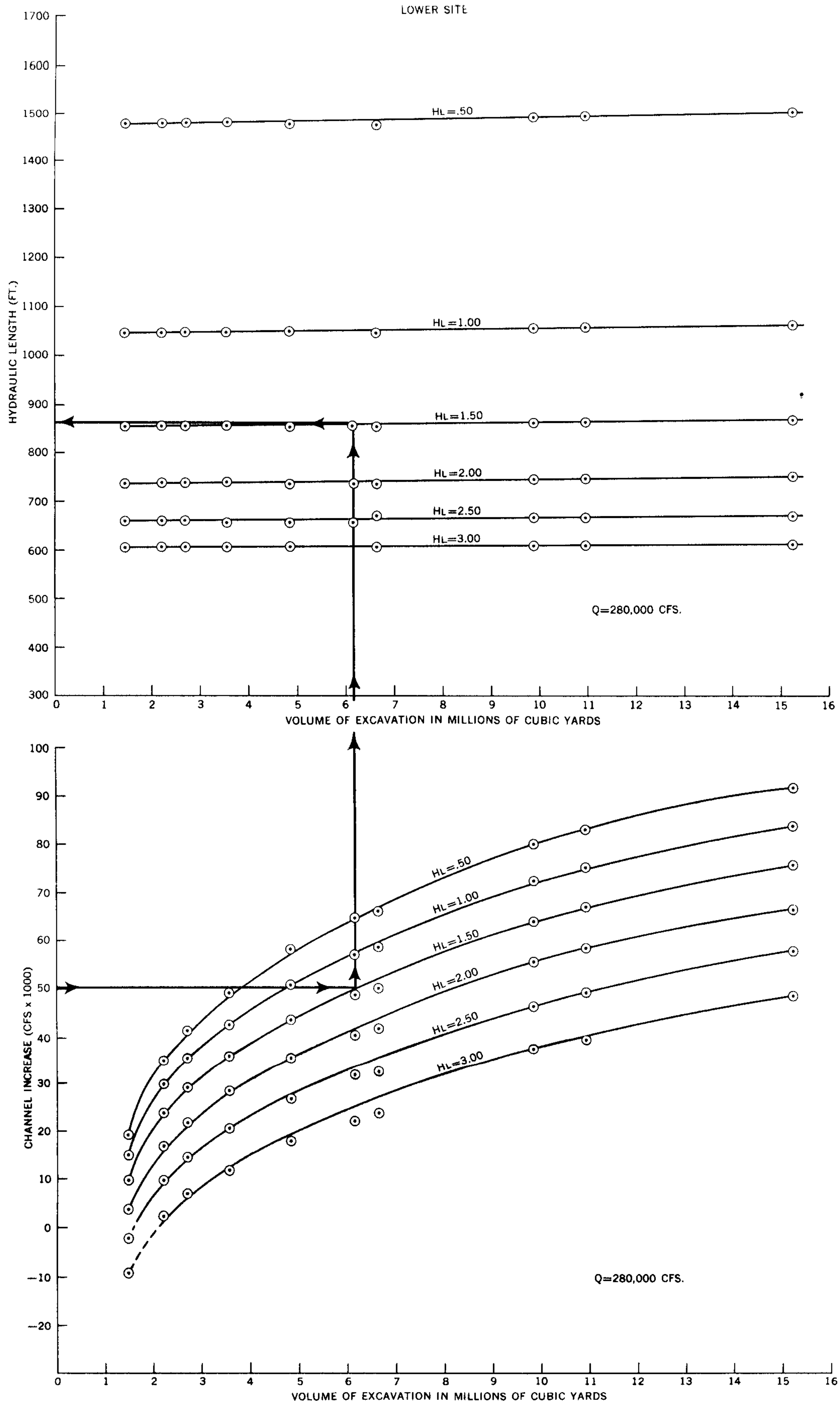


Figure G-70
UPPER NIAGARA RIVER—RELATIONSHIP BETWEEN CHANNEL CAPACITY INCREASE, VOLUME OF EXCAVATION AND NET HYDRAULIC LENGTH OF PROPOSED CONTROL STRUCTURE AT LOWER SITE

2. Unit Cost. As indicated in previous studies, the cost of excavation was approximately 60 percent of the total cost of regulatory facilities. Bearing this in mind, the unit cost of excavation would have a significant effect on the overall cost.

The following considerations were taken into account when selecting the method of dredging and dredge disposal: (1) open-lake dumping is generally more practical than on-land dumping, (2) on consultation with various environmental experts, the rock in the Niagara River was found to be clean and would have no adverse effects on the biological regime of Lake Erie, (3) the excavated rock should be put to some useful purpose, (4) velocities in this reach of the river are in the order of 12 feet per second. Weighing all factors and on consultation with various consulting/contracting marine engineering firms, open-lake dumping and excavation in-the-wet were adopted. The excavated rock would be transported within a five-mile radius, dumped and rehandled to form an island, and/or harbour protection works or any other useful configuration.

The four basic operations are: (1) mobilization and demobilization of equipment, (2) drilling, blasting and line drilling, (3) loading, hauling and dumping, and (4) sweeping, cleanup and miscellaneous. A sample of equipment costs is shown on Table G-10 for an average annual output of 850,000 cubic yards. Of course, equipment costs vary with the amount of excavation and the time frame available. Material costs varied from $0.66 million to $0.82 million depending on the quantity and depth of excavation. Mobilization and demobilization costs varied with the equipment used. As an example, it is estimated that the cost of mobilization and demobilization of the equipment listed on Table G-10 would be $1 million. Contingencies were estimated at 20 percent while indirect costs were estimated at 10 percent. Based on these studies, a plot of unit cost of rock excavation versus quantity of rock excavation was developed, as shown on Figure G-71, and utilized in these studies.

Shore Protection Works: With the control structure located at the lower site, higher than natural water levels would occur between the structure and Lake Erie (for the shoreline development affected, refer to Figure G-72). This involves the raising of existing shore protection works and the construction of new works in other areas. With the control structure located at the upper site, it may be advantageous to improve existing shore protection works; however, the costs of such work would be minimal and is considered part of the contingency estimates. The following paragraphs summarize these studies.

1. Existing Works. Existing works on the Canadian shore are: (1) a stone masonry wall, with concrete cap, extending from about 7,700 feet above the Peace Bridge, at which point the top elevation is 578.2 feet, to a point about 800 feet downstream of the Peace Bridge where the wall gradually slopes to elevation 572.9 feet, and (2) masonry wall extending about 100 feet above and below the International Railway Bridge. Existing works on the U. S. shoreline are: (1) a stone and concrete breakwater, at or about elevation 572 feet, extending from Lake Erie to a point about 2,300 feet downstream of the Peace Bridge separating the river from Black Rock Canal, (2)

Table G-10

DESIGN OF NIAGARA RIVER REGULATORY WORKS
EQUIPMENT COSTS FOR CHANNEL EXCAVATION

Operation and Equipment	No. Required	$ Per Hour	Costs* $ Millions Per Year
1. Drill, blast and line drill			
(2,080 hrs/year) (90-110 linear feet per drillboat per hour)			
Drillboats	3	216	1.35
Tugs (1200 h.p.)	2	116	0.48
Work boats	1	14	0.03
2. Load, Haul and dump			
(4,032 hrs/year) (100-120 cu. yds. per dredger per hour)			
Dipper dredge (9 cu. yds.)	2	203	1.64
Tugs (1000 h.p.)	2	105	0.85
Dump Scows	6	34	0.82
Launches	1	33	0.13
Work boats	2	13	0.10
3. Sweeping, cleanup, misc.			
(4,032 hrs/year) (100-120 cu. yds. per dredger per hour)			
Derrick boats	1	88	0.35
Tugs (1200 h.p.)	1	99	0.40
Deck scows	2	20	0.16

TOTAL	$6.31 million
Average Annual Output	0.85 million cu. yds.
Average Unit Rate	$7.42/cu. yd.

*Based on 1971 price levels

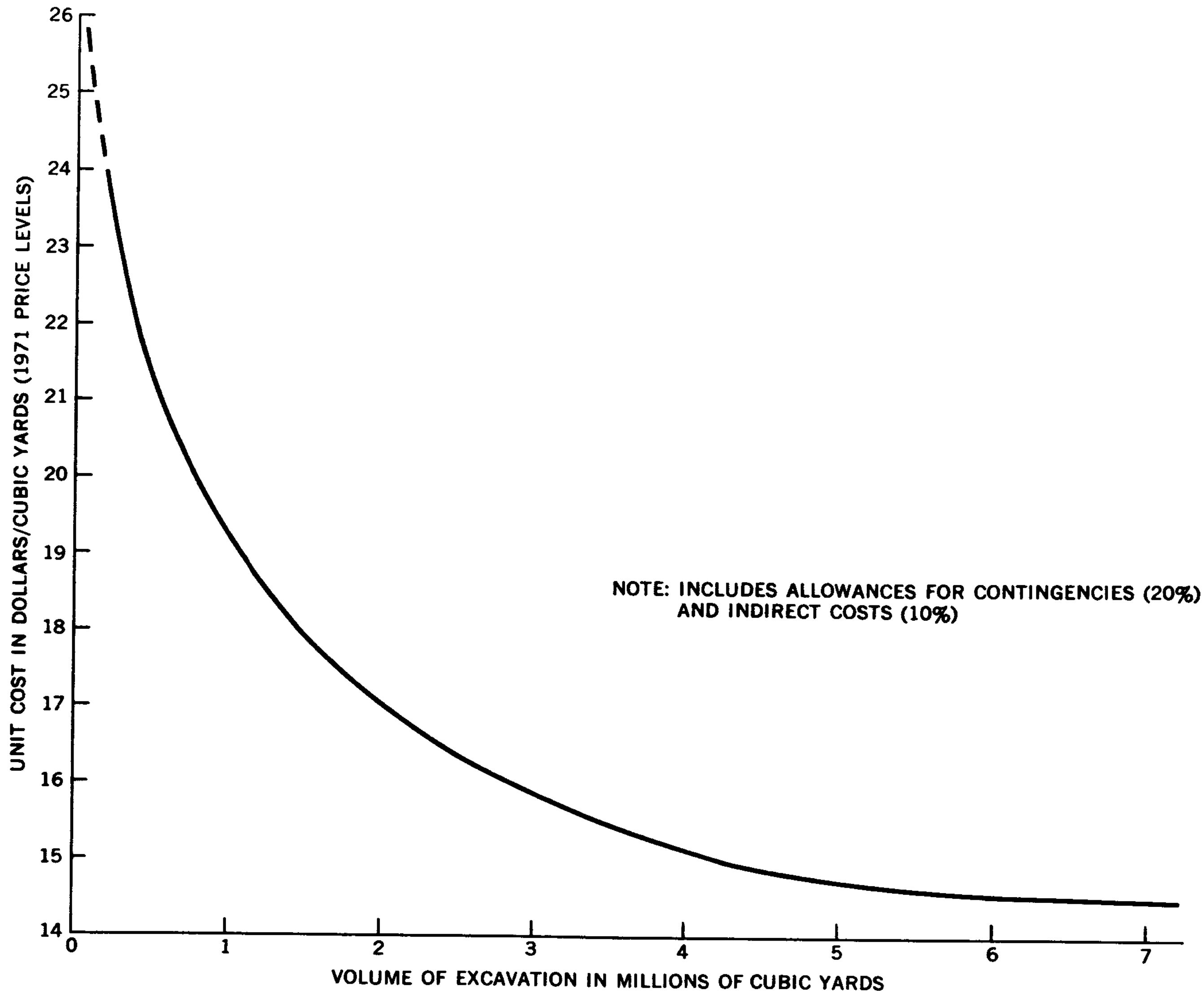


Figure G-71
UPPER NIAGARA RIVER—RELATIONSHIP BETWEEN UNIT COST OF EXCAVATION AND VOLUME OF EXCAVATION

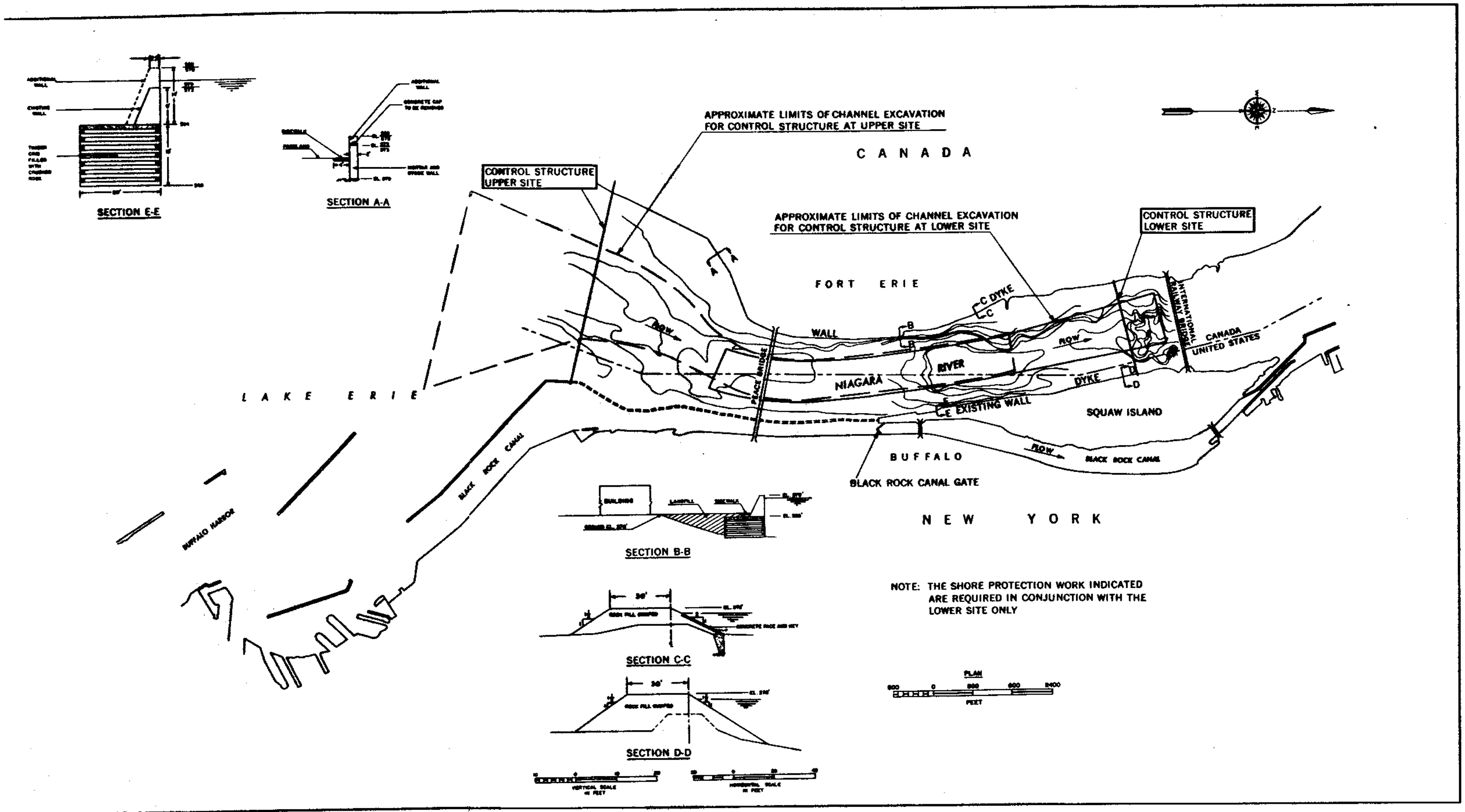


Figure G-72
UPPER NIAGARA RIVER— PROPOSED SHORE PROTECTIVE WORKS

a 3,100 foot long concrete wall, along Squaw Island with top elevation of 573 feet, protecting Brodrick Park and the Buffalo Sewage treatment plant. The concrete wall is about nine feet high and rests on a 20-foot wide timber crib filled with crushed stone, (3) a crushed stone breakwater extends from this wall forming the edge of Squaw Island, and (4) masonry walls 100 feet above and below the abutment of the International Railway Bridge.

2. Works Required. The following works are illustrated on Figure G-72.

a. Canadian Shore:

(1) Raising existing masonry wall by removing concrete cap and forming concrete on top

(2) Extension of this wall downstream for a distance of 1,750 feet to protect shops and other properties consisting of a concrete gravity wall on timber crib, backfilled and landscaped

(3) Land acquisition of properties falling inside the above wall extension

(4) Rock fill dyke on top of abandoned railway siding

(5) Raising and/or widening Niagara Boulevard along sections of shoreline

(6) Provision of concrete face slab for dykes (4) and (5) above keyed into bedrock to form impervious cut off to prevent possible raising of groundwater table in Fort Erie

(7) Raising of existing storm water outfalls, provision of proper outlets and filling of non-return valves to prevent flooding from river

b. United States Shore:

(1) Raising of the existing stone/concrete breakwater above and below the Peace Bridge

(2) Raising of the concrete wall along Squaw Island

(3) Provision of a rock fill dyke on top of existing crushed stone breakwater along Squaw Island

(4) Provision of 110-foot x 30-foot tainter gate in two flaps across Black Rock Canal to prevent flooding downstream under seiche conditions; this would not interfere with navigation since it usually is halted under seiche conditions.

(5) Modifications to the outfall facilities of the Buffalo Sewage Treatment Plant

One of the considerations associated with these particular shore protection works, especially on the Canadian shoreline, is aesthetics. It is normal practice to adopt a five-foot free-board for dykes of this nature; however, in view of the imposition of massive walls acting as a barrier between the river and those people inhabiting downtown Fort Erie, a freeboard of one foot was adopted. Thus, the top elevations of the dykes along both shorelines are sloped from elevation 580 feet at their upstream extremities to elevation 578 feet at the control structure. It may also be considered that any extension above the maximum elevation of the existing wall, 578.2 feet, should not be charged against regulation.

3. Cost estimates. Cost estimates were formulated for three alternate heights of shore protection works as shown on Table G-11. The costs include estimates of 25% for contingencies and 10% for engineering design, supervision and administration. The height of wall selected, varying from 580.0 feet to 578.0 feet, was interpolated to cost $4.5 million. It should be pointed out that this is a fixed cost, that is, it does not vary with the type of plan under consideration unless of course a drastic change is made on the level regime of Lake Erie.

TABLE G-11

DESIGN OF NIAGARA RIVER REGULATORY WORKS
ESTIMATED COST SHORE PROTECTIVE WORKS (1971 PRICE LEVELS)

($ MILLIONS)

Top Level	Canadian Side	United States Side	Total
584	3.00	3.49	6.49
580	2.13	2.82	4.95
578	1.87	2.43	4.30

Optimization Studies: By utilizing the graphs in correlation of Figure G-70, the cost curve as presented in Figure G-69 for varying length of structure and the unit cost of rock excavation as presented in curve from Figure G-71, the total costs of regulatory facilities required to meet various hydraulic requirements were optimized.

1. Procedure. Optimization of the total cost of regulatory facilities was carried out as follows:

(1) The channel capacity increase, plotted on the ordinate of the lower part of the monograph, Figure G-70, was selected.

(2) For each head loss across the structure, varying in magnitude from 0.50 foot to 3.00 feet, the volume of channel excavation and hydraulic length of structure were selected as illustrated in Figure G-70.

(3) An adjustment was made to the volume of channel excavation since the length of control structure affects the location and extent of channel excavation required at or near the structure.

(4) The cost of channel excavation was then determined by applying the unit cost of rock excavation as determined by Figure G-71.

(5) The cost of the control structure was determined by applying Figure G-69.

(6) In the case of the control structure at the lower site, a cost of $4.5 million was added to the total cost to account for the required shore protection works.

(7) The total cost of regulatory facilities was plotted versus head loss across the structure, and the optimum was selected.

(8) Steps 1 through 7 were repeated for each channel capacity increase, ranging from 0 to 60,000 cfs in increments of 10,000 cfs, and for flows from 160,000 to 320,000 cfs in increments of 40,000 cfs.

2. Preliminary Cost Curves. Based upon the procedure as outlined above, a set of preliminary cost curves was prepared to be utilized in the formulation of preliminary regulation plans. The cost curves for the lower and upper sites are illustrated on Figures G-73 and G-74, respectively. A comparison of cost between the two sites, for a design flow of 280,000 cfs, as shown on Figure G-75, reveals that the total first cost (1971 price levels) of the lower site is approximately $29 million to $35 million, dependent on the hydraulic requirements, lower than that for the upper site. The lower site was therefore selected for further design and cost estimates.

4.4 Methodology - Partial Regulation

Various methods were explored for reducing high lake levels on Lake Erie by increasing the discharge capacity at these sites:

1. Welland Canal
2. New York State Barge Canal
3. Black Rock Canal

Due to the limited capacity and erosion problems in the Welland Canal and the New York State Barge Canal, no further consideration was given to those two sites as a means of "partial regulation" of Lake Erie levels. The most promising method, utilizing the Black Rock Canal, is described below.

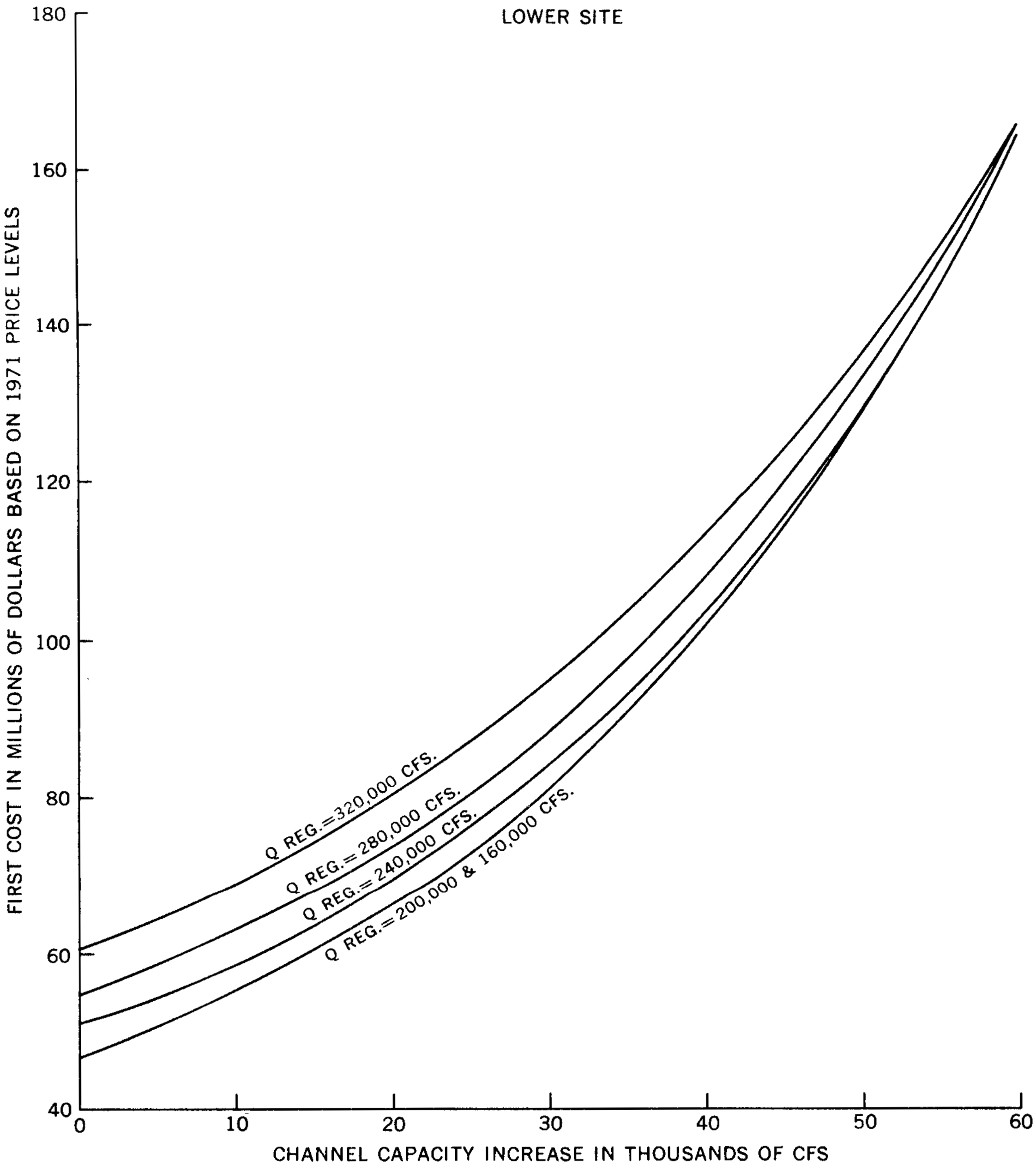


Figure G-73
UPPER NIAGARA RIVER—RELATIONSHIP BETWEEN TOTAL FIRST COSTS, CHANNEL CAPACITY INCREASE AND DESIGN FLOW—LOWER SITE

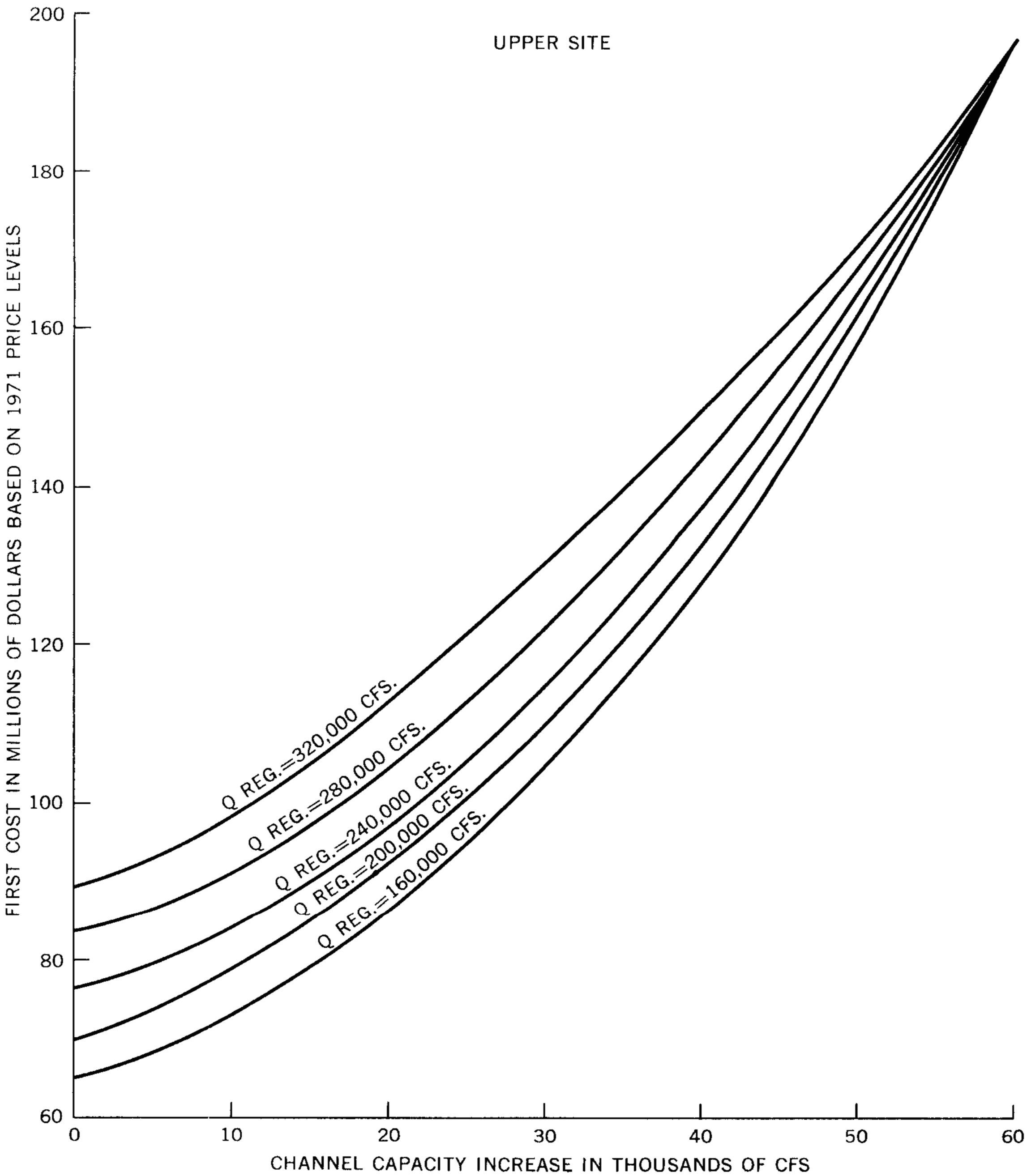


Figure G-74
UPPER NIAGARA RIVER—RELATIONSHIP BETWEEN TOTAL FIRST COSTS, CHANNEL CAPACITY INCREASE AND DESIGN FLOW—UPPER SITE

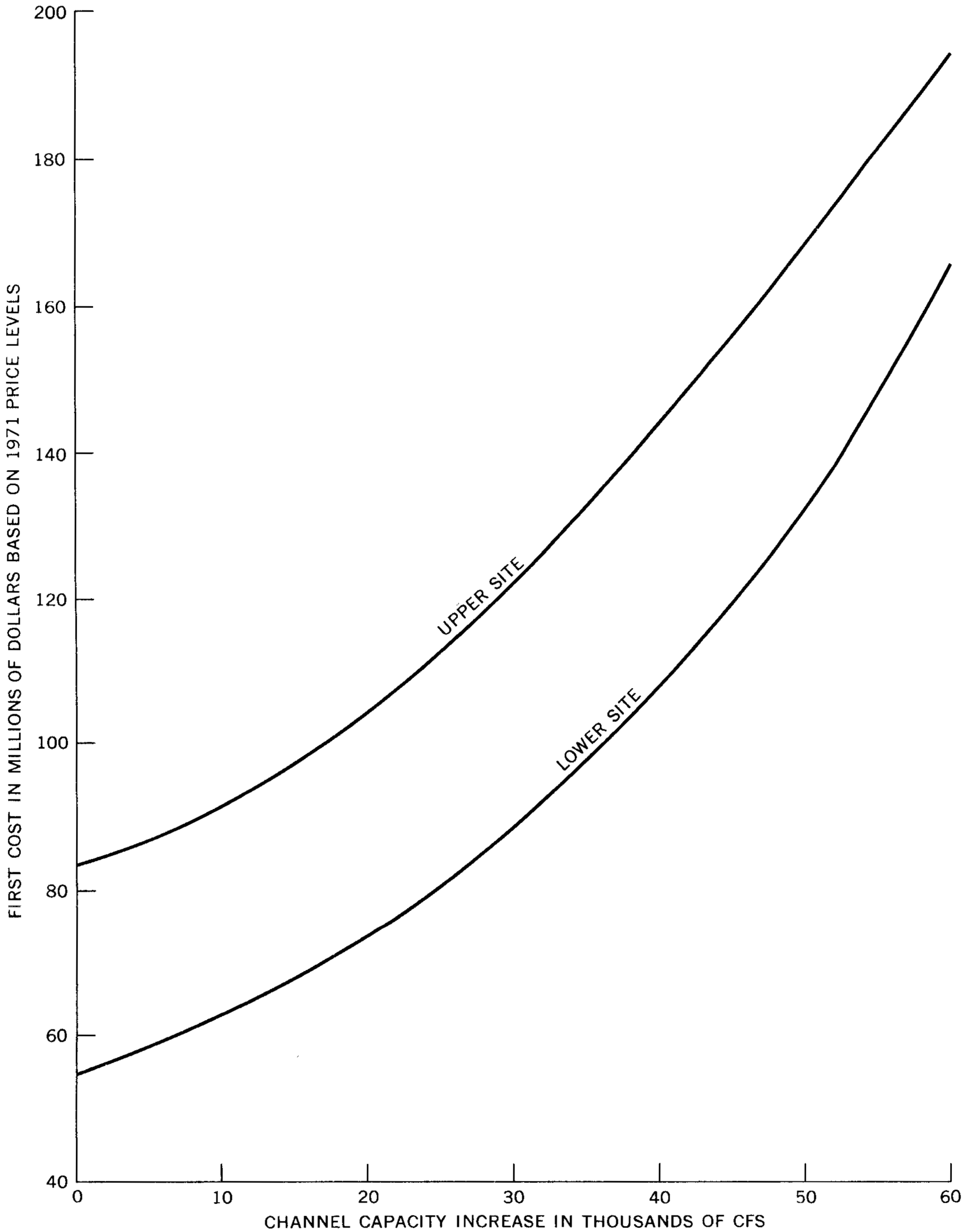


Figure G-75
UPPER NIAGARA RIVER—COMPARISON BETWEEN TOTAL FIRST COSTS OF UPPER AND LOWER SITES—PROPOSED CONTROL STRUCTURE

A promising measure, in terms of channel capacity increase, is the discharge of additional water down the Black Rock Canal thus bypassing the constriction of the river at or near the Peace Highway Bridge. There are essentially three alternative schemes: (1) breaching Bird Island Pier, which is a stone/concrete breakwater founded on a timber crib extending from Squaw Island upstream to Buffalo Harbour, and (2) excavating a diversion channel across Squaw Island just upstream of the Black Rock Lock; (3) discharging water through the Black Rock Lock chamber would increase the outflow about 15,000 cfs. This would require changing the type of gates presently in use. (Both sets of gates cannot be operated simultaneously against the 5-foot head without incurring structural changes.)

It was determined that an uncontrolled breaching of Bird Island Pier would seriously hamper navigation in the Canal. The cost of modifying the Black Rock Lock gates would cost about $1,720,000, excluding costs for protecting lock walls against higher water velocities. Accordingly, the alternative proposal of a diversion channel through Squaw Island with an attendant regulatory structure, was deemed to be the more practicable method of discharging additional water through the Black Rock Canal. The following is a summary of studies related to this proposal.

Introduction: The location of the proposed diversion channel is shown on Figure G-76. The diversion flows would be routed through the Black Rock Canal and thence through the proposed diversion channel bisecting Squaw Island between the International Railroad Bridge and Black Rock Lock. A control structure would be located near the downstream end of the diversion channel. Thus, by closing the structure, Lake Erie outflows could be returned to preproject conditions when lake levels and supply conditions fall below established threshold limits. Three alternative sizes of the diversion channel and control structure were investigated to provide a relationship between channel capacity increase and cost. This allows for the rapid determination of cost for a range of partial regulation plans having a variety of channel capacity increase requirements.

Layout: The alignment of each of the three schemes differs slightly in order to fit the different channel widths into the existing topography. The layouts of each of the three schemes, ranging from the smallest to the largest, are shown on Figures G-77, G-78 and G-79. In each case, the distance between the confluence of the proposed diversion channel with the Black Rock Canal and the entrance to the Black Rock Lock was made as great as possible to ensure that entrance velocities to the proposed diversion channel would have minimal impact on navigation. For the same reason, the control structure was located as close as possible to the Niagara River. Typical cross-sections of the proposed diversion channel upstream and downstream of the control structure are shown on Figure G-80. Earth dykes, with a top width of 10 feet, would be constructed on either side of the diversion channel to elevation 580.0, upstream of the control structure, to provide adequate freeboard. Downstream of the control structure, the earth dykes would be constructed to elevation 575.0, which is considered adequate in view of the drop in stage between Lake Erie levels and those of the Niagara River at its confluence with the proposed diversion channel. The diversion channel

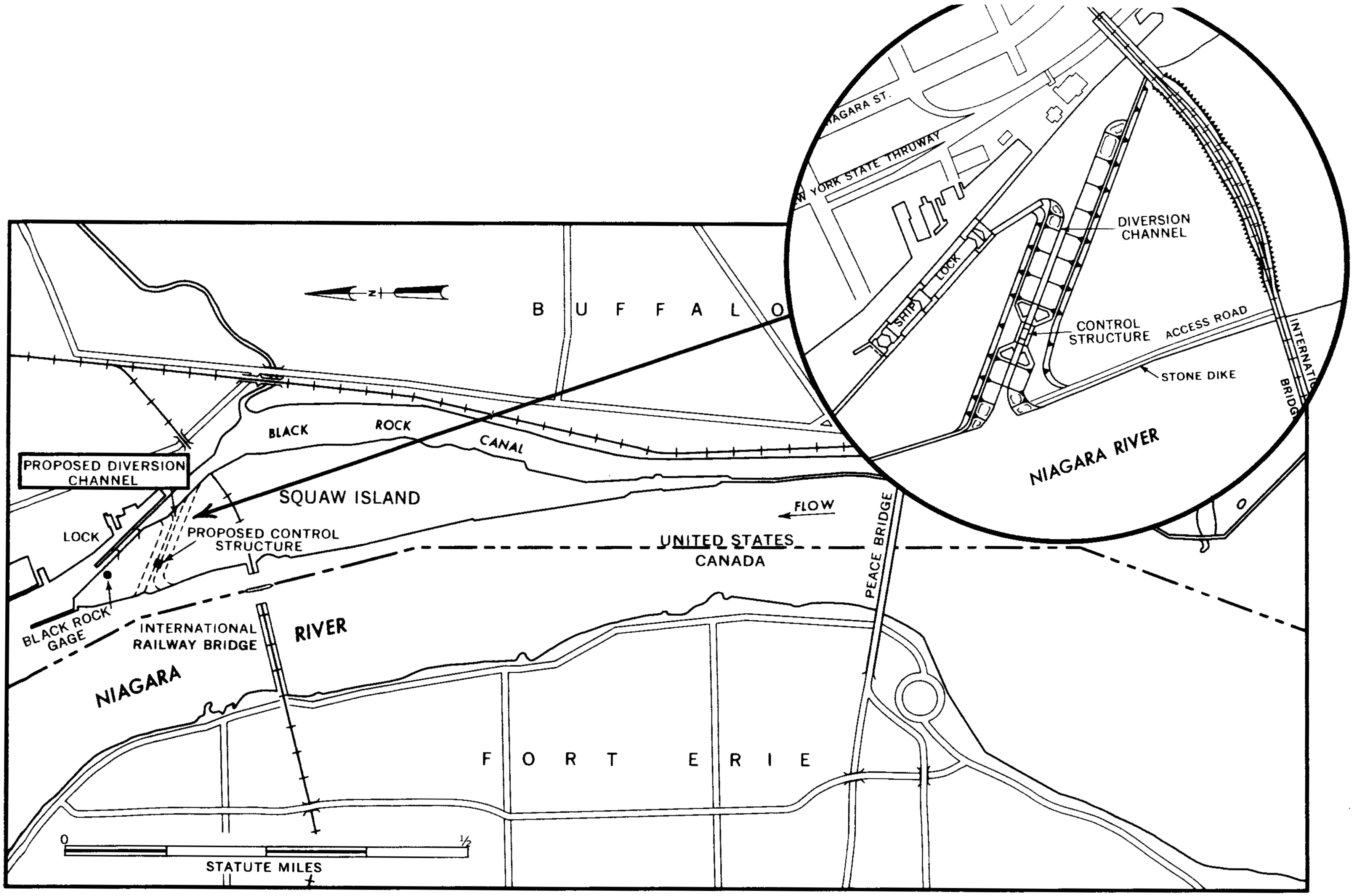


Figure G-76
PARTIAL REGULATION OF LAKE ERIE—LAKE ERIE DIVERSION VIA BLACK ROCK NAVIGATION CANAL—LOCATION MAP

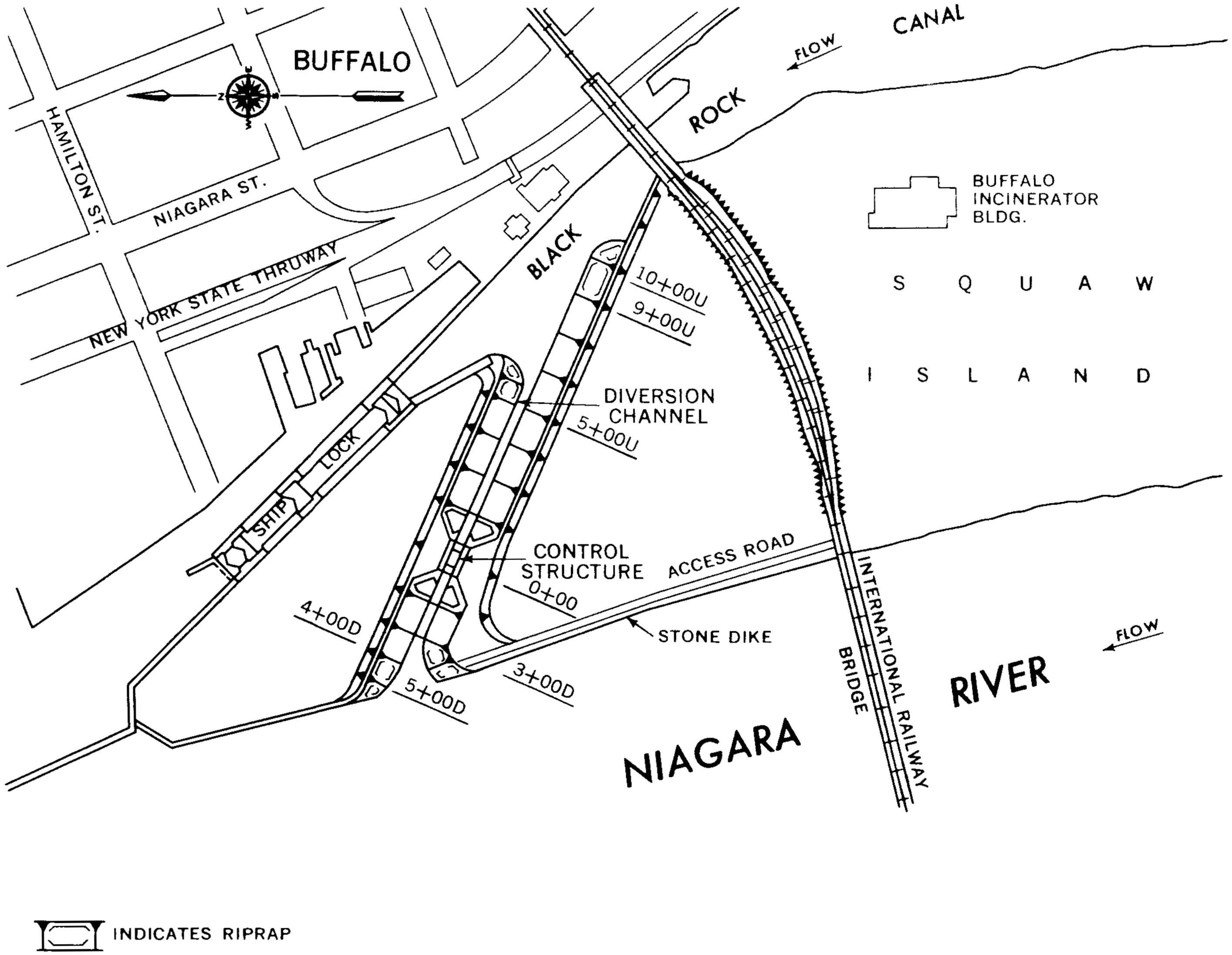


Figure G-77
LAKE ERIE DIVERSION VIA BLACK ROCK NAVIGATION CANAL—SCHEME A

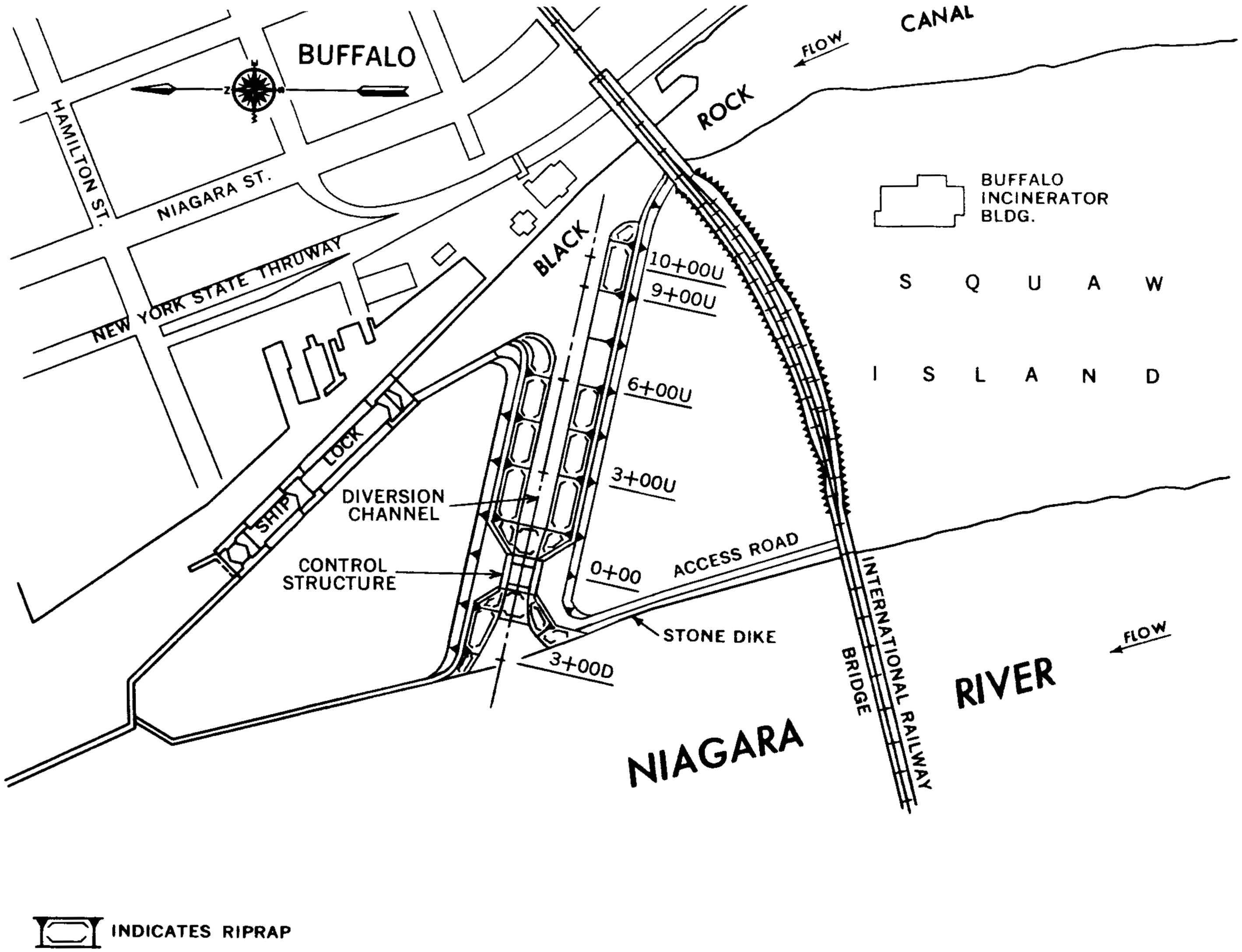


Figure G-78
LAKE ERIE DIVERSION VIA BLACK ROCK NAVIGATION CANAL—SCHEME B

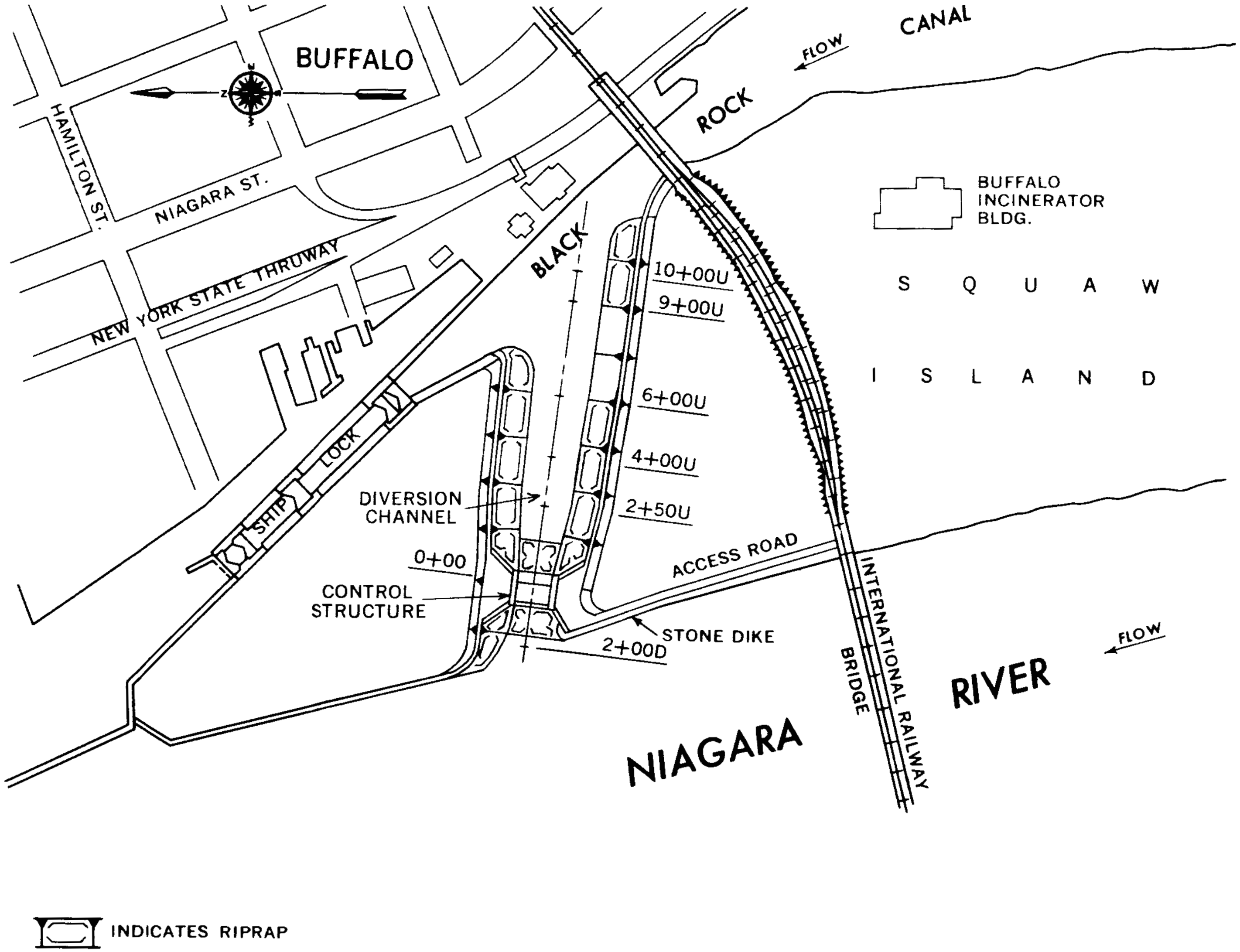
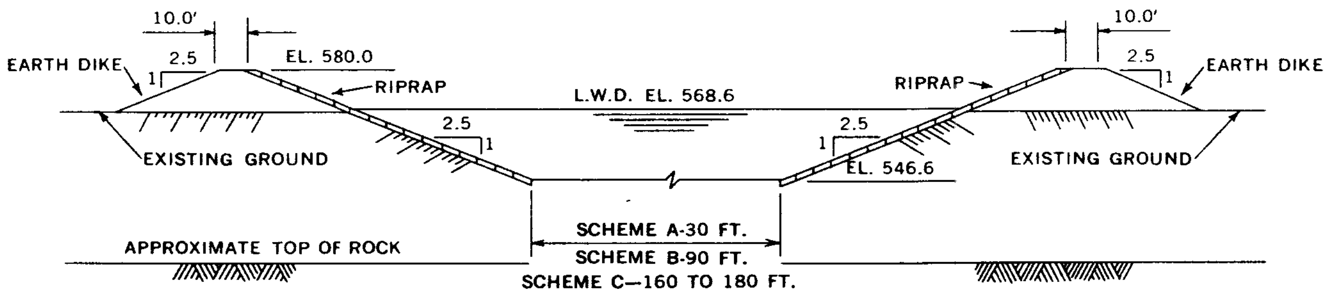
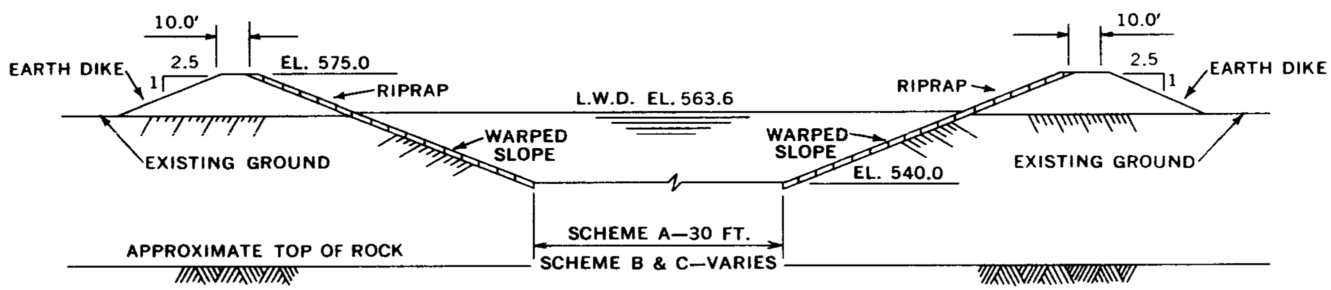


Figure G-79
LAKE ERIE DIVERSION VIA BLACK ROCK NAVIGATION CANAL—SCHEME C



TYPICAL SECTION OF DIVERSION CHANNEL
UPSTREAM OF CONTROL STRUCTURE



TYPICAL SECTION OF DIVERSION CHANNEL
DOWNSTREAM OF CONTROL STRUCTURE

Figure G-80
LAKE ERIE DIVERSION VIA BLACK ROCK NAVIGATION CANAL—TYPICAL CROSS SECTIONS OF PROPOSED SQUAW ISLAND DIVERSION CHANNEL

would be riprapped or otherwise protected as necessary to prevent bank erosion and gradually trained to and from the control structure to minimize excessive head losses due to contraction and expansion.

Hydraulic Considerations: The capacity of the Black Rock Canal--Squaw Island diversion channel was determined by applying the standard step method of backwater calculation with Manning's equation being applied to estimate friction losses. Because no prototype data were available, an estimate of Manning's roughness coefficient was made. Based on a review of relevant literature, the relative roughness of the diversion channel and of the Black Rock Canal could be approximated by a Manning's roughness coefficient varying from 0.025 to 0.030. The latter value was adopted for design purposes. Entrance losses from the Niagara River to the Black Rock Canal and from the Black Rock Canal to Squaw Island diversion channel were estimated by applying a coefficient of 0.25 to the change in velocity head across the entrance. An analysis of the geometry of the three major bends in the Black Rock Canal--Squaw Island diversion channel revealed that bend losses were insignificant. Because of the uniformity of the Black Rock Canal and the Squaw Island diversion channel, except at the control structure, energy losses due to expansion and contraction were considered to be insignificant. The conservative estimate of the Manning's roughness coefficient would correct for any errors implicit in this assumption.

The channel capacity increase for the Black Rock Canal--Squaw Island diversion channel was obtained by balancing the flows through the canal and the Niagara River for the water level conditions at their confluence. The channel capacity increase was therefore determined using the Niagara River mathematical model, as described in Section 4.3.1, in combination with the above described method of determining the capacity of the Black Rock Canal--Squaw Island diversion channel.

One of the factors that affects the ability of the diversion schemes to discharge additional flow is the available head. To present a range of hydraulic conditions that might result from partial regulation plans, two Niagara River flows (upstream of the confluence), 200,000 cfs and 248,000 cfs, were used for design purposes. The results of these investigations for three alternative schemes are summarized on Table G-12.

Design Considerations: A submersible tainter gate was selected for design purposes. Ice and debris could be passed over the submerged gate or, if desired, could be retained by a partially raised gate, thus providing operational flexibility. The tainter gate would be chain operated with electric hoists and provided with seal heaters and an air bubbler system for year-round operation. The control structure would be operated from the Black Rock Ship Lock by remote control, using closed circuit television and existing lock personnel. A centerline profile of the control structure is shown on Figure G-81. The diversion channel would be riprapped upstream and downstream of the control structure as necessary to prevent bank erosion. Earth levees, constructed to elevations 580.0 and 575.0 would be upstream and downstream of the control structure, respectively, would be provided to protect against overtopping during extreme high levels. It was assumed that,

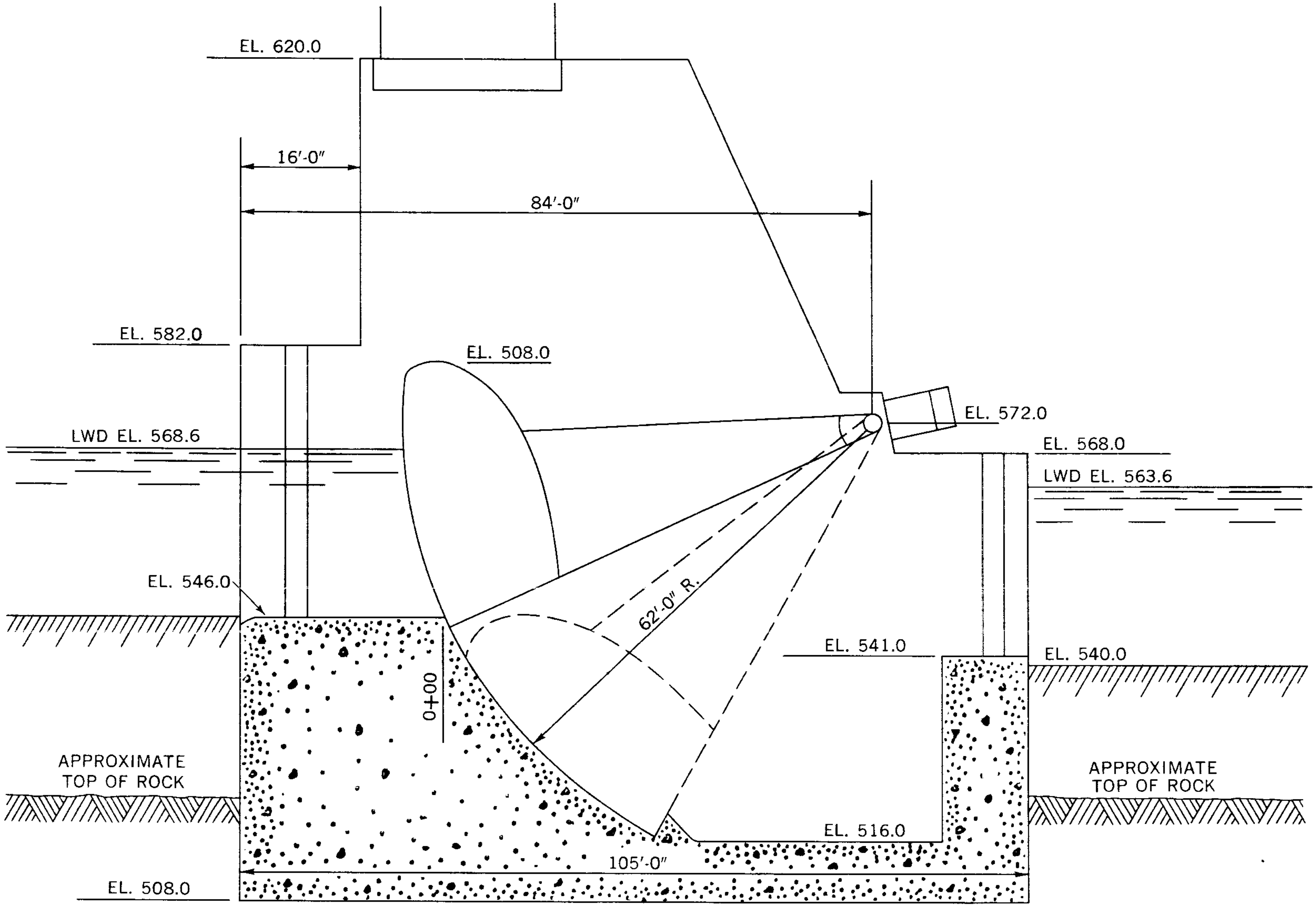


Figure G-81
LAKE ERIE DIVERSION VIA BLACK ROCK NAVIGATION CANAL—CENTERLINE PROFILE OF CONTROL STRUCTURE

for the flows considered in this study, no protective works to prevent erosion or undermining along the banks of the Black Rock Canal would be required. Removal of part of an existing sheet pile wall along the Black Rock Canal and part of an existing stone dyke along the Niagara River would be required to complete the diversion channel. Miscellaneous items which would be required include the following: (1) stop logs to provide for dewatering of the control structure, (2) a foot bridge across the control structure to provide access for maintenance personnel, and (3) a floating log boom at the upstream end of the diversion channel to prevent small pleasure craft from entering the channel.

Cost Estimates: In order that a valid comparison of cost between each of the three schemes could be made, common design criteria were utilized throughout. Costs were based on past projects in the Buffalo area expressed in 1971 price levels. An estimate of 25 percent was applied as an allowance for contingencies to obtain the total direct costs of works. Indirect costs which include allowances for detailed investigations, foundation explorations, engineering designs and construction supervision and administration were estimated at 15 percent of the total direct costs and added to obtain the total estimated first costs. A summary of the cost estimates for each of the three schemes is shown on Table G-13. A plot of channel capacity increase versus total first costs is shown on Figure G-82. In view of the requirement for winter operation, operation and maintenance costs are estimated at 0.5 percent of the total first costs. The construction period is estimated at less than one year and, therefore, interest charges during construction were neglected.

Additional studies are needed to determine:

1. Actual effects of increased currents in the canal on navigation.

2. Effects of diverting additional water on the operation of the Chippawa-Grass Island Pool.

3. Erosion of shoreline adjacent and downstream of the diversion channel.

4.5 Data

The following paragraphs list the data pertinent to the design and cost estimates of Lake Erie regulatory works.

4.5.1 Basic Data

The following paragraphs list the basic data which existed prior to the study and the collected data obtained during the course of the study.

Existing Data:

(1) Plans for regulation of Levels of Lake Erie, Technical Report No. 2-456, U. S. Army Corps of Engineers Waterways Experimental Station, Vicksburg, Mississippi, June 1957.

Table G-12

PARTIAL REGULATION OF LAKE ERIE
DETERMINATION OF CHANNEL CAPACITY INCREASE
FOR VARYING SIZE OF SQUAW ISLAND DIVERSION CHANNEL

	Scheme A	Scheme B	Scheme C
Q = 200,000 cfs			
Elevation at Black Rock Gauge*	565.65	566.01	566.18
Elevation at Buffalo Gauge	570.94	571.19	571.32
Channel Capacity Increase (cfs)	7,200	15,700	20,200
Q = 248,000 cfs			
Elevation at Black Rock Gauge*	567.59	567.98	568.19
Elevation at Buffalo Gauge	573.23	573.50	573.67
Channel Capacity Increase (cfs)	8,200	18,100	23,700

*This elevation is based on the flow in the Niagara River downstream of the confluence with the proposed diversion channel. See Figure G-76 for gauge location.

Table G-13

SUMMARY OF COST ESTIMATES - BLACK ROCK CANAL-SQUAW ISLAND DIVERSION SCHEMES

Item	Estimated First Cost (Dollars)		
	Scheme A	Scheme B	Scheme C
1. Excavation: Common	363,000	397,000	520,000
Rock	73,000	138,000	188,000
2. Levee Embankment	295,000	198,000	197,000
3. Riprap	142,000	319,000	298,000
4. Water Control During Construction	221,000	221,000	221,000
5. Reinforced Concrete	1,239,000	1,695,000	2,053,000
6. Steel Sheet Piling	279,000	260,000	260,000
7. Tainter Gate	270,000	675,000	991,000
8. Tainter Gate Machinery	88,000	132,000	199,000
9. Electrical Facilities	76,000	101,000	108,000
10. Stop Logs	125,000	312,000	457,000
11. Miscellaneous Items	162,000	184,000	193,000
SUB-TOTAL	3,333,000	4,632,000	5,685,000
25 Percent Contingencies	833,000	1,158,000	1,421,000
TOTAL DIRECT COSTS	4,166,000	5,790,000	7,106,000
15 Percent Indirect Costs	625,000	869,000	1,066,000
TOTAL ESTIMATED FIRST COSTS	4,791,000	6,659,000	8,172,000

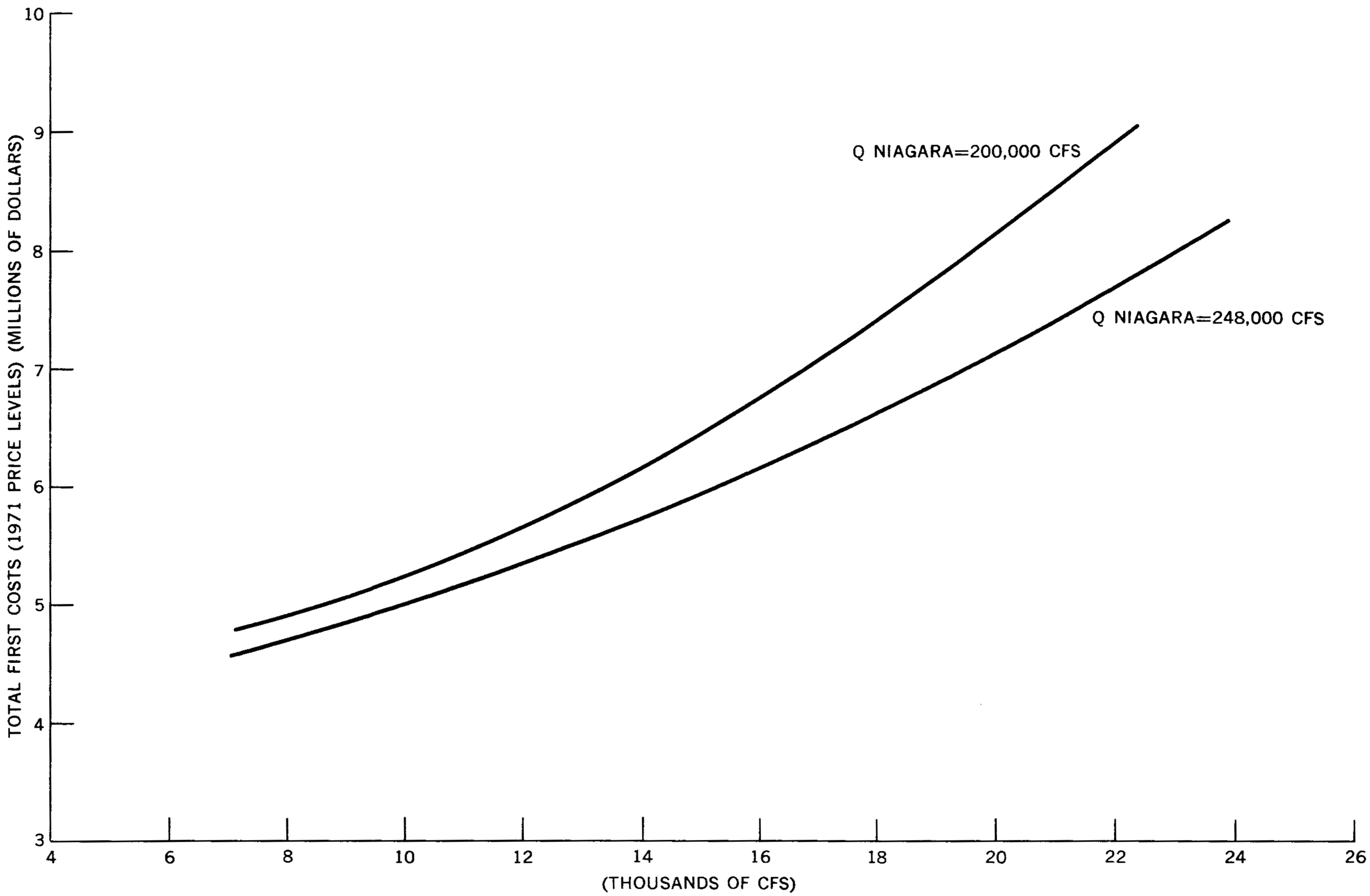


Figure G-82
LAKE ERIE DIVERSION VIA BLACK ROCK NAVIGATION CANAL—RELATIONSHIP BETWEEN CHANNEL CAPACITY INCREASE, DESIGN FLOW AND COST

(2) Water Levels of the Great Lakes, Report on Lake Regulation, U. S. Army Engineer Division, North Central, Corps of Engineers, Chicago, Illinois, December 1965.

(3) Soundings of the Upper Niagara River, U. S. Lake Survey Field Chart 1-1778, Scale 1/500. Soundings in feet, U. S. Lake Survey, 1932. 1940.

(4) Hydrographic Chart, Upper Niagara River, U. S. Lake Survey, No. 312, Scale 1/30,000, 1968 Edition.

(5) Topographic maps, Niagara Falls, Fort Erie, Surveys and Mapping Branch, Department of Energy, Mines and Resources, Scale 1/25,000, 10-foot contour interval.

(6) Photogrammetric mapping of United States and Canadian shorelines of the Niagara River between the International Railway Bridge and the head of the River, Department of Public Works, State of New York, Scale 1"-200', 5-foot contour interval, September 1962.

(7) Aerial photographs of the Upper Niagara River and adjacent shoreline, Survey and Mapping Branch, Department of Energy, Mines and Resources.

(8) Probings extending from the head of the Niagara River to two miles downstream, U. S. Army Corps of Engineers.

(9) Hydraulic Design Criteria, U. S. Army Corps of Engineers, Waterways Experimental Station, Vicksburg, Mississippi.

(10) Discharge Coefficients for Irregular Overfall Spillways, Engineering Monograph No. 9, U. S. Bureau of Reclamation.

(11) Movable Barrier Dams Across Rivers, 10th International Conference on Large Dams, Rex Madoux, and others, 1970.

(12) Manual of Standards and Procedures for Planning Water Resources Projects in Ontario, Canada--Ontario Committee on CWCAA Programs, July 1969.

(13) Table of Quantities for Estimating Cellular Coefferdams-2870, Special Projects Branch, Ministry of Transport, August 1951.

(14) Effects of Power Operations on the Niagara River and Lake Erie, Follow-up Report to the International Niagara Board of Control, April 1969.

Collected Data

(1) Preliminary Foundation Investigations, Niagara River Flow Control Structure and Appurtenances, William Trow (Hamilton) Limited, March 1970.

(2) Orthophotographic Mapping of the Area from below the International Railway Bridge to the head of the river, Lockwood Survey Corporation Limited, Scale 1" = 400 feet, set of two maps taken on 8 April 1970.

(3) Real Estate Appraisal of land adjacent to the Niagara River (Canada Side), Projected to 1975 price levels, Canada Department of Public Works, June 1970.

(4) Flow measurements and corresponding levels at several gauges, Lake Survey District, U. S. Army Corps of Engineers, and Water Survey of Canada, Environment Canada, 1967, 1968 and 1969.

(5) Profile along the river bank and adjacent land area and details of storm drainage, water intakes and sewage disposal pipeline. Water Planning and Management Branch, Environment Canada, Field Notes 1-71, September 1971.

(6) Investigation of Seiche Effects on the River, U. S. Army Corps of Engineers, Waterways Experimental Station, Vicksburg, Mississippi, December 1970.

4.5.2 Derived Data

The following list of data was derived from both the existing and collected data listed above.

(1) Cross-sectional areas, length and width of selected cross sections in the Upper Niagara River, Detroit District, U. S. Army Corps of Engineers.

(2) Mannings roughness coefficients for the various reaches in the Upper Niagara River, Water Planning and Management Branch, Environment Canada.

(3) Topographic and hydrographic map of the Upper Niagara River between the Black Rock gauge and the head of the river and adjacent land area, Water Planning and Management Branch, Environment Canada, Scale 1" = 200 feet, contour interval 5 feet.

(4) Hydrographic map of the Upper Niagara illustrating river bottom contours and rock contours, Water Planning and Management Branch, Environment Canada, Scale 1" = 100 feet, contour interval 5 feet.

(5) Relationship between depth of rock excavation and unit cost, Water Planning and Management Branch, Environment Canada, and U. S. Army Corps of Engineers, Buffalo District.

(6) Relationships between cost of gated structure versus length at both the upper and lower sites, Water Planning and Management Branch, Environment Canada.

(7) Relationships between channel capacity increase, volume of excavation and length of structure, Water Planning and Management Branch, Environment Canada.

(8) Relationships between channel capacity increase, and total first cost of structure, ancillary works and channel excavation, Water Planning and Management Branch, Environment Canada.

Section 5

ST. LAWRENCE RIVER SYSTEM

5.1 Description of the System

The St. Lawrence River forms the natural outlet of the Great Lakes Drainage basin. From its headwaters on Lake Ontario at Kingston, Ontario, the River flows generally in a northeasterly direction to its outlet on the Gulf of St. Lawrence, at Father Point, Quebec, a distance of some 530 miles. Between Kingston, Ontario and St. Regis, New York, the River establishes the international boundary between Canada and the United States. Downstream of Cornwall, Ontario, and St. Regis, New York, the River lies wholly within the Province of Quebec. A location map of the St. Lawrence River is shown on Figure G-83.

5.1.1 General

The St. Lawrence River possesses some advantages not shared by many rivers of comparable size and importance. The regulating effect of the Great Lakes results in a remarkably uniform flow in the St. Lawrence; the ratio of maximum to minimum flow at its headwaters on Lake Ontario being about 2:1 as compared, for example, to the Mississippi River with a corresponding ratio of about 40:1. Over the period 1900-1967, the mean recorded flow was 232,000 cfs, the maximum 305,000 cfs and the minimum 154,000 cfs.

From its outlet of Lake Ontario at Kingston, Ontario, to Father Point, Quebec, which marks its transition to the Gulf of St. Lawrence, the St. Lawrence River falls approximately 245 feet. Throughout the first 67 miles of its length, the river is characterized by numerous rocky islands and reefs from which the name, Thousand Islands Reach, is derived. With the construction of the St. Lawrence Seaway and Power Projects, between 1954 and 1959, the physical features of the River between Iroquois and Cornwall, Ontario, have been considerably changed. With the construction of the Saunders-Moses hydro-electric plants and appurtenances, a large man-made lake, named Lake St. Lawrence, was formed flooding out areas where entire villages had been located. Inhabitants of the area which were flooded were subsequently relocated, the costs of which were borne by the project.

Below the power dam, the river divides into two channels around Cornwall Island which then reunite to form Lake St. Francis. Downstream of Lake St. Francis, the river flows through the Beauharnois Canal and Cedars complex to Lake St. Louis. The Beauharnois powerhouse is located at the end of the canal. At the outlet of Lake St. Louis the river drops through the Lachine Rapids into the Laprairie Basin and thence through the short, swift flowing section near Victoria Bridge to Montreal Harbour, falling a distance of about 50 feet. In the 160 miles of river between Montreal and Quebec City the fall is about 25 feet at low tide. The range of tide at Quebec City averages about 16 feet, but extreme high spring tides have exceeded 21 feet. The tidal effect diminishes upstream until the range is only about 1-1/2 feet

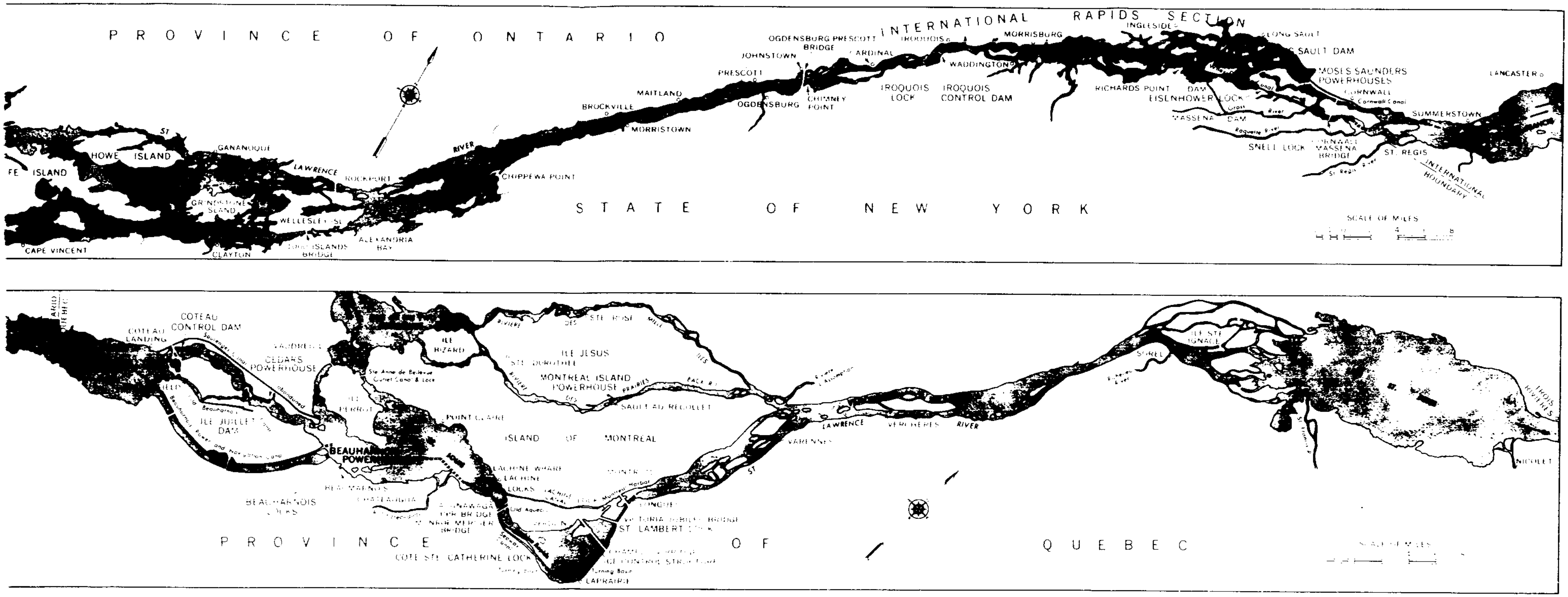


Figure G-83
ST. LAWRENCE RIVER--LOCATION MAP

maximum at Trois Rivieres and 1/2 foot maximum at the upper end of Lake St. Peter. Below Quebec City the river gradually forms its transition into the St. Lawrence estuary and finally the Gulf of St. Lawrence.

International Reach: For a distance of about 112 miles from Lake Ontario to St. Regis, New York, the St. Lawrence River is partly in Canada and in the United States and, therefore, is subject to the terms of the Boundary Waters Treaty of 1909 between the two countries. In its first 68 miles downstream to Chimney Point, New York, the river falls only about 1 foot. The river varies from one to four miles in width and is slow moving and generally deep. The numerous islands and shoals form the Thousand Islands. In the 44 miles from Chimney Point to St. Regis, the river falls approximately 92 feet. Prior to the St. Lawrence Seaway and Power Project, this amount was concentrated in a series of rapids between Chimney Point and Long Sault. However, after the completion of the St. Lawrence Seaway and Power Project, a major portion of the fall occurs at the Moses-Saunders power generation station. Three locks are provided for navigation, one at Iroquois and two in the power development area. Channel excavation has been carried out in this section in order to meet the criteria in the Order of Approval in 1955, issued by the International Joint Commission approving construction of the project. The project was designed so that water velocities in the section do not exceed 4 fps during the navigation season or 2.25 fps during the winter in order to minimize the difficulties of power generation.

Canadian Reach: Downstream from the International Rapids Section, the St. Lawrence River lies wholly in Canada and all improvements for navigation downstream to Montreal have been carried out by the St. Lawrence Seaway Authority. Below Cornwall Island, the navigation channel crosses Lake St. Francis for a distance of 31 miles to the head of the Beauharnois Power Canal. The water level of Lake St. Francis is maintained very closely to 152 feet IGLD through operation of the Beauharnois Cedars Complex by Hydro-Quebec. In authorizing diversions of water for power purposes at Beauharnois, the Government of Canada passed legislation in 1932 specifying certain conditions which would enable the power canal to be used ultimately as part of the Seaway System. Hydro-Quebec has been required to maintain the canal to give a clear width of 600 feet on the bottom, a depth of 27 feet at low water datum stage, and to provide adequate cross-sectional area so as to produce average velocities not exceeding 2.25 fps under any condition of operation.

Two Seaway locks overcome the 84 feet fall between Lake St. Francis and Lake St. Louis. Downstream of Beauharnois the river widens into Lake St. Louis which extends for ten miles to the Lachine Rapids. Below the rapids the river widens to form Laprairie Basin and then passes through a short, swift-flowing section to Montreal Harbour where water levels are about 50 feet lower than those in Lake St. Louis. The Ottawa River joins the St. Lawrence in the area southwest of Montreal. A portion of the Ottawa River flow enters through the Vandreuil and St. Anne Channels to Lake St. Louis and the remainder passes north of Montreal Island and joins the St. Lawrence below Montreal. Navigation bypasses the Lachine Rapids and reaches Montreal through Seaway facilities which consist of two locks: one at Cote Ste. Catherine; the other at St. Lambert.

5.1.2 Existing Regulatory and Power Facilities

There are four major installations in the St. Lawrence River between the outlet of Lake Ontario and Montreal. These are the Iroquois Dam, Long Sault Dam, Saunders-Moses Plants and Beauharnois-Cedars complex. In addition, channel enlargements were carried out for the seaway and power projects. Each will be briefly described in the following paragraphs.

Iroquois Dam: About 1,980 feet long, Iroquois Dam extends from Point Rockway in the United States to the Canadian shore near Iroquois. The structure is equipped with thirty-two 50-foot sluices designed to pass a maximum lake outflow in excess of the maximum flow of 310,000 cfs as specified by the current regulation plan (1958-D). The sluice gates are operated by two 350-ton travelling gantry cranes. Elevation of top of sills is 200.0 feet IGLD. If necessary, the dam can be operated to control and regulate the outflow from Lake Ontario, replacing the natural control provided by a rock ledge which existed near Galop Island prior to improvements associated with the project. The pattern of gate settings for the dam was developed from hydraulic model tests and it has been selected so as to minimize adverse currents in the navigation channel at the lower approach to Iroquois Lock. During periods of strong westerly winds, the gates may be dipped to prevent excessive buildup of water levels in Lake St. Lawrence. The gates are also used during ice formation to assist in promoting a stable ice cover.

Long Sault Dam: Long Sault Dam is located below the foot of Long Sault Island, about 25 miles downstream of the Iroquois Dam. It measures about 2,960 feet along its curved axis. Besides a non-overflow section, it also has a sluiceway section which consists of thirty 50-foot sluiceways. The sluices discharge flows in excess of requirements at the Saunders-Moses plants. It also can effectively control the river flows and water levels within specified ranges in the event that flows cannot be discharged through the Moses-Saunders plants. The spillway crest elevation is 217.0 feet IGLD.

Saunders-Moses Plants: The Saunders-Moses Plants are located about 3.5 miles downstream from Long Sault Dam and about 2 miles west of Cornwall, Ontario. The semi-outdoor plant, 3,300 feet long, with a rated head of 81 feet consists of thirty-two 57,000 kilowatt capacity generators. Sixteen generators are operated by the Power Authority of the State of New York while the other sixteen are operated by the Hydro Electric Power Commission of Ontario. Impounded behind the concrete gravity dam of the power plants is the man-made Lake St. Lawrence, which extends upstream to Iroquois Dam. The lake has a capacity of about 750,000 acre-feet at a normal elevation of 241.0 feet. Covering an area of 37,500 acres, it is confined by a system of earth embankments at the lower end totalling about 16 miles in length.

Beauharnois and Cedars Complex: At the lower end of Lake St. Francis, about 32 miles east of Cornwall, Ontario, the major part of the St. Lawrence is diverted through a 15-mile navigation and power canal to Hydro-Quebec's generating station at Beauharnois. The Beauharnois powerhouse has thirty-six main generating units with a total capacity of 1,574,260 kilowatts at a rated head of 80 feet. The navigation channel in the Beauharnois canal consists of a strip 600 feet wide with a minimum depth of 27 feet, running along the left bank of the canal. Two locks at the confluence with Lake St. Louis allow ships to enter the canal.

The remaining portion of St. Lawrence flow leaves Lake St. Francis through the Coteau Control Dam down the natural river channel. Most of this water is utilized by generating station at Cedars which is also operated by Hydro-Quebec. The Cedars powerhouse has 18 generating units with a total capacity of 162,000 kilowatts at a rated head of 35 feet.

Channel Enlargements: An integral part of the St. Lawrence Seaway - Power Project was the channel dredging and excavations carried out to: (1) provide a channel depth, width, alignment and water velocity for 27-foot navigation, (2) reduce velocities to induce ice cover over most of the river thus minimizing operational problems and enhancing the channel carrying capacity of the river subsequent to the ice forming period, (3) distribute the flow in such a way as not to interfere with navigation and (4) more important from the standpoint of lake regulation, reducing head losses at specific point to increase the channel capacity and to maximize the head available for hydro-electric power generation. For the most part, channel enlargements carried out for one interest were beneficial to the other interests.

The International Joint Commission, in its 1952 Orders of Approval, specified that the Power Entities were required to undertake channel enlargements which would ensure that velocities through the Galop do not exceed 4 fps and below Galop down to Morrisburg, not to exceed 2.25 fps during the ice forming period. Minimum depths of 29.5 feet upstream of and 28.5 feet downstream of Iroquois were required. The Power Entities carried out channel enlargements in 9 principal areas while the Navigation Agencies carried out dredging in 3. The principal locations of channel enlargements, carried out by the Power Entities, were at Chimney Island, Galop Island, Lalone - Lotus Islands, Sparrow Hawk Point - Toussaints Island, Iroquois, Point Three Points, Ogden Island, headrace of Long Sault Dam and tailrace of the Moses-Saunders Dam. The principal location of channel enlargements carried out specifically for navigation were at the Iroquois Lock, Wiley - Dondero Ship Channel and North and South of Cornwall Island.

A total of approximately 107 million cubic yards of material was excavated. The excavations carried out by the power entities totaled 63 million cubic yards; the major locations being in the vicinity of Sparrow Point - Toussaints Island (12 million), Gaiop Island (16 million) and Point Three Points - Ogden Island (11 million). The excavations carried out by the navigation agencies totalled 44 million cubic yards, with the principal locations being Wiley--Dondero Channel (25 million) and north and south of Cornwall Island (15 million).

As an example of the channel capacity increase attained by the project, a flow of 350,000 cfs was discharged out of Lake Ontario during part of 1973. During the latter part of the summer of 1973, this was about 19,000 cfs in excess of the flow that would have discharged prior to the project. More water could have been physically discharged out of Lake Ontario; however, it would have had very serious effects on navigation, shorefront properties both upstream on Lake St. Lawrence and downstream to Montreal and on the generation of power on the St. Lawrence.

5.1.3 Navigation Facilities

Works of the Federal Seaway agencies of Canada and the United States provide a 27-foot navigation channel through the river between Lake Ontario and Montreal Harbour. At and below Montreal a 35-foot navigation channel is maintained by the Canadian Ministry of Transport.

St. Lawrence Seaway: From Montreal to Lake Ontario, a vessel travels 182 miles and rises over 225 feet. This distance may be considered to consist of five sections, three of which are solely in Canadian waters, the others in international boundary waters.

The first section, about 31 miles in length, contains the St. Lambert and Cote-Ste-Catherine Locks, which enable ships to bypass the Lachine Rapids and to rise 50 feet above the level of Montreal Harbour. After moving through Lake St. Louis, ships enter the second section, the Soulanges Section, which extends for a distance of 16 miles into Lake St. Francis. The Lower and Upper Beauharnois Locks lift ships a total of 82 feet above Lake St. Louis. The third section, Lake St. Francis, is 29 miles long and terminates just east of Cornwall, Ontario.

The international section of the Seaway is entered at the upstream end of Lake St. Francis and extends to a point just east of Ogdensburg, New York. It is mainly the man-made Lake St. Lawrence resulting from the construction of the Moses-Saunders power complex. The difference in elevation is overcome by the United States Snell and Eisenhower locks near Massena and the Canadian lock at Iroquois. The remaining section extends from hereon over 68 miles into Lake Ontario. It is also known as the Thousand Island Section.

Of the seven locks mentioned above, five are operated by the St. Lawrence Seaway Authority of Canada and two are operated by the United States St. Lawrence Seaway Development Corporation.

Montreal Harbour and St. Lawrence Ship Channel: The Montreal seaport, located some one thousand miles from the Atlantic Coast, is operated by the National Harbours Board, a semi-autonomous agency of the Canadian Marine Transportation Administration, Ministry of Transport. Its deep harbour (about 35 feet at low water datum corresponding to a water surface elevation of 18.0 feet IGLD); its strategic location; and its facilities valued in excess of $150 million, exclusive of two major bridges across the St. Lawrence, have altogether made Montreal an important seaport to serve its immediate population of two million and a vast hinterland. There are about 6,000 vessel arrivals per year on an average. Cargo tonnage in 1970 was slightly over 25 million tons. Montreal can be generally considered open to shipping all year except during the most severe climatic conditions when ice jams in the river bring navigation to a halt.

The St. Lawrence Ship Canal, some 200 miles in length, refers to the main sailing course of the St. Lawrence River between the Port of Montreal and 40 miles downstream of Quebec City at which point the river is naturally deep. The main navigation channel has a maintained depth of 35 feet below low water datum and a minimum width of 800 feet. The normal fall in water surface elevation between Montreal and Quebec City is 25 feet. Velocities range from eight to ten feet per second in certain critical reaches of the ship canal but are as low at 1.5 feet per second in the Lake St. Peter reach. The portion of the St. Lawrence River downstream of Lake St. Peter is considered tidal, with a range varying from 1 foot at Trois-Rivieres to 16 feet at Quebec City. A small tidal effect can be detected at Montreal.

From the Port of Quebec downstream to Ile aux Coudres, a distance of approximately 60 miles, the St. Lawrence Ship Channel passes south of Ile d'Orleans and then swings northward through the North Traverse to take advantage of the deep water close to the north shore. The North Traverse lies in a siltation area and considerable channel dredging has been required to provide and maintain advertized channel depths. Below this reach there is naturally deep water. Other interesting features of the Ship Channel are the non-existence of locks, the stability of the river bed, practically no siltation in the channel upstream of Quebec City, and the varying character of the river bottom from solid rock at or close to the surface in several areas, to an overlying soft clay material of marine origin some 200 feet deep in Lake St. Peter.

Recreational Navigation: There is recreational boating throughout the St. Lawrence River. Among the most popular areas are: Thousand Islands; Lake St. Lawrence; Lake St. Francis; Lake St. Louis and various reaches between Montreal and Quebec City. Facilities such as marinas, launching ramps and private yachting clubs are well established in these areas. Navigational aides are established and maintained in many of the shallow draft channels serving recreational boating. There are a total of 36 marinas and yacht clubs along the St. Lawrence River.

5.1.4 Bridges, Wharves, Ferries and Other Facilities

There are 15 bridges spanning the St. Lawrence River all of which provide a vertical clearance of at least 120 feet above high water to accommodate commercial vessels. The Louis Hippolyte Lafontaine tunnel carries vehicular traffic under the St. Lawrence River at the head of Boucherville Islands, downstream of Montreal. Another tunnel carries vehicular traffic under the Lower Beauharnois lock at Melocheville, Quebec.

Two commercial wharves with a depth of 27 feet below low water datum are located in Montreal, namely Port de Valleyfield and Lower Lakes Terminal. There are 46 wharves with a maintained depth of less than 27 feet, of which 44 are located in Canada and two in the United States.

There are a total of 11 ferry routes on the St. Lawrence. Below Quebec City, ferries traverse the River between: Quebec City and Levis; Riviere-Du Loup and St. Simeon; Trois Pistoles and Les Escoumins; Rimouski and Baie Comeau; Matane and Godbout and Ste. Anne-des-Monts and Sept-Iles. Above Quebec City there are ferry crossings between: Sorel and Berthierville; the City of Dorval and Ile Dorval; Kingston and Wolfe Island with stops at Simcoe and Garden Islands; Simcoe Island and Wolfe Island; and Wolfe Island and Cape Vincent, New York. In addition, there are several scenic boat tours in operation during the tourist season throughout the river system.

There are numerous submarine cable and overhead transmission lines across the St. Lawrence River; these being 16 submarine cables and four major overhead transmission lines.

5.1.5 Ice Problems

Ice problems in the St. Lawrence can in general be related to the restrictive effects of the ice on river discharge, the magnitude of which varies from reach to reach depending on the configuration and hydraulic conditions of the river. For example, the formation of ice jams upstream of a generating station can seriously reduce the flow to the turbines, resulting in a loss of generated power, while at the same time causing flooding above the jam. Therefore, to overcome these problems the formation of a stable and relatively smooth ice cover early in winter is an important factor. Below Montreal the problem is somewhat different in that the aim is to maintain an open channel for navigation and flood prevention. One of the attendant difficulties is to keep flushing the ice downstream through areas where flow velocities are low.

General: At the present time the only measures taken to control ice in the St. Lawrence between Lake Ontario and Montreal relate to the requirements of hydroelectric power development at the Moses-Saunders plants and the Beauharnois-cedars complex. At and downstream of Montreal, operations are aimed at controlling ice to prevent flooding and to assist navigation. The use of floating wooden ice booms is a proven method of establishing stable ice cover conditions in certain critical reaches, particularly in relation to power development. Downstream of Montreal, where the emphasis is on maintaining an open channel for flood control and navigation, systematic ice-breaking operations provide the main control. Some experimental work with ice booms is also being conducted.

Ice Booms in International Reach: The St. Lawrence Power Project was put into operation in July 1958. During the first winter prevailing ice conditions were such that a heavy ice jam formed near Cardinal, Ontario, resulting in severely restricted flows into Lake St. Lawrence. Water levels at the Moses-Saunders Power Dam dropped about four feet, and generation was reduced by as much as 20 percent. The effect of this jam was also felt in the Montreal area, where, for a short period, several water intakes were uncovered.

To forestall a recurrence of the ice restriction experience, from Januar through to March 1959, the power entities obtained approval from the International St. Lawrence River Joint Board of Engineers, a Board established by the governments of Canada and United States, to install a series of booms to control the movement of ice in the reach between Ogdensburg and Cardinal. The first booms were placed at the beginning of the 1959-60 winter period and an additional boom was installed a year later. The overall layout of the booms has remained unchanged since the winter of 1961-62 and consists of a boom across the river at Ogdensburg-Prescott, a short section at Chimney Point and four booms in the Galops Reach.

These boom installations have in general successfully retained ice and have averted ice jams such as those experienced during the winter of 1958-59. The winter discharges prescribed by the International Joint Commission plan of regulation for Lake Ontario have accordingly been released without difficulty, and no reductions in power generation experienced.

Ice Booms in Canadian Reach: Ice booms are at present utilized in two areas in the Canadian reach. Booms are placed each winter in the Beauharnois Canal by Hydro-Quebec, and in the St. Lawrence River downstream of Montreal by the Ministry of Transport.

To prevent ice generated in Lake St. Francis and in the upper reaches of the Beauharnois Canal from moving downstream to jam against the powerhouse and in the approach to the Upper Beauharnois Lock, resulting in a considerable loss of energy to Hydro-Quebec during the remainder of the winter, a series of eight booms were installed to stabilize the ice cover. Five of these booms extended across the full width of the canal and closed off the portion of the canal used by navigation. Initial installation of the ice booms in Beauharnois Canal was made in December 1954.

With the advent of winter navigation to Montreal, additional measures are being taken by the Ministry of Transport, in conjunction with icebreaking operations, to effect better control of ice in the St. Lawrence Ship Channel. About 50 miles downstream of Montreal the St. Lawrence widens to form a shallow lake called Lake St. Peter, some eight miles wide and twenty miles long. The problem here is that once a channel has been opened by icebreakers there is a tendency for large pieces of the ice cover to break off through waves generated by wind and passing ships. These large masses of ice then move into the channel and effectively block it at the narrow outlet of the lake and an ice jam may be formed. To control this breakup of the ice cover, a series of booms were installed in the most critical area on an experimental basis in November 1967 and each subsequent winter. Indications are that these booms do materially contribute to ice control in Lake St. Peter.

As a further experiment, an ice boom has been installed at Lavaltrie to control ice moving through the channel north of Vercheres Islands. This boom was installed for the first time in November 1969. All booms are removed and stored on land each spring.

Montreal Ice Control Structures: Construction of the site for EXPO 67 at Montreal, involving development of St. Helen's Island and the creation of Ile Notre Dame along the Seaway Dyke, resulted in restricting the river section in that area. Although compensating dredging was carried out to ensure hydraulic conditions during open water remained relatively unchanged, it was believed that the river regime had been altered to the extent that there was a danger of more severe ice jams in Montreal Harbour which could result in greater flooding. For this reason, the Ice Control Structure was built in Laprairie Basin following unanimous agreement by a Federal interdepartmental committee on the World's Fair Site.

Constructed in 1964-65 by the Department of Public Works, the Ice Control Structure was put to a preliminary test during the winter of 1965-66 and taken over by the Ministry of Transport in October, 1966. The structure spans the St. Lawrence River three miles above the EXPO site, parallel to and approximately one thousand feet upstream of the Champlain Bridge. It has a total length of 6,698 feet between abutments and consists of 72 concrete piers, founded five feet into bedrock and designed to withstand an ice thrust of ten Kips per lineal foot of structure. Three spans, both of which are approximately 176 feet in width, were incorporated in the Ice Control Structure to provide an early opening channel for evacuating ice in the spring. Floating steel stop-logs, measuring 6 feet x 4 feet and 83 feet 1 in. long and weighing 40 tons each are placed between the piers. Rollers fitted to the ends of the stop-logs allow them to move freely in the vertical guide slots in response to water level fluctuations. The stop-logs are designed for an ice thrust of five Kips per lineal foot and are ballasted with concrete to float with a draft of four feet. When not in use, the stop-logs are lifted out of the guide slots and stored on the piers.

The early evacuation opening channel is fitted with special floating steel booms anchored to the piers by 3-1/2 in. diameter cables.

The ice control structure is operated in conjunction with ice breaking operations downstream of Montreal. The Montreal harbour section of the St. Lawrence River does not freeze over due to the high velocity of the current. Laprairie Basin and the lower part of Lake St. Louis do not generally freeze over but produce large quantities of frazil and loose ice, which float down past Montreal.

Ice jams will form readily in the section immediately downstream of Montreal. These ice jams, if not broken up by icebreakers, will proceed upstream being fed by floating loose ice and frazil and will eventually reach the Montreal harbour section. In such an event, the ice control structure can be operated to cause an ice cover to form on Laprairie Basin and loose ice can be stored under the ice cover thus slowing down the progress upstream of ice jams and giving more time to the icebreakers to demolish them.

5.1.6 Current Operating Plan

Appendix B, Lake Regulation, discusses details of the current operating plan.

5.2 Assumptions

All regulation plans are designed to meet the capacities of the existing works in the St. Lawrence River as described in Appendix B.

Based upon water supplies for the study period 1900-1967, the existing regulatory works and channel capacities of the St. Lawrence River were judged to be adequate for the regulation of Lake Ontario under the existing Orders of Approval of the International Joint Commission. However, even with the extraordinary discretionary deviation from Plan 1958-D, it was not possible to accommodate the record high supplies of 1972-73 and meet all the regulation criteria and other requirements of the Orders. Recent studies of the International St. Lawrence River Board of Control have confirmed that it is not practicable within existing physical constraints to design a plan which will meet all such criteria and other requirements under the maximum supplies received to date.

Section 6

COST EVALUATION OF SELECTED REGULATION PLANS

6.1 Introduction

Of the numerous regulation plans developed during the International Great Lakes Levels Board Study, six were selected for detailed evaluation, namely: SO-901, SMHO-11, SEO-901, SEO-33, SEO-42P and SMHEO-38. The alphabetic designation specifies the lakes that would be regulated under the plan, e.g., a SEO plan specifies that Lakes Superior, Erie and Ontario would be regulated. The numerical designation is used only to identify the particular plan selected from the array of plans developed. Each regulation plan has different regulatory requirements which may include: channel improvements, regulatory structures, modifications to existing regulatory facilities or combinations of any of these three. A brief summary of regulatory works requirements for each of the selected plans is shown on Table G-14. The following Sections summarize the cost estimates of regulatory facilities for each of the selected regulation plans. All cost figures are expressed at 1971 price levels.

6.2 Regulation Plan SO-901

Of the SO-series of regulation plans, SO-901 was selected for detailed benefit and cost evaluation. The following is a discussion of the hydraulic and operational requirements of the plan and the design and cost estimates of the facilities needed to meet those requirements.

No major modifications to existing channels or regulatory facilities at Sault Ste. Marie were required for the implementation of the selected regulation plan. However, frequent and reliable winter operation of the Lake Superior regulatory works would be required. The hydraulic capacity of the St. Marys River was found to be adequate. The required modifications to the existing works for winter operation and the results of the check on the adequacy of the existing river channels are summarized in the ensuing Sections.

The existing Lake Ontario regulatory facilities are adequate to meet the changed hydraulic regime resulting from the re-regulation of Lake Superior under Plan SO-901.

6.2.1 Modifications to Existing Regulatory Works

The recommended facilities for winter operation are discussed in detail under Section 2.3.4. These include: electrical tubular heaters for gate and gain heating, electric power and telephone lines to the structure, modifications to provide motorized drives for all 16 sets of gate hoist machinery, a metal-clad enclosure over 10 sets of gates, and hinged sheet steel covers over open gearing of all 16 gates. The estimated capital costs of these works is $574,000, which when amortized over a project life of 50 years allowing for replacement and salvage at an interest rate of 7% represents an equivalent annual cost of $43,000. It is estimated that the annual operation and maintenance costs, including hydraulic monitoring and communications, would be $27,000. The costs of the required modifications to the Lake Superior control works are summarized on Table G-15.

6.2.2 Capacity of Existing Channels and Regulatory Facilities

The capacity of the existing regulatory facilities would be adequate both to retard the flow during periods of low supply and to pass the maximum discharge under the hydraulic conditions required by Plan SO-901. The maximum water surface elevation at the U. S. Slip Gauge was computed to be 582.1 IGLD for Plan SO-901 as compared to the maximum allowable elevation below the locks of 582.9 IGLD as specified in the 1914 IJC Orders of Approval. Therefore, the capacity of the lower St. Marys River would be adequate for Plan SO-901.

6.2.3 Summary and Costs

No major capital expenditures would be required at the Lake Superior compensatory works for Plan SO-901. However, since normal winter operation of the control works at Sault Ste. Marie could require frequent changes in gate settings, modifications would be necessary so that such changes could be made more effectively and safely. The total first cost of the required modifications was estimated at $574,000 which, when amortized over a project life of 50 years, allowing for replacement and salvage, at an interest rate of 7%, corresponds to an annual cost of $43,000. The total annual costs including Operations and Maintenance costs were estimated at $70,000. No capital expenditure would be required for the Lake Ontario regulatory facilities as they are already adequate to meet the changed regime resulting from the re-regulation of Lake Superior under Plan SO-901.

The capacity of all existing regulatory works, in both the St. Marys and the St. Lawrence Rivers, were checked and found to be adequate to accommodate the level and flow regime that would result from the implementation of Plan SO-901.

6.3 Regulation Plan SMHO-11

Regulation Plan SMHO-11 was selected from the SMHO-series of regulation plans for detailed benefit and cost evaluation. Unlike the SO-series, Plan SMHO-11 requires extensive regulatory works in the St. Clair-Detroit River system. The following sections describe the works required, with corresponding cost estimates for the St. Marys and St. Clair-Detroit River systems.

No major capital expenditures would be required for the St. Marys River system to ensure compliance under Plan SMHO-11. Winter operation of the control gates is required similar to the requirements of Plan SO-901 and for the same costs.

The capacity of the Lower St. Marys River was checked and found to be adequate. The maximum elevation below the locks was computed to be 582.1 IGLD which is less than the maximum allowable elevation of 582.9 IGLD as specified in the 1914 IJC Orders of Approval.

Table G-14
SUMMARY OF REGULATORY WORKS REQUIREMENTS

Selected Regulation Plan	St. Marys River	St. Clair and Detroit Rivers	Niagara River	St. Lawrence Ri
SO-901	3,4	0	0	4
SMHO-11	3,4	1,2	0	4
SEO-901	3,4	0	1	4
SEO-33	3.4	0	1,2	4
SEO-42-P	3,4	0	1,2	4
SMHEO-38	3,4	1,2	1,2	4

LEGEND: 0 - No Regulatory Works Required
1 - Channel Improvements Required
2 - Regulatory Structure Required
3 - Modifications (Heating and Mechanization) to existing regulatory structure required
4 - Hydraulic capacity of River and Regulatory Structures

Table G-15
SUMMARY OF ESTIMATED COSTS OF MODIFICATIONS
TO LAKE SUPERIOR COMPENSATING WORKS FOR PLAN SO-901

Total First Costs (1971 Price Levels)	$574,000
Interest and Amortization 50-year Project Period at a 7% Interest Rate	$ 43,000
Annual Operation and Maintenance Costs	$ 27,000
TOTAL ANNUAL COSTS	$ 70,000

The existing Lake Ontario regulatory facilities would be adequate to meet the changed hydraulic regime resulting from the regulation of Lakes Superior and Michigan-Huron under Plan SMHO-11.

6.3.1 Lakes Michigan-Huron Regulatory Works

The design and cost estimates of regulatory works required for the regulation of Lakes Michigan-Huron that are summarized below for Plan SMHO-11 were developed from the methodology presented in Section 3.

Channel Capacity Increase Requirements: The maximum channel capacity increases for the St. Clair and Detroit Rivers under Plan SMHO-11 were determined by comparing the regulated flows with the flows that would be obtained from the regulated levels under 1933 outlet conditions. The critical design levels and flows are summarized on Table G-16.

The maximum channel capacity increase for the St. Clair River, discussed in Section 3.3.7, was determined to be 11,000 cfs. Since this is already built-in due to the 25- and 27-foot navigation improvements carried out in the St. Clair River subsequent to 1933, the existing channels are considered adequate. Consequently, no dredging would be required in the St. Clair River for Regulation Plan SMHO-11.

Similarly, the maximum channel capacity increase for the Detroit River was determined to be 11,000 cfs. However, only 6,000 cfs of this amount is built-in relative to 1933 outlet conditions because of compensation provided in the lower Detroit River as discussed in Section 3.1.7. Therefore, dredging would be required in the Detroit River, the most optimal location of which is in the Trenton Channel.

Channel Capacity Decrease Requirements: The monthly mean regulated flows resulting from Plan SMHO-11 were compared with the flows that would be detained from the regulated levels of Plan SMHO-11 applied to the 1933 outlet conditions. The critical design levels and flows are summarized in Table G-17.

The maximum channel capacity decrease required for both the St. Clair and Detroit Rivers was computed to be 29,000 cfs. However, additional retardation is required to compensate for the 11,000 cfs channel capacity increase requirements of the plan. Therefore, the maximum channel capacity decrease required for Plan SMHO-11 is 40,000 cfs for the St. Clair and Detroit Rivers.

Regulatory Structures Required: Tests made utilizing the mathematical models of the St. Clair and Detroit Rivers indicated that the water surface profiles (1933 outlet conditions) generally would be obtained by construction of four structures in the St. Clair River and five structures in the Detroit River. The four structures for the St. Clair River are those located at Port Huron, Stag Island, St. Clair and North and Middle Channels as shown on the location map, Figure G-39. The five structures for the Detroit River are those located at Peach Island (North and South), West Belle Isle, Zug Island, East Fighting Island and Trenton Channel also shown on the location map, Figure G-39.

Table G-16

SUMMARY OF CRITICAL CHANNEL CAPACITY INCREASE DESIGN CONDITIONS
LAKES MICHIGAN-HURON REGULATORY WORKS
PLAN SMHO-11

A.	ST. CLAIR RIVER	
	1. Lakes Michigan-Huron Regulated Level	579.39 feet
	2. Lake St. Clair Design Level	574.58 feet
	3. St. Clair River Regulated Flow	206,000 cfs
	4. St. Clair River Flow*	195,000 cfs
	5. Maximum Channel Capacity Increase	11,000 cfs
B.	DETROIT RIVER	
	1. Lake St. Clair Design Level	574.58 feet
	2. Lake Erie Design Level	571.86 feet
	3. Detroit River Regulated Flow	211,000 cfs
	4. Detroit River Flow*	200,000 cfs
	5. Maximum Channel Capacity Increase	11,000 cfs

*Relative to 1933 outlet conditions of Lakes Michigan-Huron.

Table G-17

SUMMARY OF CRITICAL CHANNEL CAPACITY DECREASE DESIGN CONDITIONS
LAKES MICHIGAN-HURON REGULATORY WORKS
PLAN SMHO-11

A.	ST. CLAIR RIVER	
	1. Lakes Michigan-Huron Regulated Level	578.51 feet
	2. Lake St. Clair Design Level	573.41 feet
	3. St. Clair River Regulated Flow	165,000 cfs
	4. St. Clair River Flow*	194,000 cfs
	5. Maximum Channel Capacity Decrease	29,000 cfs
B.	DETROIT RIVER	
	1. Lake St. Clair Design Level	573.41 feet
	2. Lake Erie Design Level	570.27 feet
	3. Detroit River Regulated Flow	170,000 cfs
	4. Detroit River Flow*	199,000 cfs
	5. Maximum Channel Capacity Decrease	29,000 cfs

*Relative to 1933 outlet conditions of Lakes Michigan-Huron

The total first costs for the nine structures were estimated at $152.1 million.

Total First, Capital and Equivalent Annual Cost of Regulatory Works: The total first costs of regulatory works, dredging and structures, required in the St. Clair and Detroit Rivers for Plan SMHO-11 were estimated at $198.1 million. A summary of total first, capital, annual interest and amortization and Operation and Maintenance costs are shown on Table G-18.

6.3.2 Summary and Costs

Modifications to the existing compensating works at Sault Ste. Marie are required to permit safe and reliable winter operation of the control gates. The costs of these modifications are summarized in Section 6.2.3. The capacity of the existing channels and regulatory facilities at Sault Ste. Marie were checked and found to be adequate.

The works required on the St. Clair-Detroit River system consist of nine control structures located at strategic points along the river system and dredging in the Trenton Channel of the Detroit River. The costs of regulatory works for the St. Clair-Detroit River system are summarized on Table G-18.

No capital expenditure would be required for the existing regulatory facilities in the St. Lawrence River, the capacity of which were checked and found to be adequate to accommodate the changed inflow regime as a result of regulating Lakes Superior and Michigan-Huron.

The total first, capital and equivalent annual costs of regulatory work required to meet the hydraulic requirements of Regulation Plan SMHO-11 are summarized on Table G-19. The regulatory works required for SMHO-11 would require a total capital investment of approximately $240 million which, when amortized over a 50-year project life at an interest rate of 7%, would be equivalent to an annual expenditure of approximately $18 million including operation and maintenance costs.

6.4 Regulation Plan SEO-901

During the course of the study, it became evident that a large proportion of the benefits associated with the regulation of Lake Erie would not be "regulation benefits", per se, but rather would result from a lowering of the entire range of levels on Lake Erie, which, due to backwater effects, also would result in a minor lowering of Lakes Michigan-Huron. Regulation Plan SEO-901 was selected for detailed benefit and cost evaluation as being representative of this type of plan. The regulation of Lakes Superior and Ontario, under SEO-901 would follow the same operating policies as does Plan SO-901 and would require the same modifications to the compensating works.

6.4.1 Lake Erie Regulatory Works

Regulation Plan SEO-901 requires that the channel capacity of the Niagara River be increased by approximately 4,000 cfs relative to the existing

Table G-18
SUMMARY OF ESTIMATED COSTS OF LAKES MICHIGAN-HURON REGULATORY WORKS
REQUIRED FOR PLAN SMHO-11

Total First Costs (1971 Price Levels)	$198,100,000
Total Capital Costs (End of 6-year Construction Period)	239,700,000
Annual Interest and Amortization (50-year Project Period at 7%)	17,369,000
Annual Operation and Maintenance Costs	564,000
TOTAL ANNUAL COSTS	$17,933,000

Table G-19
SUMMARY OF ESTIMATED COSTS OF REGULATORY WORKS REQUIRED
FOR PLAN SMHO-11 ($ Thousands)

	LAKE OUTLET			
	Superior	Michigan-Huron	Ontario	TOTAL
Total First Costs (1971 Price Levels)	574	198,100	0	198,67
Total Capital Costs at End of Construction Period	574	239,700	0	240,27
Annual Interest & Amortization (50-year Project Period at 7%)	43	17,369	0	17,41
Annual Operation & Maintenance	27	564	0	59
TOTAL ANNUAL COSTS	70	17,933	0	18,0[illegible]

hydraulic regime. Provision of this channel capacity increase would result in a general lowering of Lake Erie levels by 0.19 foot as specified by Regulation Plan SEO-901. To accomplish this, approximately 49,000 cu. yds. of rock would have to be excavated from the Niagara River just below the Peace Bridge. The total capital costs would be $1,360,000 which when amortized over the 50-year project life at an interest rate of 7% are equivalent to an annual cost of $99,000. A summary of cost estimates for Lake Erie regulatory works is shown on Table G-20.

TABLE G-20

SUMMARY OF ESTIMATED COSTS OF
LAKE ERIE REGULATORY WORKS REQUIRED
FOR PLAN SEO-901

Total First Costs (1971 Price Levels)	$1,270,000
Total Capital Costs at end of 2-year Construction Period	1,360,000
Annual Interest and Amortization (50-year Project Period at 7%)	99,000

6.4.2 Summary and Costs

Modifications to the existing compensating works at Sault Ste. Marie are required to permit safe and reliable winter operation of the control gates. The costs of these modifications are summarized in Section 6.2.3. The capacity of the existing channels and regulatory facilities at Sault Ste. Marie were checked and found to be adequate.

Approximately 49,000 cu. yds. of channel excavation would be required in the Niagara River to increase its discharge capacity by 4,000 cfs as required under Regulation Plan SEO-901. The cost estimates of Lake Erie regulatory works are summarized in Table G-20.

No capital expenditure would be required at the existing regulatory facilities in the St. Lawrence River. The regime of outflows from Lake Erie would be essentially the same as for Plan SO-901.

The regulatory works required for Regulation Plan SEO-901 would require a total capital investment of approximately $1,934,000 which when amortized over a 50-year project period at an interest rate of 7% is equivalent to an annual cost of $142,000. The operation and maintenance costs of the Lake Superior regulatory works, for winter operation has been estimated at $27,000 bringing the total annual cost to $169,000. The cost estimates for regulatory works required for Regulation Plan SEO-901 are summarized on Table G-21.

6.5 Regulation Plan SEO-33

Of the SEO-series of regulation plans, Regulation Plan SEO-33 was selected for detailed benefit and cost evaluation. Unlike Regulation Plan SEO-901, Regulation Plan SEO-33 requires, in addition to channel excavation, a regulatory structure and shore protection works such that Lake Erie outflows may be retarded during period of low levels. The regulation of Lake Superior, under SEO-33, is similar in philosophy to SO-901.

No major capital expenditure would be required for the St. Marys River System under Plan SEO-33. Winter operation of the control works at Sault Ste. Marie would be acquired similar to the requirements of Plan SO-901 and could be achieved for the same costs.

The capacities of the Lower St. Marys River and the compensating works at Sault Ste. Marie were checked and found to be adequate. The maximum elevation below the locks was computed to be elevation 582.0 IGLD as compared to the maximum allowable elevation of 582.9 IGLD as specified by the 1914 IJC Orders of Approval.

The existing Lake Ontario regulatory facilities are adequate to meet the changed hydraulic regime resulting from the regulation of Lakes Superior and Erie under Plan SEO-33.

6.5.1 Lake Erie Regulatory Works

An analysis of Lake Erie regulated levels and flows was carried out to determine the critical design conditions, the results of which are shown in Table G-22. In summary, the regulatory structure must be capable of decreasing the Lake Erie outflow by 40,000 cfs at a stage of 570.52 feet and the channel must be enlarged to overcome the head loss across the structure while at the same time providing a channel capacity increase of 27,000 cfs at a stage of 570.61 feet.

The total first costs of regulatory facilities were optimized, the results of which are summarized as follows:

(1) A control structure consisting of: (a) an ungated section with a span of 255 feet, (b) 8 gates (1 gate for standby) with 15 foot piers 90 feet c/c, and (c) a rock-fill dyke over the remaining 925 feet; at a total first cost of $43,326,000.

(2) Channel excavation amounting to 2,570,000 cubic yards of rock at a total first cost of $40,709,000.

(3) Shore protective works at a total first cost of $4,952,000. The total first costs of regulatory facilities would be $88,986,000. The above cost estimates are presented in detail in Table G-23.

Since it is necessary to maintain substantially the present Lake Erie stage-discharge relationship during the construction period, the dredging and control structure construction program would have to be closely integrated

Table G-21
SUMMARY OF ESTIMATED COSTS OF REGULATORY WORKS REQUIRED
FOR PLAN SEO-901 ($ Thousands)

	LAKE OUTLET			
	Superior	Erie	Ontario	TOTAL
Total First Costs (1971 Price Levels)	574	1,270	0	1,844
Total Capital Costs at End of Construction Period	574	1,360	0	1,934
Annual Interest & Amortization (50-year Project Period at 7%)	43	99	0	142
Annual Operation and Maintenance	27	0	0	27
TOTAL ANNUAL COSTS	70	99	0	169

Table G-22
SUMMARY OF CRITICAL DESIGN CONDITIONS
LAKE ERIE REGULATORY WORKS
PLAN SEO-33

	DESIGN CONDITION	
	Channel Capacity Increase	Channel Capacity Decrease
Lake Erie Regulated Level	570.61 feet	570.52 feet
Niagara River Regulated Flow	226,000 cfs	158,000 cfs
Lake Erie Natural Level	571.87 feet	568.55 feet
Niagara River Natural Flow	199,000 cfs	198,000 cfs
Hydraulic Requirements	27,000 cfs	40,000 cfs

For the purposes of this discussion, "Natural Conditions" refers to 1953 preproject outlet conditions of the Niagara River.

Table G-23

ESTIMATE OF FIRST COST (1971 PRICE LEVELS), LAKE ERIE CONTROL WORKS
REGULATION PLAN SEO-33

	ITEM	QUANTITY	UNIT	UNIT COST	AMOUNT	TOTALS
				$	$	$
	CONTROL STRUCTURE					
1.	Diversions & Cofferdams					
	a) Sheet Piling	2,615	ton	500.00	1,307,500	
	b) Fill	318,280	cu. yd.	1.00	318,300	
	c) Pumping	-	job	-	584,000	
	d) Removal	-	job	-	325,000	2,534,800
2.	Excavation					
	a) Overburden	30,000	cu. yd.	2.50	75,000	
	b) Rock	215,000	cu. yd.	6.00	1,290,000	
	c) Rock Surface Preparation	17,700	sq. yd.	3.00	53,100	1,418,100
3.	Grouting	47,750	l.f.	8.00	382,000	382,000
4.	Concrete (Including Formwork)					
	a) Base Slab	127,000	cu. yd.	40.00	5,080,000	
	b) Rollway	27,570	cu. yd.	45.00	1,240,700	
	c) Piers	67,350	cu. yd.	60.00	4,041,000	
	d) Retaining Wall	35,500	cu. yd.	55.00	1,952,500	
	e) Bridge	3,095	cu. yd.	70.00	216,700	12,530,900
5.	Reinforcing Steel	4,150	ton	400.00	1,660,000	1,660,000
6.	Gates and Hoists					
	a) Stoplog Gates	780	ton	1,360.00	1,061,000	
	b) Stoplog Guides	260	ton	1,400.00	364,000	
	c) Stoplog Crane	-	job	-	400,000	
	d) Tainter Gates	3,800	ton	1,360.00	5,168,000	
	e) Tainter Gates - Embedded Parts	1,320	ton	1,400.00	1,848,000	
	f) Tainter Gates Hoists	8	unit	300,000.00	2,400,000	11,241,000
7.	Rock Fill Dam					
	a) Stripping Common	27,400	cu. yd.	0.80	21,900	
	b) Rock Fill & Riprap	105,000	cu. yd.	6.00	630,000	
	c) Gravel Roadway	6,710	sq. yd.	4.00	26,800	678,700
8.	Control Building (5,000 sq. ft.)	-	job	-	150,000	150,000
9.	Miscellaneous, Mechanical and Electrical	-	job	-	800,000	800,000
10.	Contingencies at 20%	-	job	-	6,279,100	6,279,100
11.	Engineering Design, Supervision and Administration at 15%	-	job	-	5,651,200	5,651,200
	Total First Cost					43,325,800
	SHORE PROTECTION WORKS					
12.	Property Acquisition and Land Development	-	job	-	284,000	284,000
13.	Guard Gate for Black Rock Canal					
	a) Sector Gate	175	ton	1,360.00	238,000	
	b) Embedded Parts	62	ton	1,400.00	86,800	
	c) Concrete Guides	-	job	-	100,000	424,800

Table G-23 (Cont'd)

ESTIMATE OF FIRST COST (1971 PRICE LEVELS), LAKE ERIE CONTROL WORKS
REGULATION PLAN SEO-33

	ITEM	QUANTITY	UNIT	UNIT COST	AMOUNT	TOTALS
				$	$	$
14.	Raising of Concrete-masonry Wall (Canadian Side)					
	a) Removal of existing cap	8,500	l.f.	2.00	17,000	
	b) Concrete	1,500	cu. yd.	45.00	67,500	
	c) Reinforcing Steel	15	ton	400.00	6,000	90,500
15.	Rockfill Dykes					
	a) Stripping	14,400	cu. yd.	0.60	8,640	
	b) Rockfill	218,300	cu. yd.	6.00	1,309,800	
	c) Gravel Roadway	14,000	sq. yd.	4.00	56,000	1,374,400
16.	Concrete Facing & Key					
	a) Face Slab	1,800	cu. yd.	45.00	81,000	
	b) Key	5,200	cu. yd.	40.00	208,000	
	c) Reinforcing Steel	37	ton	400.00	14,800	303,800
17.	Crib Wall					
	a) Timber Crib	10,000	cu. yd.	30.00	300,000	
	b) Concrete Cap & Wall	10,200	cu. yd.	55.00	561,000	
	c) Reinforcing Steel	96	ton	400.00	38,400	899,400
18.	Concrete-stone Breakwater	37,400	cu. yd.	6.00	224,400	224,400
19.	Contingencies at 25%	-	job	-	900,300	900,300
20.	Engineering Design, Supervision and Administration at 10%	-	job	-	450,200	450,200
	Total First Cost					4,951,800
	CHANNEL					
21.	Rock Excavation	2,570,000	cu. yd.	12.00	30,840,000	30,840,000
22.	Contingencies at 20%	-	job	-	6,168,000	6,168,000
23.	Engineering Design, Supervision and Administration at 10%	-	job	-	3,700,800	3,700,800
	Total First Cost					40,708,800
	TOTAL FIRST COSTS FOR REGULATORY WORKS					$88,986,400

It is estimated that this restriction and those imposed by the forces of nature would extend the construction period to 6 years. At an interest rate of 7%, the total capital cost at the end of construction would be $107,673,000 which when amortized over a 50-year project period amounts to an equivalent annual cost of $7,802,000. It is estimated that the Operation and Maintenance costs would be $287,000 increasing the annual costs to $8,089,000. The cost estimates for Lake Erie regulatory works are summarized on Table G-24.

6.5.2 Summary and Costs

Modifications to the existing compensating works at Sault Ste. Marie are required to permit safe and reliable winter operation of the control gates. The costs of these modifications are summarized in Section 6.2.3. The capacity of the existing channels and regulatory facilities at Sault Ste. Marie were checked and found to be adequate.

The regulatory facilities required to satisfy the hydraulic requirements of Plan SEO-33 consist of a control structure composed of ungated, gated and dyked sections, channel excavation, and, shore protective works along both shorelines to protect low lying areas located between the structure and the head of the river. The costs of Lake Erie regulatory works are summarized on Table G-24.

No capital expenditure would be required at the existing regulatory facilities in the St. Lawrence River, the capacity of which were checked and found to be adequate for the water supplies 1900-67.

The regulatory works required for Regulation Plan SEO-33 would require a total capital investment of $108,247,000 which when amortized over the 50-year project life at an interest rate of 7% is equivalent to an annual cost of $8,159,000 including Operation and Maintenance costs estimates at $314,000. The cost estimates for regulatory works required for SEO-33 are summarized on Table G-25.

6.6 Regulation Plan SEO-42P

Of the partial regulation plans developed for Lake Erie Regulation, Plan SEO-42P was selected for detailed benefit and cost evaluation. Unlike Regulation Plan SEO-33, Plan SEO-42P does not require any channel capacity decrease relative to the existing hydraulic regime of the Niagara River. However, the regulation plan requires that flows be restored to preproject conditions when level and supply conditions permit. The regulation of Lake Superior, under Plan SEO-42P is similar in philosophy to Plan SO-901.

No major capital expenditure would be required for the St. Marys River System. Winter operation of the control works at Sault Ste. Marie is required similar to the requirements of Plan SO-901 and would be achieved at the same costs.

The capacity of the Lower St. Marys River and the compensating works at Sault Ste. Marie were checked and found to be adequate. The maximum elevation below the locks was computed to be 582.0 IGLD as compared to the

Table G-24
SUMMARY OF ESTIMATED COSTS OF LAKE ERIE REGULATORY WORKS
REQUIRED FOR PLAN SEO-33

Total First Costs (1971 Price Levels)	$88,986,000
Total Capital Costs at the End of 6-year Construction Period	107,673,000
Annual Interest and Amortization (50-year Project Period at 7%)	7,802,000
Annual Operation and Maintenance Costs	287,000
TOTAL ANNUAL COSTS	$8,089,000

Table G-25
SUMMARY OF ESTIMATED COSTS OF REGULATORY WORKS REQUIRED
FOR PLAN SEO-33 ($ Thousands)

	LAKE OUTLET			
	Superior	Erie	Ontario	TOTAL
Total First Costs (1971 Price Levels)	574	88,986	0	89,560
Total Capital Costs at End of Construction Period	574	107,673	0	108,247
Annual Interest and Amortization (50-year Project Period at 7%)	43	7,802	0	7,845
Annual Operation and Maintenance	27	287	0	314
TOTAL ANNUAL COSTS	70	8,089	0	8,159

maximum allowable elevation of 582.9 IGLD as specified by the 1914 IJC Orders of Approval.

The existing Lake Ontario regulatory facilities are adequate to meet the changed hydraulic regime resulting from the regulation of Lakes Superior and Erie under Plan SEO-42P.

6.6.1 Lake Erie Regulatory Works

Regulation Plan SEO-42P requires that the Lake Erie outflows be increased by 8,000 cfs when supply and level conditions go beyond certain threshold limits. At other times, Lake Erie outflows would be returned to preproject (1953 outlet) conditions. The required regulatory works consist of a 1,500-foot long diversion channel through Squaw Island with a bottom width of 35 feet and an attendant control structure with a 35-foot wide submersible tainter gate. Earth levees would be constructed on both sides of the diversion channel to provide adequate freeboard. The diversion channel and earth levees would be riprapped or otherwise protected to prevent bank erosion. The total capital costs of regulatory works are estimated at $4,900,000. Operation and maintenance costs are estimated at $25,000. The total annual costs are estimated at $380,000 over a 50-year project period at an interest rate of 7%. The cost estimates for Lake Erie regulatory works required for Plan SEO-42P are summarized on Table G-26.

6.6.2 Summary and Costs

Modifications to the existing compensating works at Sault Ste. Marie are required to permit safe and reliable winter operation of the control gates. The costs of these modifications are summarized in Sections 6.2.3. The capacities of the existing channels and regulatory facilities at Sault Ste. Marie were checked and found to be adequate.

The regulatory facilities required for Lake Erie consist of a gated diversion channel through Squaw Island. Flows would be routed through the Black Rock Canal and thence through the diversion channel thus bypassing the constriction in the Upper Niagara River near the Peace Bridge. The cost estimates for Lake Erie regulatory works are summarized on Table G-26.

No capital expenditure would be required at the existing regulatory facilities in the St. Lawrence River, the capacity of which were checked and found to be adequate.

The costs of regulatory works required to meet the hydraulic and operational requirements of Plan SEO-42P are summarized on Table G-27. The regulatory works required for Regulation Plan SEO-42P would require a total capital investment of $5.5 million which when amortized over a 50-year project period at an interest rate of 7% would be equivalent to an annual cost of $450,000, including operation and maintenance.

TABLE G-26
SUMMARY OF ESTIMATED COSTS OF LAKE ERIE REGULATORY WORKS
REQUIRED FOR PLAN SEO-42-P

Total First Costs (1971 Price Levels)	$4,900,000
Total Capital Costs at the End of 1-year Construction Period	4,900,000
Annual Interest & Amortization (50-year Project Period at 7%)	355,000
Annual Operation & Maintenance	25,000
TOTAL ANNUAL COSTS	$380,000

Table G-27
SUMMARY OF ESTIMATED COSTS OF REGULATORY WORKS REQUIRED
FOR PLAN SEO-42-P ($ Thousands)

	LAKE OUTLET			
	Superior	Erie	Ontario	TOTAL
Total First Costs (1971 Price Levels)	574	4,900	0	5,474
Total Capital Costs at End of Construction Period	574	4,900	0	5,474
Annual Interest & Amortization (50-year Project Period at 7%)	43	355	0	398
Annual Operation & Maintenance	27	25	0	52
TOTAL ANNUAL COSTS	70	380	0	450

6.7 Regulation Plan SMHEO-38

Plan SMHEO-38 was selected from the SMHEO-series of regulation plans for detailed benefit and cost evaluation. Unlike any of the regulation plans previously discussed, SMHEO-38 requires regulatory facilities in the outlet rivers of each of the Great Lakes including Lake St. Clair. The following Sections describe the extent of these works and corresponding cost estimates.

No major capital expenditure would be required for the St. Marys River System. Winter operation of the control works at Sault Ste. Marie is required similar to the requirements of Plan SO-901 and at the same costs.

The capacity of the St. Marys River and the compensating works at Sault Ste. Marie were checked and found to be adequate. The maximum elevation below the locks was computed to be elevation 582.0 IGLD as compared to the maximum allowable elevation of 582.9 IGLD as specified by the 1914 IJC Orders of Approval.

The existing Lake Ontario regulatory facilities are adequate to meet the changed hydraulic regime resulting from the regulation of each of the upper Great Lakes under Plan SMHEO-38.

6.7.1 Lakes Michigan-Huron Regulatory Works

The design and cost estimates of regulatory works required for the regulation of Lakes Michigan-Huron that are summarized below for Plan SMHEO-38 were developed from the methodology presented in Section 3.

Channel Capacity Increase Requirements: The maximum channel capacity increase for the St. Clair and Detroit Rivers under Regulation Plan SMHEO-38 were determined by comparing the regulated flows with the flows that would be obtained from the regulated levels under 1933 outlet conditions. The critical design levels and flows are summarized on Table G-28.

The maximum channel capacity increase for the St. Clair River was computed to be 11,000 cfs. Since this is already built-in due to the navigation improvements carried out in the St. Clair River subsequent to 1933, the existing channels are considered adequate. Consequently, no dredging is required in the St. Clair River for Regulation Plan SMHEO-38.

Similarly, the maximum channel capacity increase for the Detroit River was computed to be 11,000 cfs. However, only 6,000 cfs of this amount is built-in relative to 1933 outlet conditions because of compensation provided in the lower Detroit River as discussed in Section 3.1.7. Therefore, dredging is required in the Detroit River, the optimal location of which would be in the Trenton Channel, at a cost of $46.0 million.

Channel Capacity Decrease Requirements: The monthly mean regulated flows resulting from Plan SMHEO-38 were compared with the flows that would be obtained from the regulated levels under 1933 outlet conditions. The critical design conditions are shown on Table G-29.

Table G-28
SUMMARY OF CRITICAL CHANNEL CAPACITY INCREASE DESIGN CONDITIONS
LAKES MICHIGAN-HURON REGULATORY WORKS
PLAN SMHEO-38

A. ST. CLAIR RIVER

1. Lakes Michigan-Huron Regulated Level .... 578.99 feet
2. Lake St. Clair Design Level .... 574.17 feet
3. St. Clair River Regulated Flow .... 203,000 cfs
4. St. Clair River Flow* .... 192,000 cfs
5. Maximum Channel Capacity Increase .... 11,000 cfs

B. DETROIT RIVER

1. Lake St. Clair Design Level .... 574.17 feet
2. Lake Erie Design Level .... 571.39 feet
3. Detroit River Regulated Flow .... 208,000 cfs
4. Detroit River Flow*............................. 197,000 cfs
5. Maximum Channel Capacity Increase .... 11,000 cfs

*Relative to 1933 outlet conditions of Lakes Michigan-Huron

Table G-29
SUMMARY OF CRITICAL CHANNEL CAPACITY DECREASE DESIGN CONDITIONS
LAKES MICHIGAN-HURON REGULATORY WORKS
PLAN SMHEO-38

A. ST. CLAIR RIVER

1. Lakes Michigan-Huron Regulated Level .... 577.91 feet
2. Lake St. Clair Design Level .... 572.84 feet
3. St. Clair River Regulated Flow .... 151,000 cfs
4. St. Clair River Flow* .... 188,000 cfs
5. Maximum Channel Capacity Decrease .... 37,000 cfs

B. DETROIT RIVER

1. Lake St. Clair Design Level .... 572.84 feet
2. Lake Erie Design Level .... 569.59 feet
3. Detroit River Regulated Flow .... 156,000 cfs
4. Detroit River Flow* .... 193,000 cfs
5. Maximum Channel Capacity Decrease .... 37,000 cfs

*Relative to 1933 outlet conditions of Lakes Michigan-Huron

The maximum channel capacity decrease required for both the St. Clair and Detroit Rivers was computed to be 37,000 cfs. However, additional retardation is required to compensate for the channel capacity increase requirements of the plan. Therefore, the maximum channel capacity decrease required for Plan SMHEO-38 is 48,000 cfs for the St. Clair and Detroit Rivers.

Regulatory Structures Required: Tests made utilizing the mathematical models of the St. Clair and Detroit Rivers indicated that the water surface profiles (1933 outlet conditions) would be obtained by construction of five structures in each of the two rivers. The five structures for the St. Clair River are those located at Port Huron, Stag Island, St. Clair, and North and Middle Channels, all of which are illustrated on Figure G-39 with the fifth structure located at Fawn Island which is discussed in the following subsection. The five structures in the Detroit River are those located at Peach Island (North and South), West Belle Isle, Zug Island, West Fighting Island and Trenton Channel as shown on Figure G-39.

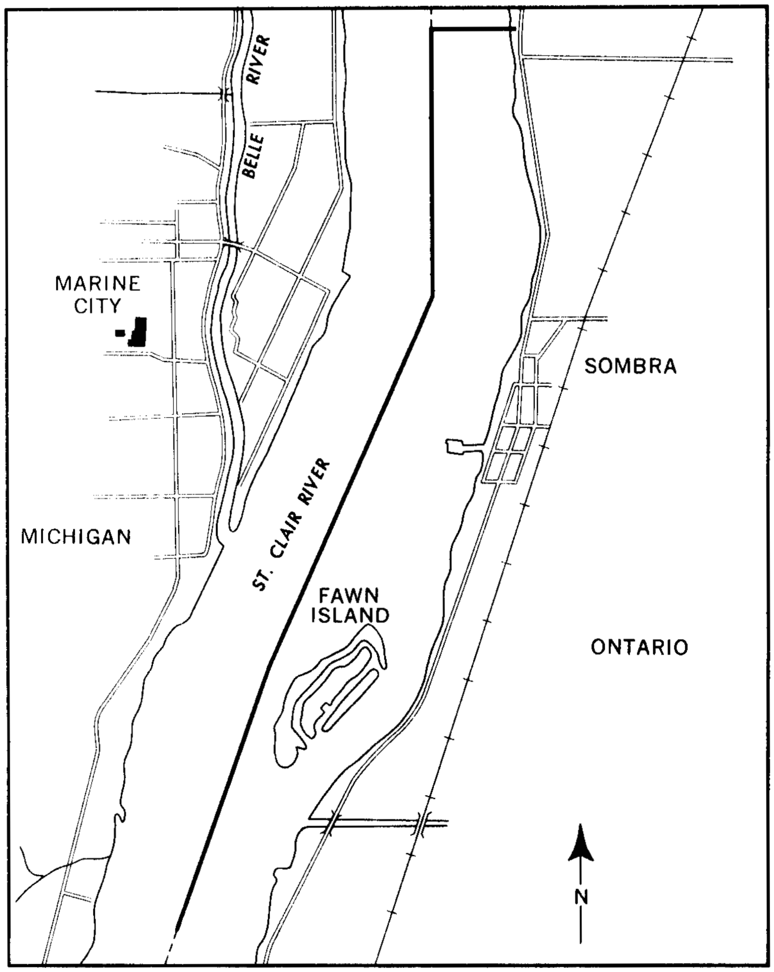
The total first costs for the ten structures are estimated at $183.1 million.

Fawn Island Structure - St. Clair River: The hydraulic studies carried out for the preparation of the preliminary cost curves considered a structure located at Fawn Island. However, the structure, incorporating a relatively short training wall, was found to be relatively ineffective as an instr ment to return the water surface profile to 1933 conditions. Furthermore, the remaining four structures provided sufficient retardation for what was assumed to be the upper limit based on the economic evaluation of very preliminary regulation plans. Since the maximum flow retardation required for Regulation Plan SMHEO-38 exceeded this upper limit established for preliminary design purposes, the Fawn Island structure was modified by extending the training wall. The design of the required works and corresponding cost estimates was carried out on the same basis, utilizing the same basic elements, as for the other nine sites described in Section 3. The location of the Fawn Island structure together with pertinent hydraulic design data are shown on Figure G-84. The total first cost of the structure was estimated at $31.0 million.

Total First, Capital and Equivalent Annual Cost of Regulatory Works: The total first costs of regulatory works, dredging and structures required in the St. Clair and Detroit Rivers for Plan SMHEO-38 were estimated at $229.1 million. A summary of the estimated costs of Lakes Michigan-Huron regulatory works is shown on Table G-30.

6.7.2 Lake Erie Regulatory Works

An analysis of Lake Erie regulated levels and flows was carried out to determine the critical design conditions, the results of which are shown in Table G-31. In summary, the regulatory structure must be capable of decreasing the outflow by 25,000 cfs at a stage of 569.78 feet and the channel must be enlarged to overcome the head loss across the structure while at the same time providing a channel capacity increase of 22,000 cfs at a stage of 571.96 feet.



ST. CLAIR RIVER CHANNEL DESIGN FOR
REGULATION OF LAKE MICHIGAN-HURON (SMHEO-38)

LOCATION OF PROPOSED CONTROL STRUCTURE	FLOW AROUND END OF STRUCTURE (C.F.S.) (CLOSED)	LENGTH OF UNGATED STRUCTURE	TOTAL LENGTH OF TRAINING WALL—FT.
FAWN ISLAND	154,000[1]	1,500	7,700

[1] EXCEPT 2000 CFS THROUGH CONTROL STRUCTURE FOR FLUSHING PURPOSES

Figure G-84
ST. CLAIR RIVER—PROPOSED REGULATORY STRUCTURE AT FAWN ISLAND

Table G-30
SUMMARY OF ESTIMATED COSTS OF LAKES MICHIGAN-HURON REGULATORY WORKS REQUIRED FOR PLAN SMHEO-38

Total First Costs (1971 Price Levels)	$229,100,000
Total Capital Costs at End of 6-year Construction Period	277,211,000
Annual Interest and Amortization (50-year Project Period at 7%)	20,087,000
Annual Operation and Maintenance Costs	687,000
TOTAL ANNUAL COSTS	20,774,000

Table G-31
SUMMARY OF CRITICAL DESIGN CONDITIONS
LAKE ERIE REGULATORY WORKS
PLAN SMHEO-38

	Channel Capacity Increase	Channel Capacity Decrease
Lake Erie Regulated Level	571.96 feet	569.78 feet
Niagara River Regulated Flow	250,000 cfs	158,000 cfs
Lake Erie Natural Level	572.95 feet	568.55 feet
Niagara River Natural Flow	228,000 cfs	183,000 cfs
Hydraulic Requirements	22,000 cfs	25,000 cfs

For purposes of this discussion, "Natural Conditions" refers to 1953 preproject outlet conditions of the Niagara River.

The total first costs of regulatory facilities were optimized, the results of which are summarized as follows:

(1) A control structure consisting of: (a) an ungated section with a span of 255 feet, (b) 8 gates (1 gate for standby) with 15 foot piers 90 feet c/c, and (c) a rock fill dyke over the remaining 925 feet; at a total first cost of $43,326,000.

(2) Channel excavation amounting to 2,090,000 cubic yards at a total first cost of $33,106,000.

(3) Shore protective works at a total first cost of $4,952,000. The above cost estimates are presented in detail on Table G-32. The total first costs of regulatory facilities would be $81,384,000.

Since it is necessary to maintain substantially the present Lake Erie stage-discharge relationship during the construction period, the dredging and control structure construction program would have to be closely integrated. It is estimated that this restriction and those imposed by the forces of nature would extend the construction period to 4 years. At an interest rate of 7%, the capital cost at the end of construction would be $92,778,000 which when amortized over a 50-year project period amounts to an equivalent annual cost of $6,723,000. It is estimated that operation and maintenance costs would be $287,000 thereby increasing the total annual cost to $7,010,000. The cost estimates for Lake Erie regulatory works are summarized on Table G-33.

6.7.3 Summary and Costs

Modifications to the existing compensating works at Sault Ste. Marie are required to permit safe and reliable winter operation of the control gates. The costs of these modifications are summarized in Section 6.2.3. The capacity of the existing channels and regulatory facilities at Sault Ste. Marie were checked and found to be adequate.

The works required in the St. Clair-Detroit River System consist of 10 control structures located at hydraulically strategic points along the river system and channel excavation in the Trenton Channel of the Detroit River. The costs of regulatory works required for the St. Clair-Detroit River System are summarized in Table G-30.

The works required in the Niagara River consist of a control structure, shore protective works on both sides of the river channel between the control structure and the head of the river in the channel excavation. The costs of regulatory works required for the Niagara River are summarized in Table G-33.

No capital expenditure is required at the existing regulatory facilities in the St. Lawrence River, the capacity of which were checked and found to be adequate for the change in inflow regime resulting from the regulation of the upstream lakes.

Table G-32

ESTIMATE OF FIRST COST (1971 PRICE LEVELS), LAKE ERIE CONTROL WORKS

REGULATION PLAN SMHEQ-38

	ITEM	QUANTITY	UNIT	UNIT COST	AMOUNT	TOTALS
	CONTROL STRUCTURE			$	$	$
1.	Diversions & Cofferdams					
	a) Sheet Piling	2,615	ton	500.00	1,307,500	
	b) Fill	318,280	cu. yd.	1.00	318,300	
	c) Pumping	-	job	-	584,000	
	d) Removal	-	job	-	325,000	2,534,
2.	Excavation					
	a) Overburden	30,000	cu. yd.	2.50	75,000	
	b) Rock	215,000	cu. yd.	6.00	1,290,000	
	c) Rock Surface Preparation	17,700	sq. yd.	3.00	53,100	1,418,
3.	Grouting	47,750	l.f.	8.00	382,000	382,
4.	Concrete (Including Formwork)					
	a) Base Slab	127,000	cu. yd.	40.00	5,080,000	
	b) Rollway	27,570	cu. yd.	45.00	1,240,700	
	c) Piers	67,350	cu. yd.	60.00	4,041,000	
	d) Retaining Wall	35,500	cu. yd.	55.00	1,952,500	
	e) Bridge	3,095	cu. yd.	70.00	216,700	12,530,9
5.	Reinforcing Steel	4,150	ton	400.00	1,660,000	1,660,
6.	Gates and Hoists					
	a) Stoplog Gates	780	ton	1,360.00	1,061,000	
	b) Stoplog Guides	260	ton	1,400.00	364,000	
	c) Stoplog Crane	-	job	-	400,000	
	d) Tainter Gates	3,800	ton	1,360.00	5,168,000	
	e) Tainter Gates - Embedded Parts	1,320	ton	1,400.00	1,848,000	
	f) Tainter Gates Hoists	8	unit	300,000.00	2,400,000	11,241,0
7.	Rock Fill Dam					
	a) Stripping Common	27,400	cu. yd	0.80	21,900	
	b) Rock Fill & Riprap	105,000	cu. yd.	6.00	630,000	
	c) Gravel Roadway	6,710	sq. yd.	4.00	26,800	678,
8.	Control Building (5,000 sq. ft.)	-	job	-	150,000	150,
9.	Miscellaneous, Mechanial and Electrical	-	job	-	800,000	800,
10.	Contingencies at 20%	-	job	-	6,279,100	6,279,
11.	Engineering Design, Supervision and Administration at 15%	-	job	-	5,651,200	5,651,
	Total First Cost					43,325,
	SHORE PROTECTION WORKS					
12.	Property Acquisition and Land Development	-	job	-	284,000	284,
13.	Guard Gate for Black Rock Canal					
	a) Sector Gate	175	ton	1,360.00	238,000	
	b) Embedded Parts	62	ton	1,400.00	86,800	
	c) Concrete Guides	-	job		100,000	424,

Table G-32 (Cont'd)
ESTIMATE OF FIRST COST (1971 PRICE LEVELS), LAKE ERIE CONTROL WORKS
REGULATION PLAN SMHEO-38

	ITEM	QUANTITY	UNIT	UNIT COST	AMOUNT	TOTALS
				$	$	$
14.	Raising of Concrete-masonry Wall (Canadian Side)					
	a) Removal of existing cap	8,500	l.f.	2.00	17,000	
	b) Concrete	1,500	cu. yd.	45.00	67,500	
	c) Reinforcing Steel	15	ton	400.00	6,000	90,500
15.	Rockfill Dykes					
	a) Stripping	14,400	cu. yd.	0.60	8,640	
	b) Rockfill	218,300	cu. yd.	6.00	1,309,800	
	c) Gravel Roadway	14,000	sq. yd.	4.00	56,000	1,374,400
16.	Concrete Facing & Key					
	a) Face Slab	1,800	cu. yd.	45.00	81,000	
	b) Key	5,200	cu. yd.	40.00	208,000	
	c) Reinforcing Steel	37	ton	400.00	14,800	303,800
17.	Crib Wall					
	a) Timber Crib	10,000	cu. yd.	30.00	300,000	
	b) Concrete Cap & Wall	10,200	cu. yd.	55.00	561,000	
	c) Reinforcing Steel	96	ton	400.00	38,400	899,400
18.	Concrete-stone Breakwater	37,400	cu. yd.	6.00	224,400	224,400
19.	Contingencies at 25%	-	job	-	900,300	900,300
20.	Engineering Design, Supervision and Administration at 10%	-	job	-	450,200	450,200
	Total First Cost					4,951,800
	CHANNEL					
21.	Rock Excavation	2,090,000	cu. yd.	12.00	25,080,000	25,080,000
22.	Contingencies at 20%	-	job	-	5,016,000	5,016,000
23.	Engineering Design, Supervision and Administration at 10%	-	job	-	3,010,000	3,010,000
	Total First Cost					33,106,000
	TOTAL FIRST COSTS FOR REGULATORY WORKS					$81,383,600

The regulatory works required for Regulation Plan SMHEO-38 would require a total capital investment of $371 million which when amortized over a 50-year project period at an interest rate of 7% is equivalent to annual costs of $27,854,000 including operation and maintenance costs estimated at $1,001,000. The estimated costs of regulatory works required for Plan SMHEO-38 are summarized on Table G-34.

Table G-33
SUMMARY OF ESTIMATED COSTS OF LAKE ERIE REGULATORY WORKS
REQUIRED FOR PLAN SMHEO-38

Total First Costs (1971 Price Levels)	$81,384,000
Total Capital Costs at End of 4-year Construction Period	92,778,000
Annual Interest and Amortization (50-year Project Period at 7%)	6,723,000
Annual Operation and Maintenance Costs	287,000
TOTAL ANNUAL COSTS	$7,010,000

Table G-34
SUMMARY OF ESTIMATED COSTS OF REGULATORY WORKS REQUIRED
FOR PLAN SMHEO-38 ($ Thousands)

	LAKE OUTLET				
	Superior	Michigan-Huron	Erie	Ontario	TOTAL
Total First Costs (1971 Price Levels)	574	229,100	81,384	0	311,058
Total Capital Costs at End of Construction Period	574	277,211	92,778	0	370,563
Annual Interest and Amortization (50-year Project Period at 7%)	43	20,087	6,723	0	26,853
Annual Operation and Maintenance Costs	27	687	287	0	1,001
TOTAL ANNUAL COSTS	70	20,774	7,010	0	27,854

ANNEX A

September 21, 1967

Terms of Reference

Regulatory Works Subcommittee

1. To prepare a detailed work program, schedule and preliminary estimates of study costs.

2. To prepare preliminary designs, plans, estimated first costs and annual costs (including operation and maintenance) of regulatory works and associated navigation locks, alterations to existing works and to channel design as may be required to discharge critical high and critical low flows at specified lake levels under various regulation plans. The works or alterations to be considered are in the following areas of the Great Lakes - St. Lawrence System:

   (a) Outlet of Lake Superior;

   (b) Between Lakes Huron and Erie;

   (c) Outlet of Lake Erie;

   (d) International Reach of St. Lawrence River; and

   (e) Canadian Reach of St. Lawrence River.

3. To perform surface and sub-surface surveys at the project sites as re quired to prepare the necessary preliminary plans, designs and cost estimates.

4. To coordinate its activities with the Subcommittees on Navigation and Regulation Subcommittee.

ANNEX B

MEMBERS AND ASSOCIATES
REGULATORY WORKS SUBCOMMITTEE (1967-1973)

NAME	AGENCY	PERIOD OF SERVICE FROM	TO
J. Bathurst[1]	Environment Canada	Sept. 1967	Completion
C. J. R. Lawrie	Ministry of Transport	Feb. 1970	Completion
K. A. Rowsell	Can. Dept. of Public Works	June 1972	Completion
J. D. Keefe	Environment Canada	Dec. 1971	Completion
J. D. Bouchard[2]	St. Lawrence Seaway Authority	Sept. 1967	Completion
B. Malamud[1]	U. S. Army Corps of Engineers	Feb. 1972	Completion
K. Hallock	U. S. Army Corps of Engineers	May 1971	Completion
J. Raoul	U. S. Army Corps of Engineers	May 1973	Completion
I. M. Korkigian (Deceased)	U. S. Army Corps of Engineers	Sept. 1967	Completion
C. A. Aune[2]	U. S. Army Corps of Engineers	Sept. 1967	Completion
G. Millar (Deceased)[3]	Can. Dept. of Public Works	Sept. 1967	June 1972
N. H. James	Environment Canada	Sept. 1967	Dec. 1971
V. G. Goelzer[3]	U. S. Army Corps of Engineers	Sept. 1967	Apr. 1970
R. H. Gallinger	U. S. Army Corps of Engineers	Sept. 1967	May 1970
A. E. Wanket[3]	U. S. Army Corps of Engineers	May 1970	Jan. 1972
R. B. McKee	U. S. Army Corps of Engineers	Apr. 1970	May 1971
S. H. Fonda, Jr.	U. S. Army Corps of Engineers	Sept. 1967	Apr. 1973

[1] Chairman, Respective Section

[2] Long-Term Associates

[3] Past Chairman, Respective Section

ANNEX C

ABSTRACTS
CONTRIBUTARY REPORTS TO APPENDIX G

TABLE OF CONTENTS

WINTER OPERATIONS AT THE LAKE SUPERIOR REGULATORY WORKS, SAULT STE. MARIE, WINTERS OF 1968-69, 1969-70, 1970-71 AND 1971-72, REGULATORY WORKS SUBCOMMITTEE, ANNUAL REPORTS

ABSTRACT

Winter gate tests were conducted at the Lake Superior compensating works during the 1968-69, 1969-70, 1970-71 and 1971-72 winter periods to determine: (1) if a maximum winter flow of 85,000 cfs is too conservative, and if so, can it be increased?, (2) if the St. Marys River can carry higher flows during the winter period or during part of it, and if so, when and to what limit?, and (3) if it is practical to change the gate settings and vary the flow as a normal procedure during the winter months, and if so, by what means and how much would it cost? The reports describe the equipment used, the maintenance carried out and the equipment problems encountered and how they were solved. The hydrologic conditions leading up to the tests were outlined and reviewed from the context of the overall planning for each winter period. The hydraulic monitoring and communication system established to identify incipient ice jamming conditions and its application are described. The actions taken under winter conditions are outlined and discussed. In general, these included preliminary tests on the mechanical equipment to ensure their adequacy, an increased flow test whereby the flow was increased beyond 85,000 cfs when feasible and a simulated emergency gate closing operation to determine if the gates could be closed in sufficient time to overt an ice jam. During the latter two winter periods, actual emergencies arose and were responded to adequately. Hydrologic, hydraulic, meteorlogical and ice survey data collected during the gate test program are presented and analyzed. The time taken to carry out each gate movement operation was analyzed in light of the prevailing conditions at the compensating works. The costs of the winter gate test program are presented and discussed. The costs of each gate movement were analyzed so as to provide a basis for the costing of the selected regulation plans.

LAKE SUPERIOR REGULATORY STRUCTURE, REPORT ON PRESENT CONDITION AND TERMS OF REFERENCE FOR FEASIBILITY STUDY OF WINTER OPERATIONS, AD HOC GROUP, REGULATORY WORKS SUBCOMMITTEE, MAY 1971

ABSTRACT

The Regulatory Works Subcommittee established an Ad Hoc Group to (a) provide an assessment of the present condition and remaining life of the Lake Superior Regulatory Structure, and (b) to formulate Terms of Reference for a study that would investigate alternate method of winter operation and other modifications which would improve the safety and efficiency of the compensating works. A field inspection was carried out on December 16, 17, 1970. A brief description of the compensating works is provided. The gates, piers, mechanical equipment and sills were investigated and reported on. Photographs are provided to document findings. It was concluded that the remaining life of the structure should be 50 years if not indefinite provided a maintenance program be undertaken to bring the Canadian portion of the structure up to the same standards as the American portion. Terms of Reference for a study of modifications to the compensating works to permit safe and reliable winter operation are also included.

LAKE SUPERIOR REGULATORY STRUCTURE, FEASIBILITY STUDY FOR IMPROVEMENTS TO LAKE SUPERIOR CONTROL WORKS, ACRES CONSULTING SERVICE, MARCH 1972

ABSTRACT

The Consulting Engineering Firm of Acres Consulting Services were engaged by the Subcommittee to investigate alternate methods, and their corresponding costs, of modifying the Lake Superior compensating works to permit safe and reliable winter operation of the control gates. The methods examined for heating the gates and/or gains included Hot Air, Electric Tubular Heaters, Air Bubblers, Radiant Heating and Steam Heating or combinations thereof. Each method was examined from the point of view of efficiency, practicability, energy source, safety and costs and related to the problem at hand. Electric tubular heaters for both gate and gain heating was selected primarily for efficiency and cost reasons but also in view of their past performance in climates not unlike those of Sault Ste. Marie. Various methods of energy supply to the site were examined including electrical power from the Great Lakes Power Company and/or Edison Sault Electric Company, Natural gas from North Central Gas Corp. Ltd., Steam Power and Diesel Generator Power. Electrical energy from the Great Lakes Power Company Limited was selected on the basis of cost and reliability as well as safety reasons. The cost of telephone service to the site was determined. An electric motor drive was chosen over the gasoline engine drive to motorize the existing manual gate moving equipment on the basis of reliability and operational ease. The gates could be moved either from a central control panel or from the bridge with the manual system retained as back-up. All open gear machinery would be covered with hinged sheet steel covers. The hoist bridge for the middle 10 gates would be fully enclosed in a continuous, weatherproof housing. A prepainted metal clad enclosure was selected over the asbestos clad enclosure on the basis of durability and lower capital and maintenance costs. Various preliminary Lake Superior Regulation Plans were costed based on the recommended facilities.

PRELIMINARY SUBSURFACE INVESTIGATION, PROPOSED FLOW CONTROL STRUCTURES, ST. CLAIR RIVER ONTARIO, 4 VOLS., WILLIAM TROW AND ASSOCIATES (HAMILTON) LIMITED, APRIL 1971

ABSTRACT

Part I is a summary of the subsurface soil investigations performed for the Subcommittee along the St. Clair River between Pt. Edward and Sombra, Ontario, for four proposed Flow Control Structures. The structures are to be located at Point Edwards, Stag Island (Corunna) St. Clair, Michigan (Mooretown) and at Fawn Island (Sombra). The subsurface soil investigations determined the geotechnical conditions at each site. The data is presented in this report. The report also covers preliminary foundation designs at the structures and includes preliminary designs of various types of training walls for each structure. Construction methods for the foundations and training walls are discussed. The character of the material to be dredged is given in the report. The material to be dredged can be summarized as loose to very loose sands, gravels and soft to stiff clays. The chemical characteristics of the ground water as water in a concrete mix was investigated. Part II of William Trow Associates work is a set of drawings showing where the subsurface soil investigations were made. The report consists of seven drawings. Part III is a set of six drawings, three of which are geotechnical profiles and three are drawings that contain a graphical representation of each boring. Part IV contains a graphical representation of each boring with shear strength data and natural moisture content data plotted opposite the boring log. This part of the report also contains all the detailed laboratory plotted test data.

SUBSURFACE INVESTIGATION, PROPOSED REGULATORY STRUCTURES BETWEEN LAKE HURON, LAKE ST. CLAIR AND LAKE ERIE, U. S. ARMY CORPS OF ENGINEERS, NOVEMBER 1971

ABSTRACT

This report covers those Regulatory Structures and the dredging areas between Port Huron and Lake Erie that are located in United States waters. The report covers site geology for the St. Clair and the Detroit Rivers and the subsurface soil investigations. Boring logs with the corresponding soil data plotted adjacent to the boring logs and individual laboratory test data results are included in the report. This report also covers a foundation analysis and recommendations for foundation design for the flow control structures and training walls. The report discusses the type of material to be dredged in the channels that are proposed to be deepened. The majority of the material to be dredged in order to achieve required channel depths, will be a loose to medium dense sand to silty sands and soft to medium clays.

SUPPLEMENT, PRELIMINARY SUBSURFACE INVESTIGATION, PROPOSED REGULATORY STRUCTURES FOR DETROIT RIVER, U. S. ARMY CORPS OF ENGINEERS, DECEMBER 1971

ABSTRACT

This report was prepared to supplement the November 1971 Subsurface Investigation report and covers the Detroit River proposed regulatory structures. The purpose of the subsurface soil investigations was to determine the soil conditions at each structure site and to analyze the subsurface conditions at each site. The report covers structures at the Belle Isle, Zug Island, and Grass Island locations and training dikes at each structure location. Site Geology of the Detroit River Basin is included in the report. Foundation conditions and recommended foundation design, dredging and channel excavation conditions are described. The material that would be excavated consists of an alluvium and soft to medium glacio - lacustrine material. The alluvium consists of sandy silt, silty clay and combinations of soft to medium gray clay.

REGULATORY STRUCTURES ON THE ST. CLAIR - DETROIT RIVERS, DEVELOPMENT OF CONCEPTUAL DESIGNS, TERMS OF REFERENCE, AD HOC GROUP, REGULATORY WORKS SUBCOMMITTEE

ABSTRACT

The purpose of the "Terms of Reference" was to provide potential consulting firms the background fundamental information necessary to develop designs and cost estimates of Regulatory Structures on the St. Clair - Detroit River System.

The document included a brief description of the regulation studies, a description of the river system, objectives required for design, data required including hydraulic results, design criteria and finally general administrative conditions.

ST. CLAIR - DETROIT RIVERS, CONCEPT OF REGULATORY STRUCTURES, VOLUME 1 - CONCEPTUAL DESIGN AND ESTIMATES, VOLUME II - CRITERIA AND SELECTION OF ELEMENTS, ACRES CONSULTING SERVICES, APRIL 1972

ABSTRACT

The necessary engineering studies to prepare structure designs and cost estimates for regulatory structures in the St. Clair - Detroit River System were provided by the consulting firm of Acres Consulting Services Limited, Niagara Falls, Canada. Terms of reference for the study and concepts were provided by the Regulatory Works Subcommittee in a document entitled "Regulatory Structures on the St. Clair - Detroit Rivers, Development of Conceptual Design, Terms of Reference," an abstract of which is contained in this section on Appendix G.

This study was presented in two volumes. Volume 1, entitled "Conceptual Design and Estimates," contains the layouts, conceptual designs and cost estimates for the regulatory structures and their associated works such as the small boat passage, shore protection works, etc., at each of nine sites. Volume II, entitled "Criteria and Selection of Elements," presents factors and criteria that govern the design of the parts or elements of the structures. Various concepts of the elements of the structures are described, and background environmental considerations and construction methods that relate to the concepts are referred to.

ST. CLAIR-DETROIT REGULATORY WORKS, REPORT ON CHANNELIZATION REQUIREMENTS FOR THE ST. CLAIR-DETROIT RIVER SYSTEM, AD HOC GROUP, REGULATORY WORKS SUBCOMMITTEE, JANUARY 1974

ABSTRACT

The paper presents the dredging requirements in the St. Clair - Detroit River system in order to increase the hydraulic capacity of the system. Types of material to be dredged and proposed methods of dredging are discussed. Maps showing the locations of reaches to be dredged, and locations of dredge disposal sites are shown. Disposal sites are selected in order to minimize transportation cost as well as adverse effects on marine environment. Methods to prepare and enhance disposal sites are discussed. The total costs of dredging, which include hauling, dumping, site preparation, utility relocation, etc., are summarized. The paper also discusses the effects of dredging and proposed regulatory works on the environment, and concludes with a recommendation that a working model be built to access and evaluate the effects of regulation on the environment.

DEVELOPMENT, CALIBRATION AND APPLICATION OF MATHEMATICAL MODELS FOR THE ST. CLAIR, DETROIT AND NIAGARA RIVERS, REGULATORY WORKS SUBCOMMITTEE, JANUARY 1974

ABSTRACT

Mathematical models, or digitized abstractions of the hydraulic characteristics of the Upper Niagara and St. Clair/Detroit Rivers, were employed by the Subcommittee to determine the location, nature and extent of regulatory works required to satisfy certain hydraulic requirements. Basic open channel flow relationships were used. Manning's roughness coefficients were applied to estimate friction losses while variable coefficients were applied to estimate losses due to expansion and contraction. A kinetic energy coefficient was applied to the Velocity head at each section to approximate for the use of mean Velocity. The basic hydraulic characteristics were determined at each of the selected sections based on the most up-to-date hydrographic and topographic data. The models were calibrated using recent discharge and corresponding water level measurements taken in each river. The results of the calibration are indicated. Tests were also made to determine the applicability of the model over a wider range of flow using adopted open-water gauge relationships. The application of the models in determining the optimal location and quantity of the proposed dredging as well as the location of the proposed flow control and appurtenant structures is discussed for each river system. It was assumed that Manning's roughness coefficient would not change significantly under slightly modified channel hydrography.

PRELIMINARY FOUNDATION INVESTIGATION, NIAGARA RIVER FLOW CONTROL STRUCTURE AND APPURTENANCES, FORT ERIE, ONTARIO, WILLIAM TROW ASSOCIATES (HAMILTON) LIMITED, MARCH 1970

ABSTRACT

The field work consisted of core drilling 7 boreholes into bedrock, 4 of which were located near the axis of the proposed control structure at the lower site and 3 along the Canadian shoreline where shore protective works would be required. In addition, 25 boreholes were augered to refusal along the Canadian shoreline. Soil and rock samples were analyzed to determine the textural classification and geologic origin. Standard tests were conducted to determine the prevailing geotechnical conditions. All collected and analyzed data are contained in the report. Three types of soil were encountered: (a) alluvium consisting of very loose to dense silts, sands and gravels, (b) glacial till consisting of very stiff to hard silty clay and clayey silt sand, (c) sanitary land fill on the U. S. shoreline on Squaw Island and sandy silt, ashes, bricks, etc., on a portion of the Canadian shoreline. The surficial bedrock at the structure site consisted of cyclically deposited anhydrite/gypsum, dolomite and shale of the salina formation. The top 10 feet is weathered, stress relieved and water bearing. Considerable quantities of anhydrite-gypsum evaporites are present which may be subject to internal erosion as a result of a differential head across the structure. An echo sounder and sparker survey was conducted in the vicinity of the flow control structure to establish the river bottom and bedrock profiles. Recommendations concerning the nature of the structure foundation and shore protective works were made.

PRELIMINARY COST STUDIES, LAKE ERIE REGULATION, ENVIRONMENT CANADA, APRIL 1972

ABSTRACT

The design and cost estimates of two alternate control structures and appurtenances required for the regulation of Lake Erie outflows are presented. Common design criteria were used throughout so that a valid comparison could be made. Both structures were designed over a wide range of hydraulic requirements, ±60,000 cfs. The Niagara River mathematical model was used to determine the nature and extent of the required dredging and to access the hydraulic adequacy of the regulatory works, both structure and dredging. The predominant forces which influenced the selection of the basic components of the control structure were those imposed by ice and seiche. A submersible Tainter gate was selected in view of the requirement to pass ice with thicknesses up to 20 feet. At each site, the span was made as large as practicable to ensure the passage, to the degree possible, of ice floes. In view of the requirements of the 1950 Niagara Diversion Treaty, an ungated section of the structure was designed for thus realizing significant cost savings and providing a relatively large opening for the passage of ice flows. A sill block, 20 feet in thickness, would be required for the base of the structure to provide an adequate safety factor against floatation. Other features of the control structure included 15-foot concrete piers recessed upstream and downstream of the tainter gate for stoplogs that would be required for maintenance. The layout of the structure at either site would consist of: a rock filled dyke extending from the Canadian shore overlain by a gravel roadway for site access and a series of piers and tainter gates over which a bridge deck and crane would be provided and an ungated section. The gated and ungated section would be founded on a 20-foot thick concrete sill tied into splayed wing walls at either end. A control building would be constructed on the Canadian shoreline. Unit costs for both the structure and channel excavation were developed from past projects conducted in the Great Lakes area. Cost curves were developed relating the length of gated and ungated structure required to their cost. These cost curves were then integrated with channel excavation costs and related to the range of hydraulic requirements. The lower site, 1000 feet south of the International Bridge, was selected on the basis of a cost saving of approximately $30 million. Shore protective works were required along both shorelines to prevent innundation of low lying properties as a result of the changed hydraulic regime upstream of the control structure. These studies formed a basis for the final design of selected regulation plans as well as for the development of preliminary regulation plans.

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