ST. JOHN'S

REGIONAL WATER SYSTEM STUDY

Volume II

CANADA
DEPARTMENT OF REGIONAL
ECONOMIC EXPANSION

GOVERNMENT OF NEWFOUNDLAND
AND LABRADOR
DEPARTMENT OF MUNICIPAL
AFFAIRS & HOUSING

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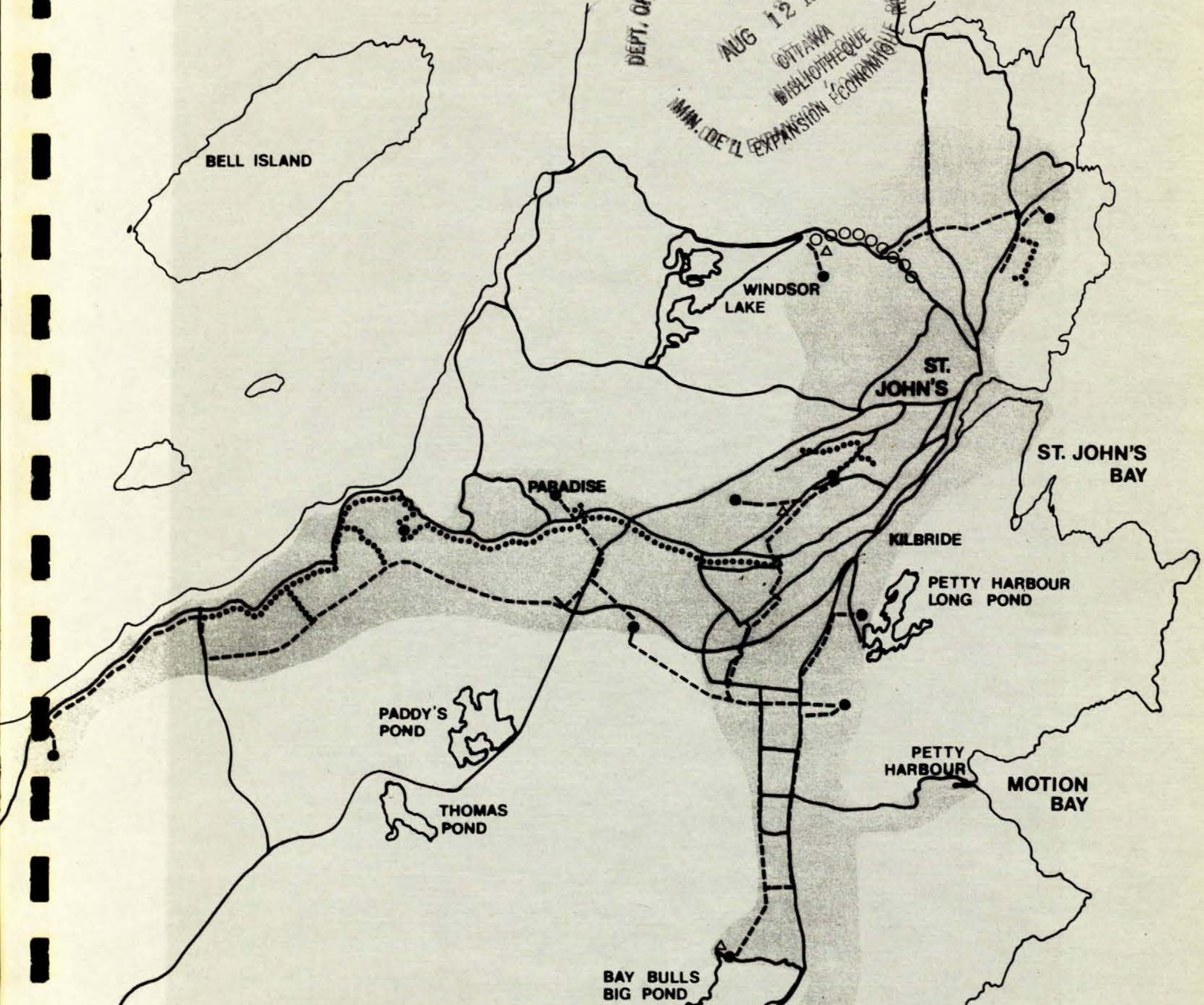
ST. JOHN'S REGIONAL WATER SYSTEM STUDY

St. John's Special Area Project 3.1

Volume II

Chapters 4, 5, 6



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December 1974

Abbreviations

acre-foot .... acre-ft
average .... avg

biochemical oxygen
demand .... BOD
brake horsepower .... bhp

capita .... cap
cubic .... cu
cubic centimeter (s) ..... cu cm = ml
cubic feet per day .... cfd
cubic feet per hour ...... cfh
cubic feet per minute .... cfm
cubic feet per second .... cfs
cubic foot (feet) .... cu ft.
cubic inch(es) .... cu in.
cubic yard(s) .... cu yd

degree(s) .... deg
degree(s) Centigrade
(Celsius) .... ^{o}C
degree(s) Fahrenheit ..... ^{o}F
diameter .... dia
dissolved oxygen .... DO
dissolved solids .... DS

elevation .... el
equation .... eq
exponential .... exp

feet .... ft
figure(s) .... Fig
foot .... ft

gallon(s) US .... US gal
gallon(s) (Imperial)
(Imp.)............. gal or Igal
gallon(s) per capita per
day (Imp.)....... gpcd or Igpcd

gallon(s) per day
(Imp.) .... gpd or Igpd
gallon per day per
acre (Imp.) .... gpd/acre
gallons per day per
capita (Imp.) ..... gpd/cap
gallons per day per
square foot(Imp.).. gpd/sq ft
gallon(s) per
hour (Imp.) .... gph
gallon(s) per minute
(Imp.) .... gpm
gallon(s) per second
(Imp.) .... gps
grams per liter .... g/l

horsepower .... hp
horsepower-hour(s) .... hp-hr
hour(s).................. hr
hydrogen ion concentration
(-log [H+]) .... pH

inch(es) .... in.

Jackson turbidity
units .... Jtu

kilovolt(s).............. kv
kilowatt(s) .... kw
kilowatt-hour(s) .... kwh

linear foot .... lin ft
liters l
logarithm (common-
base 10) .... log
logarithm (natural-
base e) .... ln

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man-hour(s)	man-hr
maximum	max
membrane filter	MF
meter(s)	m
mho(s)	mho
microgram(s)	µg
microgram(s) per liter	µg/l
microliter	µl
micron(s)	µ
mile(s)	mi
milligram(s)	mg
milligrams per liter	mg/l
milliliter(s)	ml
million gallons (Imp.)	mil gal or MG
million gallons per day (Imp.)	mgd
million gallons per day per acre (Imp.)	mgd/acre
minimum	min.
minute(s)	min
most probable number	MPN
number(s)	No.
part(s) per billion	ppb=µg/l
part(s) per million	ppm = mg/l
percent	% or percent
pound(s)	lb
pound(s) per square inch	psi
pound(s) square inch absolute	psia
pound(s) per square inch gage	psig
revolution(s) per minute	rpm
revolution(s) per second	rps
second(s)	sec
second feet (cubic feet per second)	cfs
square	sq
square foot (feet)	sq ft
square inch(es)	sq in
volume	vol
weight	wt
yard(s)	yd
year(s)	yr

These symbols may be used in conjunction with numerical values or in mathematical expression.

greater than	> or G
less than	< or L
infinity	∞

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CHAPTER 4

FUNDAMENTAL DESIGN CONSIDERATIONS

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FUNDAMENTAL DESIGN CONSIDERATIONS

SYNOPSIS

Fundamental design factors for consideration in the planning of waterworks have been presented and analysed in this Chapter. A summary of the findings and recommendations is as follows:

- Design Period: To the year 1995.

- Study Region: Has been tentatively assumed to extend from Torbay, Portugal Cove Road, and Portugal Cove in the North, to the Goulds and Seal Cove in the South. A systems economic analysis (carried out under the works included in Chapter 6) will accurately define the study region.

- Communities in Study Region: Have been grouped into the following categories:

  (a) Regional Centre - St. John's and expansion zones, Mount Pearl, New Town, Kilbride, Wedgewood Park, Shea Heights, (Blackhead Road).

  (b) Sub-Regional Centre - Conception Bay South Area (Seal Cove, Gullies, Kelligrews, Foxtrap, Long Pond, Manuels, Chamberlains, Topsail).

(c) Local Centres "A" - Paradise, Topsail Road, Torbay, Torbay Road, Penetanguishene, Goulds, Petty Harbour.

(d) Local Centres "B" - St. Phillips, Portugal Cove, Portugal Cove Road, Thorburn Road.

- Population: Based on the St. John's Urban Regional Plan Study - S.J.U.R.P.S. - the 1995 population projections, relative to community categories, are:

(a)	Regional Centre	-	159,000
(b)	Sub-Regional Centre	-	17,410
(c)	Local Centres "A"	-	17,770
(d)	Local Centres "B"	-	10,090

- Land Use: In accordance with S.J.U.R.P.S., with Industrial Park developments at:
  - Donovan's, 992 acres
  - Kenmount, 496 acres
  - White Hills, 496 acres

- Water Use: 1995 average daily annual projections are:
  - Urban St. John's, 135 GPCD
  - Suburban Developments, 90 GPCD
  - Industrial Parks, 2,500 GPAD

These projections result in a total average daily demand of:

(a)	Regional Centre	-	18.43 MGD
(b)	Sub-Regional Centre	-	1.57 MGD
(c)	Local Centres "A"	-	1.60 MGD
(d)	Local Centres "B"	-	0.91 MGD
(e)	Industrial Parks	-	4.96 MGD

- Water Use Maximum Factors:

  - Factor for maximum day relative to average annual use is 1.3, and 1.5 for urban St. John's, and suburban developments, respectively.

  - Factor for maximum hour relative to average annual use is 1.7, and 2.0 for urban St. John's, and suburban developments, respectively.

- Fire Demand:

  - 3,500 GPM for urban St. John's for a four hour duration.

  - 1,000 GPM for residential areas of suburban developments, such as Conception Bay South Area, for a two hour duration.

  - 2,500 GPM for suburban developments with large mall shopping centres such as Mount Pearl and New Town, for a three hour duration.

  - 3,500 GPM for industrial parks, for a four hour duration.

- Service Reservoirs: Will have the capacity to provide the maximum hourly fluctuations in water need, the fire demand, and a reserve capacity of one half maximum day water requirement.

- Water Quality: Will be in accordance with the "Canadian Drinking Water Standards and Objectives, 1968". This document differentiates between "objectives" and "acceptable limits" of water constituents. It is recommended that the water quality "objectives" be adopted as the goals for the design period.

- Pressure Zones: The following pressure zones are recommended:

  - Low pressure zone to extend between elevations of 0 and 210 feet (above sea level). The pressure elevation will be 300 feet.

  - Intermediate pressure zone to extend between elevations 215 and 425 feet. The pressure elevation will be 515 feet.

  - High pressure zone to extend between elevations 425 and 625 feet. The pressure elevation will be 715 feet.

- Metering: Financially, it is not advisable to consider metering of residential customers at this stage.

I. INTRODUCTION

Waterworks planning concerns itself with the typical factors that may influence customer water use. Some of the principal factors in this respect include the nature and administrative organization of the service area, its land use, the nature of industry, business and institutional service, population, quality and adequacy of water supply sources, standard of living, status of economy, climatological conditions, and level of water rate changes.

This chapter presents an assessment of current trends in customer water use and an analysis of fundamental design parameters for the projection of future water requirements, to ensure that the available sources of water would be allocated according to needs and would be exploited to the best advantage in the interest of all regional water consumers.

As explained in Chapter 6, it has been tentatively assumed that the supply region will extend from Torbay, Portugal Cove Road and Portugal Cove in the North, to the Goulds and Seal Cove in the South. Based on the results of the systems economics analysis, as included in Chapter 6, the boundary of the supply region will be defined and accordingly a final adjustment will be made to the water requirements projection of this region. With this basic concept in mind, future forecasts for population and water use of the communities in the study region have been grouped into the following categories:

(a) Regional Centre - St. John's and expansion zones, Mount Pearl, New Town, Kilbride, Wedgewood Park, Shea Heights (Blackhead Road).

(b) Sub-Regional Centre - Conception Bay South Area (Seal Cove, Gullies, Kelligrews, Foxtrap, Long Pond, Manuels, Chamberlains, Topsail).

(c) Local Centres "A" - Paradise, Topsail Road, Torbay, Torbay Road, Penetanguishene, Goulds Petty Harbour.

(d) Local Centres "B" - St. Phillips, Portugal Cove, Portugal Cove Road, Thorburn Road.

II. DESIGN PERIOD

It is customary to design water supply schemes for requirements 20 years hence, or greater period, giving consideration to the potentialities of the water sources for a longer design period.

It is recommended that the 20 year design period postulated for this project commence at the time of first stage construction thus making the design period effective for twenty full years. We have assumed 1975 to be the earliest practical year for the first stage construction. Accordingly, the study design period will extend to the year 1995.

III. AREA WITHIN SUPPLY REGION

1. General

The area within the study region contains a variety of structural (and non-structural) political boundaries. In the region, extending from Torbay in the north to the Goulds and Seal Cove in the south, 13 incorporated areas and 2 non-incorporated areas can be identified. As can be expected, of the autonomous areas in the region, only the City of St. John's has a central water supply and distribution system, save for a minor system in Petty Harbour. By special arrangement with adjacent incorporated areas, the City has extended its supply mains to provide water to Mount Pearl, Kilbride, Wedgewood Park and Shea Heights. The other coastal and inland autonomous areas do not have municipal water systems.

2. Service Area and Supply

It has been recognized that limitations due to political boundaries, or other causes, such as topography and economics, tend to restrict a water utility from expanding into growing adjacent areas. These facets have to be borne in mind as they will have a material influence on future water use predictions. It is apparent, however, that amalgamation of water utilities can more readily permit greater overall efficiency and reliability, and further ensures both the quality and adequacy of the water supply. With such an "amalgamation" there would be an obvious influence in an increased requirement of "regional" water.

Normally, small and fragmented local water supply systems can result in poorly planned, inadequately supervised, and generally unsatisfactory operations. Conversely, a regional authority, detached from political and jurisdictional boundaries, can have the resources to approach the water supply requirements on a highly technical, economical, and rational basis, and would be able to provide the necessary level of management competency. The optimum size of a regional water supply system, which will function efficiently and economically, cannot be standardized. In order to establish and define the optimum size of the "St. John's Regional Water Supply System" (S.J.R.W.S.S.), and consequently water requirements for the design period, we have developed an econometric model, described and discussed in Chapter 6, which accommodates local conditions and provides the tool to optimize the S.J.R.W.S.S.

IV. POPULATION

1. General

The study region, common to most urbanized areas, is growing in population thus making the adequacy of any plan for waterworks largely dependent upon the rate of population growth. The big problem, accordingly, is to forecast population in the design year, namely twenty years hence.

A basic source of population figures can be found in the Statistics Canada census, carried out every five years. Projection of future population, from this basic source, can be done in several different methods. However, for this project it was decided that population figures and projections would be based upon data assembled for the St. John's Urban Regional Plan Study (S.J.U.R.P.S.) [2].

The Municipal Services Plan of this study recommended population distribution and projection, for that study, for the years 1971 to 1991, after careful consideration of four criteria, namely:

(i) Existing and required road capacity.
(ii) Infilling capacity.
(iii) School Capacity.
(iv) The necessity to provide for the needs of the local population that wishes to live there.

2. Municipal Services Plan Projections

The 1991 projections were based on an estimate of the 1971 population. The 1971 census population, published at a later date than this estimate, revealed, in some cases, considerable divergence from those actually estimated and included in the report. As a result, the S.J.U.R.P.S. postulates that since the 1971 regional population has been overestimated, the projected 1991 population may not in fact be reached until 1997. However, with their basic criteria and applying an appropriate statistical rationale, the S.J.U.R.P.S. has, in our opinion, made a sufficiently accurate prediction of that study region population to suit all purposes of this project.

Based on the above cited premise, we have accepted the S.J.U.R.W.S. projected horizon population of 215,000 (plus or minus 15,000) for the entire region for the proposed design period, i.e. for the year 1995. From the 1971 census population, and using the horizon population above, our future projections for five yearly incremental population concentrations for each community, were estimated.

3. Recommended Design Population

Table 4.1 summarizes our recommended population for the design period, identified by communities and reflecting a five-year growth period. An "S" shaped growth curve, known as the logistic curve, which describes a theory of P. F. Verhulst [8] in mathematical terms, was assumed to be representative of conditions in the study area. These future growth patterns were reviewed and evaluated by local officials responsible for planning the key communities.

As a consequence, their views on the anticipated future growth patterns have been given due consideration in the development strategy of the area as well as the planned dispersion of the future population.

Population growth for the region for the design period is forecast at an average (compound) rate of 2 percent per annum. Following the "S" shaped growth pattern indicated above, this growth will start at 3.2 percent per annum until the year 1980 (such a growth rate has recently been experienced in St. John's environs), thereafter tapering off to about 2 percent per annum between 1980 and 1985, and 1 percent per annum between 1990 and 1995, resulting in a total average growth of 2 percent per annum.

It should be noted that the S.J.U.R.P.S. [2] estimated the total population increase for the "Urban Areas" over the study period to be 54,000. These Urban Areas are identified as the North Expansion Zone, South Expansion Zone, Remainder of City, New Town Mount Pearl, Kilbride, and Shea Heights. However, the various municipal plans for the individual areas show a projected population increase of 68,500 which is 14,500 or 27 percent above their estimate. The S.J.U.R.P.S. further explored the various reasons for this inconsistency but nevertheless concluded that the population increase figure for the Urban Areas through the study period should be 54,000, distributed as follows:

TABLE 4.1

PROJECTIONS OF POPULATION GROWTH

Community	1971 Population: S.J.U.R.P.S.[2] Estimate	1971 Population: Census	Projected Population: 1975	1980	1985	1990	1995
1. Regional Centre							
City of St. John's	85,130		86,000	86,500	86,890	86,890	86,890
North Expansion Zone	-	88,102	2,000	6,000	8,000	10,000	12,000
South Expansion Zone	4,000		5,000	8,500	11,000	13,500	16,000
Wedgewood Park	380		500	600	700	800	910
Donovans	550		550	550	550	550	550
Shea Heights	1,440	4,967	1,630	2,050	2,300	2,600	2,850
Kilbride	1,870		2,120	3,500	5,000	5,000	5,000
Pippy Park	1,000		1,000	1,000	1,000	1,000	-
Mount Pearl	7,730	7,211	7,800	7,800	7,800	7,800	7,800
New Town	-		6,000	15,000	20,000	24,000	27,000
Total Category 1	102,100	100,280	112,600	131,500	143,240	152,140	159,000
2. Sub-Regional Centre							
Seal Cove	1,080	706	780	950	1,100	1,170	1,220
Gullies	1,160	728	920	1,350	1,450	1,520	1,570
Kelligrews	2,010	2,046	2,300	2,690	2,690	2,690	2,690
Foxtrap	1,630	908	1,100	1,250	1,600	2,050	2,110
Long Pond	2,050	1,758	1,900	2,200	2,500	2,850	3,320
Manuels	980	1,200	1,220	1,240	1,260	1,270	1,280
Chamberlains & Topsail	3,950	3,078	4,100	4,500	4,800	5,000	5,220
Total Categories 1 and 2	114,960	110,704	124,920	145,680	158,640	168,690	176,410

TABLE 4.1 CONTINUED

Community	1971 Population S.J.U.R.P.S. [2] Estimate	1971 Population Census	Projected Population 1975	1980	1985	1990	1995
3. Local Centres "A"							
Paradise	1,250	524	650	900	1,200	1,350	1,430
Topsail Road	1,525	N.A.	1,750	2,000	2,500	2,800	3,000
Torbay	2,010	2,090	2,250	2,300	2,350	2,400	2,440
Torbay Road	1,665	N.A.	2,665	4,100	5,300	5,500	5,550
Penetanguishene	1,080	1,080	1,200	1,350	1,500	1,600	1,670
Goulds	2,550	2,119	2,300	2,550	2,550	2,550	2,550
Petty Harbour	960	940	970	1,020	1,070	1,100	1,130
Total Categories 1, 2 and 3	126,000		136,705	159,900	175,110	185,990	194,180
4. Local Centres "B"							
St. Phillips	1,360	573	700	1,000	1,300	1,500	1,630
Hogans Pond	130	N.A.	130	130	130	130	130
Portugal Cove	790	1,411	1,400	1,400	1,400	1,400	1,330
Portugal Cove Road	2,990	N.A.	3,300	3,500	3,750	3,850	4,000
Thorburn Road	1,740	N.A.	2,000	2,400	2,600	2,800	3,000
Total Categories 1,2,3 and 4	133,010		144,235	168,330	184,290	195,670	204,270

N.A. Signifies Not Available.

Development Area	Population Increase
North Expansion Zone	12,000
South Expansion Zone	12,000
New Town (Mount Pearl)	24,000
Others	6,000
Total	54,000

The above population allocation has been adopted in principle for this project, with an adjustment having been made to the New Town population to reflect the most recent planning concepts.

There are certain development areas in the study region which are not specifically identified in the Population Distribution Table given in the S.J.U.R.P.S.[2] but are rather lumped together under the heading "Balance (e.g. Torbay Road, Topsail Road, etc.). Since such development areas do require proper identification as water demand centres, we have extracted the 1971 estimated population and the 1996 projected population for each development area from the S.J.U.R.P.S. "Balance" totals, and analysed it in accordance with the above cited growth pattern that we have assumed for the study region.

V. LAND USE

1. General

In assessing the land use of the region, having regard for present and future water use by locational and quantitive criteria, a review has been carried out of the S.J.U.R.P.S. [2], Plan 91 for the City of St. John's [8], and other plans for the development of the region including those of the Town of Mount Pearl [9], New Town [10], Kilbride [11], and the Goulds [12].

The Regional Plan [2] proposed acts as a basis for the establishment of a land policy for the region as an entity. It recognizes that more detailed plans for each particular area should be established, but that in general the individual detailed plans should conform to the policies proposed by the Regional Plan, otherwise regional disconformities will exist which will only serve to detract from the potential of the region as a whole.

The Regional Plan has designated certain areas within the Region for major land uses which can be categorized under two headings, namely:

(i) Urban Development
(ii) Non-Urban Development

It is proposed in this section to outline the major land uses which have been considered, for purposes of assessing the present and future water requirements of the Region, in terms of each of these two main categories. These land uses are shown on Drawing 4.1 [2].

2. Urban Development

The land uses within an Urban Development, which are considered to be the most important in terms of water requirements, are for residential, industrial, commercial and institutional purposes. These land uses have been reviewed for the major centres of existing and proposed urban developments within the region for:

(a) Present uses, and

(b) Future uses.

a. Urban Development - Present Uses

(i) City of St. John's

The focal point of the region, the city of St. John's is the regional centre of business and commerce.

Extracted from a 1969 land use survey of the City [8], Table 4.2, shows the population density (in terms of people per acre) of the city to be low medium, with an average of 16.68 persons per developed acre and 36.76 persons per residential acre. The overall ratio of units per acre is 7.2 Single family detached dwellings occupy 87 percent of the residential acreage and as a consequence shows a rather low ratio of 2.59 dwelling units per acre. The remaining 3 percent of the residential acreage, on the other hand, contains more than half the dwelling units of the city in the form of row housing, flats or apartments, and has led to the development of an average density of 31.5 dwelling units per acre. The clustered development that has resulted from this phenomenon of "building into a building" particularly because of the very steep and prevailing terrain, provides one of the ingredients which makes St. John's unique and distinctive amongst North American cities.

Over 30 percent of the land within the city is vacant, although this is not unusual in North American cities. However, it has been claimed that substantial portions of this land are not economically developable or available for urban land use purpose, having regard for present day land use costs and available funds.

The 1969 land use survey indicated that at this point in time St. John's could not be classified as an industrial city since only some 8 percent of the developed land is being used for this purpose. The study further states that St. John's is probably short of space for industrial development. This view is also postulated in the M. V. Jones "St. John's Metropolitan Area Industrial Study" of 1967.

The roads leading out of the City of St. John's to the regional communities display substantial ribbon development invariably on both sides of the roads, with residential and commercial uses intermixed and scattered.

LAND USE IN ST. JOHN'S MARCH 1969

ACREAGES AND PERCENTAGE OF TOTAL, WITH AND WITHOUT VACANT LAND, FOR THIRTY-ONE CATEGORIES OF LAND USE

TABLE 4.2

LAND USE ANALYSIS - ST. JOHN'S, MARCH 1969

	TOTAL ACRES	PERCENT	ACRE/100 POPULATION
RESIDENTIAL			
Single Family	1984.2	40.2	2.370
Multiple	164.9	3.3	.167
Apartments	116.4	2.4	.139
Mixed	12.3	.3	.015
	2277.8	46.2	2.691
INSTITUTIONS	248.5	5.0	.297
SCHOOLS	288.8	5.8	.345
COMMERCIAL	350.6	7.2	.419
INDUSTRIAL	398.6	8.0	.476
TRANSPORT & UTILITIES	98.8	2.0	.118
OPEN SPACE	1272.5	25.8	1.520
WATER	247.4		.295
VACANT	2260.2		2.699
TOTAL	7443.2	100.0%	8.886

16.68 persons per developed acre

36.76 persons per residential acre

NOTE: - Streets and roads are included as part of the adjacent land use.

- Population Base 86,452

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TABLE 4.3

SUMMARY OF PROPORTIONAL USE OF LAND, ST. JOHN'S, MARCH 1969

CATEGORIES *	ACRES	PERCENTAGE	PERCENT WITHOUT VACANT
RESIDENTIAL	2277.8	30.6	46.2
Single Family	1984.2	26.7	40.2
Multiple	164.9	2.2	3.3
Apartments	116.4	1.5	2.4
Mixed	12.3	.2	.3
INSTITUTIONS	248.5	3.3	5.0
Churches	29.1	.4	.6
Religious Orders	4.4	-	-
Hospitals	112.0	1.5	2.3
Cemetery	56.4	.8	1.2
Recreational	12.3	.2	.3
Public Service	29.1	.4	.6
Orphanages	3.0	-	-
Prisons	2.2	-	-
SCHOOLS	288.8	3.8	5.8
Elementary and Secondary	159.6	2.1	3.2
Higher Education	129.2	1.7	2.6
COMMERCIAL	350.6	4.7	7.2
Commercial (Retail)	248.8	3.3	5.1
Commercial with Residence	7.0	.1	.1
Offices	52.9	.7	1.1
Service Stations	41.9	.6	.9

TABLE 4.3 CONTINUED

SUMMARY OF PROPORTIONAL USE OF LAND, ST. JOHN'S, MARCH 1969

CATEGORIES *	ACRES	PERCENTAGE	PERCENT WITHOUT VACANT
INDUSTRIAL	398.6	5.3	8.0
Industrial	187.9	2.5	3.8
Storage and Warehousing	202.0	2.7	4.1
Quarries and Sandpits	3.5	-	-
Industry without Buildings	5.2	.1	.1
TRANSPORT AND UTILITIES	98.8	1.5	2.0
Railroad	40.6	.6	.8
Wharf	4.4	.1	.1
Parking	14.1	.2	.3
Bus Depot	5.3	.1	.1
Utilities	34.4	.5	.7
OPEN SPACE	1272.5	17.1	25.8
Parks	1135.8	15.3	23.0
Sports	17.2	.2	.4
Golf	119.5	1.6	2.4
WATER	247.4	3.3	-
VACANT	2260.2	30.4	-
TOTAL	7443.2	100.0	100.0

* Streets and roads are included as part of the adjacent land use.

The development generally extends only for one or two houses back from the highways, although in the more populated areas particularly in Conception Bay South, more concentrated nuclei do exist.

It should be borne in mind that until recent times, except for the City, the region did not have a zoning or land use plan. This of course has permitted the "unstructured" type of development now prevailing. It is recognized however, that future developments can now be controlled and our plans will recognize this condition.

(ii) <u>The Town of Mount Pearl</u>

From information provided by the Town of Mount Pearl, the total area of the Town is some 745 acres. An analysis of this area shows the distribution of land uses to be given in Table 4.4

The area shown in the above Table for Rural Reservation is planned for development as outlined in the "Development Scheme Town of Mount Pearl" [9].

Development of the Town as outlined in this Report will mean that the Town can be considered essentially as a residential community.

TABLE 4.4

LAND USE IN MOUNT PEARL

CATEGORIES *	ACREAGE	PERCENTAGES OF TOTAL
Residential	365	49.0
Rural Reservation	228	30.6
Conservation	75	10.1
Industrial	43	5.8
Commercial	20	2.7
Schools	8.3	1.1
Parks	3.4	0.4
Municipal	2.3	0.3
	745.0	100.0

* Streets and roads are included as part of the adjacent land uses.

(iii) New Town (Mount Pearl)

This recently constituted development area, located on the South West suburbs of St. John's, is a new development being undertaken under the auspices of the Newfoundland and Labrador Housing Corporation.

New Town will be a balanced community consisting of residential and industrial development. Five hundred residential lots are now under active development, an additional seven hundred are planned to be available by the end of 1974 and a further one thousand are in the planning stage and should be developed by 1975.

In the New Town's Industrial Park at Donovans, one hundred gross acres were developed in 1973 and another one hundred and ten will be available late in 1974.

(iv) Kilbride

The community of Kilbride, situated in the South West environs of the City of St. John's, is mainly a residential community, surrounded by fine agricultural land on one side and Petty Harbour Long Pond watershed on the other.

A development plan for the area has been prepared for the St. John's Metropolitan Area Board by the Provincial Planning Office.

The plan basically proposes that future residential development should be located in an area East of Old Bay Bulls Road with residual infilling of the existing ribbon development on Bay Bulls Road and old Petty Harbour Road. This would eventually lead to a concentrated residential community, with the necessary attendant local commercial and institutional facilities required by the residents.

(v) Shea Heights (formerly Blackhead Road)

The Blackhead sub-division plan was developed jointly by the Federal and Provincial Governments under the aegis of Blackhead Road Urban Renewal Scheme, and is now essentially completed.

The community is situated on high land to the South of and overlooking the City of St. John's, and is largely residential in nature although some local institutional needs are catered to within the development area.

(vi) Northern and Western Sectors

In these parts of the study area, the main centres of development are St. Phillips, Portugal Cove, Torbay and Penetanguishene.

St. Phillips and Portugal Cove were originally established as fishing settlements. Although some fishing is still carried on, they are now essentially residential dormitory type communities for the City of St. John's.

Due to constraints imposed by topography, Portugal Cove has been forced to develop essentially along, or close to, the roads leading into the community.

The St. Phillips area on the other hand, enjoys a more conducive landscape which allows houses to be built on the valley slopes and along the coastline, affording scenic views of Conception Bay. Within each of these two communities there are limited commercial land uses serving only minor local needs for the residents of the area.

The Town of Torbay is located on rolling cleared land, and is a widely dispersed community with areas of unused and arable land located between the scattered dwellings.

Being so close to the City of St. John's, Torbay acts as a residential dormitory suburb for St. John's with minor commercial facilities which serve only local day to day needs of the residents.

Penetanguishene, located on Portugal Cove Road, is essentially a residential community and is from a practical, but not necessarily political point of view, a suburb of St. John's.

(vii) Southern Shore Section

The main development nodes in this sector of the study are the Goulds and Petty Harbour. The prevailing landscape, comprising rolling and cleared land, rugged hills and ponds, is best suited for agrarian, conservation and recreational uses. The National Park located at Cape Spear has tourist potential, being identified as containing the most easterly point of land in North America.

The Goulds consists mainly of ribbon development centered on the intersection of Petty Harbour Road and Bay Bulls Road (Route 5.). Historically, the community probably grew up as an agricultural settlement, but it has recently become an attractive dormitory, conveniently located, to St. John's, some five miles to the north.

Petty Harbour is still an active fishing community. In common with quite a few of the outlying communities in the region, restricted site conditions are forcing new development to form ribbons for the community of Petty Harbour compared with its original compact development. Most of those residents not working locally in the fishing industry travel daily to and from work in St. John's. Most of the services are obtained from St. John's.

(viii) Conception Bay Sector

This sector can be considered to comprise two parts, in terms of Urban Development, i.e. Topsail to Seal Cove portion along Route 3 and the Local Improvement District of Paradise. The communities of Topsail, Chamberlains, Manuels, Long Pond, Foxtrap, Kelligrews, Gullies and Seal Cove are so homogeneously developed and geographically similar that it is difficult to identify the beginning and end of each community.

The above mentioned communities, with the exception of Foxtrap, combined in 1972 to form the Local Improvement District of Conception Bay South.

Although originally a series of small rural communities contributing to the agricultural needs of St. John's, the area presently can be neither classified as classically rural nor urban development, but now comprises a conglomerate of old and new houses, commercial and institutional uses (including schools, churches, restaurants, small stores and gas stations), pasture land, vacant land, and summer cottages. The Local Improvement District of Conception Bay South is now attacking the problem of how to best create a planned and orderly development in the area.

Most of the working inhabitants of the area travel daily to St. John's and other parts of the region (e.g. Holyrood) and in common with other regional "suburban" residents do the bulk of their shopping in St. John's, except for their day to day needs which they obtain locally.

In the Local Improvement District of Paradise most of the development is in the form of residential ribbon development extending along Topsail Road and a road adjacent to it, and is considered by the Urban Region Study to be a residential suburb of the capital city, St. John's.

b. <u>Urban Development - Future Uses (to 1996)</u>

The S.J.U.R.P.S. [8] proposed a series of regional, sub-regional and local centres for the region. These are as follows:

(i) <u>Regional Centre</u> - St. John's and expansion areas, Mount Pearl - New Town, Kilbride and Shea Heights (Blackhead Road).

(ii) <u>Sub-Regional Centre</u> - Conception Bay South (Gullies, Kelligrews, Foxtrap, Long Pond, Manuels, Chamberlains and Topsail).

(iii) <u>Local Centres</u> - Paradise, St. Phillips, Portugal Cove, Torbay, Penetanguishene, Petty Harbour and Goulds.

(i) The Regional Centre

In the growth concentration area of the Regional Centre, the plan [2] proposes that land uses shall include the full range of typical residential land use densities, regional local and highway commercial facilities, heavy and light industrial uses (where appropriate), all types of institutional and park facilities, a full range of institutional and cultural facilities, and the wide variety of other activities commonly associated with a major metropolitan centre.

Also proposed is the policy that the detailed development of the Regional Centre be carried out through the preparation and adoption of primary and secondary plans which shall conform to the basic structure established in the Regional Plan. Such local plans shall define detailed development policies for different types of land use and may define within the Regional Centre areas that should not be developed, for one reason or another.

On this basis the future land uses which have been considered in formulating water requirements in the Regional Centre are those contained in:-

(a) Plan 91 for the City of St. John's [8]

Comprising also plans for the residential and industrial expansion areas of the North Expansion Zone and South Expansion Zone of the City of St.

John's. The 1975 to 1995 population increase for each of these expansion zones is taken to be 12,000 and the industrial acreages which have been assumed to be developed by 1995 are 496 for each zone.

(b) Development Scheme - Town of Mount Pearl [9]

The future development of the Town of Mount Pearl in terms of infilling the existing structure and the development of the vacant lands within the Town Boundary by the Newfoundland and Labrador Housing Corporation.

(c) Mount Pearl-New Town Report [10]

The development proposals contained in this report for the establishment of a residential, fully serviced community and an Industrial Park at Donovans. The 1995 population allowed for is 27,000 and the 1995 developed industrial acreage allowed for is 992.

(d) Kilbride Development Plan [11]

The development proposals contained in this plan are for the consolidation and co-ordination of development in Kilbride. This allows for a 1995 population total of 5,000.

(e) The Blackhead Road Urban Renewal Scheme

The land use and population proposals are contained in the Urban Region Study and the Provincial

Planning Office Report for the development of this community (now substantially completed). The 1995 population allowed for in the community is 2,850.

(ii) The Sub-Regional Centre of Conception Bay South

The Regional Plan Policy for this centre is that the development of a full range of local services consistent with the anticipated size of the community should be encouraged. When municipal services are installed the permitted uses of the land shall include a wide range of residential land use densities compatible with prevailing socio-economic capabilities and needs, sub-regional, local and highway commercial facilities; parks and community facilities, elementary and high school facilities and other uses compatible with a "sub-regional" centre.

Once the maximum level of infilling in a specific part of the Conception Bay South area has been achieved, then the development of limited local additional areas may be permitted provided it is within the urban service area designated on the Regional Plan and provided such development is accompanied by the installation of appropriate urban services, such as sewers, water mains and roads. The Provincial Planning Office is preparing a detailed plan for this area but it is not available as yet. Based on their advice the extent and type of development used for the purposes of this study and for this particular area, are those contained in the Urban Region Study and Plan.

(iii) Local Centres

The Regional Plan proposes that for the local centres the pattern of future development should be as follows:

(a) Residential development to be established at low densities.

(b) Commercial development to be limited and local but is to include both retail and highway commercial functions.

(c) Industrial activities are to be limited and are to reflect local needs or specialties, e.g. the Fishing Industry.

One of the prime aims for the development of these local centres is to encourage infilling and consolidation of the presently semi-developed areas.

3. Non-Urban Development

The main land uses covered by this category are Agricultural Uses, Rural Uses, Restricted Development and Public Open Space.

The policies outlined in the Regional plan for control of the use of these land use categories are such that no municipal servicing is to be required. No consideration for servicing of these lands has, therefore, been included in this study.

VI. DEMAND FOR WATER - PAST

1. General

Water use records reflect the composite effect of all factors influencing customer usage at a given time. These records are universally adopted as the key component in predicting future water requirements.

From historic data of water use, average consumption(s) and trends can be interpolated. However, without examining them in depth, and the circumstances of supply and demand invalid predictions may arise. To avoid this pitfall, the nature of recent trends in water use as experienced in the last decade, were examined along the lines described in this section.

As can be seen from the previous sections of this Chapter, the study region comprises an urbanized area, principally the City of St. John's where extensive commercial properties and industry are mixed within residential areas, and suburban type developments such as Mount Pearl, Conception Bay South, the developing New Town, South and North Expansion Zones, where industrial parks have been designated for commercial and industrial properties, ranging from extensive to small scale depending on the character and/or growth predictions of the various areas. Accordingly, we find it appropriate that water usage and trends can be differentiated between urban St. John's (within its present built-up boundary) and the suburban areas of development within the study region. Analysis of water usage data pertaining to St. John's will be used to establish trends and projections for future requirements in the City, whereas those data pertaining to Mount Pearl will serve the same purposes for all other suburban developments. Industrial parks will be individually assigned a separate usage rate, having regard for size and potential land use applications appertaining.

2. Water Usage 1963-1973

Table 4.5 indicates monthly water usage for the years 1963-1973 as measured at the existing sources of supply, Windsor Lake and Petty Harbour Long Pond. The data included in this Table has been evaluated to determine two parameters, namely:

(a) Monthly fluctuations in water usage.
(b) Annual trends in water usage.

a. Monthly Fluctuations

Water usage changes with the seasons, normally with larger volumes of water being drawn during summer heat

TABLE 4.5

MONTHLY WATER USAGE 1963 - 1973
(IMPERIAL MILLION GALLONS)

	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973
January	304.6	330.9	357.7	340.9	347.3	313.6	325.3	355.2	386.2	380.8	464.1
February	300.9	312.2	338.2	319.4	328.2	302.1	288.9	303.8	331.0	395.5	416.9
March	347.8	336.1	368.2	346.2	356.4	321.0	319.0	333.0	367.4	431.2	463.0
April	304.8	312.7	354.6	323.8	339.4	217.8	298.4	320.8	324.4	397.6	425.0
May	296.0	307.0	352.7	334.2	334.6	277.6	306.0	320.2	336.8	384.8	423.6
June	278.4	288.3	340.4	312.1	330.6	270.5	312.8	318.6	336.4	416.4	423.6
July	295.2	309.8	355.2	307.4	328.0	281.9	339.0	355.0	355.8	415.8	457.5
August	293.0	311.0	352.4	337.0	320.0	327.9	332.8	358.9	364.6	405.0	451.1
September	283.7	292.8	318.4	308.2	301.4	326.2	310.4	342.6	339.6	391.3	437.4
October	295.8	316.4	297.6	309.3	302.3	348.1	341.4	359.4	355.6	391.7	473.3
November	292.6	325.4	279.9	312.7	291.2	328.6	332.8	330.6	338.0	391.1	430.0
December	307.2	345.4	318.9	340.6	317.4	321.8	331.6	348.8	355.2	436.2	434.4
Total Annual Use	3,600.0	3,788.0	4,034.2	3,891.8	3,896.8	3,637.0	3,838.4	4,046.9	4,191.0	4,837.4	5,299.8
Average Monthly Use	300.0	315.7	336.2	324.2	324.7	303.1	319.9	337.2	349.2	403.2	441.7
Maximum Month : Average Month	1.16:1	1.1:1	1.1:1	1.07:1	1.1:1	1.15:1	1.07:1	1.07:1	1.11:1	1.08:1	1.07:1

and drought. The monthly records of water usage do not reflect this common trend. In the eleven year test period, the maximum monthly water usage occured four times in March, four times in October, twice in December and once in January. Since, in general, the industrial and commercial use of water did not increase in the winter months, the above cited trends can be explained as follows:

(i) Water is run to waste in order to keep household services and pipes from freezing.

(ii) High leakage rate occasioned by joint contractions due to lower ambient and water temperature.

Normal fluctuation in water use, between average monthly annual rate and average maximum monthly rate, is generally considered to be in the range of 1.2 to 1.5. The comparable fluctuation from records presented in Table 4.5 shows the range in the City to be between 1.07 and 1.16. This low range is attributed to water usage practices experienced in the winter months, and consistently high leakage which has the effect of "dampening" peak factors.

For purposes of comparison, we reviewed the monthly fluctuations in water usage experienced in Halifax, N.S., where all customers are metered. From the 1971 annual report of the Public Service Commission of Halifax [4], for the areas it serves, it was found that water usage trends of a similar pattern to that experienced in St. John's occurred. The maximum monthly water usage occurred in winter (December and February); the ratio between the average maximum monthly water rate and the average monthly annual water rate was 1.1:1.

b. <u>Annual Trends in Water Usage</u>

Historical water usage indicates a continuing trend of increasing annual consumption. The degree of this increase in water usage depends on the principal factors mentioned in Section I of this Chapter as they relate to local conditions.

Analysis of the annual trends in water use for the test period reveals the following:

(i) The increase from the year 1963 to the year 1965 was consistent with an increase in population served.

(ii) A decline in consumption occurred in 1966. Accounting for population growth this decrease was quite moderate. Interesting to note is the general trend for decrease of water use in every month of that year. This can probably be attributed to reduction in waste, as can be implied from the records of repairs of water main breaks (or leaks) referred to in Appendix 1.

(iii) Essentially, water use was the same in 1967 as in 1966. Of interest here is the decrease in water use during the months of August through December. Two major factors affected this decrease, namely:

- Three consecutive months (June, July, August) of the most severe drought recorded since 1874. Consequently, a water use conservation policy was adopted and widely publicised.

- At the end of October 1967 a main pressure reducing valve was adjusted to maintain lower pressures, in the low lying service areas, in the supply system from Windsor Lake.

(iv) A moderate decrease in consumption in 1968. This could be attributed to the effects of continuing to repair water main breaks or leaks (Appendix I), and lower pressures in parts of the supply system.

(v) An increase occurred in the period between 1969 and 1971. Accounting for population growth, this increase would be considered as reasonably gradual.

(vi) A substantial increase occurred in 1972. The records show that there is a continuous increase occurring in 1973. These sudden increases have occurred after an apparent period of stabilization in water usage trends, established during the years 1969-1971.

As can be seen from Table 4.5 a substantial increase in water usage started in February 1972.

and continued consistently for the remainder of 1972 and throughout 1973. Meteorological records show the prevalence of abnormally cold temperatures which averaged below normal figures, 4.1° in January, 6.2° in February. Record low temperatures occurred on January 7, 3°; January 28, 1°; February 2, -5°; February 23, -8°. In summary, there was a record of 1250 degree days of temperature below 32° in 1971-2 and 1123 degree days in 1972-3. These abnormally cold conditions would cause a frost depth penetration [3], varying from about 48 inches in asphalt paved surfaces to about 60 inches in undisturbed snow covered areas. In our judgement these conditions caused an excessive underground leakage that apparently still continues.

c. <u>Discussion and Recommendations</u>

The above analysis has examined the two major parameters that should be used in the projection of future water requirements, and in summary we conclude:

(i) Water usage can be reduced with reasonable control measures. Control of pressure and systematic repairs of water main breaks are common measures which were used from the years 1968 through 1971. (Repairing of water main breaks is part of the regular maintenance program). Accordingly, we recommend that the trends established during those years serve as the basis for projecting future water requirements. Furthermore, in order to conclusively determine the cause for the sudden high increase in water usage in 1972 - 1973, and to facilitate the restoration of the supply system to its previous condition, a water waste survey is being undertaken, a course of action certainly warranted, in our opinion.

(ii) The average maximum monthly water usage factor for urban St. John's is within the minimum range as experienced and reported in waterworks literature. This same pattern has been reported for Halifax and its environs. It is accordingly recommended that a factor of 1.2 be used to correlate the urban St. John's average maximum monthly water usage to the average monthly annual usage.

3. Water Use by Customer Class

In order to provide a basis for evaluation and projection of water use, it has become customary to express usage on a per capita basis. It should be realized, however, that this convenient approach is prone to misapplication of data because its validity hinges on the accuracy of population estimates, and is consequently not a sensitive index.

The rationale developed below was determined to predict a more sensitive means of assessing user demands by various types.

An analysis of the water usage records presented in Table 4.5 for the average daily annual rate per capita, gives the results shown in Table 4.6. The population for this analysis was taken from the census for the years 1961, 1966 and 1971. Population estimates for the intermediate years were based on the assumption that the growth occurred at a compound rate.

Table 4.6 was further analysed for the three representative years, 1969-71 to differentiate between the per capita water usage in urban St. John's and the suburban developments supplied from the system. The results are presented in Table 4.7.

The per capita water usage as given in Tables 4.6 and 4.7 is as measured at the source of supply, and it includes usage by residential customers, industry and large commercial properties, public buildings and waste. It is, therefore, warranted to analyse each of these classes separately, and accordingly project future water requirements as a total of these individual component users. We have consequently considered the following classes of customers:

(a) Residential
(b) Industry and Commercial
(c) Public
(d) Waste
(e) Unaccounted for

TABLE 4.6

AVERAGE ANNUAL WATER USAGE

GALLONS PER CAPITA PER DAY

	1963	1964	1965	1966[1]	1967	1968	1969	1970	1971[1]	1972	1973
1. Average Daily Annual use MGD	9.85	10.37	11.04	10.65	10.67	9.96	10.51	11.08	11.47	13.25	15.0
2. Population											
a. St. John's	69,670	72,925	76,225	79,884	81,460	83,080	84,720	86,400	88,102	89,840	
b. Mount Pearl	3,350	3,675	4,030	4,428	4,880	5,380	5,930	6,540	7,211	7,800	
c. Shea Heights								1,000[1]	2,000	2,000	
d. Wedgewood Park								200[2]	400	400	
e. Total	73,020	76,600	80,255	84,312	86,340	88,460	90,650	94,140	97,713	100,040	101,500
3. Average Daily Annual use GPCD	135	135	137.5	126	123.5	112.5	116	117.5	117.5	132.5	147.5

(1) Census Year.

(2) These communities were connected to St. John's water supply system in the Spring of 1970. It was therefore assumed that half the population used water for the year of 1970.

TABLE 4.7

AVERAGE ANNUAL WATER USAGE
FOR URBAN & SUBURBAN AREAS

	1969		1970		1971	
	MGD	GPCD	MGD	GPCD	MGD	GPCD
Mount Pearl & Suburbia	0.4	67 (1)	0.52	67 (1)	0.65	67 (1)
Urban St. John's	10.11	119	10.56	122	10.82	123

(1) See Section 3 (d) of this Chapter.

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a. <u>Residential Customers</u>

As mentioned in Chapter 3 of the report, some residential customers have service water meters. These meters, installed during the years 1967-69, have not yet, however, been officially read. We have selected at random a sample of 75 metered households in urban St. John's, reasonably distributed amongst different income levels, and surveyed them to carry out a statistical analysis of residential water use. A detailed presentation of the statistical analysis is contained in Appendix 2. The findings of this analysis indicated a mean residential water use of 43.6 GPCD, and the 90 percent confidence interval to be between 39.4 and 47.8 GPCD. It should be realized, however, that these consumption figures relate to a period of time extending from the date the meters were installed to May 1973, the time of their survey. For practical purposes, it can be assumed that current residential water use is in the order of magnitude of 45 GPCD (CF 43.6) to reflect the general tendency to increase "consumption" with higher economical conditions. This is the figure that will be used as the basis for the projection of future water use by customers in urban St. John's.

A random sample of 50 metered households in Mount Pearl whose recorded water use extended over the years 1968-69 was also analysed statistically. The findings show a mean residential consumption of 34 GPCD. The fact that this figure is lower than the City's figure can be explained partly due to Mount Pearl being a dormitory of St. John's.

It is of interest to note that the average residential use of metered water at Halifax amounted to 40 GPCD in 1971 [4].

b. Industrial and Commercial Customers

Customers under this category are basically as defined in the "Existing Land Use Plan (1972) prepared by the City Planning Office.

All these customers are metered, including, in addition, the harbour and airport.

The 1951 Pitometer survey shows the water usage of these customers to be equivalent to an average of 40 GPCD. Comparable consumption figures determined in the 1966 Pitometer survey and from the City water tariff accounts for 1972 was 28 GPCD and 27 GPCD, respectively. The higher water usage per capita in 1951 could be attributed to a higher ratio between industry and commercial properties to population, and also due to a relatively low rate of charges in 1951, to these customers. The order of magnitude of water usage in 1966-1972 has been adopted as the basis for projection of use by this customer class.

In Mount Pearl, the water usage for the metered industrial and commercial customers in 1972 was equivalent to 16 GPCD. We propose that this value be used with discretion for this customer class in all other suburban developments in the region.

Agricultural water usage, as experienced at Conception Bay South, will be considered as part of the water requirements by this customer class.

Industrial parks will be considered as independent water users. Murray V. Jones and Associates Limited [5] recommended that a figure of 2,500 gallons per acre per day (GPAD) be used for the calculation of Donovan's Park Industrial water needs.

Based on water usage figures by industry in other areas of the City, we find the above value of 2,500 GPAD as valid for industrial parks in the study area at large.

c. Public Customers

Public customers, which include such facilities as schools, hospitals, and institutions such as religious, orphanages, old people's homes, prisons, fire protection street washing and similar public services, are not metered, and their water use has to be estimated.

A normal range of water usage by public customers is considered to be between 5 and 20 GPCD. It can be assumed that current consumption by this customer class is in the order of magnitude of 10 GPCD and 5 GPCD for urban St. John's and the suburban developments, respectively.

d. Waste

Waste is that portion of the total output of water which is not used by the consumers due to leakage from the supply system.

The 1951 Pitometer survey indicates waste, as defined above, to account for 13.9 percent of the total water usage. The comparable waste found in the 1966 Pitometer survey amounted to 19.5 percent. These rates of waste are equivalent to a consumption in the range of 20 to 25 GPCD. As discussed previously in this chapter, it is possible that the waste has increased beyond the above cited range due to the deep frost penetration experienced in 1972. The rate of waste in Mount Pearl is estimated at 18 percent, which is equivalent to a consumption of 12 GPCD, based on the following premises:

- Total water usage (measured)	67 GPCD
- Residential use (a sample of metered households)	34 GPCD
- Industrial and Commercial use (measured)	16 GPCD
- Public use (assumed)	5 GPCD
- Waste (balance)	12 GPCD

e. Unaccounted for

Water measured at the source of supply but unaccounted for by the previous four customer classes, is covered by this category.

The Pitometer surveys of 1951 and 1966 for urban St. John's show that water unaccounted for its use, amounted to about 25 percent of the total output. This high percentage can be attributed to house waste due to faulty fixtures, under-registration of meters, assumptions and residual error. This phenomenon is not noticeable in Mount Pearl.

It is felt that the proportion of water unaccounted for in relation to the total output should be small. Based on the analysis presented in this section, and taking current demand as an average annual water use of 120 GPCD (see Table 4.7), the breakdown into the five customer classes will be as shown in Table 4.8.

4. <u>Water Use - Maximum Factors</u>

a. <u>General and Present Conditions</u>

In order to properly dimension the constituent structures of a water supply system, such as treatment facilities, pump stations, trunk mains, service reservoirs, and distributing pipes, normal variations in water usage must be known. The average monthly annual rate discussed in the previous sections while useful, does not provide the required design parameters. Maximum water use factors related to the average monthly annual rate have to be determined.

Figure 4.1 is a general presentation of the constituent structures in a water supply system and the capacity for which each of these consituents should be sized.

TABLE 4.8

WATER USE PER CUSTOMER CLASS - URBAN AREAS

Customer Class		Percent of Total	GPCD
1.	Residential	37.5	45
2.	Industry and Commercial	22.5	27
3.	Public	10.0	12
4.	Waste	20.8	25
5.	Unaccounted for	9.2	11
6.	Total	100.0%	120 GPCD

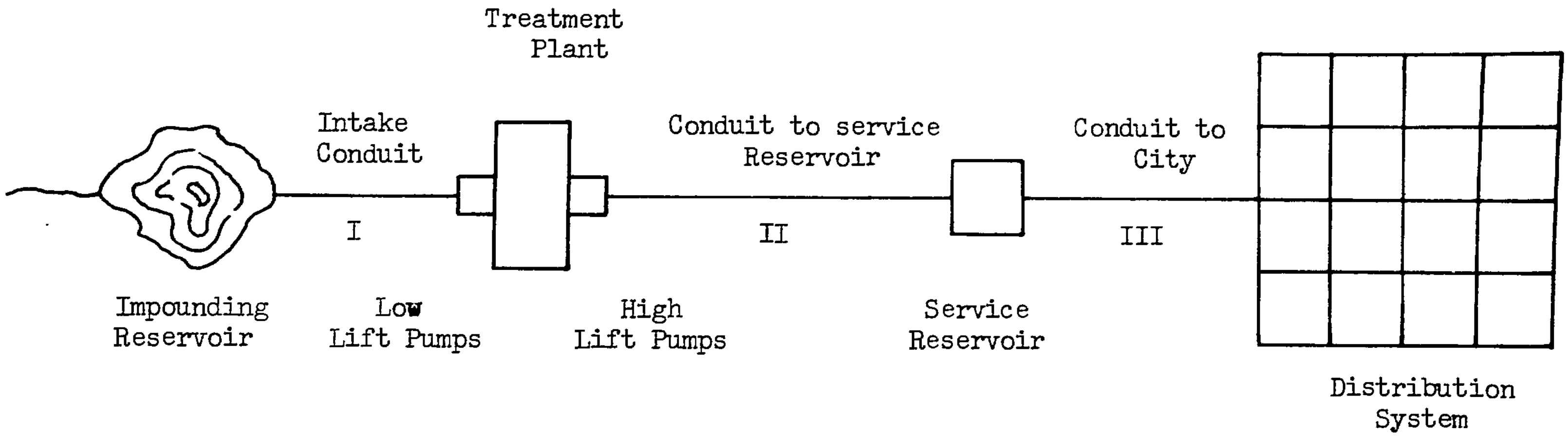

FIGURE 4.1

CAPACITY REQUIREMENTS FOR CONSTITUENTS OF TYPICAL WATER SUPPLY SYSTEM

STRUCTURE	REQUIRED CAPACITY
1. Impounding Reservoir	Maximum day at severe drought
2. Conduit I	Maximum day
3. Conduit II	Maximum day
4. Conduit III	Maximum day and fire
5. Low-lift pumps	Maximum day plus stand by
6. High-lift pumps	Maximum day plus stand by
7. Treatment Plant	Maximum day
8. Service Reservoir	Maximum hourly fluctuations + fire + reserve
9. Distribution System	Maximum day and fire

Accordingly, it can be seen that the factors for maximum daily and maximum hourly consumptions, as well as the fire demand, must be established.

Analysis of the water supply records for the test period gives a range of values for each maximum factor, the upper limit of this range being as follows:

- Maximum daily rate: Average 1.26:1
- Maximum hourly rate: Average 1.53:1

b. Discussion and Recommendations

The maximum water use factors derived for the test period relate predominantly to urban St. John's. The equivalent maximum factors for the suburban developments could be expected to be higher due to the different nature of these areas. Also, we feel that some flexibility for future fluctuations should be included in these factors. Accordingly, it is recommended that for future purposes maximum water use factors as presented in Table 4.9 be used.

TABLE 4.9

WATER USE - MAXIMUM FACTORS

Factor		Urban St. John's	Suburban Developments
Maximum Day :	Ave.	1.3:1	1.5:1
Maximum Hour:	Ave.	1.7:1	2.0:1

Water use data for industrial customers show, at present, a maximum factor of 1.5. We recommend that this factor be used for industrial parks. Maximum hourly factors would have to be established separately for each type of industry once its nature is known.

As can be seen from Figure 4.1, service reservoirs are assumed to take care of maximum hourly fluctuations in water use, fire demand, and if warranted, emergency reserve.

Requirements for the supply and storage of water to combat fire are discussed in the following section. To allow for the fluctuations in maximum hourly demand, it is ordinarily required to provide storage of 10 to 30 percent of the maximum daily water use. Figure 4.2 is a typical hydrograph of the system showing the variation in water use throughout a maximum day. The shaded area which represents the minimum storage capacity required to balance hourly maximum demands is equal to 10 percent of the maximum daily demand at a supply rate equal to this maximum daily demand. For

TABLE 4.10

FIRE FLOW REQUIREMENTS BY C.U.A.

Fire Area		Description of Building	Fire Flow Requirement
Residential	(1)	Detached dwellings, one and two storeys	400 GPM
	(2)	Old large dwellings to 3 storyes or duplexes with exposing rear sheds, etc.	1,000 GPM
	(3)	Three storey frame, 6 to 8 suites, unexposed	1,000 GPM
	(4)	Three storey frame, 10,000 sq.ft. unexposed	3,500 GPM
Institutional (schools, etc.)	(1)	Old ordinary brick, 3 storey, unexposed 25,000 sq.ft.	3,500 GPM
	(2)	Fire resistive, 2 storey, unexposed, 25,000 sq.ft.	2,500 GPM
Industrial	(1)	Typical industrial park, moderate fire load, non-combustible, 40,000 sq.ft.	3,000 GPM
	(2)	Frame warehouse, moderate fireload, 20,000 sq.ft.	3,500 GPM
Commercial	(1)	Old business district blocks, brick 2-4 storey	3,000 - 5,000 GPM
	(2)	Large mall shopping centre in one area of 150,000 sq.ft. non-combustible	5,000 GPM
	(3)	Small open centre or part of large one, 50,000 sq.ft. non-combustible	2,500 GPM
	(4)	High-rise office building, 10,000 sq.ft. floors cut off.	2,000 GPM

design purposes, we recommend that a storage capacity equal to 10 percent of the maximum daily water use, including requirements for industrial parks where applicable, be provided to balance the hourly maximum demands.

5. Fire Demand

a. General

It is generally accepted that water distribution systems be designed to provide sufficient water for the maximum daily demand plus the requirements to fight fires.

Fire flow requirements used to be determined as a function of the population served by application of the following equation:

$$G = 1{,}020\ P^{\frac{1}{2}}\ (1 - 0.01P^{\frac{1}{2}})$$

where: G = required fire flow in USGPM

P = population in thousands

It became apparent that with the new patterns of development, this equation was no longer representative. Population is not necessarily the guide for use in estimating fire flow requirements. Instead, the Insurance Services Office (I.S.O.) has developed and issued a "Guide for Determination of Required Fire Flow" based on the most hazardous building, its floor area and material of construction. Accordingly,

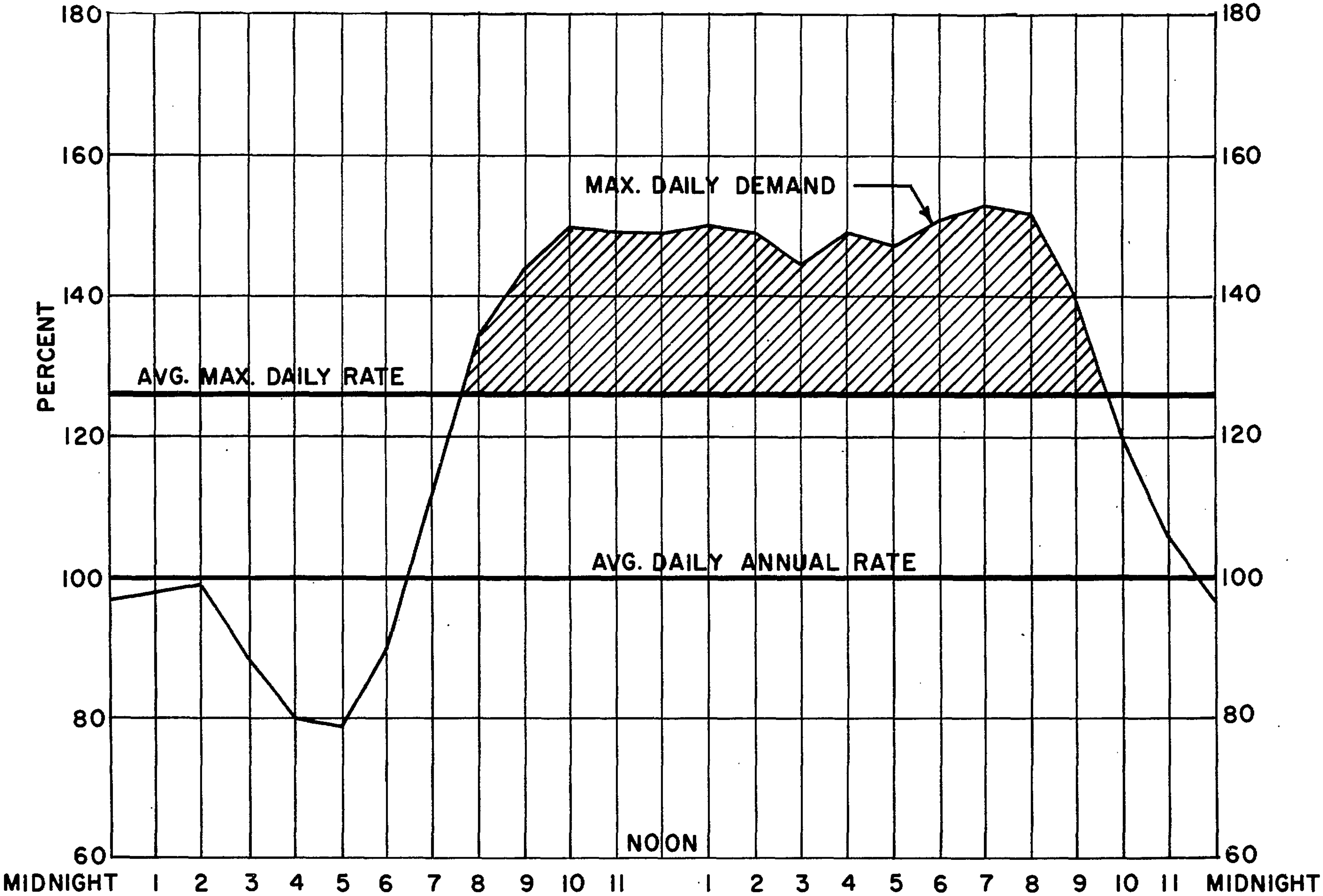

FIG. 4.2 HOURLY WATER USE AS PERCENT OF AVERAGE DAILY ANNUAL RATE.

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estimates of the fire flow required for a given fire area are now determined by the formula:

$$F = 18C\ (A)^{0.5}$$

where: F = the required fire flow in USGPM

C = co-efficient related to the type of construction

C = 1.5 for wood frame construction
= 1.0 for ordinary construction
= 0.8 for non-combustible construction
= 0.6 for fire-resistive construction

A = the total floor area (including all storeys, but excluding basements) in the building being considered.

A complete guide on "How the I.S.O. Estimates Fire Flow Requirements", as published in the May 1973 issue of the Journal of the American Water Works Association, is contained in Appendix III.

The Canadian Underwriters Association (C.U.A.) who have adopted the I.S.O. method as described above and in Appendix III, calculated fire flow requirements, for some specific situations. These are given in Table 4.10, and will be used to determine fire demands for this project.

Fire flow requirements are presented in Table 4.10 for durations of 2 hours for flows up to 1,000 GPM, 3 hours for flows between 1,000 and 3,000 GPM, and 4 hours for flows between 3,000 and 5,000 GPM, in pumped supply systems. This water will be stored in service reservoirs.

VII. DEMAND FOR WATER - FUTURE

Projection of requirements for the design period will be based on the analysis and trends of water use relative to customer classes as discussed in Section VI, and land use and population as presented in Sections IV and V, respectively.

1. Residential Customers

The largest single consumer of water is the residential class. In projecting this class's future requirements for water, the following principal factors that influence increase and decrease in water use have been considered.

a. Factors influencing increase in water use

Economic conditions within a community have a significant effect on residential customer water use trends. Compared with other Metropolitan areas, the study region has not been known to have a favourable economic climate (see Chapter 2). Recent development trends, however, indicate that efforts are being made to accelerate the economic conditions of the region. Experience has shown that water use will follow the general trend in the economic situation of a region.

Supply limitations due to political boundaries, quality and adequacy of the water supply sources, suitability of the distribution system, all have a restricting effect on water use. A regional water supply system will tend to remove these restrictions.

b. <u>Factors influencing decrease in water use</u>

The most significant factor to influence decrease in water use is user charges. However, such decrease in use will depend on the current level of water usage, the level of charges and the degree of increase in rates.

Changes in residential living patterns such as a marked shift from single family dwellings to multi-family units, and changes in established area occupancy such as urban redevelopment, are other factors that may cause a decrease in water use.

After assessing the above cited factors, we recommend that for design purposes of a regional system an increase in residential water use during the design period be allowed. This increase, we recommend, will be as follows:

(i) Urban St. John's, a compound increase of about 1.5 percent per year. Under this assumption the residential water use by the end of the design period will amount to 60 GPCD compared to the present 45 GPCD (of residential customers).

(ii) All suburban developments, a compound increase of about 2 percent per year. Under this premise the residential water use at the end of the design period will be 54 GPCD compared to the current 34 GPCD as presently experienced in the Town of Mount Pearl.

It would be of interest to note here that in the 1971 annual report of the Public Service Commission of Halifax [4], the general manager states "...it must be noted that the average daily water demand is increasing".

2. Industrial and Commercial Customers

This customer class in urban St. John's has been using water equivalent to 27 GPCD. In projecting its requirements for the design period we have considered the possibilities that certain industries will relocate to one or other of the new industrial parks, that the anticipated economic situation will result in extensive development of commercial properties, such as hotels and the Atlantic Place Development, that the managing organization for the regional system will effectively service the water meters. Accordingly, we recommend a gradual increase in water use by this customer class to 35 GPCD.

Industrial and commercial customers in the Town of Mount Pearl have been using water equivalent to 16 GPCD. Since Mount Pearl has basically reached saturation in its development, we recommend that this value of 16 GPCD be used through the design period for Mount Pearl and all other similar suburban development areas.

This same value of 16 GPCD will be applied to suburban areas using water for agricultural needs in lieu of industry.

Industrial parks will be allotted 2,500 gallons of water per acre per day.

3. Public Customers

We have assumed that basically the ratio of public service to population as it exists presently will be maintained through the design period. Accordingly, the water use by this customer class will remain in the order of magnitude of 10 GPCD and 5 GPCD for urban St. John's and the suburban developments, respectively.

4. Waste

All water supply systems experience waste through underground leakage. The aggregate amount of this waste may be from 10 to 25 percent, or more, of the total output. A system experiencing leakage of about 10 percent of its output would be considered a well managed one.

Experience within St. John's supply system as determined from the 1966 Pitometer survey shows that the bulk of the leaks (about 70 percent) occurs at pipe joints, and that six leaky pipe joints may account for about 40 percent of the total waste. A summary of this data is contained in Appendix IV. Based on this information, and under the supposition that a regional managing organization will be equipped and staffed to adequately repair pipe leaks, and accounting for a longer system of pipe lines but with controlled pressures, we have assumed that waste could be maintained at no more than 15 percent of the total output. To provide flexibility in design, we recommend that about 17 percent of the total water output be allotted to this "use".

5. Unaccounted for

A regional managing organization equipped to service meters regularly; to enforce by-laws and regulations which ensure good workmanship and the provision of high quality service pipes, fittings and fixtures; to inspect such installations and to survey all premises supplied; to monitor the supply systems; to regulate and control unauthorized use of water, will decrease the volume of water unaccounted for to an insignificant portion of the total output, say 3 percent.

For suburban developments no allowance is made for water unaccounted for in use.

6. Fire Demand

In determining the fire demand for the study region we have distinguished between the following situations:

a. Fire flow requirements for suburban developments consisting of residential areas such as in the Conception Bay South area. A flow rate of 1,000 GPM would be adequate for fire fighting purposes, provided that a duration of two hours can be sustained at this flow. Accordingly a storage capacity for fire protection of 120,000 gallons will be provided in service reservoirs for suburban residential developments.

b. For suburban developments, such as Mount Pearl and the New Town fire flows will be provided to accommodate the requirements of large mall shopping centres of non-combustible construction material. Table 4.10 indicates that the ordinary fire demand for such a complex is 5,000 GPM. However, experience has shown that these types of malls have a light fire loading for which 25 percent deduction in fire demand is warranted. Also an additional 25 percent can be deducted from the original demand if the mall is equipped with an automatic sprinkler system which usually is the case. Accordingly, a fire flow of 2,500 GPM for a 3 hour duration, will be provided for suburban developments with large mall shopping centres. Service reservoirs planned for these developments will contain a storage capacity of 450,000 gallons allotted for fire protection.

c. Industrial Parks will be considered as containing large complexes of non-combustible construction, a moderate fire load, and an automatic sprinkler system. Accordingly, a fire flow of 3,500 GPM for a 4 hour duration will be provided to these areas. The same fire flow and duration time will be allotted to urban St. John's. Where service reservoirs are included in the supply system, the storage capacity for fire protection will be 840,000 gallons.

7. Summary

A summary of the basic design parameters as analyzed and recommended in the previous sections of this chapter is presented herein.

a. Rate of Water Use - 1995

TABLE 4. 11

Customer Class	Urban St. John's	Suburban Developments	Industrial Parks
1. Residential	60 GPCD	54 GPCD	-
2. Industrial & Commercial	35 GPCD	16 GPCD	2,500 GPAD
3. Public	12 GPCD	5 GPCD	-
4. Waste	24 GPCD	15 GPCD	-
5. Unaccounted for	4 GPCD	-	-
6. Total	135 GPCD	90 GPCD	2,500 GPAD
7. Fire demand	3,500 GPM	1,000 GPM to 2,500 GPM	3,500 GPM

An analysis of water use by customer class as percent of the total output is contained in Appendix V.

b. Average Daily Annual Water Use

The summary of forecasts for average water use in the years 1975-1995 is presented in Tables 4.12 and 4.13 in the following two forms:

(i) Relative to categories of regional, sub-regional, and local community centres (as proposed in the S.J.U.R.P.S. and referred to previously in this chapter).

(ii) Relative to categories of need, such as existing serviced areas, immediate development areas, health areas, and other areas. It was felt that a division (and presentation) of these categories will assist the decision making authorities in assigning priorities to the staging of waterworks construction. Further discussions along this concept are presented in Chapters 5 and 6 of the report.

Areas that require early supply of water to accommodate immediate growth and expansion have been included under the "immediate development" category.

Areas that need improved supply of water at an early stage, due to deteriorating sanitary conditions, have been identified as "health areas".

Areas, except for the existing serviced areas, whose present supply of water is considered to be adequate, but which would require additional or improved supply within the 20 year design period, due to social and orderly developments over the years, have been included under the "other areas" category.

Projection of the average daily annual use of water is also presented in a diagramatic form in Figure 4.3.

It should be noted here that the projected water use in the above two tables is based on the rate of water consumption by customer classes as discussed and presented previously in the chapter. However, excessive withdrawal of water was experienced in the years 1972-1973, which exceeds our forecasts by about 2.5-3.0 MGD. This excess quantity of water has been attributed primarily to waste, and as previously noted, we have assumed that the water waste survey which is being undertaken will facilitate the restoration of the supply system to its previous conditions.

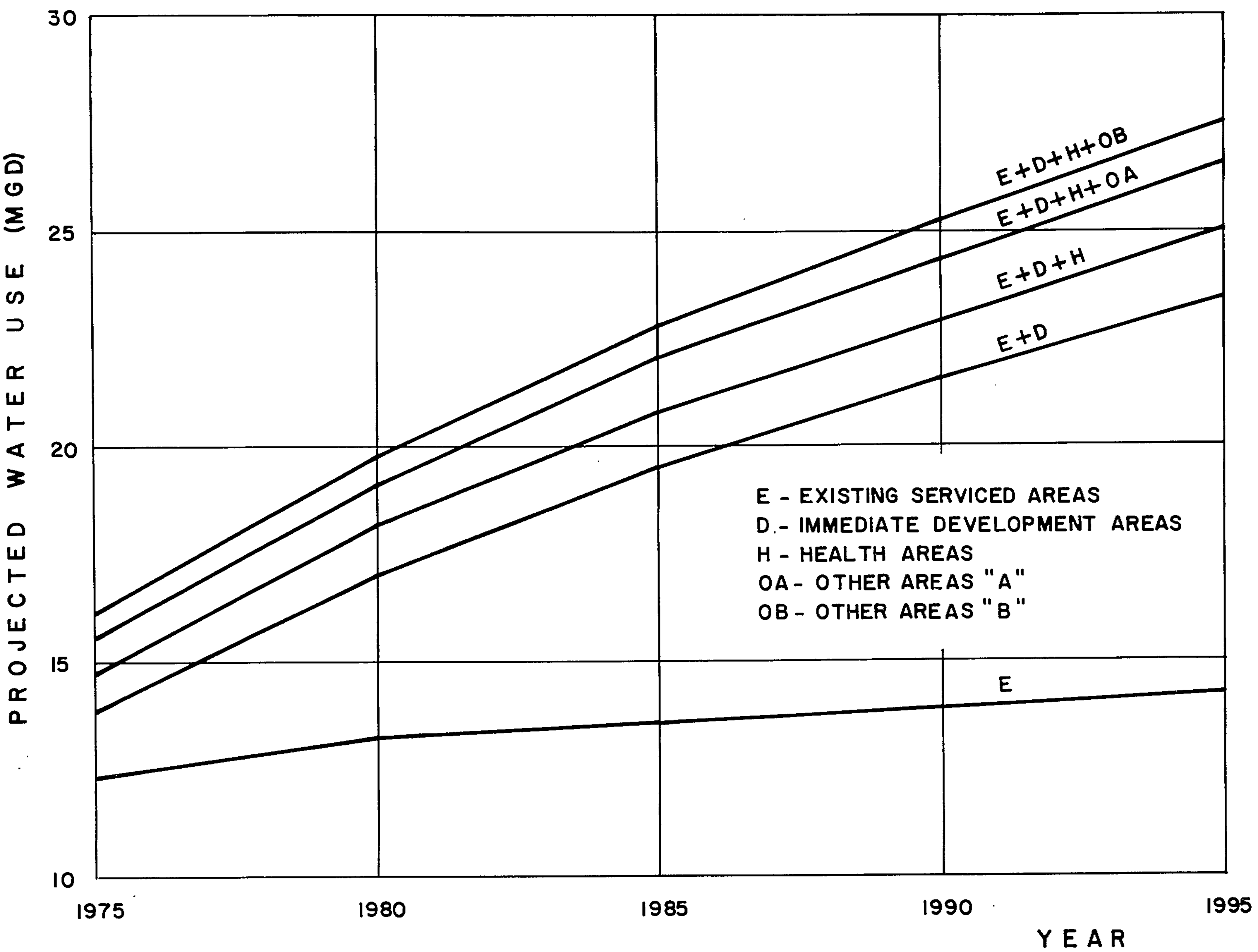

FIGURE 4·3

AVERAGE DAILY ANNUAL WATER USE

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TABLE 4.12

AVERAGE DAILY ANNUAL WATER USE
(Relative to Community Centres)

Community	Projected Water Use in MGD 1975	1980	1985	1990	1995
1. Regional Centre					
Urban St. John's	11.00	11.50	11.85	12.10	12.30
North Expansion Zone	0.14	0.45	0.64	0.85	1.08
South Expansion Zone	0.07	0.30	0.56	0.81	1.08
Wedgewood Park	0.04	0.05	0.06	0.07	0.08
Donovans	0.04	0.04	0.04	0.05	0.05
Shea Heights	0.12	0.15	0.18	0.22	0.26
Kilbride	0.15	0.26	0.40	0.43	0.45
Mount Pearl	0.56	0.59	0.62	0.66	0.70
New Town	0.44	1.12	1.60	2.04	2.43
Total Category 1	12.56	14.46	15.95	17.23	18.43
2. Sub-Regional Centre					
Seal Cove	0.06	0.07	0.09	0.10	0.11
Gullies	0.07	0.10	0.12	0.13	0.14
Kelligrews	0.16	0.20	0.22	0.23	0.24
Foxtrap	0.08	0.09	0.13	0.17	0.19
Long Pond	0.14	0.16	0.20	0.24	0.30
Manuels	0.09	0.09	0.10	0.11	0.12
Chamberlains & Topsail	0.30	0.34	0.38	0.42	0.47
Total Categories 1 and 2	13.46	15.51	17.19	18.63	20.00

TABLE 4.12 CONTINUED

AVERAGE DAILY ANNUAL WATER USE

(Relative to Community Centres)

Community	Projected Water Use in MGD				
	1975	1980	1985	1990	1995
3. Local Centres "A"					
Paradise	0.05	0.07	0.10	0.11	0.13
Topsail Road	0.13	0.15	0.20	0.24	0.27
Torbay	0.16	0.17	0.19	0.20	0.22
Torbay Road	0.19	0.31	0.42	0.47	0.50
Pentanguishene	0.09	0.10	0.12	0.14	0.15
Goulds	0.16	0.19	0.20	0.22	0.23
Petty Harbour	0.07	0.08	0.09	0.09	0.10
Total Categories 1, 2 and 3	14.31	16.58	18.51	20.10	21.60
4. Local Centres "B"					
St. Phillips	0.05	0.08	0.10	0.13	0.15
Hogans Pond	0.01	0.01	0.01	0.01	0.01
Portugal Cove	0.10	0.11	0.11	0.12	0.12
Portugal Cove Road	0.24	0.26	0.30	0.33	0.36
Thorburn Road	0.14	0.18	0.21	0.24	0.27
Total Categories 1,2,3 and 4	14.85	17.22	19.24	20.93	22.51
5. Industrial Parks					
Donovan's (992 acreage)	0.60	1.40	2.00	2.25	2.48
Kenmount (496 acreage)	0.40	0.60	0.80	1.00	1.24
White Hill (496 acreage)	0.25	0.50	0.20	1.00	1.24
Total Categories 1 to 5	16.10	19.72	22.74	25.18	27.47
Total Categories 1,2,3 and 5	15.56	19.08	22.01	24.35	26.56

TABLE 4.13

AVERAGE DAILY ANNUAL WATER USE

(relative to categories of need)

Community	Projected Water Use in MGD 1975	1980	1985	1990	1995
1. Existing Serviced Areas					
Urban St. John's	11.00	11.50	11.85	12.10	12.30
Mount Pearl	0.56	0.59	0.62	0.66	0.70
Kilbride	0.15	0.26	0.26	0.26	0.26
Shea Heights	0.12	0.15	0.18	0.22	0.26
Wedgewood Park	0.04	0.05	0.06	0.07	0.08
Kenmount Ind. Pk.	0.40	0.60	0.60	0.60	0.60
Total Category 1	12.27	13.15	13.57	13.91	14.20
2. Immediate Develop't Areas					
New Town	0.44	1.12	1.60	2.04	2.43
South Expansion Zone	0.07	0.30	0.56	0.81	1.08
North Expansion Zone	0.14	0.45	0.64	0.85	1.08
Torbay Road	0.07	0.15	0.24	0.27	0.29
Donovan's Ind. Park	0.60	1.40	2.00	2.25	2.48
Kenmount Ind. Park	-	-	0.20	0.40	0.64
White Hills Ind. Park	0.25	0.50	0.70	1.00	1.24
Total Categories 1 and 2	13.84	17.07	19.51	21.53	23.44

Community	Projected Water Use in MGD				
	1975	1980	1985	1990	1995
3. Health Areas					
Seal Cove	0.06	0.07	0.09	0.10	0.11
Gullies	0.07	0.10	0.12	0.13	0.14
Kelligrews	0.16	0.20	0.22	0.23	0.24
Foxtrap	0.08	0.09	0.13	0.17	0.19
Long Pond	0.14	0.16	0.20	0.24	0.30
Manuels	0.09	0.09	0.10	0.11	0.12
Chamberlains & Topsail	0.30	0.34	0.38	0.42	0.47
Total Categories 1,2,& 3	14.74	18.12	20.75	22.93	25.01
4. Other Areas "A"					
Paradise	0.05	0.07	0.10	0.11	0.13
Topsail Road	0.13	0.15	0.20	0.24	0.27
Torbay	0.16	0.17	0.19	0.20	0.22
Torbay Road	0.12	0.16	0.18	0.20	0.21
Penatanguishene	0.09	0.10	0.12	0.14	0.15
Goulds	0.16	0.19	0.20	0.22	0.23
Petty Harbour	0.07	0.08	0.09	0.09	0.10
Kilbride	-	-	0.14	0.17	0.19
Donovans	0.04	0.04	0.04	0.05	0.05
Total Categories 1,2,3,& 4	15.56	19.08	22.01	24.35	26.56
5. Other Areas "B"					
St. Phillips	0.05	0.08	0.10	0.13	0.15
Hogans Pond	0.01	0.01	0.01	0.01	0.01
Portugal Cove	0.10	0.11	0.11	0.12	0.12
Portugal Cove Rd.	0.24	0.26	0.30	0.33	0.36
Thorburn Rd.	0.14	0.18	0.21	0.24	0.27
Total Categories 1 - 5	16.10	19.72	22.74	25.18	27.47

TABLE 4.14

MAXIMUM DAILY WATER USE

(relative to categories of need)

Community	Projected Water Use in MGD				
	1975	1980	1985	1990	1995
1. Existing Serviced Areas					
Urban St. John's	14.30	14.95	15.41	15.73	15.99
Mount Pearl	0.84	0.89	0.93	0.99	1.05
Kilbride	0.23	0.39	0.39	0.39	0.39
Shea Heights	0.18	0.23	0.27	0.33	0.39
Wedgewood Park	0.06	0.08	0.09	0.10	0.12
Kenmount Ind. Pk.	0.60	0.90	0.90	0.90	0.90
Total Category 1	16.21	17.44	17.99	18.44	18.84
2. Immediate Develop't Areas					
New Town	0.66	1.68	2.40	3.06	3.65
South Expansion Zone	0.10	0.45	0.84	1.22	1.62
North Expansion Zone	0.21	0.68	0.96	1.28	1.62
Torbay Road	0.10	0.23	0.36	0.40	0.44
Donovan's Ind. Park	0.90	2.10	3.00	3.38	3.72
Kenmount Ind. Park	-	-	0.30	0.60	0.96
White Hills Ind. Park	0.38	0.75	1.05	1.50	1.86
Total Categories 1 and 2	18.56	23.33	26.90	29.88	32.71

Community	Projected Water Use in MGD 1975	1980	1985	1990	1995
3. Health Areas					
Seal Cove	0.09	0.10	0.14	0.15	0.17
Gullies	0.10	0.15	0.18	0.20	0.21
Kelligrews	0.24	0.30	0.33	0.35	0.36
Foxtrap	0.12	0.14	0.20	0.25	0.29
Long Pond	0.21	0.24	0.30	0.36	0.45
Manuels	0.14	0.14	0.15	0.17	0.18
Chamberlains & Topsail	0.45	0.51	0.57	0.63	0.71
Total Categories 1,2,& 3	19.91	24.91	28.77	31.99	35.08
4. Other Areas "A"					
Paradise	0.08	0.10	0.15	0.16	0.20
Topsail Road	0.20	0.23	0.30	0.36	0.40
Torbay	0.24	0.25	0.29	0.30	0.33
Torbay Road	0.18	0.24	0.27	0.30	0.32
Penatanguishene	0.14	0.15	0.18	0.21	0.23
Goulds	0.24	0.29	0.30	0.33	0.35
Petty Harbour	0.10	0.12	0.14	0.14	0.15
Kilbride	-	-	0.21	0.25	0.29
Donovans	0.06	0.06	0.06	0.08	0.08
Total Categories 1,2,3,& 4	21.15	26.35	30.67	34.12	37.43
5. Other Areas "B"					
St. Phillips	0.08	0.12	0.15	0.20	0.22
Hogans Pond	0.02	0.02	0.02	0.02	0.02
Portugal Cove	0.15	0.16	0.16	0.18	0.18
Portugal Cove Rd.	0.36	0.39	0.45	0.50	0.54
Thorburn Rd.	0.21	0.27	0.32	0.36	0.40
Total Categories 1 - 5	21.97	27.31	31.77	35.38	38.79

c. Water Use - Maximum Factors

	Urban St.John's	Suburban Developments	Industrial Parks
1. Average Max. Monthly: Average Annual	1.2:1	1.3:1	1.3:1
2. Maximum Day: Average Annual	1.3:1	1.5:1	1.5:1
3. Maximum Hour: Average Annual	1.7:1	2.0:1	Depending on type of Industry 2.0:1

d. Maximum Daily Water Use

The summary of forecasts for maximum daily water use in the years 1975 - 1995 is presented in Table 4.14 relative to categories of need (i.e. existing serviced areas; immediate development areas; health areas, other areas).

VIII. WATER QUALITY

The definition of a functionally ideal water, as used by the American Water Works Association (AWWA)[6] is as follows: "Ideally, water delivered to the consumer should be clear, colourless, tasteless and odourless. It should contain no pathogenic organisms and be free of biological forms which may be harmful to human health or aesthetically objectionable, or economically damaging. The water should not be corrosive or incrusting to, or leave deposits on, water conveying structures through which it passes, or in which it may be retained, including pipes, tanks, water heaters, and plumbing fixtures. The water should be adequately protected by natural processes, or by treatment processes, which insure consistency in quality".

The "Canadian Drinking Water Standards and Objectives, 1968", which are patterned after AWWA standards, has been adopted as the water quality criteria for this project. Table 4.15 summarizes these criteria.

As noted, Table 4.15 includes quality criteria designated as "objectives" and "acceptable limits". The latter is regarded as the minimum and immediate requirements for attaining safe and acceptable drinking water quality, whereas long-range planning for water quality rather than minimum requirements is referred to as "objectives". Accordingly, we recommend that the water quality "objectives" contained in Table 4.15 be adopted as the goals for the design period. In practice this would mean that a source of supply whose water quality is within the "acceptable limits" will be considered for treatment to achieve the "objectives" goal during the second half of the design period, i.e. after 1985. In the interim such a source will be carefully surveyed and should its quality deteriorate to exceed the "acceptable limits", an earlier construction of treatment works would be required. Conversely, a source of supply whose water quality parameters presently exceed the "acceptable limits" will be considered for treatment as soon as it is tapped for supply.

TABLE 4.15

WATER QUALITY CRITERIA

Characteristic	Objective	Acceptable Limits
Physical Factors		
Colour - TCU(1)	L 5	15
Odour - T.O.N.(2)	0	4
Taste	Inoffensive	Inoffensive
Turbidity - JTU(3)	L 1	5
Temperature - ^{o}C	L 10	15
Chemical Factors (in mg./l)		
Ammonia as N	0.01	0.5
Calcium as Ca	L 75	200
Chloride as Cl	L 250	250
Copper as Cu	L 0.01	1.0
Iron (dissolved) as Fe	0.05	0.3
Magnesium as Mg	L 50	150
Manganese as Mn	L 0.01	0.05
Methylene Blue Active Substances (MBAS)	L 0.2	0.5
Phenolic Substances as Phenol.	Not detectable	0.002
Phosphates as PO_4 (inorganic)	L 0.2	0.2
Total Dissolved Solids	L 500	1,000
Organics as CCE + CAE(4)	L 0.05	0.2
Sulphate as SO_4	L 250	500
Sulphide as H_2S	Not detectable	0.3
Uranyl Ion as UO_2	L 1.0	5.0
Zinc as Zn	L 1.0	5.0

TABLE 4.15

WATER QUALITY CRITERIA

Characteristic	Objective	Acceptable Limits
Corrosion & Incrustation Factors		
Alkalinity as $CaCo_3$	Should be stable; change of not more than 1 mg/l (see Appendix 6).	
Hardness as $CaCo_3$	120 mg/l; a balance between deposition and corrosion characteristics is necessary (see Appendix 6).	
pH Units	6.5 - 8.3	
Bacteriologic Factors		
Coliform organisms (by Membrane Filter Method)	No coliform organisms	(a) At least 95 percent of samples in any consecutive 30 day period should be "negative" for coliform organisms. (b) None of the samples "positive" for total coliform organisms should have an MF count greater than 4 per 200 ml or 10 per 500 ml portions.
Coliform Organisms (by Most-Probable-Number method)	No coliform organisms	(a) At least 95 percent of samples in any consecutive 30 day period should be "negative" to coliform organism. (b) None of the samples "positive" for total coliform organisms should have an MPN Index greater than 4 per 100 ml.

TABLE 4.15
WATER QUALITY CRITERIA

Characteristic	Objective	Acceptable Limits
Toxic Factors (in mg./l)		
Arsenic as As	Not Detectable	0.01
Barium as Ba	Not Detectable	L 1.0
Boron as B	-	L 5.0
Cadmium as Cd	Not Detectable	L 0.01
Chromium as Cr^{+6}	Not Detectable	L 0.05
Cyanide as CN	Not Detectable	0.01
Lead as Pb	Not Detectable	L 0.05
Nitrate + Nitrite as N	L 10.0	
Selenium as Se	Not Detectable	L 0.01
Biocidic Factors		
All dissolved biocides	Not Detectable	
Radiologic Factors (in pc/l) (5)		
Cross Radioactivity	L 10	

(1) True Colour Unit, Platinum - Cobalt Scale

(2) Threshold Odour Number

(3) Jackson Turbidity Units

(4) Total of carbon chloroform and carbon alcohol extracibles

(5) Picocuries per litre

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The rationale behind the water standards and objectives presented in this section, as discussed in the "Canadian Drinking Water Standards and Objectives, 1968" and the AWWA "Quality Goals for Potable Water" [6], is contained in Appendix 6.

IX PRESSURE ZONES

1. General

There are certain desirable conditions to aim at when designing a new and/or strengthening an existing water supply system which would result in an economic and efficient system. The conditions which apply to pressure zones can be summarized as follows [7] :-

(i) The division of the system into convenient pressure zones so that control of the quantity used in each zone is facilitated.

(ii) The employment of as low a pressure as possible consistent with an adequate supply, and fulfilment of statutory obligations.

(iii) The maintaining of as uniform a pressure as possible (say ± 20 percent variation)

(iv) The provision of alternative means of supply for each zone (ring mains help to achieve this).

(v) The making of cross-connections between pressure zones.

(vi) The elimination of both partially open valves between pressure zones and of normally closed valves within a zone.

Within the study region, the above cited conditions should be particulary considered for St. John's and environs, Mount Pearl and New Town, including the industrial parks.

2. Proposed Pressure Zones

Opinion appears to favour a service pressure range for each zone of about 100 to 150 feet. However, a higher range may not only be permissible but may, for economic reasons be unavoidable in hilly country. We consider this to be the case in St. John's and environs and the pressure range for each of the three zones, as recommended in the S.J.U.R.P.S. [2] and shown in Table 4.16 is supported by us.

TABLE 4.16

PROPOSED PRESSURE ZONES

Condition	Zone Range in Feet above Sea: Low Pressure Zone	Intermediate Pressure Zone	High Pressure Zone
Maximum Service Elevation	210	425	625
Minimum Service Elevation	0	215	425
Hydraulic Gradient Elevation	300	515	715

It can be readily seen from Table 4.16 that the total pressure in each zone varies between approximately 40 psi. and 130 psi.

Whereas a net supply pressure of 40 psi is adequate, 130 psi may prove to be somewhat excessive. This factor will be taken into consideration in the laying out of new supply lines or the strengthening of existing ones. Ideally, the trunk main will feed the pressure zone at the designated hydraulic gradient from its highest elevation, thereby minimizing head losses and preserving to the best possible extent the 40 psi pressure. The high head available for the low-lying parts of the zone will then be utilized for economical sizing of the major ring mains. However, since the ideal is sometimes unattainable low or excessive pressures will be overcome by boosting or by the use of pressure-reducing valves, as economic conditions dictate and having regard for the best economic solution.

X. METERING

1. General Definitions

The subject of metering cannot be properly addressed until the term metering is defined and discussed.

It is necessary to separate the accounts to be metered into classes and to define each class. Following are the basic classes of metering:

(i) Metering source withdrawals and/or water treatment plant production.

(ii) Metering industrial and commercial establishments.

(iii) Metering public customers such as hospitals, churches, schools.

(iv) Metering apartments and multiple family units.

(v) Metering individual homes.

(vi) Metering fire protection services, standpipes, hydrants.

The difference between metering records under class (i) and the sum records of classes (ii) through (vi) provides management with the magnitude of leakage through the system. This knowledge could and would be used for more efficient system operation and maintenance.

Metering under classes (ii) through (v) provides management with a metered-billing basis, as customers are charged for each unit of water used. This approach enhances industry and commercial institutions to initiate more stringent water use practices, and encourages residential customers to correct service leakage and eliminate wasteful usage. In this latter regard, class (v) metering would be more effective than class (iv) since each household customer will be paying only his share of water use, benefiting from any conservation measure that he has undertaken. Under class (iv) metering, a household customer may be required to pay for his neighbours wastes or to subsidise his neighbours costs.

Class (vi) metering provides management with a tool to detect illegal use of water.

It follows, then, that metering encourages the prudent use of water. The extent of the effect of metering on reductions in customer usage of water depends on local trends in water use and level of rates.

2. Experience With Metered Systems

Reports of studies on the conversion of water systems from a flat rate to a metered-billing basis and the consequent effect on customer water usage draw the attention to three areas of significant importance. These areas are as follows:

(i) That often the conversion to a metered basis is coupled with a rate increase.

(ii) That water usage per customer, with the introduction of metering and increasing in rate, often tends to decrease. However, such decrease in use may be temporary or permanent, depending on the prevailing level of water usage, the level of rates, and the degree of increase.

(iii) That residential use of water is relatively inelastic. Basic uses of water, such as for hygienic purposes, drinking, cooking, laundrying and other beneficial purposes are usually unaffected by metering or rates.

Experience has shown that, aside from service leakage, residential water use can effectively and permanently be reduced by metering (and rates) in two areas, namely:

(a) Where maintenance of residential lawns is dependent upon irrigation.

(b) Where water consuming luxury appliances are in use. For instance, metering and rates have been instrumental in causing water customers to switch from water-consuming, evaporative-type house cooling to electric refrigeration cooling.

Case histories of metered systems and the resulting effect on water use can be found in the literature. A classic example is the case history of Philadelphia and Newark; both comparably large cities, in the same geographical area and subject to the same economic forces. Both cities have their water system completely metered. Yet, whereas the total water requirements in Philadelphia had dropped by about 11.5 percent (two years after the metering program was completed), during the same period Newark saw its water needs increase roughly 10 percent.

The case of Halifax indicates that water consumption is on the increase, exceeding 100 GPCD in 1971. Projections for the next ten years call for the water needs to increase by more than 10 percent. It would appear now that the reduction in water use experienced for a number of years after the metering of the water system, had been of a temporary nature. In the case of Halifax, one has also to bear in mind the present quality of the water which does not enhance substantial increase in use.

Summing-up the above presentation, we are of the opinion that water metering and rates are indeed tools to reduce or curtail excessive usage, depending on local conditions in each specific community or region.

3. St. John's Conditions

Based on discussions in the previous sections, an assessment is presented here on the possible effects of metering on water use in St. John's. It is felt that this assessment will be valid for the entire study region.

Major customers of water are the residential class, the industrial and commercial class, and waste. A detailed analysis of water use by customer class is contained in Appendix V of this Chapter. A summary pertaining to the above three classes indicates the following:

Customer Class	FENCO Analysis Based on Present Use of 120 GPCD	FENCO Analysis Based on Projected Use of 135 GPCD
Residential	37.5% of Total	44.5% of Total
Industrial & Commercial	22.5% of Total	26.0% of Total
Waste (including unaccounted for)	30.0% of Total	20.5% of Total

It can readily be seen that the above three customer classes account for about 90 percent of the total water use. A discussion relative to each of these customer classes is presented below.

a. Residential Class

The two major factors that may effectively and permanently reduce water needs, namely sprinkling of lawns and use of luxurious appliances do not apply, or at best have a very limited application, to St. John's. The only area where water saving may be experienced as a result of metering is correction of service leakage. However, as analysed in Appendix II of this Chapter, residential water use is in the order of magnitude of 45 GPCD (compared with 40 GPCD in the completely metered system of Halifax). We do not consider this use as being excessive. At best it may be possible to reduce it by 10-15 percent (based on Halifax experience). However, for future projections an allowance would have to be made for increase in consumption.

b. Industrial and Commercial Class

This customer class is being metered, resulting in an equivalent water use of 27 GPCD. To further reduce this demand water rates would have to be increased to such a level that water conservation and re-use practices will become competitive, provided of course these practices are feasible. At the same time, it appears conceivable to assume that a higher rate of industrialization in St. John's and the study region will tend to increase the equivalent per capita water use.

c. Waste

As indicated previously in this Chapter, water waste through pipeline leakage (primarily pipe joints) is excessive. This appears to be one area where reduction in water output can be experienced, and projections along this concept have been made. Source withdrawals are being metered. Periodic flow measurements (at hydrants for example) will provide an effective measure to control waste. System pressure control and systematic programs to correct leaks are other tools to keep leakage at a relatively low level.

d. Discussion

Reduction in water use, which in turn may result in a low outlay for supplying water, can be experienced by controlling waste and metering of the residential customers.

Using the population figure projected for the end of the design period, and a density of 4.3 persons per household, some 44,000 residential meters would be required to completely service the system. Assuming an annual cost of $12.50 per meter (see breakdown in Table 4.17) for amortization, administration of reading and billing, operation and maintenance, the total annual cost for a metered system will amount to $550,000. The benefit of this expenditure should now be assessed.

TABLE 4.17

ANNUAL COST OF A METERED SYSTEM

Description of Expenditure	Annual Cost $/Meter
1. Capital payment of meter ($75 @ 8% over 20 years)	7.46
2. Maintenance	0.94
3. Reading and Billing	3.10
4. Miscellaneous Services	1.00
TOTAL	12.50

The financial analysis contained in Chapter 10 of Volume IV estimates the total annual cost of a 26.5 MGD system to be at $5.175 million. Similarly, the financial analysis shows that if 13 MGD were supplied, the annual cost would be $3.592 million. If meters are to be economically viable, the water usage in the system must be decreased so that the total annual cost of the reduced water supply facilities and that of the meters will be less than or equal to the annual cost of the proposed 26.5 MGD system. In other words, the annual cost of the smaller supply system must not exceed $4.625 million ($5.175 million-$0.550 million). From the financial analysis data presented above, this amount of money could provide a supply system with an average capacity of about 22 MGD. Thus, reduction in water use would have to be about 17 per cent of the projected total need. However, as pointed out earlier, this reduction would have to come by tighter control of waste and decrease in residential use. Allowance for the former has already been made in our water requirements projection. Assuming that a further reduction in waste of 5 percent may occur, this leaves 12 percent of the projected total water use to be saved by the residential customers. Since this customer class accounts in our projections for 44.5 percent of the total use, water saving in excess of 25 percent would have to be encountered by the residential customers. This means that the projected 1995 water use of this customer class would have to be at the 1973 level. We believe this to be impractical and unattainable.

The inference of the above analysis is as follows:

(i) The cost of water to the residential customer will be less with a metered system. This is due to a reduction in the amount of water he uses, and consequently the smaller and less costly supply facilities that would be provided.

(ii) On top of the cost of water he has used, the residential customer will have to pay a meter charge, bringing his total payment to the same flat rate that he would pay should the supply system be larger and his use of water be in accordance with our projections.

It is, therefore, recommended that water meters not be considered at this stage. Management of the regional system will include surveys of water use. Should these surveys indicate the actual use of water to exceed our projections, measures could be taken to reduce the demand. We envisage the initial measures to include educational and public relations programs. At such time that the capacity of the proposed regional supply system (and sources) will reach its ultimate utilization, and water use will exceed our projections, meters could be considered as an alternative to the development of a new source of supply which may be of poorer quality or more remotely located.

REFERENCES

(1) City of St. John's Report on Water Supply System, April 1966, Canadian-British Engineering Consultants.

(2) St. John's Urban Region Study, Municipal Services Plan, January 1973, Proctor and Redfern Limited.

(3) Estimating the depth of Pavement Frost and Thaw Penetration, Report No. CBED-6-266, February 1973, Canada Ministry of Transport.

(4) Twenty-seventh Annual Report, December 31, 1971, Public Service Commission of Halifax.

(5) Donovan's Industrial Park, November 1970, Murray V. Jones and Associates Limited.

(6) Quality Goals for Potable Water, AWWA Task Force 2640 P Report, Journal AWWA, December 1968.

(7) Manual of British Water Engineering Practice, Vol II: Engineering Practice.

(8) City of St. John's, Draft Master Plan Report, Sunderland and Simard, Vols. I - VII.

(9) Development Scheme Town of Mount Pearl, M.V. Jones and Associates Limited, in association with Newfoundland Design Associates Limited, June, 1972.

(10) Mount Pearl New Town Report, May 1971, prepared for the Newfoundland and Labrador Housing Corporation and Central Mortgage and Housing Corporation.

(11) Kilbride Development Plan, Preliminary Report prepared by the Provincial Planning Office for the St. John's Metropolitan Area Board.

(12) Town of Goulds, Interim Plan, prepared by the Provincial Planning Office.

APPENDIX 1

REPAIRS OF WATER MAIN BREAKS

	1964*	1965*	1966	1967	1968	1969	1970
January			-	5	5	1	2
February			-	-	4	2	1
March			1	1	2	-	4
April			-	-	3	-	3
May			2	1	3	-	1
June			2	-	-	1	-
July			5	2	-	1	2
August			5	-	-	4	1
September			7	6	-	-	6
October			4	9	-	-	2
November			4	1	1	3	2
December			1	3	4	3	7
Total Repairs	5	12	31	28	22	15	31

* Monthly breakdown not analyzed.

APPENDIX II

RESIDENTIAL WATER USE

The basic problem associated with calculating the residential use of water in the St. John's system arose from a lack of available data. To overcome this problem we conducted a statistical analysis of data collected by FENCO in a water meter survey.

The St. John's Municipal Council initiated a programme of installation of domestic water meters during the period 1967 to 1969. The meters have never been officially read. The date of their installation however was known and the meters had a zero setting at that time. Thus the amount of water used could be calculated, by reading the meters, knowing the duration of the metering period (from the installation date to May 1973, the time of this survey), and determining the number of consumers using water supplied through the meter.

The number of meters to be read was set at 75. This was done to minimize costs and also to obtain a statistically valid sample size. The metered households were chosen randomly so as to best represent the average "residential" user.

The sample mean was computed to equal 43.6 G.P.C.D. The sampling distribution of means is a normal distribution in a practical sampling situation (where the sample size is relatively large). It has a mean equal to the population mean.

The sample standard deviation and the sample size are used to represent the standard error of the population distribution. Both are found without any knowledge of the parent population.

These principles can be used to estimate the unknown population mean with a defined degree of confidence.

The sample standard deviation is calculated from:

$$S = \sqrt{\frac{(x_i - \bar{x})^2}{n-1}}$$

FENCO

Where S = Sample standard deviation

x_i = the ith sample

$\bar{x}$ = Sample mean

n = Sample size (75)

The sample standard deviation was calculated as 22.34 from the above formula.
If the estimate of the actual mean is to be made with 90% confidence, the important positions on the sampling distribution are:

$$u = \bar{x} + 1.667 \frac{S}{\sqrt{n}} \text{ and } u = \bar{x} - 1.667 \frac{S}{\sqrt{n}}$$

Where u = Actual mean of population

S = Sample standard deviation

n = Sample size (75)

$\bar{x}$ = Sample mean

and 1.667 is the t distribution variate for 90% probability with $n = 75$

Applying this formula to our collected data we find that the 90% confidence interval is:

$$39.3 < u < 47.9$$

It should be borne in mind that we do not know in practice in which position the sample mean lies in respect to the population mean. It is only possible to assess the probability of the actual mean within certain limits. From this assessment we can establish with a probability of 90% that the actual use of water is no less than 39.3 G.P.C.D. and no more than 47.9 G.P.C.D. and therefore we have taken the residential water use to be 45 G.P.C.D.

Appendix III

How the ISO Estimates Fire-Flow Requirements

INSURANCE SERVICES OFFICE

GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOW

1. An estimate of the fire flow required for a given fire area may be determined by the formula:

$$F = 18\,C\,(A)^{0.5}$$

where

F = the required fire flow in gpm
C = coefficient related to the type of construction
C = 1.5 for wood frame construction
= 1.0 for ordinary construction
= 0.8 for noncombustible construction
= 0.6 for fire-resistive construction

Note: For types of construction that do not fall within the categories given, use a coefficient reflecting the differences. Such coefficients shall not be greater than 1.5 nor less than 0.6 and may be determined by interpolation.

A = the total floor area (including all stories, but excluding basements) in the building being considered. For fire-resistive buildings consider the 6 largest successive floor areas if the vertical openings are unprotected; if the vertical openings are properly protected, consider only the 3 largest successive floor areas.

The fire flow as determined by the above shall not exceed

8,000 gpm for wood frame construction
8,000 gpm for ordinary construction
6,000 gpm for noncombustible construction
6,000 gpm for fire-resistive construction

except that for a normal 1-story building of any type of construction the fire flow shall not exceed 6,000 gpm.

The fire flow shall not be less than 500 gpm.

For 1-family and small 2-family dwellings not exceeding 2 stories in height see note 10.

2. The value obtained in No. 1 above may be reduced by up to a 25% credit for occupancies having a light fire loading or may be increased by up to a 25% surcharge for occupancies having a high fire loading. As a guide for determining low or high fire loadings, lists of light hazard and extra hazard occupancies as given in National Fire Protection Association Standard No. 13 are included in the Appendix.

The fire flow shall not be less than 500 gpm.

3. The value obtained in No. 2 above may be reduced by up to a 25% credit for complete automatic sprinkler protection. For buildings of fire-resistive or noncombustible construction having a light fire loading the reduction may be up to 50%. The percentage reduction that can be made for an automatic sprinkler system will depend upon the extent to which the automatic sprinkler system is judged to reduce the probability of fires spreading within and beyond the fire area. Normally this reduction will not exceed 25 percent.

4. To the value obtained in No. 2 above a surcharge should be added for structures exposed within 150 feet by the fire area under consideration. The degree of this charge shall depend upon the height, area, and construction of the building(s) being exposed, the separation, openings in the exposed buildings(s), the length of exposure, the provision of automatic sprinklers and/or outside sprinklers in the building(s) exposed, the occupancy of the exposed building(s), and the effect of hillside locations on the possible spread of fire.

The charge for any one side generally should not exceed the following limits for the separations shown:

Separation	Charge
0-10 feet	25%
11-30	20
31-60	15
61-100	10
101-150	5

The total percentage surcharge shall be the sum of the charges for all sides, but shall not exceed 75%.

5. The value obtained in No. 2 above is reduced by the credit (if any) determined in No. 3 above and increased by the surcharge (if any) determined in No. 4 above.

The fire flow shall not exceed 12,000 gpm nor be less than 500 gpm.

Note 1: The guide is not expected to necessarily provide an adequate value for lumber yards, petroleum storage, refineries, grain elevators, and large chemical plants but may indicate a minimum value for these hazards.

Note 2: Judgment must be used for business, industrial, and other occupancies not specifically mentioned.

Note 3: Consideration should be given to the configuration of the building(s) being considered and to the fire department accessibility.

Note 4: Wood frame structures separated by less than 10 feet shall be considered as one fire area.

Note 5: Party Walls:- Normally an unpierced party (common) wall may warrant up to a 10% exposure charge.

Note 6: High one-story buildings:- When a building is stated as 1 = 2, or more stories, the number of stories to be used in the formula depends upon the use being made of the building. For example consider a 1 = 3-story building. If the building is being used for high-piled stock, or for rack storage, the building would probably be considered as 3 stories and, in addition, an occupancy surcharge may be warranted. However, if the building is being used for steel fabrication and the extra height is provided only to facilitate movement of objects by a crane, the building would probably be considered as a 1-story building and an occupancy credit may be warranted.

Note 7: If a building is exposed within 150 feet, normally some surcharge for exposure will be made.

Note 8: Where wood shingle roofs could contribute to spreading fires, add 500 gpm.

Note 9: Any noncombustible building is considered to warrant an 0.8 coefficient.

Note 10: Dwellings;- For groupings of 1-family and small 2-family dwellings not exceeding 2 stories in height, the following short method may be used. (For other residential buildings, the regular method should be used.)

Exposure distances	Suggested required fire flow
Over 100'	500 gpm
31-100'	750-1000
11-30'	1000-1500
10' or less	1500-2000*

*If the buildings are continuous, use a minimum of 2500 gpm.

Also consider Note 8.

Outline of Procedure

A. Determine the type of construction.

B. Determine the ground floor area.

C. Determine the height in stories.

D. Using tables in Appendix, determine required fire flow to the nearest 250 gpm.

E. Determine the credit or surcharge for occupancy and apply to the value obtained in D above. Do not round off the answer.

F. Determine the credit, if any, for automatic sprinkler protection. Do not round off the value.

G. Determine the total surcharge for exposures. Do not round off the value.

H. To the answer obtained in E, subtract the value obtained in F and add the value obtained in G.

Round off the final answer to the nearest 250 gpm if less than 2500 gpm and to the nearest 500 gpm if greater than 2500 gpm.

Use of Tables (Steps A, B, C, D)

The tables use the GROUND AREA of the building and the height of the building in stories. Using the table corresponding to the type of construction, look under the number of stories and locate the ground area of the building(s) being considered between two ground areas given in the table. The corresponding fire flow is found in the left column.

Examples

a. Given: A 3- story building of ordinary construction of 7300 square feet (ground area). Using the table C = 1.0, in the 3-story column, 7300 square feet falls between 7100 and 8500 square feet and the corresponding fire flow is 2750 gpm.

b. Given: A 3-story building of ordinary construction of 7300 square feet (ground area) communicating to a 5-story building of ordinary construction of 9700 square feet (ground area) for a total ground area of 17,000 square feet. Determine the total floor area which equals 3 (7300) + 5 (9700) = 70,400 square feet. Using the table C = 1.0, under the one story column for 70,400 square feet the corresponding fire flow is 4750 gpm.

c. Given: A 3-story wood frame building of 7300 square feet (ground area) communicating with a 5-story building of ordinary construction of 9700 square feet (ground area) for a total ground area of 17,000 square feet.

Determine the total floor area for each type of construction and for the fire area which is 3 (7300) = 21,900 square feet of wood frame construction, 5 (9700) = 48,500 square feet of ordinary construction, and a total area of 70,400 square feet with 31% being of wood frame construction and 69% being of ordinary construction. Under the one-story column in the wood frame construction table (C = 1.5), an area of 70,400 square feet has a corresponding fire flow of 7250 gpm. Similarly, under the one-story column in the ordinary construction table (C = 1.0), an area of 70,400 square feet has a corresponding fire flow of 4750 gpm. In this case, the fire flow will be 31% (7250) + 69% (4750)= 2250 + 3280 = 5530 gpm or, to the nearest 250 gpm. = 5500 gpm.

d. Given: A 2-story building of ordinary construction of 105,000 square feet (ground area) communicates with a 1-story building of noncombustible construction of 80,000 square feet (ground area). Normally the required fire flow would be determined by proportioning as in "c" above. This would result in a required fire flow of 7460 gpm, or 7500 gpm. However, it is to be noted that the total area of the 2-story building alone results in a fire flow of 8,000 gpm and, of course, the logical answer would be 8,000 gpm. Any time the total area results in the use of an upper limit for fire flow, the possibility of a portion of the fire area justifying the upper limit must be investigated.

e. Given: A normal 1-story building of ordinary construction of 210,000 square feet (ground area). The table gives a required fire flow of 8,000 gpm, however, since this is a normal 1-story building, the maximum fire flow is 6,000 gpm.

f. Given: A normal 1-story building of ordinary construction of 80,000 square feet communicates with a normal 1-story building of noncombustible construction of 85,000 square feet. Normally the required fire flow would be determined by proportioning as in "c" above. This would result in a required fire flow of 6480 gpm, or 6500 gpm. However, since these are normal 1-story buildings the maximum fire flow is 6,000 gpm.

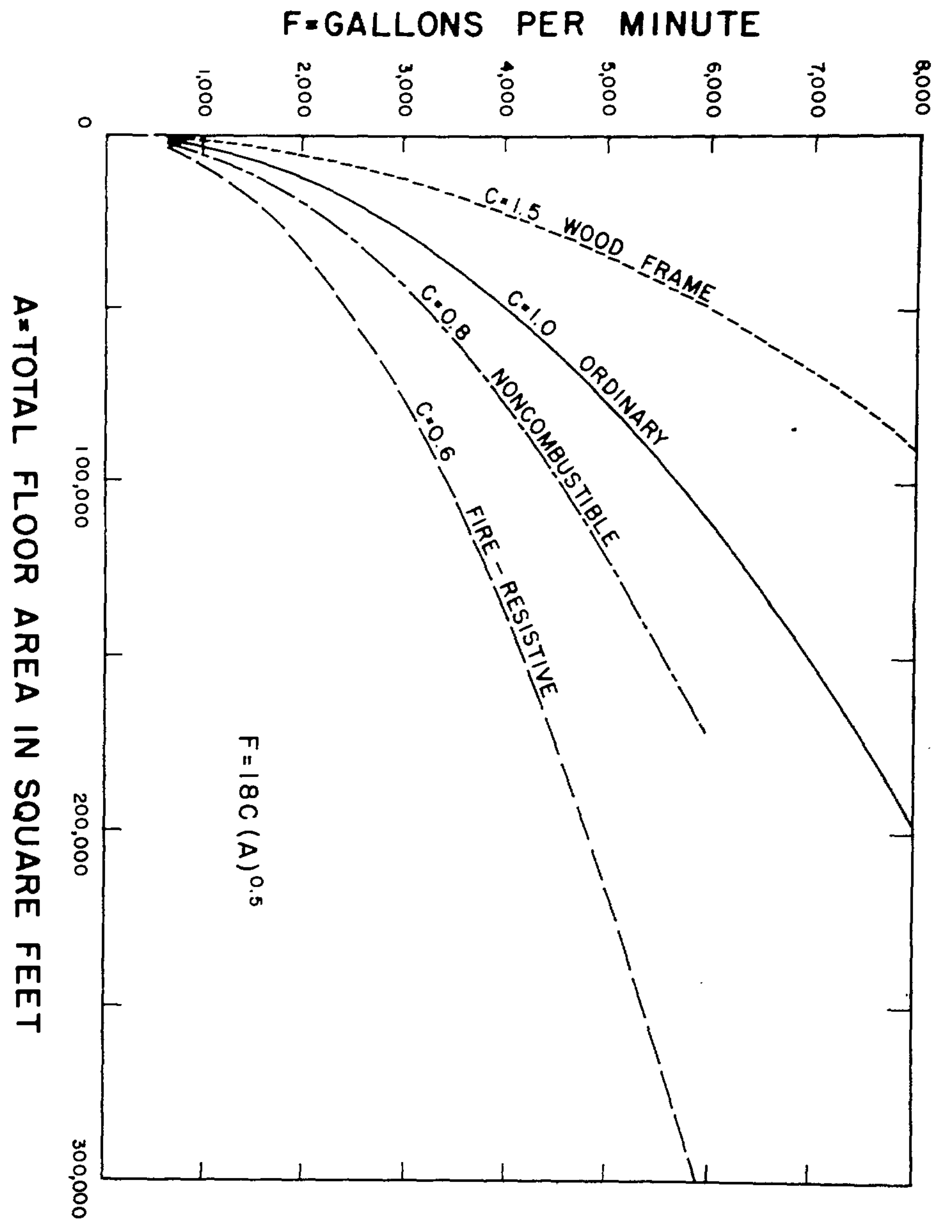
APPENDIX

NFPA No. 13 -1971, Paragraph 1311. Light Hazard Occupancies:

Apartments	Libraries, except Large Stack Room Areas
Asylums	Museums
Churches	Nursing, Convalescent and Care Homes
Clubs	Office Buildings
Colleges and Universities	Prisons
Dormitories	Public Buildings
Dwellings	Rooming Houses
Hospitals	Schools
Hotels	Tenements
Institutions	

NFPA No. 13 -1971, Paragraph 1331. Extra Hazard Occupancies:

Aircraft Hangers
Chemical Works - Extra Hazard
Cotton Picker and Opening Operations
Explosives and Pyrotechnics Manufacturing
High Piled Combustible Storage in excess of 21 feet high
Linoleum and Oilcloth Manufacturing
Linseed Oil Mills
Oil Refineries
Paint Shops
Pyroxylin Plastic Manufacturing and Processing
Shade Cloth Manufacturing
Solvent Extracting
Varnish Works
and other occupancies involving processing, mixing, storage and dispensing flammable and/or combustible liquids.



INSURANCE SERVICES OFFICE

C=1.5

FIRE FLOW VS GROUND AREA

Wood Frame Construction

(ground area in square feet)

$F=18C(A)^{0.5}$

F=gpm; C=1.5

A=area in sq. ft.

gpm	1	2	3	4	5	6 Stories
500						
	500	300	200	100	100	100
750						
	1,100	600	400	300	200	200
1000						
	1,700	900	600	400	300	300
1250						
	2,600	1,300	900	700	500	400
1500						
	3,600	1,800	1,200	900	700	600
1750						
	4,800	2,400	1,600	1,200	1,000	800
2000						
	6,200	3,100	2,100	1,600	1,200	1,000
2250						
	7,700	3,900	2,600	1,900	1,500	1,300
2500						
	9,400	4,700	3,100	2,400	1,900	1,600
2750						
	11,300	5,700	3,800	2,800	2,300	1,900
3000						
	13,400	6,700	4,500	3,400	2,700	2,200
3250						
	15,600	7,800	5,200	3,900	3,100	2,600
3500						
	18,000	9,000	6,000	4,500	3,600	3,000
3750						
	20,600	10,300	6,900	5,200	4,100	3,400
4000						
	23,300	11,700	7,800	5,800	4,700	3,900
4250						
	26,300	13,200	8,800	6,600	5,300	4,400
4500						
	29,300	14,700	9,800	7,300	5,900	4,900
4750						
	32,600	16,300	10,900	8,200	6,500	5,400
5000						
	36,000	18,000	12,000	9,000	7,200	6,000
5250						
	39,600	19,800	13,200	9,900	7,900	6,600
5500						
	43,400	21,700	14,500	10,900	8,700	7,200
5750						
	47,400	23,700	15,800	11,900	9,500	7,900
6000						

WOOD FRAME

C=1.5

INSURANCE SERVICES OFFICE

FIRE FLOW VS GROUND AREA

Wood Frame Construction

(ground area in square feet)

$F=18C(A)^{0.5}$

F=gpm; C=1.5

A=area in sq. ft.

gpm	1	2	3	4	5	6 Stories
6000						
	51,500	25,800	17,200	12,900	10,300	8,600
6250						
	55,700	27,900	18,600	13,900	11,100	9,300
6500						
	60,200	30,100	20,100	15,100	12,000	10,000
6750						
	64,800	32,400	21,600	16,200	13,000	10,800
7000						
	69,600	34,800	23,200	17,400	13,900	11,600
7250						
	74,600	37,300	24,900	18,700	14,900	12,400
7500						
	79,800	39,900	26,600	20,000	16,000	13,300
7750						
	85,100	42,600	28,400	21,300	17,000	14,200
8000						

4.97

INSURANCE SERVICES OFFICE

C=1.0

FIRE FLOW VS GROUND AREA
Ordinary Construction
(ground area in square feet)

$F=18C(A)^{0.5}$
F=gpm; C=1.0
A=area in sq. ft.

gpm	1	2	3	4	5	6 Stories
500						
	1,200	600	400	300	200	200
750						
	2,400	1,200	800	600	500	400
1000						
	3,900	2,000	1,300	1,000	800	700
1250						
	5,800	2,900	1,900	1,500	1,200	1,000
1500						
	8,200	4,100	2,700	2,100	1,600	1,400
1750						
	10,900	5,500	3,600	2,700	2,200	1,800
2000						
	13,900	7,000	4,600	3,500	2,800	2,300
2250						
	17,400	8,700	5,800	4,400	3,500	2,900
2500						
	21,300	10,700	7,100	5,300	4,300	3,600
2750						
	25,500	12,800	8,500	6,400	5,100	4,300
3000						
	30,100	15,100	10,000	7,500	6,000	5,000
3250						
	35,200	17,600	11,700	8,800	7,000	5,900
3500						
	40,600	20,300	13,500	10,200	8,100	6,800
3750						
	46,400	23,200	15,500	11,600	9,300	7,700
4000						
	52,500	26,300	17,500	13,100	10,500	8,800
4250						
	59,100	29,600	19,700	14,800	11,800	9,900
4500						
	66,000	33,000	22,000	16,500	13,200	11,000
4750						
	73,300	36,700	24,400	18,300	14,700	12,200
5000						
	81,100	40,600	27,000	20,300	16,200	13,500
5250						
	89,200	44,600	29,700	22,300	17,800	14,900
5500						
	97,700	48,900	32,600	24,400	19,500	16,300
5750						
	106,500	53,300	35,500	26,600	21,300	17,800
6000						

ORDINARY

INSURANCE SERVICES OFFICE

C=1.0

FIRE FLOW VS GROUND AREA
Ordinary Construction
(ground area in square feet)

$F=18C(A)^{0.5}$
F=gpm C=1.0
A=area in sq. ft.

gpm	1	2	3	4	5	6 St
6000						
	115,800	57,900	38,600	28,900	23,200	19,300
6250						
	125,500	62,800	41,800	31,400	25,100	20,900
6500						
	135,500	67,800	45,200	33,900	27,100	22,600
6750						
	145,800	72,900	48,600	36,500	29,200	24,300
7000						
	156,700	78,400	52,200	39,200	31,300	26,100
7250						
	167,900	84,000	56,000	42,000	33,600	28,000
7500						
	179,400	89,700	59,800	44,900	35,900	29,900
7750						
	191,400	95,700	63,800	47,900	38,300	31,900
8000						

INSURANCE SERVICES OFFICE

C=0.8

FIRE FLOW VS GROUND AREA
Non-combustible Construction
(ground area in square feet)

$F=18C(A)^{0.5}$
F=gpm; C=0.8
A=area in sq. ft.

NONCOMBUSTIBLE

gpm	1	2	3	4	5	6 Stories
500						
	1,900	1,000	600	500	400	300
750						
	3,700	1,900	1,200	900	700	600
1000						
	6,100	3,100	2,000	1,500	1,200	1,000
1250						
	9,100	4,600	3,000	2,300	1,800	1,500
1500						
	12,700	6,400	4,200	3,200	2,500	2,100
1750						
	17,000	8,500	5,700	4,100	3,400	2,800
2000						
	21,800	10,900	7,300	5,500	4,400	3,600
2250						
	27,200	13,600	9,100	6,800	5,400	4,500
2500						
	33,200	16,600	11,100	8,300	6,600	5,500
2750						
	39,700	19,900	13,200	9,900	7,900	6,600
3000						
	47,100	23,600	15,700	11,800	9,400	7,900
3250						
	54,900	27,500	18,300	13,700	11,000	9,200
3500						
	63,400	31,700	21,100	15,900	12,700	10,600
3750						
	72,400	36,200	24,100	18,100	14,500	12,100
4000						
	82,100	41,200	27,400	20,500	16,400	13,700
4250						
	92,400	46,200	30,800	23,100	18,500	15,400
4500						
	103,100	51,600	34,400	25,800	20,600	17,200
4750						
	114,600	57,300	38,200	28,700	22,900	19,100
5000						
	126,700	63,400	42,200	31,700	25,300	21,100
5250						
	139,400	69,700	46,500	34,900	27,900	23,200
5500						
	152,600	76,300	50,900	38,200	30,500	25,400
5750						
	166,500	83,300	55,500	41,600	33,300	27,800
6000						

INSURANCE SERVICES OFFICE

C=0.8

FIRE-FLOW VS GROUND AREA
Fire Resistive Construction
(ground area in square feet)

$F=18C(A)^{0.5}$
F=gpm; C=0.8
A=area in sq. ft.

FIRE-RESISTIVE

gpm	1	2	3	4	5	6 Stories
500						
	3,300	1,700	1,100	800	700	600
750						
	6,600	3,300	2,200	1,700	1,300	1,100
1000						
	10,900	5,500	3,600	2,700	2,200	1,800
1250						
	16,200	8,100	5,400	4,100	3,200	2,700
1500						
	22,700	11,400	7,600	5,700	4,500	3,800
1750						
	30,200	15,100	10,100	7,600	6,000	5,000
2000						
	38,700	19,400	12,900	9,700	7,700	6,500
2250						
	48,300	24,200	16,100	12,100	9,700	8,100
2500						
	59,000	29,500	19,700	14,800	11,800	9,800
2750						
	70,900	35,500	23,600	17,700	14,200	11,800
3000						
	83,700	41,900	27,900	20,900	16,800	13,900
3250						
	97,700	48,900	32,600	24,400	19,500	16,300
3500						
	112,700	56,400	37,600	28,200	22,500	18,800
3750						
	128,700	64,400	42,900	32,200	25,700	21,500
4000						
	145,900	73,000	48,600	36,500	29,200	24,300
4250						
	164,200	82,100	54,700	41,100	32,800	27,400
4500						
	183,400	91,700	61,100	45,900	36,700	30,600
4750						
	203,700	101,900	67,900	50,900	40,700	34,000
5000						
	225,200	112,600	75,100	56,300	45,000	37,600
5250						
	247,700	123,900	82,600	61,900	49,500	41,300
5500						
	271,200	135,600	90,400	67,800	54,200	45,200
5750						
	295,900	148,000	98,600	74,000	59,200	49,300
6000						

APPENDIX IV

DETAILS OF UNDERGROUND LEAKAGE

PITOMETER SURVEY (1966)

PITOMETER SURVEY DISTRICT	MAIN LEAKS	JOINT LEAKS	SERVICE LEAKS	MISC.(1) LEAKS	LEAKAGE G.P.D.
1	-	1	2	-	90,000
2	-	1	9	1	197,000
3	-	3	3	1	243,000
4	1	2	-	-	575,000
5	-	11 (2)	1	1	995,000
6	-	-	1	-	15,000
7	-	1	2	1	95,000
TOTAL	1	19	18	4	
GPD	440,000 (3)	1,400,000	305,000	65,000	2,210,000
Unavoidable leakage (4)					310,000

(1) Includes fire hydrants, abandoned air vent; bonnet leak.

(2) Includes 6 excessive joint leaks of 100,000; 125,000; 125,000; 140,000; 150,000; 180,000 GPD

(3) Abnormal leak promptly fixed; excluded from analysis as not typical.

(4) Based on 2,500 gallons per day per mile of main as the amount of leakage which would cost more to locate and repair than to permit to exist.

APPENDIX V

ANALYSIS OF WATER USE BY CUSTOMER CLASS

AS PERCENT OF TOTAL OUTPUT

Customer Class	Percent (%) of Total Water Use					
	1951 Pitometer Survey	1966 (1) Pitometer Survey	Corrected 1966 (2) Pitometer Survey	FENCO Analysis Based on 120 GPCD (3)	FENCO Projection Based on 135 GPCD (4)	FENCO Projection Based on 90 GPCD
1. Residential (Domestic)	33.2 (6)	30.0 (7)	31.3 (9)	37.5	44.5	60.0
2. Industrial & Commercial (Metered Consumption)	27.8	20.9 (8)	20.9 (8)	22.5	26.0	18.0
3. Public (Eleemosynary and Municipal Use)		5.2	10.0	10.0	9.0	5.0
4. Waste (Underground leakage)	13.9	19.5	19.5	20.8	17.5	17.0
5. Unaccounted for (House waste and under-registration of meters, residual error)	25.1	24.4 (8)	18.3	9.2	3.0	-
6. Total	100.0	100.0	100.0	100.0	100.0	100.0

FOOTNOTES TO APPENDIX V

(1) In reviewing the 1966 Pitometer survey, it was noted that a big main leak of 440,000 GPD had been detected and repaired. This was regarded as a temporary consumption and therefore excluded from the water usage analysis.

(2) For comparison purposes, the 1966 Pitometer breakdown of water usage according to customer class was corrected to account for:

(a) Water use by Mount Pearl consumers.

(b) Water use by consumers other than those metered but not read. These consumers include public services such as schools, hospitals, street flushing, and institutions such as religious, orphanage, old people's homes, prisons. To account for all of these public consumers we have doubled Pitometer estimate (see Section VI).

(3) FENCO's analysis of current trends of water usage shows an average of 120 GPCD for urban St. John's, see Tables 4.5 and 4.7.

(4) FENCO's analysis of trends for future use of water resulted in a projection of 135 GPCD for urban St. John's (see Section VII).

(5) FENCO's analysis of trends for future use of water resulted in a projection of 90 GPCD for suburban developments (See Section VII).

(6) Based on an estimate of 50 GPCD.

(7) Based on an estimate of 40 GPCD.

(8) 590,000 GPD used by Job's Fish Plant was recorded by Pitometer as "under-registration by meter". To obtain a representative water usage by Industry we have excluded this consumption from Customer Class 5 and added it to Customer Class 2.

(9) Based on an estimate of 40 GPCD for St. John's and 30 GPCD for Mount Pearl.

APPENDIX VI

RATIONALE FOR WATER QUALITY

STANDARDS AND OBJECTIVES

COLOUR

Colour limitations in drinking water are primarily to meet aesthetic satisfaction, and secondly, to prevent possible staining of clothes, food, and fixtures. Excessive colour may also indicate the presence of undesirable organic substances. Colour of less than 3 TC units will not be noticed, even in a filled bathtub, whereas colour of 5 TC units may be noted by many. When the colour value approaches the acceptable limit of 15 TC units, it may be desirable to investigate the cause and nature of such colour to determine the acceptability of the water supply.

TURBIDITY

Turbidity above 1JT units may be objected to by the majority of the consumers. There is evidence that freedom from disease organisms is associated with freedom from turbidity, and that freedom from taste and odour requires clarification of the water.

ODOUR

Odour is a nebulous characteristic difficult to quantify.

Agreement is seldom obtainable as to the presence of odour or its character in a given potable water. The objective of water utilities should be elimination of all odour.

TASTE

Taste is also a nebulous characteristic whose determination is complicated by the variability of perception of individuals from day to day. It is generally agreed that all potable waters do have some taste. If the taste is mild and not offensive in character, most individuals become accustomed to it.

TEMPERATURE

Organic growths in distribution lines and odour-taste characteristics of water may be intensified at temperatures above approximately 15^{O}C.

ALUMIMUM

At a level exceeding 0.05 mg/l, precipitation may take place on standing, or in the distribution system. Turbidity and nonfilterable residue will be affected.

AMMONIA

Ammonia can react readily with chlorine to form compounds with markedly less disinfecting efficiencies. It may also promote the growth of organisms and corrosion in the distribution system.

CALCIUM

Calcium limits are desirable, otherwise it may be detrimental to domestic uses such as washing, bathing and laundering, and because the calcium tends to form incrustations on cooling coils, utensils, water heaters and other fixtures.

CHLORIDE

Chloride has an effect on taste, and on such household uses such as coffee brewing.

COPPER

Copper content of 0.5 mg/l, or even less in some soft waters, will cause staining of procelain. In contents of 1-5 mg/l it will impart undesirable taste of water.

IRON

Iron is a highly objectionable element in water supplies for domestic uses.

With an iron content exceeding 0.05 mg/l, some colour may develop, staining to fixtures and laundry items may occur, and precipitates may form. The magnitude of such phenomena are directly proportional to the concentration of iron in the water.

MANGANESE

In concentrations of only a few hundredth milligrams per litre, manganese will cause build-up of coatings in distribution piping, which slough off.
It causes staining of laundry items (in brown blotches) and forms black precipitates, objectionable to consumers.

METHYLENE BLUE ACTIVE SUBSTANCES (MBAS)

This classification replaces the designation of ABS (Alkyl-Benzene-Sulphonate) previously in use. This change was required because of changes in composition of new detergents. There is need for limits in order to prevent the possible occurrence of: foaming, excessive turbidity, interference in water treatment processes and adverse effects to taste and odour.

ORGANICS (CARBON CHLOROFORM AND ALCOHOL EXTRACTIBLES - CCE + CAE)

Tastes and odours often may be correlated with the amounts of chloroform-soluble materials present, these materials having excessive odour threshold. Most of the chloroform-soluble materials derive from man-made wastes. Waters from sources remote from concentrated industrial activities or human populations usually show CCE concentrations less than 0.04 mg/l. Where concentrations of CCE of 0.2 mg/l are found, the taste and odour of the water is always poor.

PHENOLIC SUBSTANCES

The threshold concentrations, as they affect taste or odour facets produced by these substances, may be as low as 0.01 mg/l per phenol, and 0.00001 mg/l as phenol in chlorinated water.

PHOSPHORUS

It is recognized that phosphates, in general, may stimulate the growth of photosynthetic organisms, resulting in problems of odour and tastes and other detrimental effects. It appears that even a concentration of 0.2 mg/l as PO_4 may be high under some conditions.

SULPHATES

Water with sulphate concentration above 500 mg/l as SO_4 may not be usuable for drinking purposes since users may experience gastrointestinal irritation and catharsis. Objectionable taste may also occur.

SULPHIDE

Concentration greater than 0.05 mg/l as H_2S may produce taste and odour objectionable to the majority of users.

TOTAL DISSOLVED SOLIDS (Filterable Residue)

In general, concentrations of total dissolved solids in excess of 500 mg/l in drinking water may not be acceptable on grounds of undesirable taste, and, perhaps, also laxative effects.

URANYL ION

This chemical may produce objectionable taste and colour in water. It is also suspected as being capable of producing damage to kidneys. The set limit of 5.0 mg/l as UO_2 is based on colour and taste considerations. Health considerations would apply for concentrations of 10 mg/l or higher.

ZINC

In concentrations of 5 mg/l, a disagreeable taste may be noted. Zinc is also undesirable in water passing through piping systems, as it may aid corrosion.

ALKALINITY

Alkalinity is expressed in terms of the equivalent amount of calcium carbonate. The maintenance of calcium carbonate stability is the most effective method of preventing corrosive action on iron water mains.
"Undersaturation" will result in reactions causing iron pick-up and development of "red water". "Oversaturation" will result in carbonate deposition in utensils, water heaters, household piping, and even in water mains. The point of stability is quite variable in different waters. Various methods have been utilized to determine the point

of stability, including the Enslow stability indicator, the Langelier index, the Ryzmar index. The measure of alkalinity decrease or increase in the distribution system, measured over a period of time, and also from a sample left to stand for 12 hours (at 130^{o}F in a closed plastic bottle) followed by filtration will indicate, in a practical way, that the alkalinity might be stable.

HARDNESS

Hardness of water is a relative expression. It has been accepted to classify water of hardness less than 80 mg/l as $CaCo_3$ as very good, whereas a hardness of between 80 and 120 mg/l is considered as good water. To the average water consumer, hardness of 80 - 100 mg/l is not objectionable. It is important, however, that the hardness should be maintained at a uniform level. The higher the hardness, the greater the treatment cost to individual consumers to obtain "soft" water; the less the hardness, unless corrected, the greater the corrosion tendencies, and the greater the relative cost for the treatment.

HYDROGEN ION CONCENTRATION (pH)

Water in the pH range of 6.5 to 8.3 is acceptable provided other conditions are satisfactory. At higher pH's there is a progressive decrease in the effectiveness of chlorine disinfection processes. Lower pH's will enhance corrosion.

COLIFORM ORGANISMS

Many water utilities have adopted high standards of operation and their water supplies have shown only a fraction of one coliform per litre over periods of many years. Municipalities with such high bacteriological quality have established much improved health conditions with respect to certain significant illnesses, such as intestinal disturbances. Modern disinfection control procedures are such that a practical objective can be the destruction of all coliform organisms.

MACROSCOPIC AND NUISANCE ORGANISMS

It is obvious that macroscopic organisms such as larvae, crustacea, and numerous algae that may affect appearance should not be present. Nuisance organisms may affect appearance, taste or odour. They include, among others, the iron bacteria, sulphur bacteria and slime growth.

NITRATES AND NITRITES

The set limit on nitrate plus nitrite as nitrogen is based on the relationship established between this chemical and the possible occurrence of infantile methaemoglobinemia (a disease characterized by specific blood changes and cyanosis).

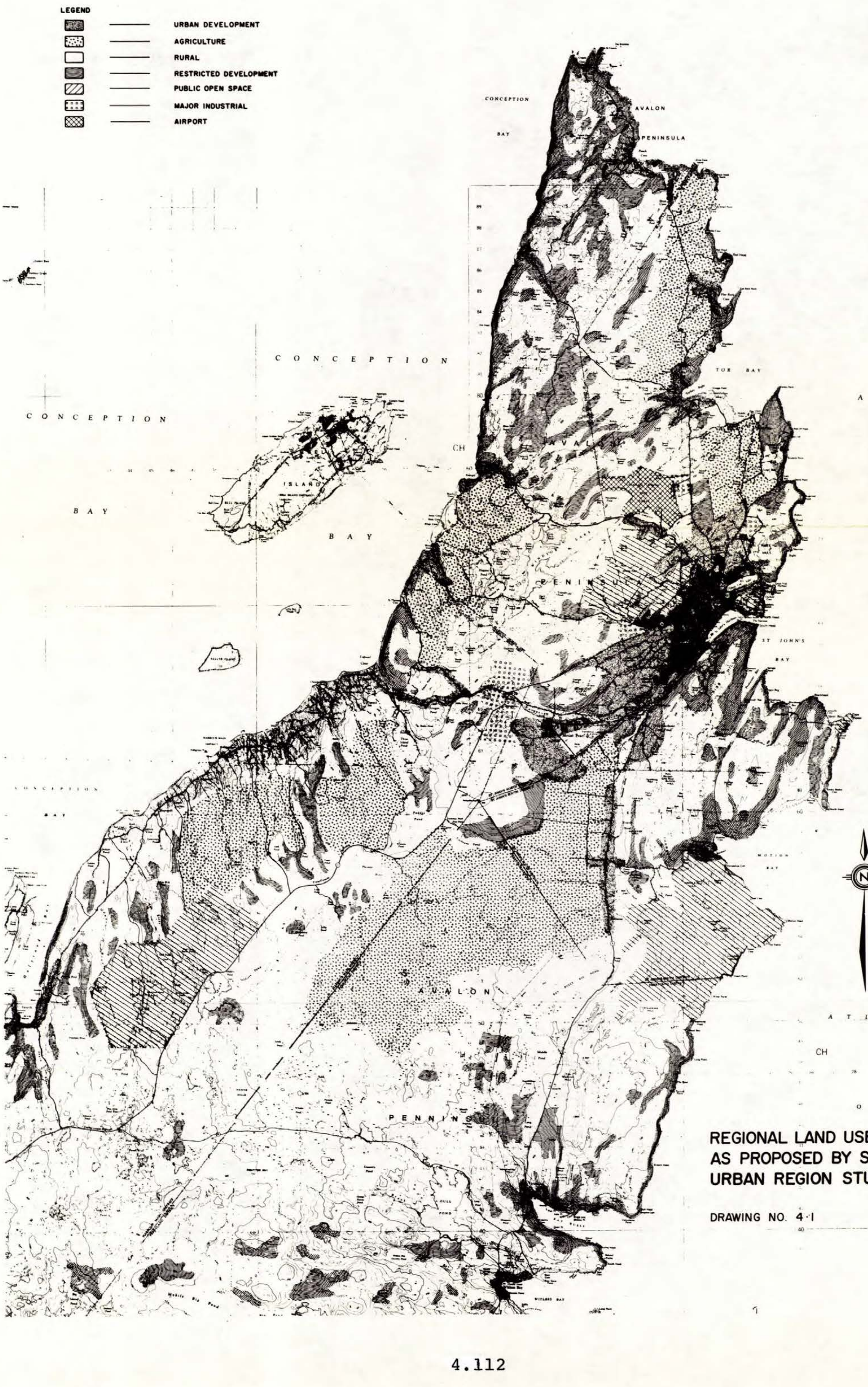
BIOCIDES

The term "biocides" is used to include all organic chemical agents employed for the control of pests, disease vectors and nuisance organisms on land and in water that may:

(a) have toxicological and health effects on man on the basis of long-term exposure, and
(b) have taste and odour effects on water.

GROSS RADIOACTIVITY

All evidence indicates the effects of radioactivity to be entirely harmful rather than benefical. Therefore, it appears desirable to limit the intake of radioactivity as much as possible. The natural background in most areas is only about 10 pc/l.



REGIONAL LAND USE PLAN 1991
AS PROPOSED BY ST. JOHN'S
URBAN REGION STUDY

DRAWING NO. 4·1

FENCO

Chapter 5

CHAPTER 5

SOURCES OF WATER SUPPLY

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CHAPTER 5

SOURCES OF WATER SUPPLY

SYNOPSIS

Alternative sources of water supply, that will meet the requirements projected in Chapter 4, have been considered and evaluated in this Chapter. A summary of the findings and recommendations is as follows:

- Windsor Lake, Petty Harbour Long Pond, Bay Bulls Big Pond, and Thomas Pond are the four (4) sources of supply considered for the St. John's Regional Water System. The first two sources are presently in use; the last two sources have been considered as supplementary sources.

- Windsor Lake catchment area is 6.4 square miles. The reliable yield of Windsor Lake is in the order of magnitude of 10 MGD. The quality of the lake water is generally within the "acceptable limits" of the "Canadian Drinking Water Standards and Objectives".

- Petty Harbour Long Pond catchment area is 3.38 square miles. Its reliable yield is about 4 MGD. Taste and odour problems are experienced with these waters during the algae blooming season.

- Bay Bulls Big Pond catchment area is 14.5 square miles. Its reliable yeild is in the order of magnitude of 23 MGD. The large storage capacity of the pond contributes to this relatively high yield. The water is of a quality that requires treatment to primarily remove colour, coliform organisms, and algae.

- Thomas Pond catchment is 15.8 square miles. Due to a small storage capacity, its reliable yield is only in the order of magnitude of 11 MGD. Thomas Pond water is of a much inferior quality when compared with Windsor Lake and Bay Bulls Big Pond. It requires treatment to remove (in addition to coliform organisms) colour, iron and manganese, the last two being highly objectionable elements in water supply.

- Little Powers Pond is presently used for augmentation of Windsor Lake. Its reliable yield (with an existing limited storage capacity) has been assessed to be in the order of magnitude of 1 MGD.

- Treatment concepts have been assessed for the water quality of Windsor Lake, Bay Bulls Big Pond and Thomas Pond. It is recommended that a test program be undertaken for Bay Bulls Big Pond water to establish the treatment process most effective and economical for this source water.

I. SOURCES CONSIDERED

As mentioned in Chapter 3, the requirement to augment the water supplied to the St. John's distribution system from Windsor Lake and Petty Harbour Long Pond arose in the early 60's, when an acute water shortage was experienced. Consequently, the level of Windsor Lake was raised three feet thereby increasing its reported reliable yield to 9.5 MGD.

In a review of the Metropolitan Area Water Supply Programme [9], Newfoundland Design Associates Limited, in association with M. V. Jones and Associates, recommended the following:-

> "Considering that the increasing population along the Bay Bulls Road will no doubt in future form local governments, and also that the long-term expansion of the City will most likely be towards the Southwest, we strongly suggest that this alternative (the use of Bay Bulls Big Pond as the supplementary source of water supply to the Metropolitan Area) be investigated in detail."

Subsequently, Newfoundland Design Associates Limited were retained to carry out this detailed investigation, and in their report of November 1967 [6], they have proposed to tap the water of Bay Bulls Big Pond, and following local treatment by screening, chlorination, and lime addition, to pump it into the St. John's distribution system, via a 24 inch diameter pipeline that would be connected to the Petty Harbour Long Pond outlet.

In the context of the St. John's Urban Region Planning Study [7], the water demand of the region was projected to be 28 MGD by the year 1991. That report also assessed the yield of the existing sources of supply, i.e. Windsor Lake and Petty Harbour Long Pond, to be 15 MGD resulting in a deficit of 13 MGD in the horizon year. The report [7] stated that "only a few of the drainage areas can supply this demand and the one best placed by far is Bay Bulls Big Pond."

The report also adopted a different strategy from that outlined by Newfoundland Design Associates Limited [6]. Whereas, the latter recommended that Bay Bulls Big Pond be connected directly to St. John's distribution system through a 24 inch diameter pipeline, the Municipal Services Plan [7] considered it desirable that this pipeline be used to augment the existing source at Petty Harbour Long Pond. With such an arrangement, any treatment requirements would be centred at the Petty Harbour Long Pond site.

Quantitative, qualitative and economic factors of developing water sources, relative to the supply region, should firstly be considered and evaluated. In this respect, we concur with the recommendation in the Municipal Service Plan [7], that there should be no more than two regional sources of water supply with adjacent treatment facilities, as required. It is self evident that one of the sources is Windsor Lake. We believe, however, that considerable thought should be given to the selection of the second source. We are of the opinion that as an alternative to Bay Bulls Big Pond-Petty Harbour Long Pond system, a supply source more centrally located relative to Conception Bay South and Newtown - Mount Pearl - Donovan's Industrial Park should be considered. Thomas Pond, with a catchment area of 15.8 square miles and an elevation of 480 feet appears to answer this requirement. Accordingly, four catchments - Windsor Lake, Petty Harbour Long Pond, Bay Bulls Big Pond, and Thomas Pond have been evaluated.

It will be worthwhile noting here that attention was given to two other catchments, namely, Paddys Pond and Broad Cove River.

Paddys Pond [11] is fed by overflows from Thomas Pond and Cochrane Pond. At its full storage elevation of 430 feet Paddys Pond occupies a surface area of 538 acres. The catchment area is about 20 square miles. The terrain ranges from heavily-wooded to marshy loglands. Activities in the catchment area include the following:

(i) Use as a reservoir for the hydroelectric generating station at Topsail.

(ii) Use as a sea-plane base and heliport by the Government of Newfoundland and Labrador Air Ambulance Service.

(iii) Use as a sea-plane base by several private interests.

(iv) Use as a Forest Fire Patrol depot by the Provincial Department of Forestry and Agriculture.

(v) Use as a recreational centre for angling, boating, swimming, picnicking, camping, and some water fowl hunting.

(vi) Use as a modest summer cabin development.

(vii) Use as a Community Pasture Project by the Provincial Department of Forestry and Agriculture.

The above practised activities, most notably, the uses under items (ii) and (iii), the low elevation of the pond relative to Thomas Pond, and the fact that Trans-Canada-Highway runs parallel to the pond for a distance of about one mile, led us to discard Paddys Pond from consideration as an alternative source of supply.

Little Powers Pond on the Broad Cove River system is being used by the City of St. John's for augmentation of Windsor Lake Water. The pond itself is very small; at its full storage elevation of 378 feet it occupies a surface area of some 30 acres. The catchment area is about 4.3 square miles. This fact coupled with the preference for a southern source of supply (as previously described in this section, and as financially justified in Chapter 10 of this Report), ruled out further pursuit of Broad Cove as an alternative source of supply. However, we did assess, in the ensuing sections of this Chapter, the yield of Little Powers Pond.

It should be noted that both Bay Bulls Big Pond and Thomas Pond are at present used by the Newfoundland Light and Power Company as sources of water for the generation of hydroelectric power. The use of either of them as a water supply source would mean a loss of power potential and, therefore, payment of compensation costs will most likely be necessary.

II. WATERSHED CHARACTERISTICS

1. General

The four watersheds under consideration in general are rather similar topographically. They form part of the region's rolling plateau land and are located at approximately 400 to 600 feet elevation. Knolls within these catchments project a few hundred feet higher. The area is covered with innumerable thickets, bogs and shallow ponds and streams. Much of it is wooded, but the trees are small and the streams draining the area are precipitious and afford opportunities for the local development of hydro-electric power. Several such streams have been harnessed, as already mentioned, to form the Topsail, Petty Harbour and Seal Cove Hydro-electric development schemes, owned and operated by the Newfoundland Light & Power Company.

There is good evidence that the area was heavily glaciated during the Pleistocene period by ice moving radially from the central part of the Avalon Peninsula. A considerable thickness of relatively unweathered glacial debris of local origin is common. The original soil was pushed into the sea leaving behind sterile sands and gravels. The thin soil cover is leached and therefore acidic, and coupled with the climate - short cool summers and long cold winters - is suitable only for coniferous forests. Peat bogs are common and rest on the eroded surface of the bed rock.

2. Windsor Lake Catchment

The drainage area of Windsor Lake forms the pond of the same name located at latitude and departure 47° 36'N, 52° 48'W, respectively. The basin has a surface area of 6.4 square miles of which 2.2 square miles are water area. There are several small ponds in the natural catchment, all of which drain into Windsor Lake.

Highway 19 (Thorburn Road) and Highway 20 (Portugal Cove Road) run through the catchment and skirt Windsor Lake along its southern and northern boundary, respectively. There are residential developments along both of these highways. However, there is no recreational activities in the catchment, and there is no evidence of further residential developments, both of which are restricted by law [10].

Facilities have recently been constructed to divert the waters of Little Powers Pond, (catchment area 4.3 square miles) into Windsor Lake. The characteristics of this catchment area are essentially the same as of Windsor Lake and restrictions regarding residential developments and recreational activities are also similar.

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Geologically, the rocks in the drainage basin date from the Pre-Cambrian era, are of sedimentary origin and belong to the Conception Bay Group. The rocks are relatively unaltered, fine grained and composed of materials derived from the underlying Harbour Main Group. Thus, they consist mainly of siltstone, sandstone and slate.

The catchment area is heavily wooded and contains some peat bogs. The general elevation in the area does not vary much. The full supply elevation of Windsor Lake is approximately 496 feet.

3. Petty Harbour Long Pond Catchment [2]

The drainage area of Petty Harbour Long Pond is 3.38 square miles of which 1.4 square miles are water surface area. The pond itself has no large streams draining into it, but rather receives runoff from Rocky Pond which in turn is fed by a diversion from Bear Pond.

The catchment area lies at a high altitude, ranging from 537 feet to 638 feet in general, but it is over 700 feet high on its western edge. The topography of the land is extremely hilly and the surface is barren with practically no bogs. Only the immediate surrounding area of Long Pond is heavily wooded. The forest comprises of fir, spruce and birch trees and it extends to the water edge.

Geologically, the catchment is in a region of Pre-Cambrian sedimentary and volcanic rocks and has been classified as belonging to the Cabot Group of the Signal Hill Formation. The typical rocks are arkosic sandstone and conglomerate, quartzitic sandstone with thin beds of argilite, siltstone and slate.

There is very little residential development in the basin and no recreational activities. Both are stringently restricted [10].

4. Bay Bulls Big Pond Catchment

The Bay Bulls Big Pond watershed covers a total area of 14.5 square miles of which approximately 3 square miles are water surface area. The main pond is located at 47° 27'N, 52° 47'W.

Highway No. 5 runs through the drainage basin for about 5 miles and considerable development exists along this highway. In addition to this, approximately 124 acres of the catchment area has been developed for agricultural and dairy farm purposes. Some timber is extracted and processed at a sawmill located within the catchment.

The area is characterized by many ponds and several marshes and peat bogs. The eastern part has a steep slope which ends in a general flat region surrounding the Bay Bulls Big Pond. The main stream draining from the pond is Raymond Brook. Its elevation graduates from 402 feet to 250 feet.

Geologically, the basin has been classified as belonging to the Cabot Group with St. John's formation. It contains siltstone, arkose, conglomerate, quartzitic sandstone with thin beds of argilite and acidic to intermediate volcanic rocks.

Vegetation in the catchment is typical of the environs of St. John's, consisting of black spruce, balsam fir and tamarack.

From a recreational point of view, Bay Bulls Big Pond is extensively used being the nearest to the City of St. John's. The recreational activities include swimming, fishing, boating, wild fowl hunting, picnicking and camping.

5. Thomas Pond Catchment [1]

The Thomas Pond watershed covers a total area of 15.8 square miles of which 1.5 square miles are water area. It lies approximately in the centre of the Avalon Peninsula. The pond itself, which was dammed in 1956 to form a reservoir, is situated at 47^{o} 27'N, 52^{o} 55'W with a full supply elevation of around 482 feet.

The catchment is characterized by distinctly wooded areas, a large number of small lakes which string along the main stream, (Manuels River, which drains into Thomas Pond), several peat bogs and swamp areas. Elevation along the main stream varies from 600 feet to 480 feet.

There is no major highway in this catchment, but the Trans Canada Highway runs close to the pond for a short distance, at the downstream end of the dam. The Provincial Department of Mines, Agriculture and Resources operates two community pastures in the drainage area. The Foxtrap pasture (2,500 acres) lies totally within the catchment, while the Cochrane Pond Pasture (10,000 acres) is only partly within the drainage area. Some timber is removed from the forest of the basin.

The vegetation of the catchment is of the coniferous type and the predominant trees are black spruce, white spruce and larch, balsam and fir are also found in some areas.

Geologically, the catchment lies in an area of Pre-Cambrian sedimentary and volcanic rocks. Most of the strata in the area are of sedimentary origin and have been classified as belonging to the Conception Bay Group containing mainly siltstone, sandstone and slate.

Thomas Pond is a popular fishing pond for the residents of the area. This distinction is undoubtedly caused by virtue of its ready access and its purported, but probably diminishing, high yield of fish. In addition to angling, other recreational uses include boating, picnicking, camping and wild fowl hunting.

III. SOURCES STORAGE AND YIELD

1. General

The four sources of supply considered for this project can be classified as "impounding" reservoirs, namely, reservoirs that are filled by natural inflow (run-off) from their own catchment (together with the run-off of any other catchment diverted into them), and from which the water is drawn off to supply at a given rate. It follows then that the most important physical characteristic of these sources is their storage capacity, and that the most significant aspect of the storage is the relation between its capacity and yield. A definition of yield for impounding reservoirs used for water supply is [4]: "the uniform rate at which water can be drawn from the reservoir throughout a dry period of specified severity without depleting the contents to such an extent that withdrawal at that rate is no longer feasible."

In order to determine the reliable yield of each or any of the sources of supply, the relationship between its natural inflow (run-off) at a period of critical severity and the existing, or potential, storage capacity has to be established. The main difficulty with this technique lies in defining the extent of the dry period having the most critical severity.

With the advent of the high speed digital computer, it has become more practical to analyze longer periods of record in the determination of reliable yield. Thus, rather than studying the performance of an impounding reservoir during one dry period of severe conditions, a complete history of runoff quantities can be analyzed to determine the yield that can be withdrawn with a specific amount of reliability. This reliability of the system can be taken as the number of years that the rate of water withdrawal is met divided by the number of years in the historical record that is being analyzed. However, in either of the approaches outlined above, it is necessary to have reliable stream flow records.

2. Run-Off

The term run-off is used to denote stream flow maintained by ground and surface storage, together with surface flow resulting from heavy precipitation.

Long-continuous gauging records of run-off not only give an accurate mean flow, but also provide information regarding what may be expected in the way of successive dry years and the pattern of wet periods. It is generally recognized that several successive dry years will cause serious depletion of an impounding reservoir's storage which could result in a failure of the system to meet the required rate of water withdrawal, if appropriate provisions have not been made in the design and operation of the system. Where only a limited amount of flow records are available, there may be a need to extend the record using one of several techniques. Firstly, run-off data may be synthesized from meteorological and other physical data. Alternatively, a stochastic hydrologic model can be used to extend existing streamflow records.

Previous reports [6], [7], make reference to available gauging records of run-off which were used as a basis for determining the relationship between reliable yield and storage capacity of the reservoirs in the study region. We have approached the source agency of these records, which is the Newfoundland Light and Power Company Limited, and they have explained the methods used to determine flow quantities. These data are determined by converting the power produced each month at the particular powerstation to an equivalent amount of runoff. In addition, corrections are made to account for changes of storage in the catchment. It is self-evident that these data of run-off should be used cautiously due to:

(i) Uncertainty of the degree of accuracy since this is not a direct measure of the streamflow.

(ii) Uncertainty whether this historical record is long enough to provide an adequate description of the run-off patterns that would constitute a dry period which could be used for design purposes.

A run-off record period of 41 years (1931 to 1972) exists for the Petty Harbour power station. A study of the accuracy of these run-off determinations was carried out by others [8]. They have shown (diagramatically), that the results obtained from streamflow measurement (of the power station outflow), were usually up to 10 percent less than those determined by computations from the power output. We have, therefore, concluded that the Petty Harbour power station records could be utilized in this study provided the above limitation is recognized. Furthermore, the abundant and well distributed rainfall in the region result in a runoff pattern which is relatively reliable. This phenomenon permits the use of a relatively short flow record with a reasonable degree of accuracy. The existing record could be extended using a technique to estimate runoff from meterological and physical data, or a stochastic hydrologic model, as mentioned earlier. However, this was not deemed necessary.

3. Yield-Storage

a. Method Selected

A computer model simulating the operation of an impounding reservoir for any given rate of water withdrawal relative to an inflow hydrograph as defined by the recorded run-off data at Petty Harbour power station was the method selected for this study. Transportation of the flow quantities to each of the catchments considered as a source of supply was based on the ratio of the land areas of the study catchment to the area of the gauged catchment. Input to the computer program, other than the inflow data corrected for each specific catchment, and given rates of water withdrawal, included the amount of storage that could be utilized and the initial storage available in the reservoir. The reliability was determined as the number of years that a given rate of water withdrawal was met to the total number of years in the historical record analyzed.

In addition, checks were performed to verify the yield-storage data derived from the above selected method using two alternative methods, as follows:

(i) A. F. Meyer's [5] method to determine run-off data on the basis of meteorological and other physical data.

(ii) A. Hazen's [12] method to determine the storage to be provided in impounding reservoirs for municipal water supply.

The results obtained from the selected method and the above two alternative methods were of the same order of magnitude.

The application of the selected methodology, based on run-off data transposed from the Petty Harbour power station to the sources of supply considered for the project indicates the following:

b. Windsor Lake

The mean annual yield of this catchment has been computed to be in the order of magnitude of 11 MGD.

Figure 5-1 shows the yield versus reliability curve that was determined from the computer simulation. A usable storage of 7,600 acre-feet was used in developing this curve. This storage is in line with comparable data published in previous reports [7], [13]. It can be seen from the curve that at a reliability of 99 percent, the yield is 9.8 MGD, whereas a yield of 9.4 MGD would be provided at a reliability of 99.8 percent. These results support the previously reported reliable yield as being 9.5 MGD.

c. Bay Bulls Big Pond

The largest storage volume available at any of the four catchments is at Bay Bulls Big Pond. More than 20,700 acre-feet of storage are available from its full storage elevation to the existing intake conduit. The catchment area is 14.5 square miles which provides a mean annual yield in the order of magnitude of 26 MGD.

The results of the computer simulation are presented in Figure 5.2 which shows the reliability of the system for various yields. Yields of 23 MGD and 22 MGD would be provided (with the existing system)

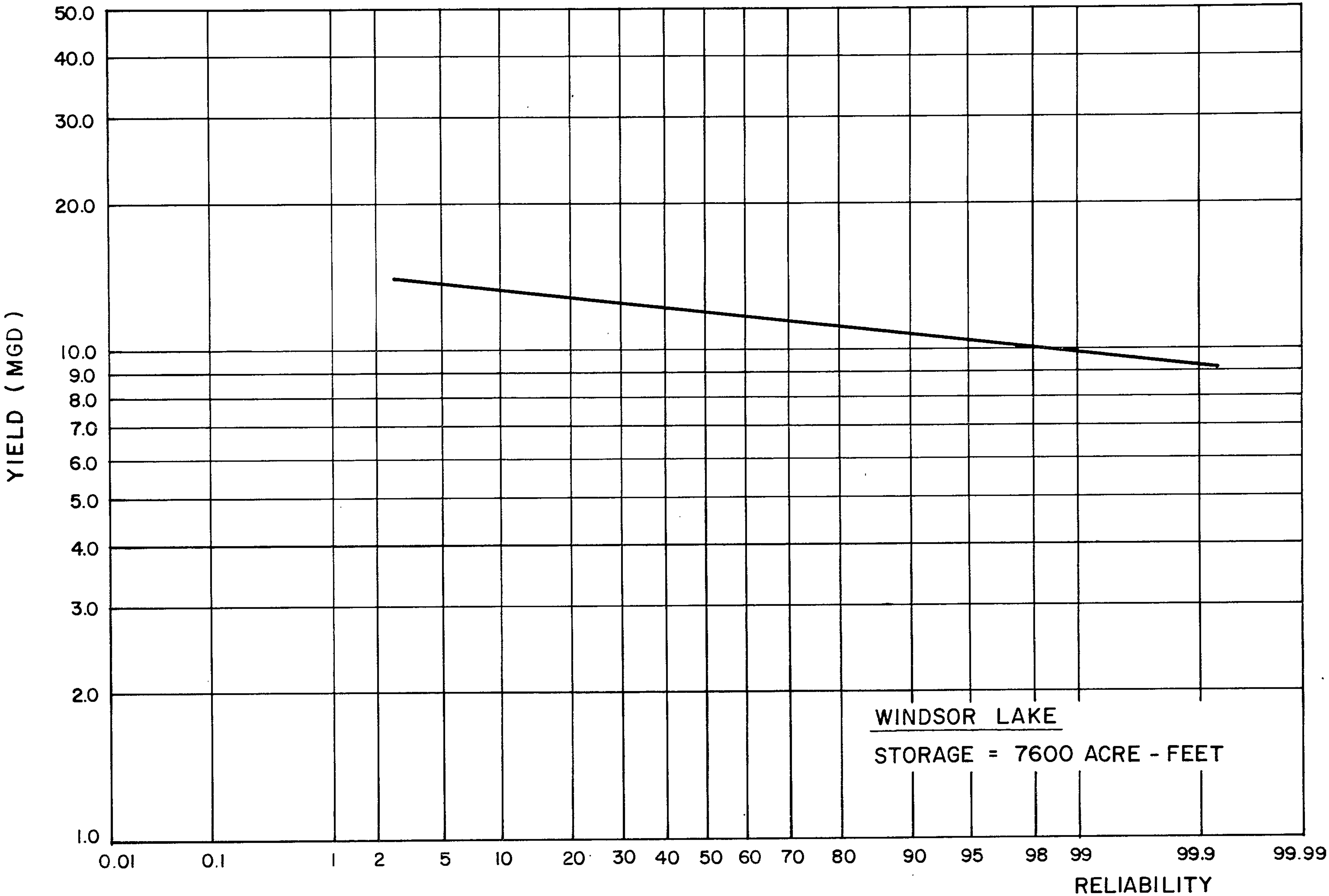

FIGURE 5·1
YIELD vs. RELIABILITY

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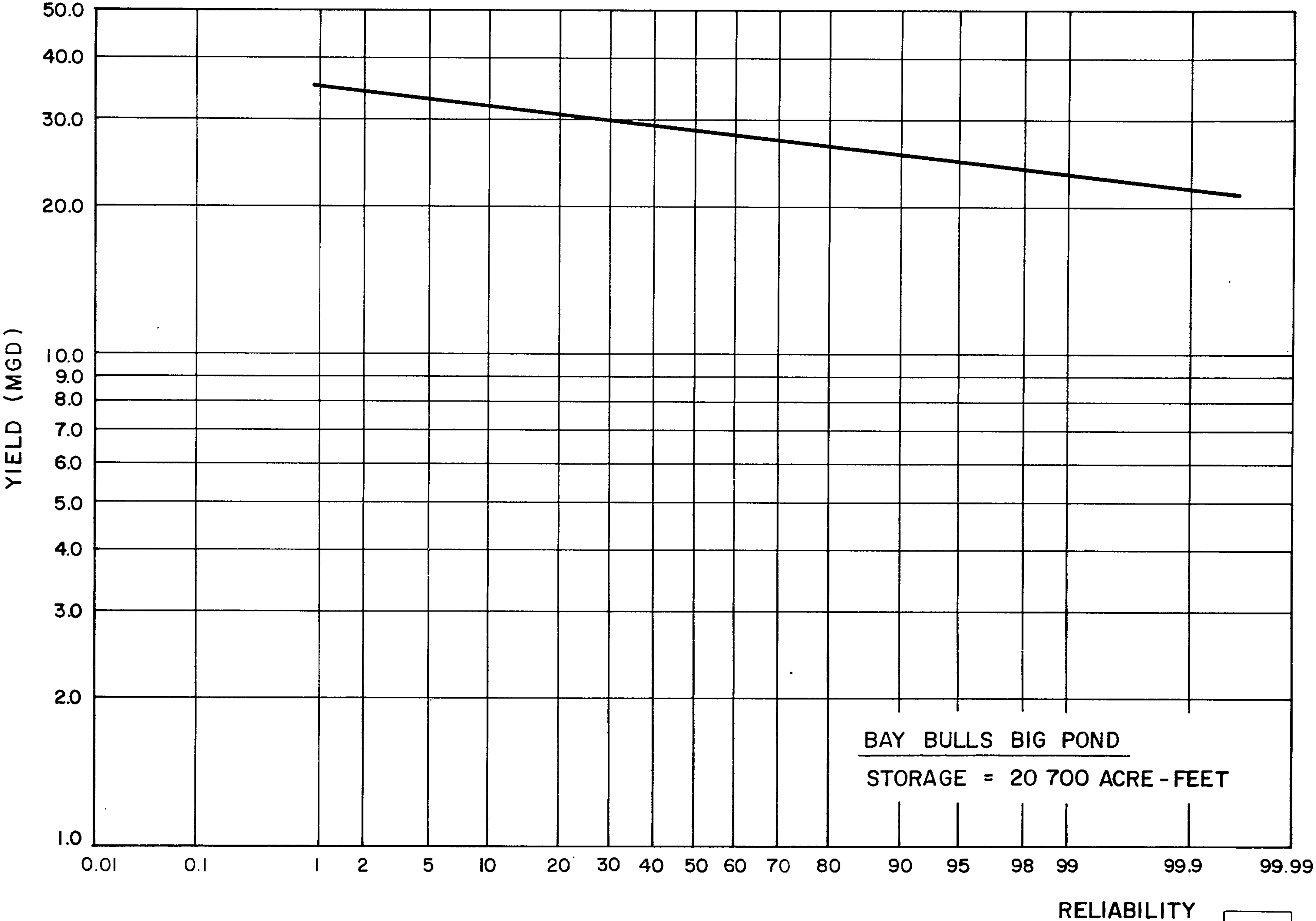

FIGURE 5·2

YIELD vs. RELIABILITY

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at a reliability of 99 per cent and 99.8 percent, respectively.

d. Thomas Pond

Thomas Pond catchment differs significantly from the two areas previously considered in that the usable storage in this pond is relatively small compared to the catchment area. Approximately 4,200 acre-feet of storage are available to modulate the run-off from the 15.8 square mile drainage basin. Consequently, while the mean annual yield is approximately 30 MGD, the computer results plotted in Figure 5.3 show that a yield of only 11 MGD can be relied on with the present storage capability.

e. Petty Harbour Long Pond

Petty Harbour Long Pond can be characterized as a reservoir with a large storage capacity relative to its catchment area. Its mean annual yield has been computed to be in the order of magnitude of 6 MGD. In the absence of more accurate data on storage capacity, we have used a volume of 2,000 acre-feet for computer simulation of the reliable yield. The curve shown in Figure 5.4 indicates that at a reliability of 99 percent or better, the yield provided would be between 3.75 and 4 MGD. For a smaller storage capacity, this reliable yield may drop to between 3.5 and 3.75 MGD.

f. Little Powers Pond

As mentioned previously in this Chapter, the computer simulation model has been applied to assess

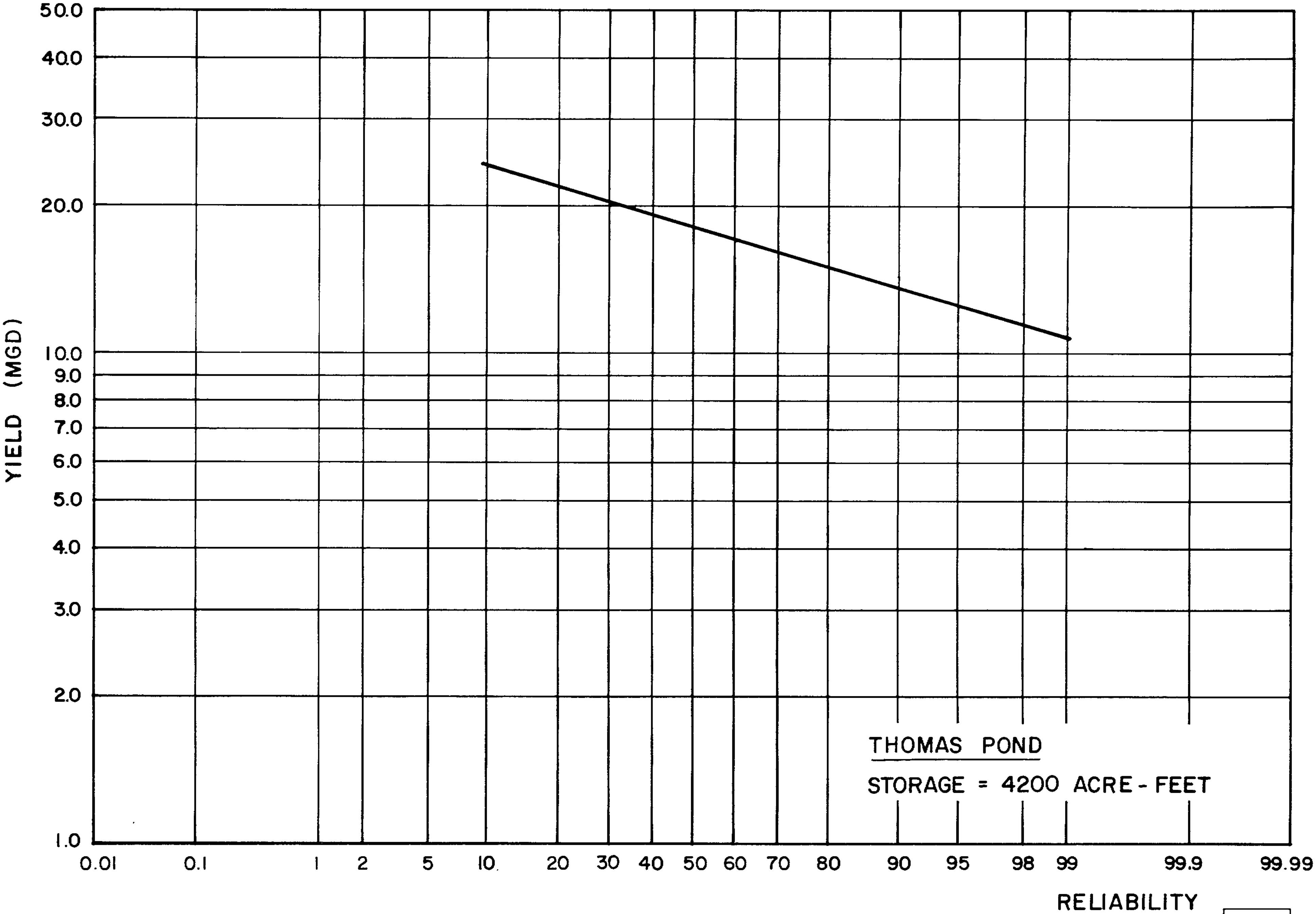

FIGURE 5·3

YIELD vs. RELIABILITY

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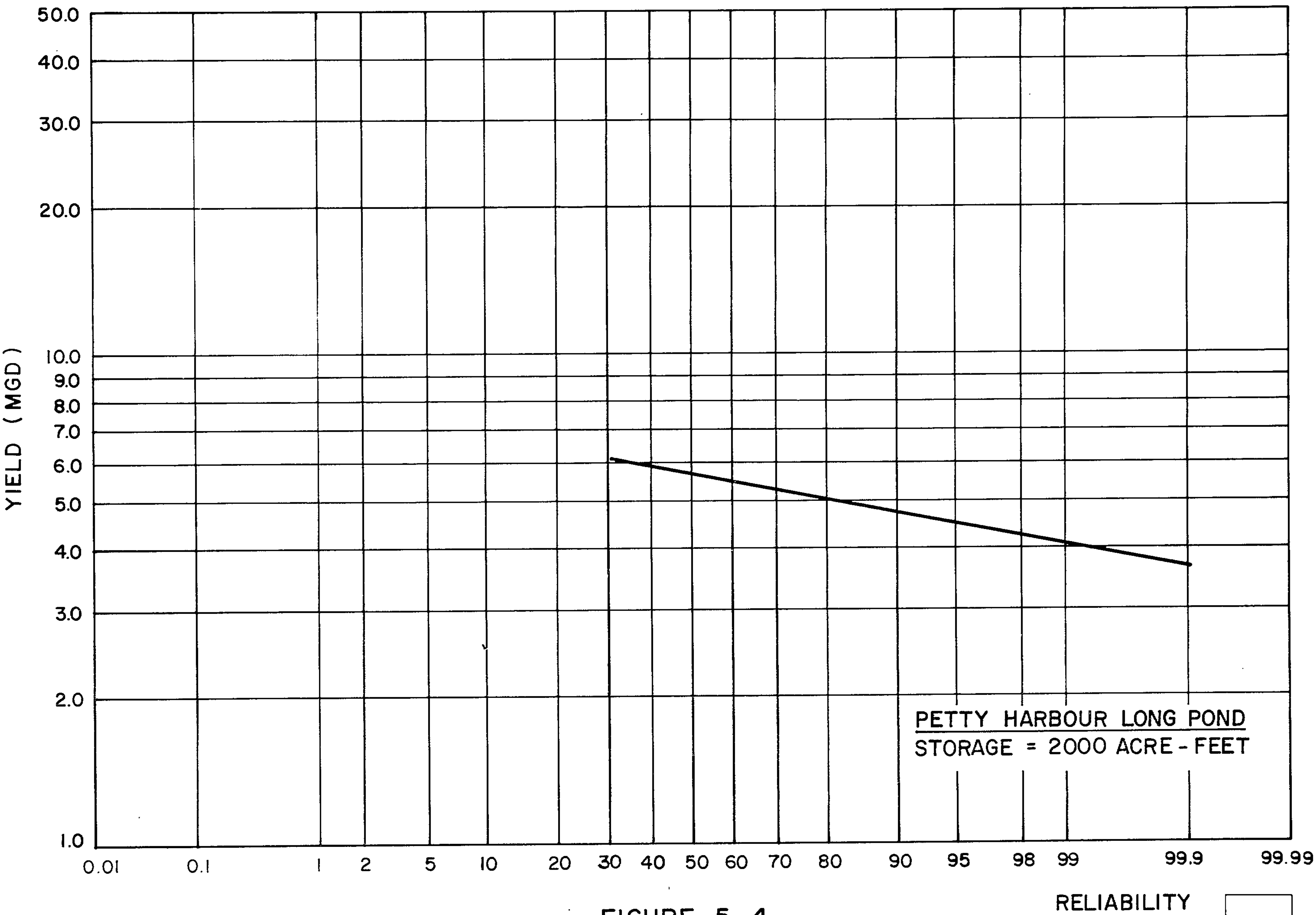

FIGURE 5·4

YIELD vs. RELIABILITY

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the reliable yield of Little Powers Pond. Due to lack of data concerning the storage capacity of the pond, three curves were plotted on Figure 5.5 which give yields comparable to storage volumes of 200, 250 and 300 acre-feet. It can readily be seen from these curves that at a reliability of 99 percent, the yield provided would be in the order of magnitude of 1 MGD with a storage capacity of between 250 and 300 acre-feet. It should be noted here that the catchment area (4.3 square miles) is large enough to produce a much higher yield; however, the limited existing storage restricts this capability.

4. Discussion

In viewing the results of the study included in this Chapter, several factors should be considered. These are the nature of the estimates of reliable yield, the rate of water withdrawal from each source, and the amount of water that will be available to maintain low flows in the streams downstream of each of the developments.

As mentioned earlier in this Chapter, the data used to simulate an inflow hydrograph was taken from the Petty Harbour power station records. This hydrograph was transposed to each study catchment by making the following adjustments:

(i) Correction for catchment area in the ratio of land area of the study catchment to the land area of the gauged catchment.

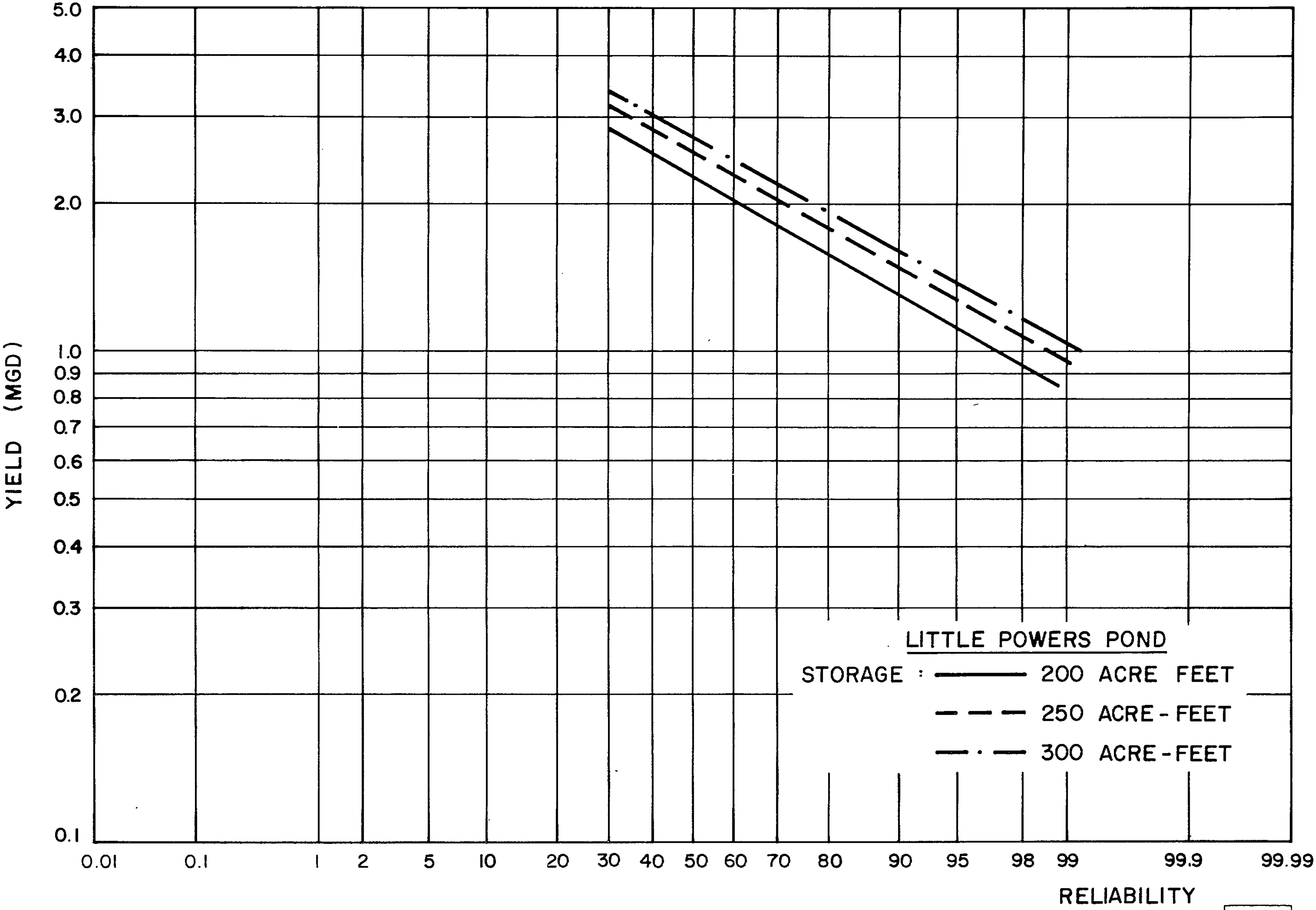

FIGURE 5·5

YIELD vs. RELIABILITY

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(ii) Correction for a safety factor of 0.9 to account for the discrepancies in converting power output to run-off, as reported by others [8].

(iii) Correction for the specific ratio of mean annual run-off to precipitation in each study catchment in accordance with the findings of a previous report [8].

The estimates we feel are conservative but realistic values of the reliable yields that can be obtained from each of the sources considered.

It should also be noted that with more information obtainable after a source is developed, it may be reasonable to revise the estimates of the reliable yield. Data of this nature may prove valuable when further development is contemplated. Accurate flow measuring equipment could be installed and operated in conjunction with any new facilities constructed as a part of management of the new regional supply system.

In comparing the reliable yield of the four sources of supply studied to the water requirements (as projected in Chapter 4), these alternative schemes appear to be viable, namely:

(i) A scheme based on Windsor Lake, Petty Harbour Long Pond and Bay Bulls Big Pond as the sources of supply.

(ii) A scheme based on Windsor Lake and Bay Bulls Big Pond as the sources of supply.

(iii) A scheme based on Windsor Lake and Thomas Pond as the sources of supply.

The water balance between the projected requirements and the above three supply alternatives is given in Table 5.1. In preparing this table, we have utilized the results of the econometric analysis, as presented in Chapter 6. In essence, this means that demand centres identified (in Chapter 4) as other areas "B" are excluded from the proposed regional system. These excluded centres are St. Phillips, Hogans Pond, Portugal Cove, Portugal Cove Road and Thorburn Road.

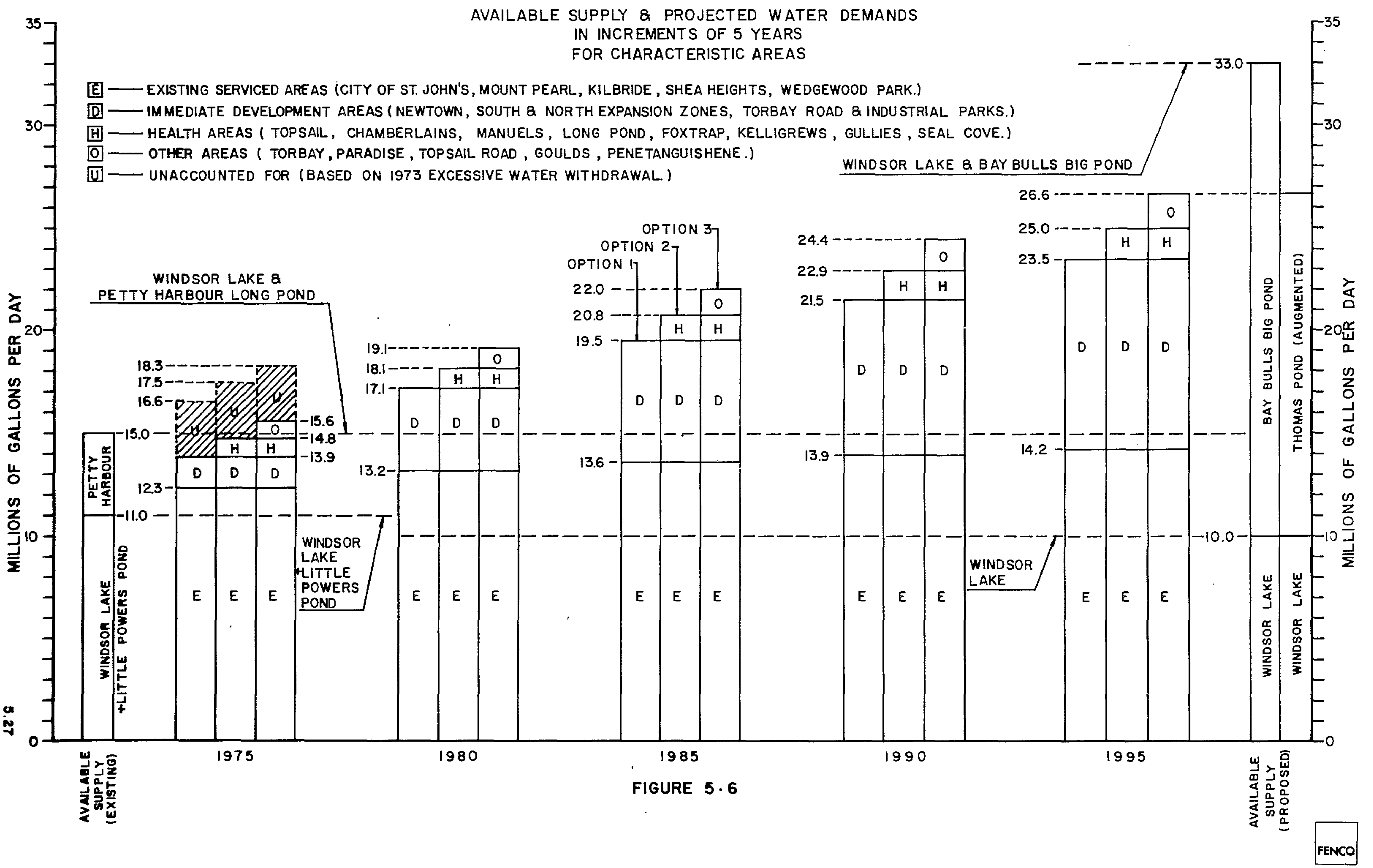
A bar-chart showing the water requirements for each category of need, i.e. the existing serviced areas, immediate development areas, health areas, and other areas "A", relative to present availability of water and water that will become available with the development of the new southern source is presented in Figure 5.6. The excessive withdrawal of water experienced in 1972 and 1973, and attributed to leakage (for further details see Chapter 4), was added to the total requirement in the year 1975. It can readily be seen from this figure that the water requirements of the region are quickly approaching the capacity of the existing supply sources. The situation may be more critical if measures are not taken to reduce excessive leakage. It should also be noted that the water requirement for 1975 as shown on the bar-chart is

TABLE 5.1

FUTURE WATER REQUIREMENTS AND SUPPLY ALTERNATIVES

	1975			1980			1985			1990			1995		
WATER REQUIREMENTS [1]	15.6			19.1			22.0			24.4			26.6		
Alternative Scheme	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
Supply Sources:															
- Windsor Lake	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10
- Petty Harbour L.P.	4	-	-	4	-	-	4	-	-	4	-	-	4	-	-
- Bay Bulls B. P.	1.6	5.6	-	5.1	9.1	-	8	12	-	10.4	14.4	-	12.6	16.6	-
- Thomas Pond	-	-	5.6	-	-	9.1	-	-	12	-	-	14.4	-	-	16.6
TOTAL SUPPLY	15.6	15.6	15.6	19.1	19.1	19.1	22.0	22.0	22.0	24.4	24.4	24.4	26.6	26.6	26.6

(1) Average daily annual



AVAILABLE SUPPLY & PROJECTED WATER DEMANDS
IN INCREMENTS OF 5 YEARS
FOR CHARACTERISTIC AREAS
E — EXISTING SERVICED AREAS (CITY OF ST. JOHN'S, MOUNT PEARL, KILBRIDE, SHEA HEIGHTS, WEDGEWOOD PARK.)
D — IMMEDIATE DEVELOPMENT AREAS (NEWTOWN, SOUTH & NORTH EXPANSION ZONES, TORBAY ROAD & INDUSTRIAL PARKS.)
H — HEALTH AREAS (TOPSAIL, CHAMBERLAINS, MANUELS, LONG POND, FOXTRAP, KELLIGREWS, GULLIES, SEAL COVE.)
O — OTHER AREAS (TORBAY, PARADISE, TOPSAIL ROAD, GOULDS, PENETANGUISHENE.)
U — UNACCOUNTED FOR (BASED ON 1973 EXCESSIVE WATER WITHDRAWAL.)
MILLIONS OF GALLONS PER DAY
WINDSOR LAKE & PETTY HARBOUR LONG POND
WINDSOR LAKE +LITTLE POWERS POND
WINDSOR LAKE
WINDSOR LAKE & BAY BULLS BIG POND
OPTION 1
OPTION 2
OPTION 3
PETTY HARBOUR
WINDSOR LAKE +LITTLE POWERS POND
BAY BULLS BIG POND
THOMAS POND (AUGMENTED)
AVAILABLE SUPPLY (EXISTING)
1975
1980
1985
1990
1995
AVAILABLE SUPPLY (PROPOSED)
FENCO

FIGURE 5·6

quite conservative. The early development of a new source of supply is, therefore, warranted and recommended.

In developing the new source of supply, it should be realized that both Bay Bulls Big Pond and Thomas Pond are multiple-use reservoirs. In addition to supplying water to the proposed regional system, some flow would be required to maintain a portion (at least) of the natural flow in the streams below the developments, and for the production of hydroelectric power.

Thomas Pond, with a small storage capacity, has a relatively small reliable yield. By providing more storage upstream of the existing pond, it would be possible to increase this amount but with a corresponding increase in the cost of supplying water. The mean annual runoff of this area is high enough to meet the multiple-use demands (including the St. John's Regional Water System) provided the use of Windsor Lake as a source of water is continued, and sufficient storage is provided in the Thomas Pond catchment to augment flow during dry periods.
Using Thomas Pond without augmentation would result in a gradual reduction of natural stream flow as the demand on the source increases. By the year 1985 the potential of this source would be completely utilized for water supply and the flows released to maintain stream flow would be unreliable and would occur only as overspill. Remedial action such as the construction of dams to provide increased storage would be required to meet the water demand

and maintain streamflow after 1985.

Bay Bulls Big Pond offers the best alternative to providing a reliable source of water supply, if it is desirable to minimize the impact of water abstraction on the natural flows in the streams, and power production. On rare occasions, when the reliable yield will be less than 23 MGD, the deficit may not be very critical. Using Windsor Lake and Bay Bulls Big Pond as the supply sources, 16.5 to 17.0 MGD of the reliable yield of 23 MGD from Bay Bulls Big Pond would be devoted to water supply. A reliable flow of up to 6 MGD may be available to maintaining flow in the stream, and for power production.

Subsequent to this analysis, a bathymetric survey of the pond was carried out, with the resulting map being shown on Drawing 5.1. Use of this information, together with data obtained from run off gauges, which should be installed in the catchment area, will ensure a well controlled and efficient water use management.

IV. SOURCES WATER QUALITY

1. General

Sources of water supply for domestic use should be examined in order to:

(i) Classify them with respect to general level of mineral constituents.

(ii) Demonstrate the absence of an excess of any particular constituent which would affect its potable quality.

(iii) Demonstrate the level of organic impurities.

(iv) Investigate behaviour, for example, corrosion potential.

(v) Determine the degree of clarity and ascertain the nature of matter in suspension.

(vi) Assess potability.

(vii) Detect and assess the degree of excremental pollution.

(viii) Assess and predict the growth intensity of native flora which may be troublesome, such as algae.

Once the above features have been satisfactorily established treatment requirements to render the source of supply safe for human consumption can be assessed.

2. Water Quality Records

Review of water quality records relative to the four sources of supply, considered for this project, shows the following:

(i) Windsor Lake: Samples for water quality determinations are taken intermittently at a frequency well below that normally required to comply with the "Canadian Drinking Water Standards and Objectives, 1968".

(ii) Petty Harbour Long Pond: Same as Windsor Lake.

(iii) Thomas Pond: Three sets of samples were collected for analysis in September 1968, June 1969, and May 1970, for incorporation in the Fisheries Service Progress Report No. 73. [1]

(iv) Bay Bulls Big Pond: Two sets of samples were collected for analysis; one on May 15, 1967 for incorporation in Newfoundland Design Associates Report [6], and the other on June 7, 1971 for Student Shoreline Program of Memorial University of Newfoundland. The latter report, unlike the former, and others, quantifies phosphates and toxic elements like copper and lead.

The limited analytical records available on the physical and chemical quality of the above four sources is contained in Appendix I.

The Municipal Service Plan report [7] does not quantify the physical and chemical quality of the water in Windsor Lake and Petty Harbour Long Pond. It does, however, present statistical information on the bacteriological quality of these waters for the years 1967 - 70. Out of a total yearly number of samples as shown in Table 5.2, only in 1969 (Windsor Lake), and in 1970 (Petty Harbour Long Pond) did these sources not meet the objective of the Canadian Drinking Water Standards for water requiring treatment solely by chlorination (see Appendix II).

TABLE 5.2

SAMPLES COLLECTED FOR BACTERIOLOGICAL TESTS

	1967	1968	1969	1970
Windsor Lake	63	71	31	79
Petty Harbour Long Pond	46	56	46	47

In view of the above cited information, the scant data on the physical and chemical characteristics of the sources water, and the necessity to develop a new source of supply, it was recommended and accepted that a sanitary survey should be carried out.

3. Sanitary Survey

A sanitary survey to establish the water quality of a source of supply is analogous to a hydrological survey carried out to determine the source's reliable yield. It will be appreciated that the importance of both of these surveys cannot be over-emphasized.

The objective of the sanitary survey is to provide the Engineer with the physical, chemical, bacteriological, and biological characteristics of the source water so that he can examine them, in accordance with the features outlined in Section IV.1 of this Chapter. This in turn will enable the Engineer to make a decision as to the suitability of the water for supply for domestic purposes, and if treatment is deemed required, the amenability of the water for treatment, the degree of treatment and the processes to be applied.

It is evident from the foregoing that the survey to establish the quality of a source water should preferably extend to include different meteorological, run-off, and catchment activity conditions, at least for some samples, depending on the facets which reveal areas of enquiry.

In accordance with the above approach, and preliminary engineering, assessment and considerations, it was decided to perform the sanitary survey on Windsor Lake and Thomas Pond waters; this was later expanded to include Bay Bulls Big Pond. The original objective was to extend this survey from the time the above impounding reservoirs were free of ice to the time they will be covered again with ice.

However, due to laboratory difficulties, the survey period lasted six months, from May 1973 to October 1973 (inclusive). The scope of the survey, which complies with the requirements of the Provincial Division of Environmental Health can be summarized as follows:

a. Bacteriological Tests

One sample taken daily Sunday through Thursday. In addition, during the first two weeks of the survey six samples were collected on Sundays from pre-selected locations.

b. Physical and Chemical Tests

One sample per week. In addition, three representative samples were collected during the first batch of sampling.

c. Tests for Toxicants and Biocides

Scan tests, one in July and one in September. Samples were taken from three locations at each reservoir, and from two different depths, except for Windsor Lake where only a surface sample was collected.

d. Biological Tests

Tests for algae, and other deleterious micro-organisms, on a bi-weekly basis.

e. $B.O.D._5$

Bi-weekly tests.

A detailed outline of the sanitary survey programme is contained in Appendix III.

Originally, it was intended that the samples would be collected from the designated point of draw-off; at Windsor Lake near the present draw-off point, at Thomas Pond near the diversion canal, and at Bay Bulls Big Pond near the outlet conduit. However, this was proven to be impractical and samples were collected from a steep section of the shoreline in a general direction of the designated intake. Once a month "depth" samples were collected for correlation with the weekly samples (except for Windsor Lake where this is impossible).

Analysis for coliform organisms, total and faecal was done by the Membrane Filter (MF) method and the Most Probable Number (MPN) multiple tube fermentation method. Experience has shown that the MF technique, although now acceptable as a standard testing procedure, does not measure precisely the same coliform spector as the MPN technique does. For this reason, both of these methods have been used to establish conformity or correlation between the two. Once this was established satisfactorily, only the MF method was used for analysis.

Physical and chemical determinations were done in accordance with the 13th edition of "Standard Methods" (APHA, AWWA, & WPCP).

4. Assessment of Water Quality

a. General

The assessment of the water quality of Windsor Lake, Bay Bulls Big Pond, and Thomas Pond, as presented in this section, is based on the analytical results of the sanitary survey previously described.

Quite typical for surface sources, the water of all of the above three reservoirs can be classified as "soft". This characteristic along with the low alkalinity and low mineral content, specifically chlorides and sulphates, render the water corrosive. The low pH value, indicative of acidic soil, requires correction prior to supply.

Previous reports [7, 8], mention that sodium ion concentration may be of concern because of the proximity of the sea and its salinity (possibly by wind blown spray). Standards for sodium concentrations have not been formulated. Usually sodium content of drinking water is considered relative to diets of patients who must restrict their intake of this cation. Sodium content in the order of magnitude of 10 mg/l is considered acceptable. The data gathered during the sanitary survey indicates a concentration below this level.

A summary of the physical, chemical and biological characteristics of the water of Windsor Lake, Bay Bulls Big Pond, and Thomas Pond is presented in Tables 5.3, 5.4, and 5.5, respectively.

TABLE 5.3

SUMMARY OF WATER QUALITY - WINDSOR LAKE

CONSTITUENT	AVERAGE VALUE	RANGE OF 90% OF SAMPLES	TOTAL NUMBER OF SAMPLES
PHYSICAL FACTORS			
Color-TCU	6	0 - 20	13
Turbidity-JTU	0.62	0.18 - 1.80	16
Specific Conductance (umho/cm)	44.8	42.9 - 46.8	16
CHEMICAL FACTORS (in mg/l)			
Calcium	0.93	0.81 - 0.98	15
Chloride	8.8	5.6 - 10.5	16
Copper	0.002	L0.001 - 0.004	13
Magnesium	0.80	0.67 - 0.94	15
Manganese	0.029	0.013 - 0.073	14
Phosphate	L0.01	N.A.	15
Dissolved Solids	34	20 - 44	14
Sulphate	4.3	3.5 - 5.0	16
Zinc	0.027	L0.002 - 0.035	14
Sodium	5.7	5.2 - 6.5	10
Floride	L0.05	N.A.	17
CORROSION & INCRUSTATION FACTORS (in mg/l)			
Alkalinity	0.72	0.30 - 1.53	15
Hardness	5.60	5.12 - 6.15	15
pH Units	N.A.	5.02 - 5.63	17
BACTERIOLOGIC FACTORS (in mg/l)			
B.O.D.	0.68	0.5 - 1.0	8
TOXIC FACTORS (in mg/l)			
Arsenic	0.008	L0.005 - 0.016	6
Cadmium	L0.001	L0.001 - 0.005	4
Chromium	L0.005	N.A.	4
Lead	0.003	L0.001 - 0.005	4
Nitrate Nitrogen	0.147	L0.005 - 0.558	11
Nitrite Nitrogen	L0.005	N.A.	11

TABLE 5.3 (cont'd)

SUMMARY OF WATER QUALITY - WINDSOR LAKE

CONSTITUENT	AVERAGE VALUE	RANGE OF 90% OF SAMPLES	TOTAL NUMBER OF SAMPLES
	BIOCIDIC FACTORS (in $\mu g/l$)		
α - BHC	L0.005	N.A.	7
Lindane	L0.005	N.A.	7
Heptachlor	L0.005	N.A.	7
Aldrin	L0.005	N.A.	7
Heptachlor Epoxide	L0.005	N.A.	7
p,p' - DDD	L0.005	N.A.	7
p,p' - DDT	L0.010	N.A.	7
p,p' - Methoxychlor	L0.050	N.A.	7
Endosulfan Isomers (α & β)	L0.005	N.A.	7
Endrin	L0.005	N.A.	7
α - Chlordane	L0.005	N.A.	7
γ - Chlordane	L0.005	N.A.	7
O,p - DDT	L0.005	N.A.	7
2,4 - D	L0.010	N.A.	5
2,4,5 - T	L0.010	N.A.	5
MCPA	L0.05	N.A.	5
Malathion	L0.020	N.A.	9
Parathion	L0.020	N.A.	9
Methyl. Parathion	L0.020	N.A.	9
Fenitrothion	L0.020	N.A.	9
p,p' - DDE	L0.005	N.A.	7
Dieldrin	L0.005	N.A.	7

TABLE 5.4

SUMMARY OF WATER QUALITY - BAY BULLS BIG POND

CONSTITUENT	AVERAGE VALUE	RANGE OF 90% OF SAMPLES	TOTAL NUMBER OF SAMPLES
		PHYSICAL FACTORS	
Color - TCU	19	5 - 30	27
Turbidity - JTU	1.03	0.34 - 2	11
Specific Conductance (umho/cm)	35.6	34 - 37	10
		CHEMICAL FACTORS (in mg/l)	
Calcium	0.80	0.72 - 0.87	11
Chloride	4.8	3.0 - 6.5	10
Copper	0.002	L0.001 - 0.004	11
Magnesium	0.56	0.43 - 0.62	11
Manganese	0.015	0.004 - 0.025	15
Phosphate	L0.01	N.A.	11
Dissolved Solids	24.3	4 - 31	11
Sulphate	2.1	L1 - 3	10
Zinc	0.007	L0.001 - 0.010	11
Sodium	4.4	4.0 - 4.8	10
Floride	L0.05	N.A.	10

TABLE 5.4 (Cont'd)

SUMMARY OF WATER QUALITY - BAY BULLS BIG POND

CONSTITUENT	AVERAGE VALUE	RANGE OF 90% OF SAMPLES	TOTAL NUMBER OF SAMPLES
CORROSION & INCRUSTATION FACTORS (in mg/l)			
Alkalinity	1.07	0.5 - 1.51	8
Hardness	4.30	3.56 - 4.72	11
pH Units	N.A.	5.3 - 6.18	17
BACTERIOLOGIC FACTORS (in mg/l)			
B.O.D.	0.68	0.1 - 1.3	15
TOXIC FACTORS (In mg/l)			
Cadmium	L0.001	L0.001 - 0.002	9
Chromium	L0.001	L0.001 - 0.001	9
Lead	L0.001	N.A.	9
Nitrate Nitrogen	0.044	L0.005 - 0.055	10
Nitrite Nitrogen	L0.005	N.A.	10

TABLE 5.5

SUMMARY OF WATER QUALITY - THOMAS POND

CONSTITUENT	AVERAGE VALUE	RANGE OF 90% OF SAMPLES	TOTAL NUMBER OF SAMPLES
	PHYSICAL FACTORS		
Color - TCU	75	40 - 110	17
Turbidity - JTU	1.62	0.68 - 2.80	17
Specific Conductance (UMHO/CM)	33.0	30.0 - 40.0	17
	CHEMICAL FACTORS (in mg/l)		
Calcium	0.92	0.79 - 1.16	15
Chloride	7.4	5.5 - 9.0	17
Copper	0.004	0.001 - 0.025	15
Magnesium	0.64	0.53 - 0.86	15
Manganese	0.089	0.002 - 0.169	17
Phosphate	0.032	0.020 - 0.138	15
Dissolved Solids	42.3	27.5 - 71.0	16
Sulphate	3.5	3.0 - 4.5	16
Zinc	0.004	L0.002 - 0.010	13
Sodium	3.7	3.5 - 4.2	10
Floride	L0.05	N.A.	17
	CORROSION & INCRUSTATION FACTORS (in mg/l)		
Alkalinity	1.15	0.30 - 2.01	15
Hardness	4.94	4.63 - 5.50	15
pH Units	N.A.	4.92 - 5.67	19
	BACTERIOLOGIC FACTORS (in mg/l)		
B.O.D.	0.81	0.3 - 1.8	8

TABLE 5.5 (Cont'd)

SUMMARY OF WATER QUALITY - THOMAS POND

CONSTITUENT	AVERAGE VALUE	RANGE OF 90% OF SAMPLES	TOTAL NUMBER OF SAMPLES
TOXIC FACTORS (in mg/l)			
Arsenic	0.009	0.005 - 0.017	12
Cadmium	L0.001	N.A.	5
Chromium	L0.005	N.A.	5
Lead	L0.005	N.A.	5
Nitrate Nitrogen	0.147	L0.005 - 0.558	11
Nitrite Nitrogen	L0.005	N.A.	11
BIOCIDIC FACTORS (in μg/l)			
α - BHC	L0.005	N.A.	17
Lindane	L0.005	N.A.	17
Heptachlor	L0.005	N.A.	17
Aldrin	L0.005	N.A.	17
Heptachlor Epoxide	L0.005	N.A.	17
p,p' - DDD	L0.005	N.A.	17
p,p' - DDT	L0.010	N.A.	17
p,p' - Methoxychlor	L0.050	N.A.	17
Endosulfan Isomers (α & β)	L0.005	N.A.	17
Endrin	L0.005	N.A.	17
α - Chlordane	L0.005	N.A.	17
γ - Chlordane	L0.005	N.A.	17
O,p - DDT	L0.005	N.A.	17
2,4 - D	L0.010	N.A.	18
2,4,5 - T	L0.010	N.A.	18
MCPA	L0.05	N.A.	18
Malathion	L0.020	N.A.	15
Parathion	L0.020	N.A.	15
Methyl. Parathion	L0.020	N.A.	15
Fenitrothion	L0.020	N.A.	15
p,p' - DDE	L0.005	N.A.	17
Dieldrin	L0.005	N.A.	17

Water quality constituents of significant importance warranting an elaborate discussion are turbidity, colour, iron, manganese, coliform organisms, and algae.

Turbidity and colour are usually objected to by the majority of the consumers. Colour primarily on aesthetic grounds, and turbidity on the basis of the evidence that freedom from disease organisms is associated with freedom from turbidity, and that freedom from taste and odour requires clarity of the water. Typical to soft surface water, the turbidity of the sources water is very low, however, the colour intensity is moderate to relatively high, especially in the case of Thomas Pond.

Iron and manganese are highly objectionable elements in water supply, and are often associated with soft surface water of relatively high colour intensity. Both of these elements cause staining of laundry and form precipitates. A remarkable concentration of these elements was found in Thomas Pond water.

Coliform organisms are indicative of hazardous contamination. Water of high bacteriologic quality results in much improved health conditions with respect to certain significant illnesses, such as intestinal disturbances.

Algae not only affect appearance, but may result in taste and odour problems and filter clogging.

Species of algae identified during the sanitary survey belong primarily to the diatom group. Some flagellates and blue-green algae were also identified. However, the algae population in all three sources surveyed was reported to exist in scant concentrations. Quantification of algae species was performed on water samples collected on August 22, 1973. The results obtained are given in Table 5.6. Since the most diversified species of algae at the highest concentration was found to exist in Bay Bulls Big Pond, a second sample for quantitative analysis was taken from this pond on October 11, 1973 (a period considered to be critical, especially for diatoms). The complete results for algae in Bay Bulls Big Pond water are given in Table 5.7.

A detailed assessment of the above constituents, relative to each source water, follows later. In addition, a statistical analysis of colour, iron and turbidity was performed for recurrence probability. The results for colour are shown on Figures 5.7, 5.8 and 5.9; for iron on Figures 5.10 and 5.11; for turbidity on Figures 5.12, 5.13, and 5.14. Coliform organism data is given in Figures 5.15, 5.16, 5.17. All other water quality constituents tested and not pointed out herein were generally found to be within the objectives adopted for this project. (see presentation in Chapter 4).

TABLE 5.6

SUMMARY OF ALGAE SPECIES AND CONCENTRATIONS

SPECIES	CONCENTRATION - AUG. 22			UNITS	GROUP	COMMENTS
	Windsor Lake	Bay Bulls Big Pond	Thomas Pond			
Tabellaria		2.7×10^4	9.6×10^2	Cells/Litre	Diatoms	Filter Clogging, Taste & Odour
Asterionella	1.7×10^3	2.9×10^4		Cells/Litre	Diatoms	Filter Clogging, Taste & Odour
Microcystis	6.0×10^3	2.9×10^4	4.8×10^3	Colonies/Litre	Blue-Green	Taste & Odour
Gloeocapsa	5.1×10^4	1.5×10^5	3.8×10^3	Cells/Litre	Blue-Green	Taste & Odour
Merismopedia	10.5×10^3			Colonies/Litre		
Mallomonas		4.8×10^3		Cells/Litre	Flagellates	Taste & Odour
Synedra	5.3×10^3	5.8×10^3		Cells/Litre	Diatoms	Filter Clogging, Taste & Odour
Flagellates		9.6×10^3	3.8×10^3	Cells/Litre	Flagellates	Taste & Odour

TABLE 5.7

SUMMARY OF ALGAE - BAY BULLS BIG POND

SPECIES	CONCENTRATION		UNITS	GROUP	COMMENTS
	Aug. 22	Oct. 11			
Tabellaria	2.7×10^4	118×10^4	Cells/Litre	Diatoms	Filter Clogging, Taste & Odour
Asterionella	2.9×10^4	33×10^4	Cells/Litre	Diatoms	Filter Clogging, Taste & Odour
Microcystis	2.9×10^4	1.27×10^4	Colonies/Litre	Blue-Green	Taste & Odour
Gloeocapsa	1.5×10^5	7×10^4	Cells/Litre	Blue-Green	Taste & Odour
Merismopedia		9×10^4	Colonies/Litre		
Desmids		1.27×10^4	Cells/Litre		
Stephanodiscus		0.72×10^4	Cells/Litre	Diatoms	Filter Clogging, Taste & Odour Surface
Mallamonas	4.8×10^3	3.6×10^4	Cells/Litre	Flagellates	Taste & Odour
Synedra	5.8×10^3		Cells/Litre	Diatoms	Filter Clogging, Taste & Odour
Flagellates	9.6×10^3		Cells/Litre	Flagellates	Taste & Odour

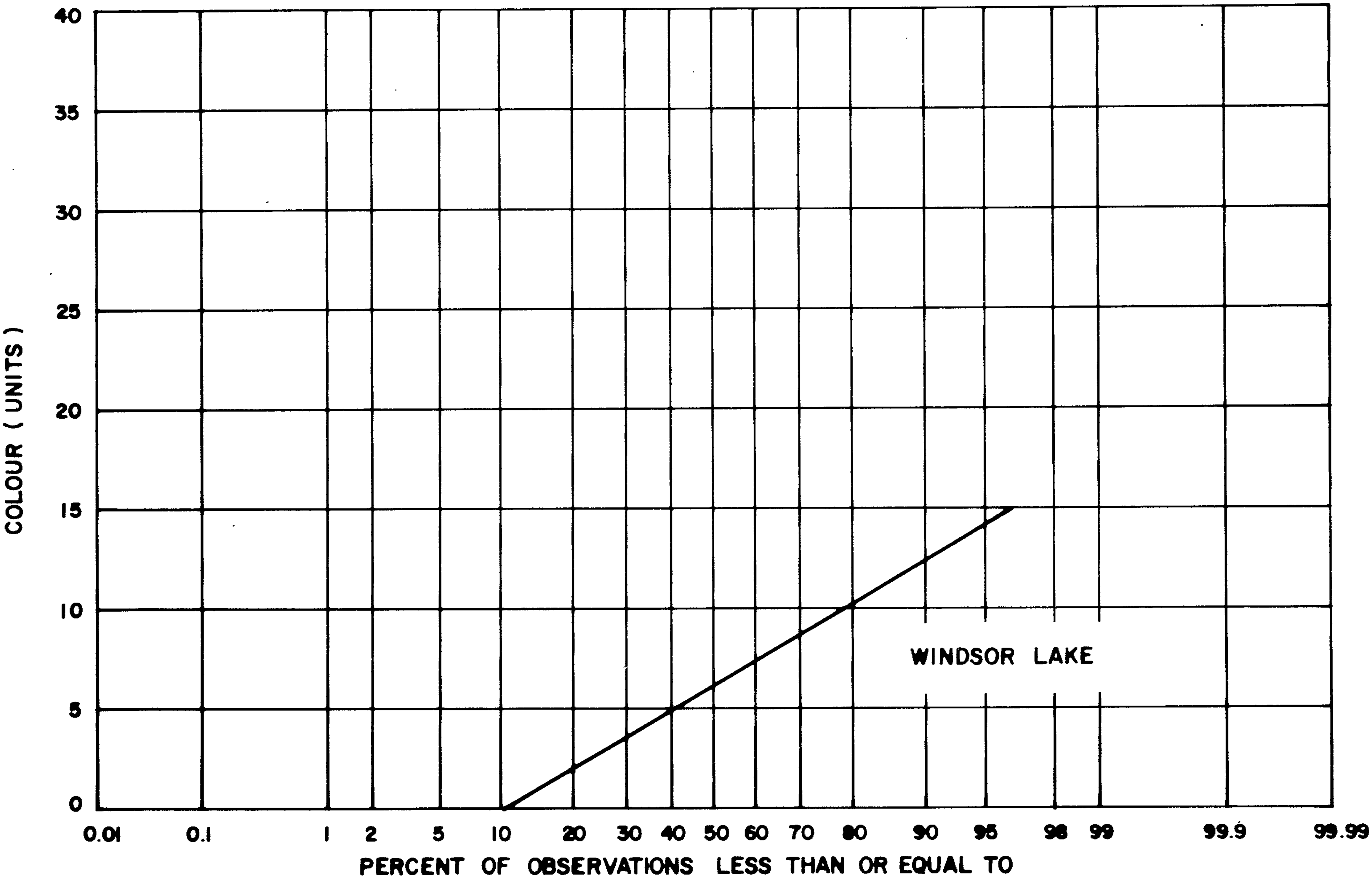


FIGURE 5·7

SUMMARY OF COLOUR ANALYSES

FENCO

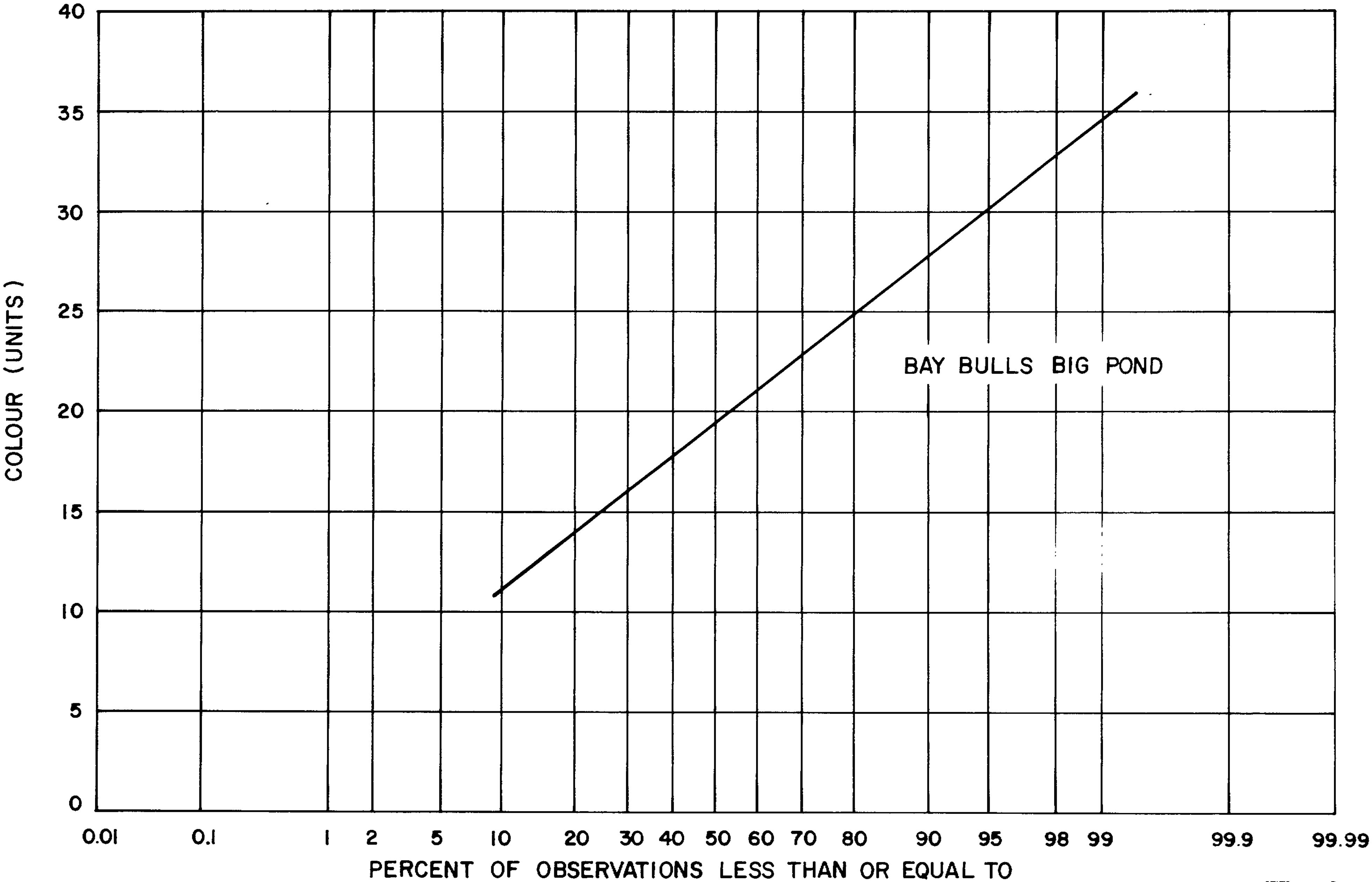


FIGURE 5·8
SUMMARY OF COLOUR ANALYSES

FENCO.

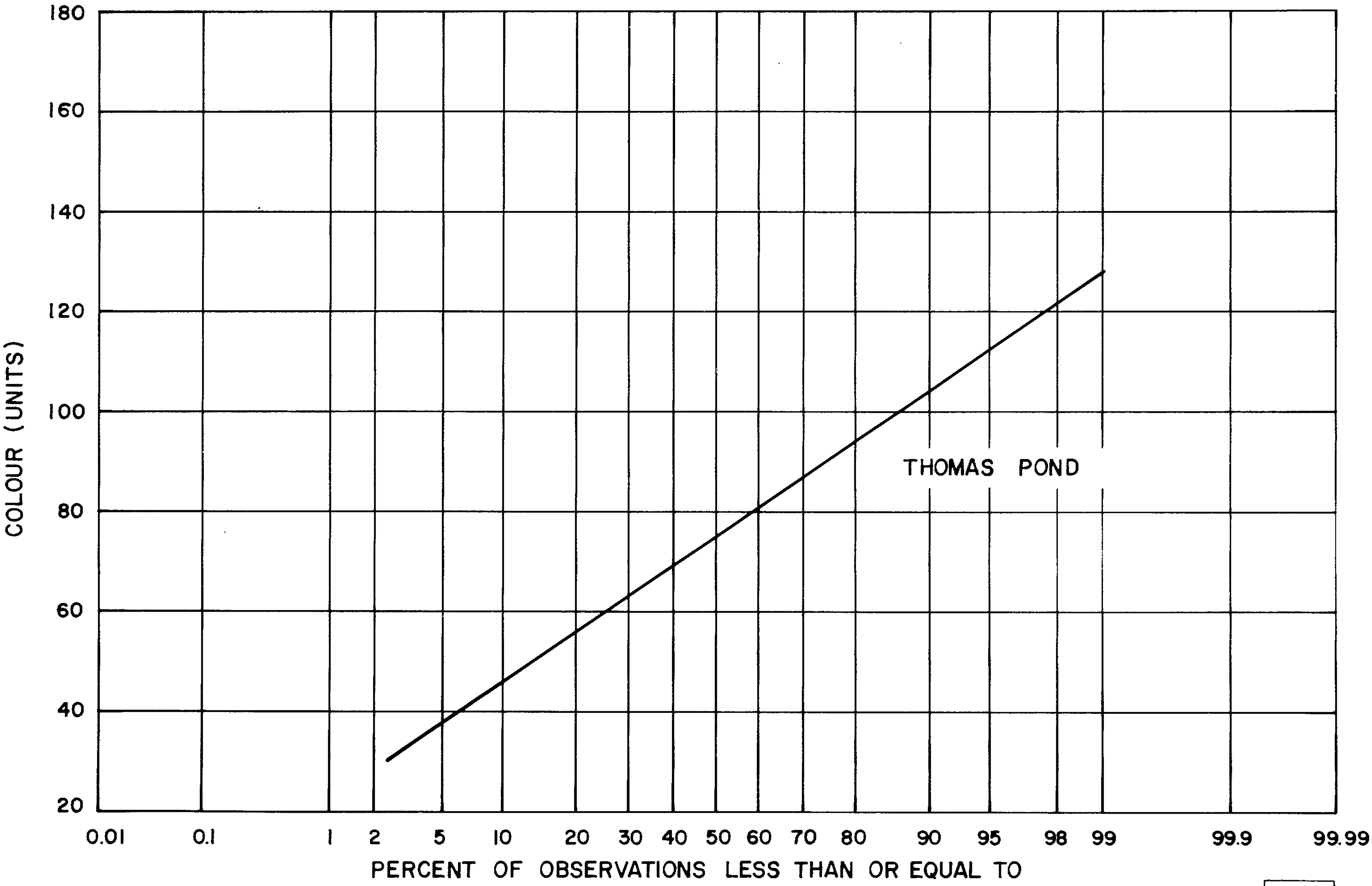


FIGURE 5·9
SUMMARY OF COLOUR ANALYSES

FENCO

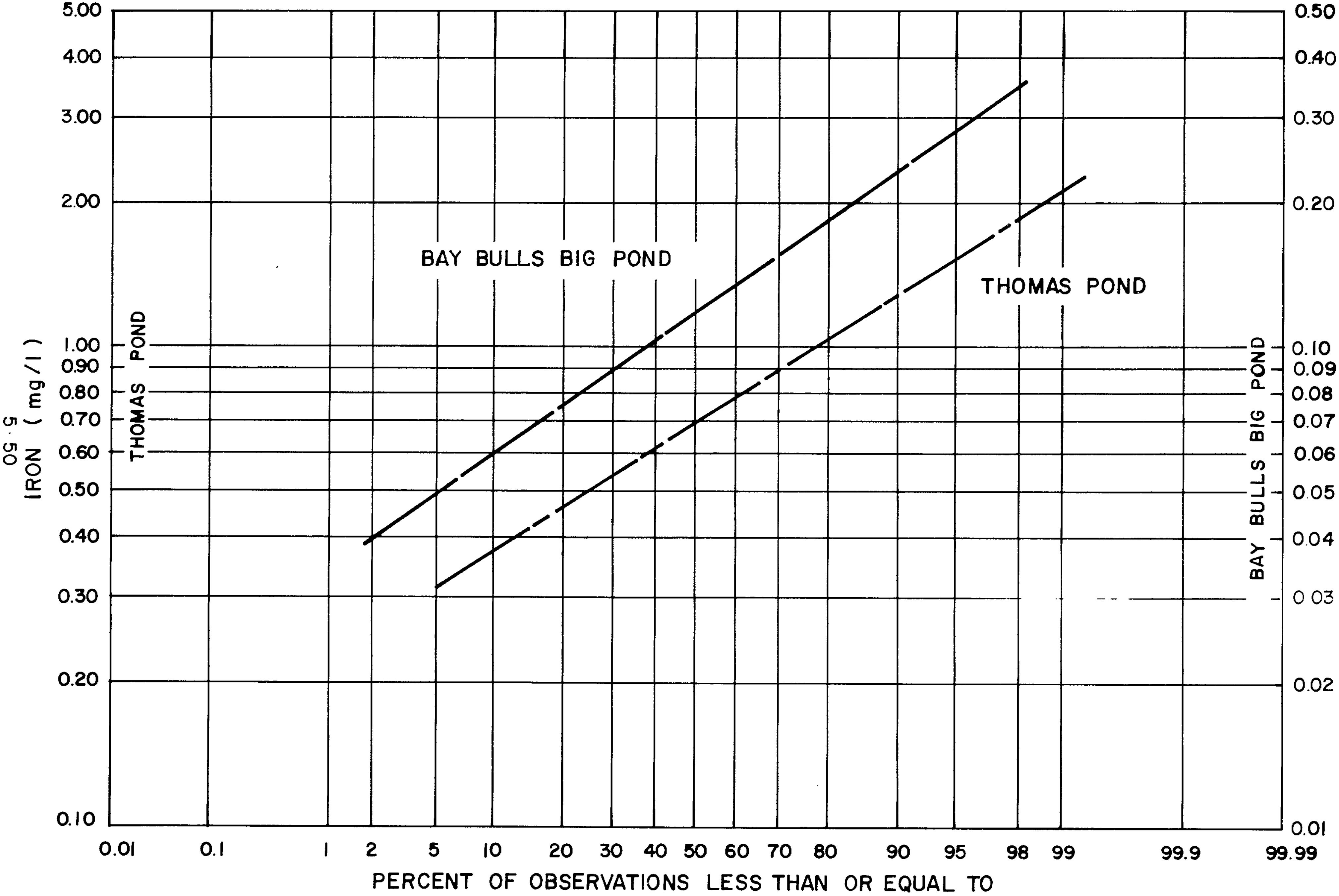


FIGURE 5·10

SUMMARY OF IRON ANALYSES

FENCO

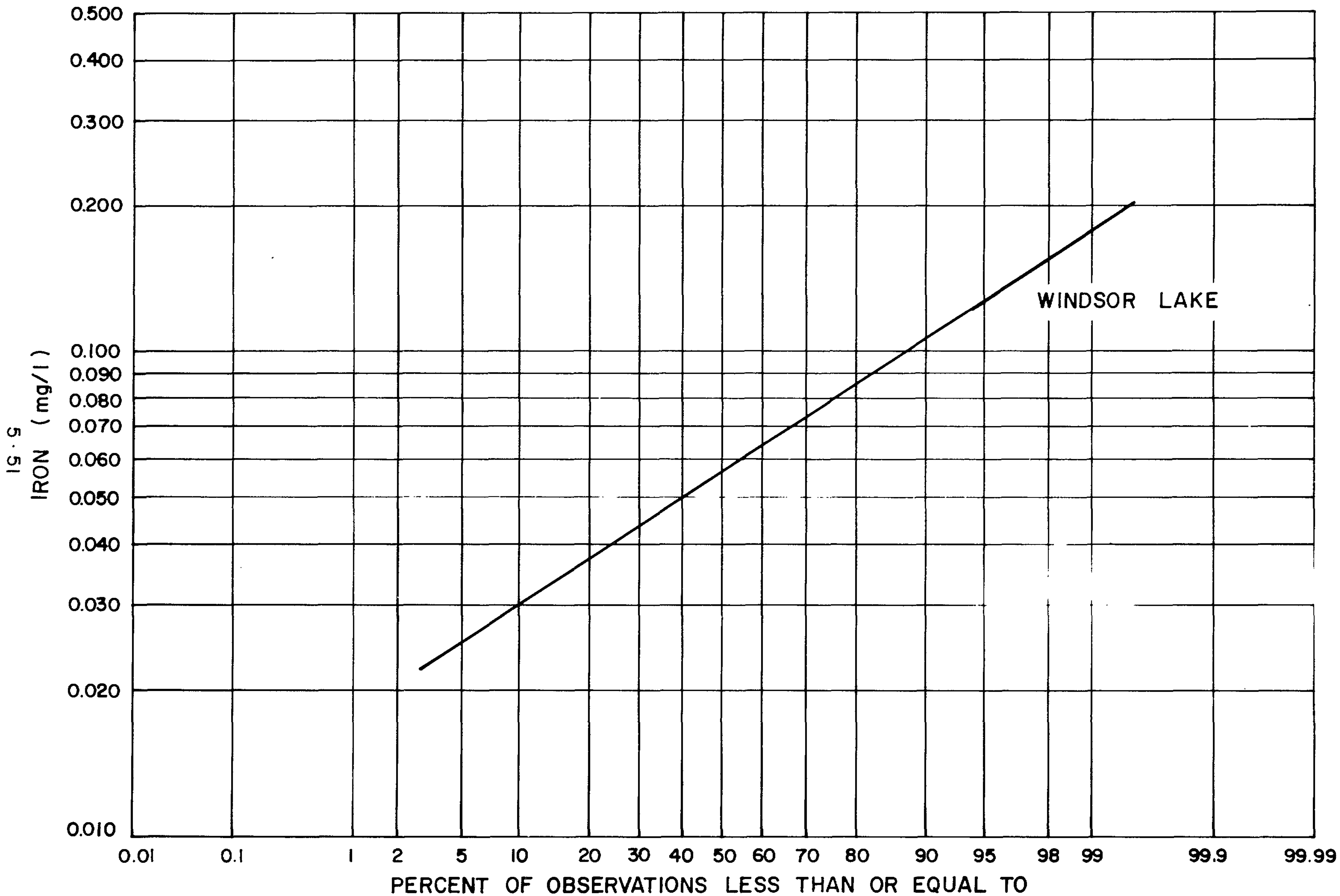


FIGURE 5·11
SUMMARY OF IRON ANALYSES

FENCO

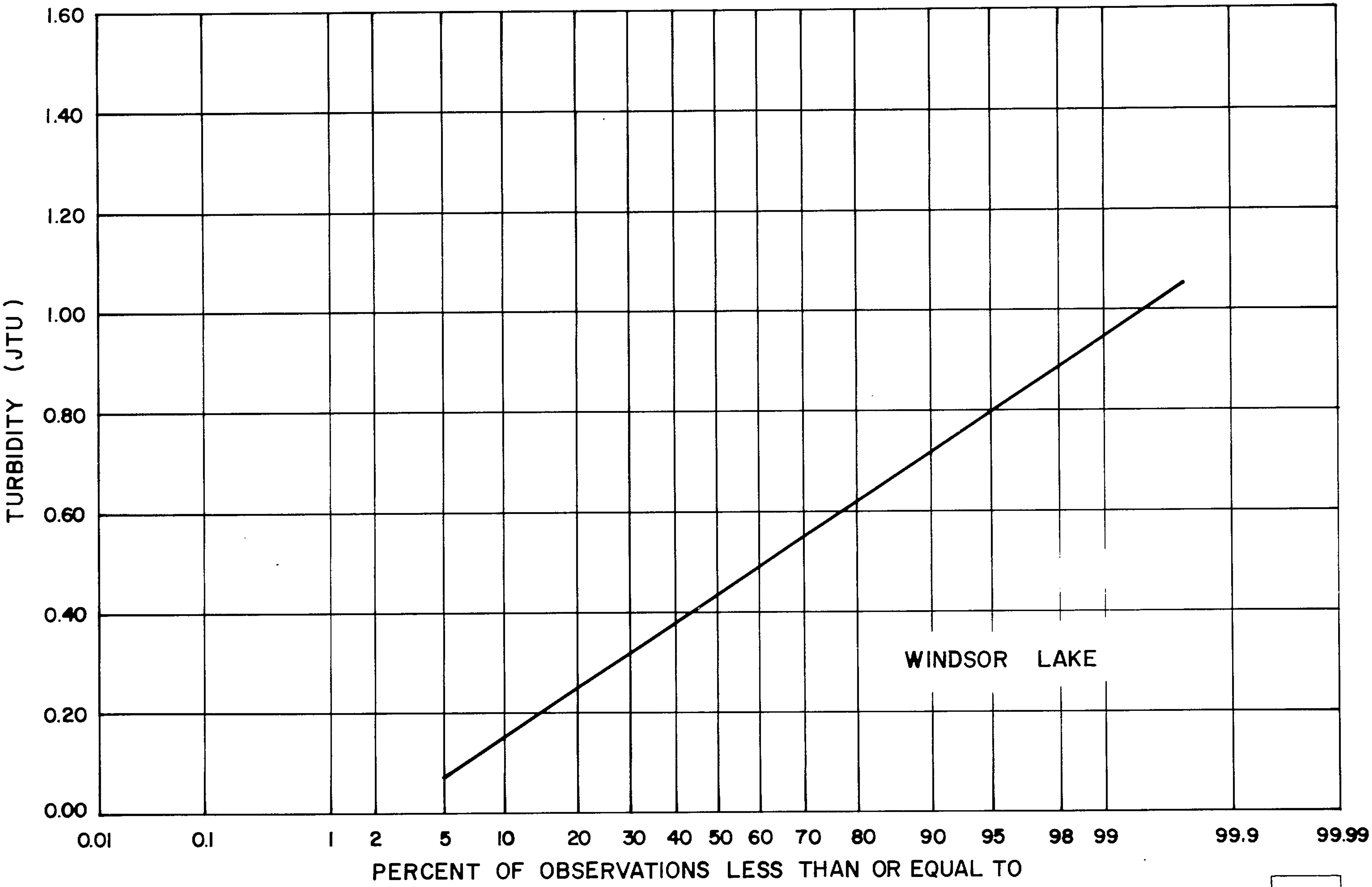


FIGURE 5·12

SUMMARY OF TURBIDITY ANALYSES

FENCO

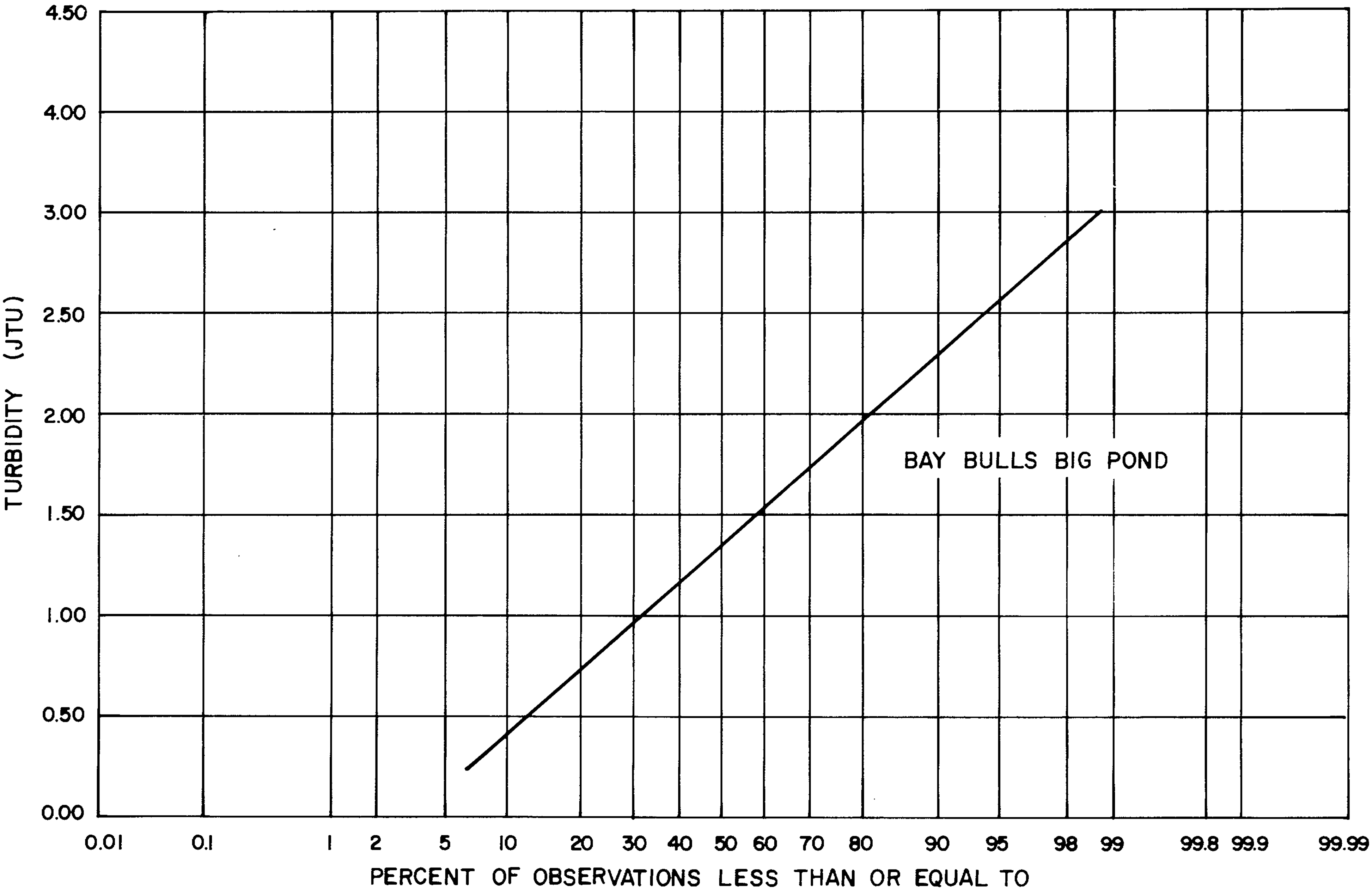


FIGURE 5·13
SUMMARY OF TURBIDITY ANALYSES

FENCO

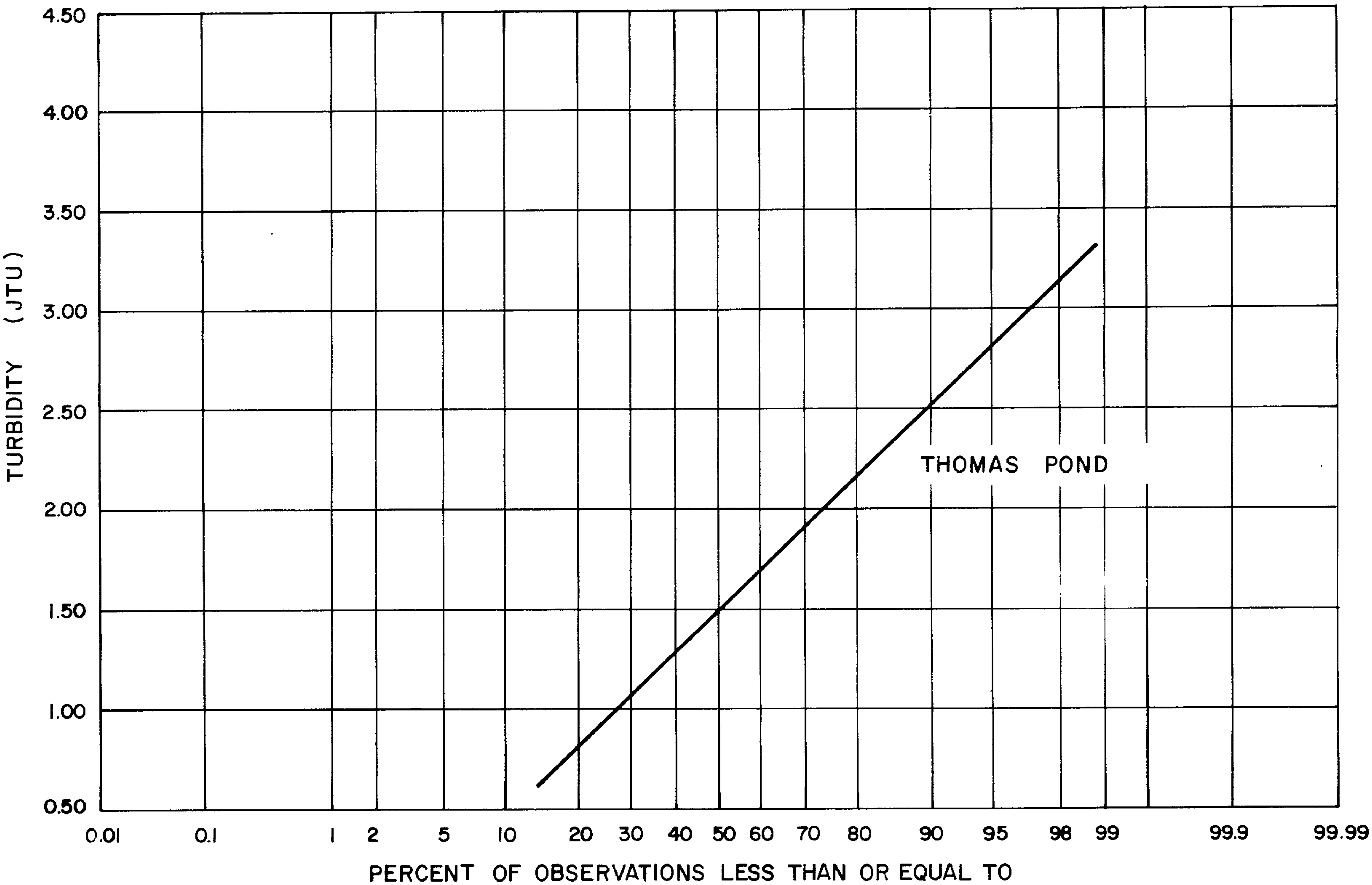


FIGURE 5·14
SUMMARY OF TURBIDITY ANALYSES

FENCO

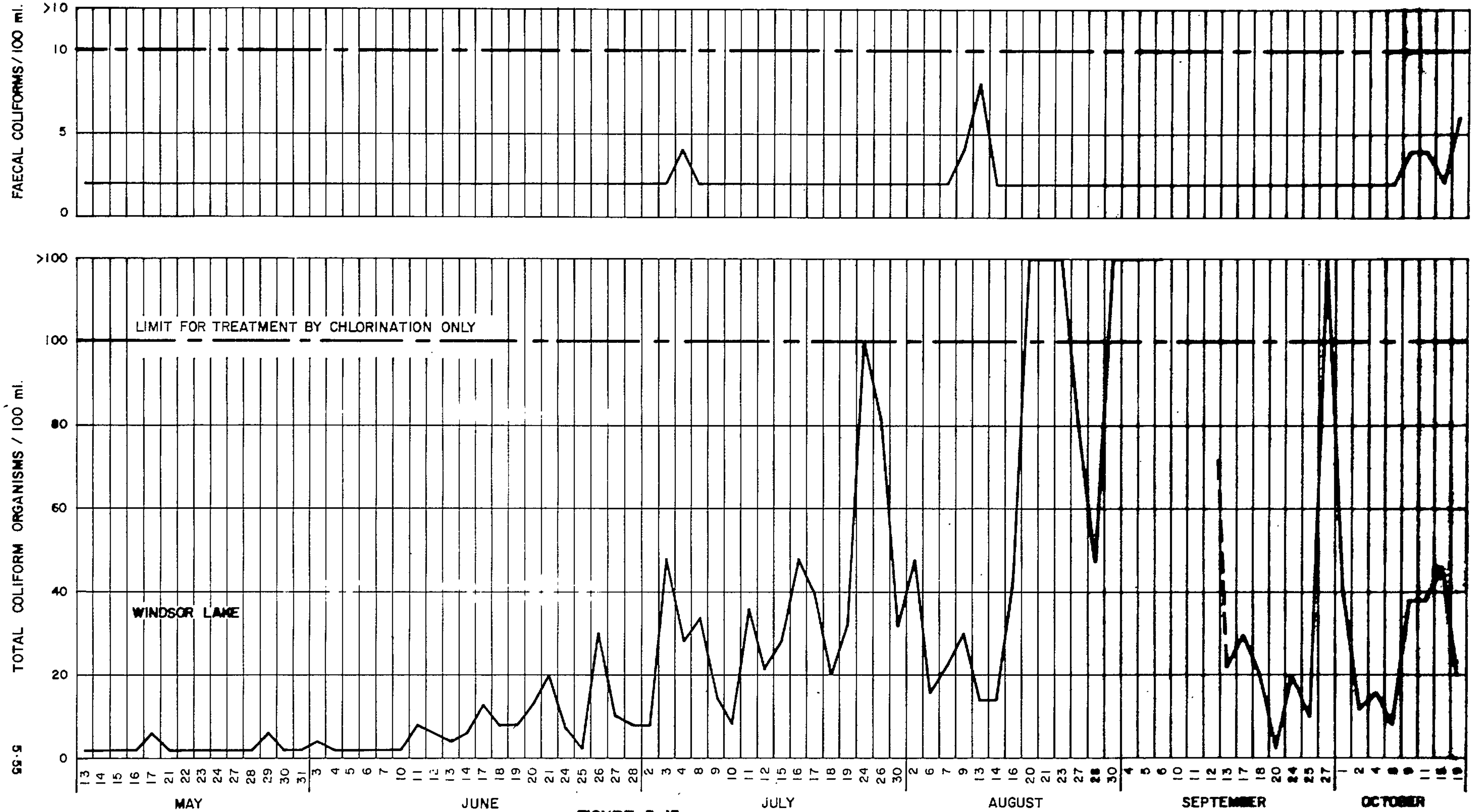


FIGURE 5-15
SUMMARY OF BACTERIOLOGIC ANALYSES

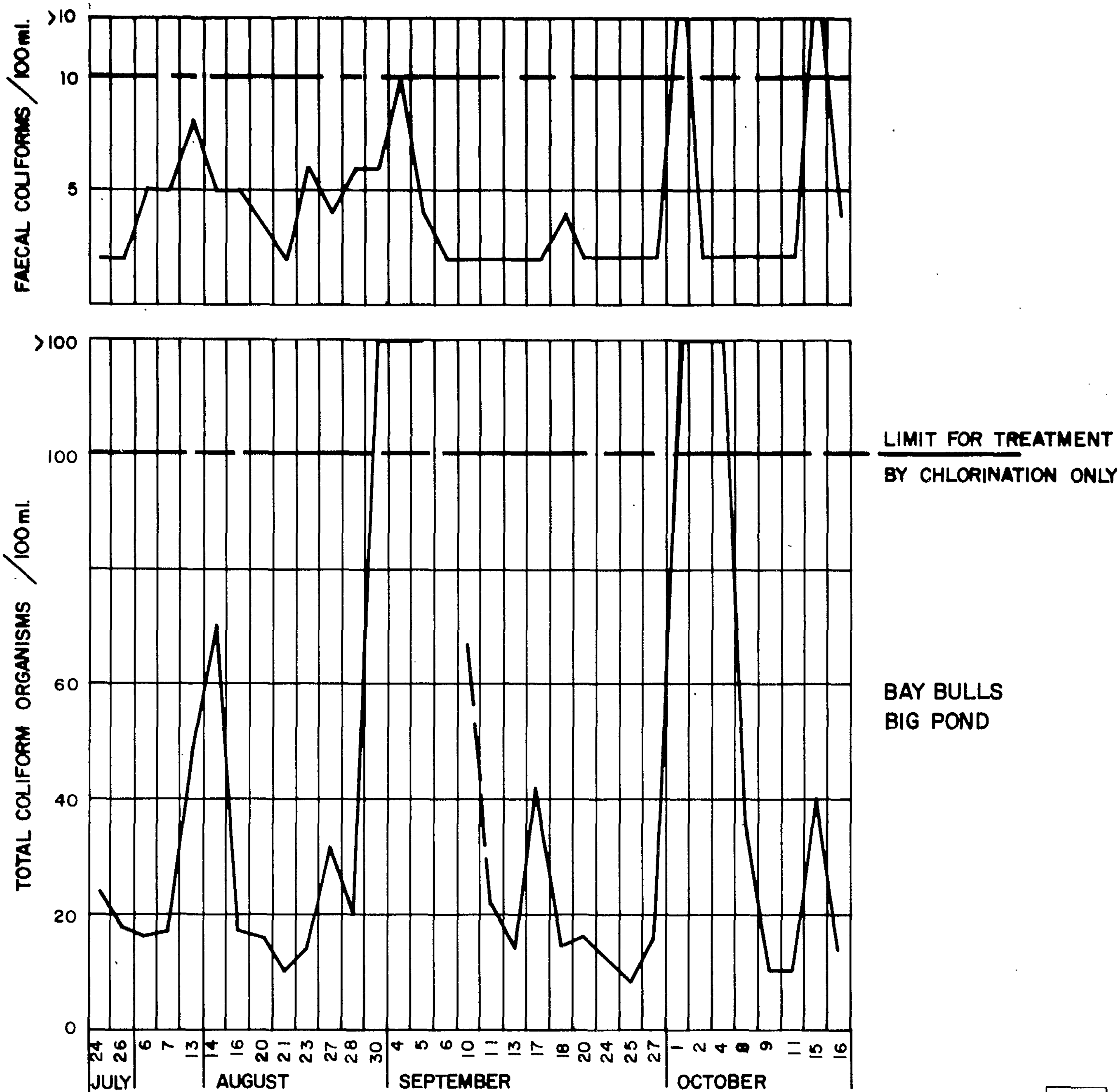


FIGURE 5·16
SUMMARY OF BACTERIOLOGIC ANALYSES

FENCO

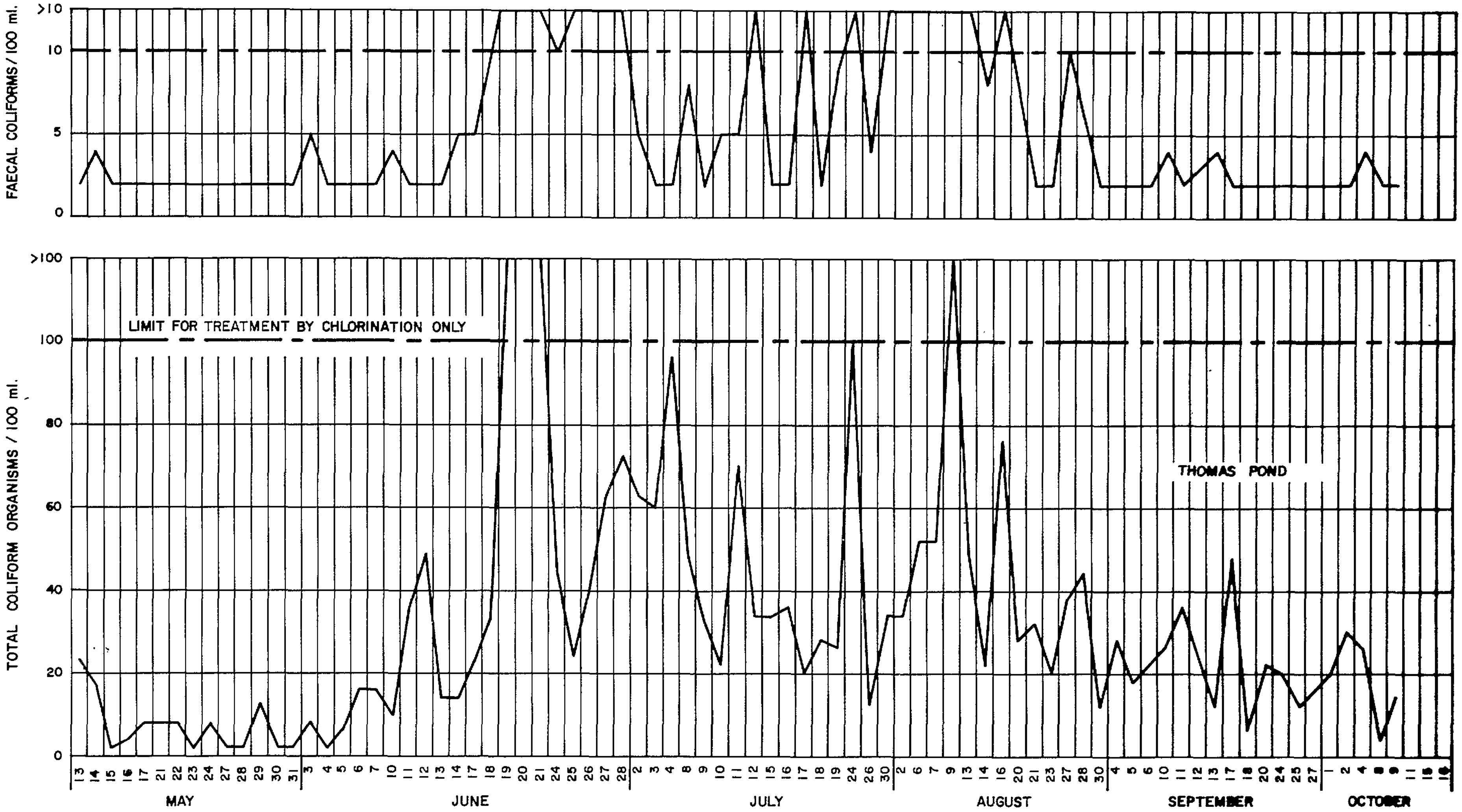


FIGURE 5·17
SUMMARY OF BACTERIOLOGIC ANALYSES

b. Windsor Lake

The quality of Windsor Lake water is by far the best of the three sources surveyed. A detailed assessment of Windsor Lake water quality suggests the following:

(i) Colour: (Platinum - Cobalt Scale) 40 percent of the time the colour intensity is less or equal to the "objective"* level of 5 units. 95 percent of the time, the colour intensity is less or equal to the "acceptable limit"* of 15 units.

(ii) Iron: 40 percent of the time the iron concentration is less or equal to the "objective" level of 0.05 mg/l. 99.9 percent of the time, the iron concentration is less or equal to the "acceptable limit" of 0.3 mg/l.

(iii) Manganese: The average concentarion of manganese is 0.029 mg/l compared with the "objective" level of 0.01 mg/l, and the "acceptable limit" of 0.05 mg/l. The range of 90 percent of the samples (14) lies between a concentration of 0.005 and 0.045 mg/l.

(iv) Turbidity: 99 percent of the time, the turbidity is less or equal to the "objective" level of 1 JTU (Jackson Turbidity Units).

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* As defined in "Canadian Drinking Water Standards and Objectives, 1968", and referred to in Chapter 4.

(v) Algae: Whereas algae species present in the water are of the type that can produce taste and odour, as well as clog filters, their concentration is generally scant, and no major problem is envisaged.

(vi) Coliform Organisms: The "objective" calls for one to two samples in any consecutive 30-day period to have a total coliform density and faecal coliform density of 100 and 10 per 100 ml., respectively. Higher densities require conventional treatment. It can be seen from Figure 5.15 that whereas faecal coliforms were within the "objective" density, total coliforms exceeded this "objective" during three different periods of time. Being a controlled catchment area, it is difficult to explain the occurrence of this high density of total coliform organisms.

c. Bay Bulls Big Pond

Bay Bulls Big Pond water can be characterized as soft, acidic, and moderately coloured. A detailed assessment of the water quality offers the following:

(i) Colour: (Platinum - Cobalt Scale). Only 25 percent of the time, the colour intensity is less or equal to the "acceptable Limit" of 15 units. 50 percent of the time the colour intensity is less or equal to 20 units. A colour intensity less or equal to 28 and 35 units can be expected to occur for 90 and 99 percent of the time, respectively.

(ii) Iron: 5 percent of the time, the iron concentration is less or equal to the "objective" level of 0.05 mg/l. An iron concentration less or equal to 0.12 and 0.23 mg/l can be expected to occur for 50 and 90 percent of the time. Only 4 percent of the time, the iron concentration exceeds the "acceptable limit" level of 0.3 mg/l.

(iii) Manganese: The average concentration of manganese is 0.015 mg/l compared with the "objective" level of 0.01 mg/l and the "acceptable limit" of 0.05 mg/l. The range of 90 percent of the samples (15) lies between a concentration of 0.004 and 0.025 mg/l.

(iv) Turbidity: 30 percent of the time the turbidity is less or equal to the "objective" level of 1.0 JTU. A turbidity level less or equal to 1.3 and 2.3 JTU can be expected to occur for 50 and 90 percent of the time, respectively. 99 percent of the time, the turbidity is less or equal to 3.0 JTU. The "acceptable limit" level is 5.0 JTU.

(v) Algae: Algae appear to be a problem in Bay Bulls Big Pond water. As can be seen in Table 5.7, the predominant species are Tabellaria and Asterionella of the diatom group. Whereas algae can be destroyed by an oxidant (chlorine or ozone), as dead organisms they can still cause tastes and odours and clog filters. In addition, diatoms which at different densities are present during all seasons of the year, have a cell wall composed principally of silica which is not subject to decomposition. Also, the most serious offenders of diatoms are Asterionella and Tabellaria, the two species which are predominant in Bay Bulls Big Pond. At concentrations in the order of magnitude that were present on October 11, 1973, these species can reduce the length of filter runs. This effect, however, is likely to occur over a period of only one to two months per year.

(vi) Coliform Organisms: It can be seen from Figure 5.16 that total coliform organisms as well as faecal coliforms exceeded the limit for treatment by chlorination only, during two different periods of time. This occurrence can be attributed to activities that take place in the catchment area.

d. Thomas Pond

Thomas Pond water is of a much inferior quality when compared with Windsor Lake, and Bay Bulls Big Pond. A detailed assessment of the water quality of Thomas Pond offers the following:

(i) Colour: (Platinum - Cobalt Scale). At all times the colour intensity exceeds the "acceptable limit" level of 15 units; its lower range being

40 units. 50 percent of the time, the colour intensity is less or equal to 75 units. A colour intensity less or equal to 105 and 125 units can be expected to occur for 90 and 99 percent of the time, respectively.

(ii) Iron: At all times, the iron concentration exceeds the "objective" level of 0.05 mg/l. Only 5 percent of the time, the iron concentration is less or equal to the "acceptable limit" level of 0.3 mg/l. 25 percent of the time, the iron concentration is less or equal to 0.5 mg/l; 50 percent of the time, less or equal to 0.7 mg/l; 90 percent of the time, less or equal to 1.3 mg/l. These concentrations of iron are excessive for municipal water supply.

(iii) Manganese: The average concentration of manganese is 0.089 mg/l compared with the "objective" level of 0.01 mg/l, and the "acceptable limit" of 0.05 mg/l. This concentration is excessive for municipal water supply. The range of 90 percent of the samples (17) lies between a concentration of 0.002 and 0.169 mg/l.

(iv) Turbidity: 25 percent of the time, the turbidity is less or equal to the "objective" level of 1 JTU. A turbidity level less or equal to 1.5 and 2.5 JTU can be expected to occur for 50 and 90 percent of the time, respectively. 99 percent of the time, the turbidity is less or equal to 3.3 units. The "acceptable limit" level is 50 JTU.

(v) Algae: Whereas algae species present in the water are of the type that can produce taste and odour, as well as clog filters, their concentration is generally scant and no major problem is envisaged. The relatively high colour intensity of the water serves as a buffer to algae growth.

(vi) Coliform Organisms: It can be seen from Figure 5.17 that total coliform organisms and faecal coliforms exceeded the limit for treatment by chlorination only, during two and seven different periods of time, respectively. The high recurrence rate of faecal coliforms can be attributed to recreational activities in the catchment area.

5. Treatment Concepts

a) General

In considering treatment concepts for water of the quality assessed in the previous section, we have classified treatment works into four categories, as follows:

(a) Disinfection
Disinfection of raw water to destroy coliform organisms may include screening and some chemical dosage for adjustment of pH, corrosiveness, etc.

(b) Direct Filtration
Raw water after screening is applied directly onto filters; this is then followed by disinfection. Depending on the characteristics of

the raw water, the filters may be preceded by pre-chlorination and chemical coagulation-flocculation.

(c) Clarification - Filtration

This system is the same as (b) above except that a clarifier, of any type of design, precedes the filter.

(d) Complete Treatment

This system is similar to (c) above except that additional chemicals and process units (e.g. ion exchange) may be used to reduce minerals and hardness were excessive.

It can readily be seen that the above four types of treatment works are in an ascending order of cost, with type (a) being the least costly.

Previous experience and case histories of water treatment were concerned primarily with turbidity removal, for which technological information is available in abundance. However, the chemistry and mechanism of colour removal are completely different, and relatively little information is available on colour removal technology. We know that chemical coagulation will precipitate colour out. Depending on the amount of precipitates, filtration (type (b) treatment), or a combination of the clarification - filtration (type (c) treatment) may be required.

It has also been established that ozone removes moderate colour by oxidation. This process may be carried out in a type (a) or (b) treatment plant, depending on the residue left in the water after oxidation with ozone.

An assessment of the type of treatment that should be considered for each source water is presented below.

b. <u>Windsor Lake</u>

Disinfection (type (a) treatment) is presently being applied to Windsor Lake water. Our assessment of the sanitary survey data indicates the quality of this water to be within the "acceptable limits". Disinfection as a treatment measure could, therefore, be considered satisfactory as the minimum and immediate requirement for attaining safe and acceptable drinking water quality. However, as noted, total coliform organisms at times exceed the acceptable limit for treatment by disinfection only, necessitating the use of type (b) treatment (filtration). Assessing this requirement versus other water constituents and relative to the need for spread of cash flow, it is recommended that tight measures be adopted to control activities in the catchment area, thus enabling deferment of the construction of type (b) treatment facilities. Periodic sanitary surveys in the catchment area will determine when these facilities should be constructed. It is further recommended that filtration, which will include dual media high-rate filters, be preceded by pre-treatment facilities to bring colour, iron, manganese and turbidity to within the "objective" levels of the Canadian Drinking Water Standards. These pre-treatment facilities could include chemical coagulation-flocculation, or ozonation. A more elaborate discussion on these alternative pre-treatment processes is presented in the following section (dealing with Bay Bulls Big Pond water).

c. <u>Bay Bulls Big Pond</u>

Since Bay Bulls Big Pond water exceeds the "acceptable limit" of colour (in 75 percent of the time); contains offensive and nuisance algae (at certain periods during the year); exceeds (at times) the limit of total coliform organisms and faecal coliforms that is acceptable for treatment by disinfection only, it is recommended that conventional treatment be provided at the time this source is tapped for supply. This conventional treatment will be type (b), (direct filtration), or type (c), (clarification-filtration) as explained later in this section. The objective of treatment will be to bring the water constituents within the "objectives" of the Canadian Drinking Water Standards which were recommended earlier (Chapter 4) for adoption as the standards of this project.

Impurities in water (in the form of solids or flocs) can be removed in filters (type (b) treatment), or in a combined system of clarifiers and filters (type (c) treatment). When filters only are used (referred to as direct filtration) the impurities are being caught and stored in the interstices of the filter media. Excessive amounts of impurities will fill this limited storage space at an accelerated rate, necessitating frequent backwashing of the filter media to clean off the collected impurities. Consequently, the shorter filter-run times may pose a serious problem, as increasingly large quantities of treated water would have to be diverted for filter backwashing in lieu of supply to the customers. Another potentially serious problem is the phenomenon of breakthrough. Gravity filters are usually designed for an operation head of about 8 feet. If the flocs stored in the filter could not stand this head, a breakthrough will occur, resulting in impurities carried over with the filtered water. It follows, therefore, that direct filtration should be evaluated carefully for each specific application, and if necessary its economics should be compared with a clarification-filtration plant (type (c) treatment). In the latter plant clarifiers are provided ahead of the filters to remove the bulk of the impurities (solids and flocs).

Applying the treatment concepts previously cited to Bay Bulls Big Pond water offers the following:

(i) Chemical coagulation-flocculation will precipitate out colour and other major impurities such as iron, manganese and turbidity. It will also help in the removal of coliform organisms and algae. However, basic information is required in order to determine the effect of the precipitates on filter performance and consequently the applicability and economics of direct filtration (type (b) treatment), and clarification-filtration (type (c) treatment) to this water.

(ii) Ozonation will remove colour, as well as iron and manganese, by oxidation. With this process only a little residue, amenable to direct filtration, is expected to remain in the ozonated water, as opposed to the case of chemical precipitation by coagulation-flocculation. Filtration would be required to improve turbidity, and help remove coliform organisms and algae. Here again, basic information on ozonation reaction rate would be required in order to evaluate the economics of ozonation versus chemical coagulation-flocculation.

We have, therefore, recommended after Bay Bulls Big Pond was selected as the new source of supply by the Econometric Model (see Chapter 6), that a test program be undertaken to establish the treatment process most effective and economical for this source water. Details of this program and description of the process and treatment facilities recommended can be found in Chapter 8, Volume III.

d. Thomas Pond

Thomas Pond water contains excessive concentrations of colour, iron, and manganese. It will be safe to assume that removal of these objectionable water constituents will require chemical pre-treatment or a combination of oxidation (by ozone) and chemical pre-treatment. In any case, precipitates will be formed, that in all likelihood would require clarifiers and filters. This type of treatment (type (c) has been considered in the system analysis of the Econometric Model (Chpater 6).

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APPENDIX I

PHYSICAL AND CHEMICAL QUALITY OF THE WATER SOURCES

FROM PREVIOUS WORK BY OTHERS

CONSTITUENT	WINDSOR LAKE	PETTY HARBOUR L.P.	BAY BULLS BIG POND	THOMAS POND
I. Physical				
Turbidity (JTU)	0.8-2.9	0.5	0.9	0.65-2.0
Colour (TCU)	5-15	10	15	40. - 45
pH Value	5.9-6.1	5.3	5.7	5.2 -5.8
II. Chemical (Mg/l)				
Hardness ($CaCO_3$)	0.7-5.8	0.7	5.5	4.2-7.3
Total Alkalinity ($CaCO_3$)	1.3-2.0	0.7	1.5	1.7-6.0
Total Iron (Fe)	0.08-0.40	0.10	0.20	0.28-0.43
Total Manganese (Mn)	N.D.*-0.03	0.05	N.D.*	0.06-0.08
Nitrate + Nitrite (N)	N.D.*-0.02	L0.01	0.001	---
Calcium (Ca)	1.5		1.4	0.8 - 0.9
Magnesium (Mg)	0.5		0.5	0.5 - 0.6
Sodium (Na)	7.5		3.2	3.8 - 3.9
Sulphate ($SO_4^{=}$)	4.5		2.0	1.1 - 2.5
Chloride (Cl)	10.7		5.8	6.4 - 6.7
Fluoride (F)	0.02-0.08	0.01	0.25	0.5
Silica ($S.O_2$)	2.0		1.2	1.1 - 1.4
Bicarbonate (CO_3)	0.8-2.0	0.4	0.9	2.2 - 2.8
Carbonate (CO_3)	NIL		NIL	
Total Dissolved Solids	26 - 36	21	18	22 - 31

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APPENDIX II

RAW WATER TOTAL COLIFORM STANDARDS*

Objective	Acceptable Limit	Maximum Permissable Limit
At least 95% of the samples in any consecutive 30 day period should have a total coliform density of less than 100 per 100 ml.	At least 90% of the samples in any consecutive 30 day period should have total coliform density of less than 1,000 per 100 ml.	At least 90% of the Samples in any consecutive 30 - day consecutive 30 day period should have a total a total coliform density of less than 5,000 per 100 ml.
Treatment by chlorination is required.	Complete or partial treatment including chlorination is required.	Complete water treatment is required.

* Canadian Drinking Water Standards and Objectives, 1968.

APPENDIX III

OUTLINE OF SANITARY SURVEY

The survey includes sample collection from Windsor Lake and Thomas Pond and their analysis for bacteriological, biological, physical, chemical, toxic and biocidic constituents. In July, 1973, this survey was extended to include Bay Bulls Big Pond.

1. Sampling Procedure

a. Bacteriological

Sampling commenced on May 13, 1973 because of the presence of ice, access difficulties and related problems at Windsor Lake and Thomas Pond. Special sterilized glass bottles protected by a paper were provided, for sample collection, by the Provincial Department of Health Laboratories at St. John's General Hospital. This same laboratory was also responsible for the analysis of the samples.

The sampling bottle was held at the base to prevent any external contamination. The bottle was then plunged horizontally below the water surface, with its neck pointing against the direction of the current. The stopper was then removed (under water) and the bottle was allowed to fill to within one inch of the top without rinsing. The stopper was replaced immediately while the bottle was still under water. The collected samples were kept refrigerated until tested.

On May 23, 27, and June 6, five additional samples were taken from different sections of Thomas Pond to comply with the requirements of the Provincial Division of Environmental Health, who are the Control Agency, described in the Canadian Drinking Water. When a sudden increase in coliform organisms density was encountered an extensive sampling programme was immediately carried out to determine the cause. This course of action was, for example, carried out during the period June 19 to 28, 1973, at Thomas Pond.

b. Physical and Chemical

Sampling commenced on May 13, 1973, and continued on a weekly basis thereafter with samples being taken every Tuesday.

Samples were collected in plastic, acid washed bottles provided by Environment Canada, Environmental Protection Service in St. John's. The samples were analyzed at the Bedford Institute of Oceanography in Dartmouth, Nova Scotia.

The sample bottle was filled with the source water to within 3/4 inch of the top, and was rinsed well with this liquid. This procedure was repeated at least three times. The bottle was then held in an inverted position, with the stopper off, and plunged below the water surface and then turned horizontally into the current and allowed to fill. The stopper was then replaced, and the collected samples shipped, as expeditiously as possible via air express, to the analyzing laboratory.

c. Toxic and Biocides

Samples for toxic chemicals and biocides were collected in accordance with guidelines provided by the Analytical Services Section of Environment Canada, Atlantic Region office in Moncton, N. B. This same office also was responsible for the analysis of the samples. The sampling and preservation guidelines are as follows:

SAMPLE CONTAINER	CONTAINER SIZE	CONTAINER TYPE	PRESERVATIVE TECHNIQUE	SAMPLING PROCEDURE	PARAMETERS TESTED
1.	250 ml.	Plastic	Nil	A	Arsenic
2.	250 ml.	Plastic	10 mg $HgCl_2$ & stored at $4^{o}C$	B	Nitrate + Nitrite
3.	250 ml.	Plastic	0.5 ml. HNO_3	C	Cadmium, Chromium, Lead
4.	1,000 ml.	Glass (Wine)	Nil	D	Chlorinated Hydrocarbons
5.	1,000 ml.	Glass (Wine)	50 ml. (Pestide G) Chloroform	E	Organo Phosphates
6.	1,000 ml.	Glass (Wine)	2 ml. H_2SO_4	F	Herbicides

A- Rinse sample container with sample once, then fill to within two inches of the top.

B- Rinse sample container with sample once, fill to within two inches of the top, add the $HgCl_2$, replace cap and shake. Loosen cap to allow the air pressure to stabilize, then retighten cap. Store at 4^{o}C (if possible).

C- Rinse sample container with sample and fill to within two inches of the top. Add HNO_3 and shake well after replacing cap.

D- Rinse the sample container with the sample, and fill to the mark. Replace cap being careful not to drop the teflon liner out of the cap.

E- Rinse the sample container with the sample, and fill to the mark. Add the chloroform, replace the cap and shake. Loosen cap to allow the air pressure to stabilize then re-tighten.

F- Rinse the sample container with the sample, and fill to the mark. Add the H_2SO_4; replace the cap and shake. Loosen cap to allow the air pressure to stabilize then re-tighten.

d. Biological

Sampling for the determination of algae and nuisance micro-organisms commenced on June 3, 1973, and continued on a weekly basis thereafter with samples being taken every Sunday (except for statutory holidays).

Sampling procedures (and bottles) were the same as those used for the bacteriological programme. The Department of Health Laboratories at St. John's General Hospital were responsible for the analysis of these samples.

Sampling for the determination of B.O.D.'s commenced on June 7, 1973, and continued thereafter on a bi-weekly basis with samples being taken every Thursday. There was one such sampling location at Thomas Pond and two at Windsor Lake. A total of six samples were collected from each sampling location.

Sampling procedures and bottles were the same as those used for the physical-chemical procedure, except that the bottles were not acid washed.

Environment Canada, Environmental Protection Service in St. John's carried out the analysis of the B.O.D.'s samples. Samples were delivered to this office for analysis as soon as they were collected.

2. Sample Analysis

Samples were generally analyzed in accordance with the 13th edition of "Standard Methods" (APHA, AWWA, & WPCF). The following tables give the analytical determinations, their respective limits, and the laboratories capabilities.

COLIFORM ORGANISMS

Organisms	Request (1)	Lab. Capability		
	Lower Limit	Method in Operation	Detection Limit	Prepared to Accept
Total Coliform	At least 95 percent of samples in any consecutive 30 day period should have a total coliform density of less than 100 per 100 ml.	MPN and MF (2)	As Requested	Yes
Faecal Coliform	At least 95 percent of the samples in any consecutive 30 day period should have a faecal coliform density of less than 100 per 100 ml.	MPN and MF (2)	As Requested	Yes

(1) As listed in Canadian Drinking Water Standards (1968)

(2) Most Probable Number multiple tube fermentation and Membrane Filter.

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PHYSICAL CONSTITUENTS

Constituent	Request (1)		Lab. Capability			
	Measured as	Lower Limit	Method in Operation	Measured as	Detection Limit	Prepared to Accept
Colour	TCU (2)	L5	Yes	TCU (2)	As Requested	Yes
Odour	TON (3)	0	Yes		As Requested	Yes
Taste		Inoffen- sive	No (5)			No (5)
Turbidity	JTU (4)	LI	Yes	JTU (4)	As Requested	Yes
Temperature	°C	LIO	Yes			No (6)
pH		6.5-8.3	Yes		As Requested	Yes

(1) As listed in Canadian Drinking Water Standards.

(2) True Colour Unit, Platinum - Cobalt Scale (1968).

(3) Threshold Odour Number.

(4) Jackson Turbidity Units.

(5) This is a comparative test not requiring laboratory equipment.

(6) Temperature will be measured at the time a sample is collected.

CHEMICAL CONSTITUENTS

Constituent	Request[1]		Lab. Capability			
	Measured as	Lower Limit mg/l	Method in Operation	Measured as	Detection Limit	Prepared to Accept
Alkalinity	$CaCO_3$		Yes	$CaCO_3$	As Requested	Yes
Ammonia	N	0.01	Yes	N	"	Yes
Calcium	Ca	L75	Yes	Ca	"	Yes
Chloride	Cl	L250	Yes	Cl	"	Yes
Copper	Cu	L0.01	Yes	Cu	"	Yes
Iron (dissolved)	Fe	L0.05	Yes	Fe	"	Yes
Magnesium	Mg	L50	Yes	Mg	"	Yes
Manganese	Mn	L0.01	Yes	Mn	"	Yes
M.B.A.S.[2]		L0.2	Yes		"	Yes
Phenolic	Phenol	L0.002	Yes	Phenol	"	Yes
Phosphate	PO_4	L0.2	Yes	PO_4	"	Yes
Total Dissolved Solids		L500	Yes		"	Yes
Total Hardness	$CaCO_3$	L120	Yes	$CaCO_3$	"	Yes
Organics	CCE + CAE[3]	L0.05	Yes	CCE + CAE	"	Yes
Sulphate	SO_4^{--}	L250	Yes	SO_4^{--}	"	Yes
Sulphide	H_2S	L0.3	Yes	H_2S	"	Yes
Uranyl Ion	UO_2^{--}	L1.0	Yes	UO_2^{--}	"	Yes
Zinc	Zn	L1.0	Yes	Zn	"	Yes

(1) As listed in Canadian Drinking Water Standards (1968).

(2) Methylene Blue Active Substances.

(3) Carbon Chloride and Carbon Alcohol Extractibles.

TOXICANTS

Toxicants [1]	Request [1]		Lab. Capability				
	Measured as	Lower Limits ppm	Method in Operation	Measured as	Detection Limit ppm	Prepared to Accept
Arsenic	As	0.01	Yes	As	0.005	Yes
Barium	Ba	L 1.0	No [2]	Ba	1.0	No
Boron	B	L 5.0	No	--	--	No
Cadmium	Cd	L 0.01	Yes	Cd	0.001	Yes
Chromium	Cr^{6}	L 0.05	Yes	Cr	0.002	Yes [3]
Cyanide	CN	0.01	No	--	--	No
Lead	Pb	L 0.05	Yes	Pb	0.001	Yes
Nitrate + Nitrite	N	L10.0	Yes	N	0.001	Yes
Silver	Ag	0.05	No [2]	Ag	0.005	No

(1) As listed in Canadian Drinking Water Standards (1968).

(2) Method & equipment available but not set-up.

(3) Not measured in the same form as requested.

BIOCIDES

Request[1]		Lab. Capability		
Biocides[1]	Lower Limits ppm	Method in Operation	Detection Limit ppb	Prepared to Accept
Chlorinated Hydrocarbons				
Aldrin	0.017	Yes	0.005	Yes
X-Chlordane)	0.003[2]	Yes	0.005	Yes
X-Chlordane)		Yes	0.005	Yes
p,p'-DDT)	0.042[2]	Yes	0.010	Yes
O,p'-DDT)		Yes	0.005	Yes
Dieldrin	0.042	Yes	0.005	Yes
Endrin	0.001	Yes	0.005	Yes
Heptachlor	0.018	Yes	0.005	Yes
Heptachlor Epoxide	0.018	Yes	0.005	Yes
Lindane	0.056	Yes	0.005	Yes
p,p'-Methoxychlor	0.035	Yes	0.050	Yes
Toxaphene	0.005	No[3]	--	No
Organophosphates				
Fenitrothion)	0.100[2]	Yes	0.020	Yes
Parathion)		Yes	0.020	Yes
Methyl Para-) thion		Yes	0.020	Yes
Carbamates	0.100[2]	No	--	No
Herbicides				
2,4-D)	0.100[2]	Yes	0.010	Yes
2,4,5-T)		Yes	0.010	Yes
2,4,5-TP)		No	--	No
Silvex)		Yes	0.010	Yes
MCPA)		Yes	0.010	Yes

(1) As listed in Canadian Drinking Water Standards (1968) with some added.

(2) Sum of all not to exceed indicated level.

(3) Method and equipment available but not set-up.

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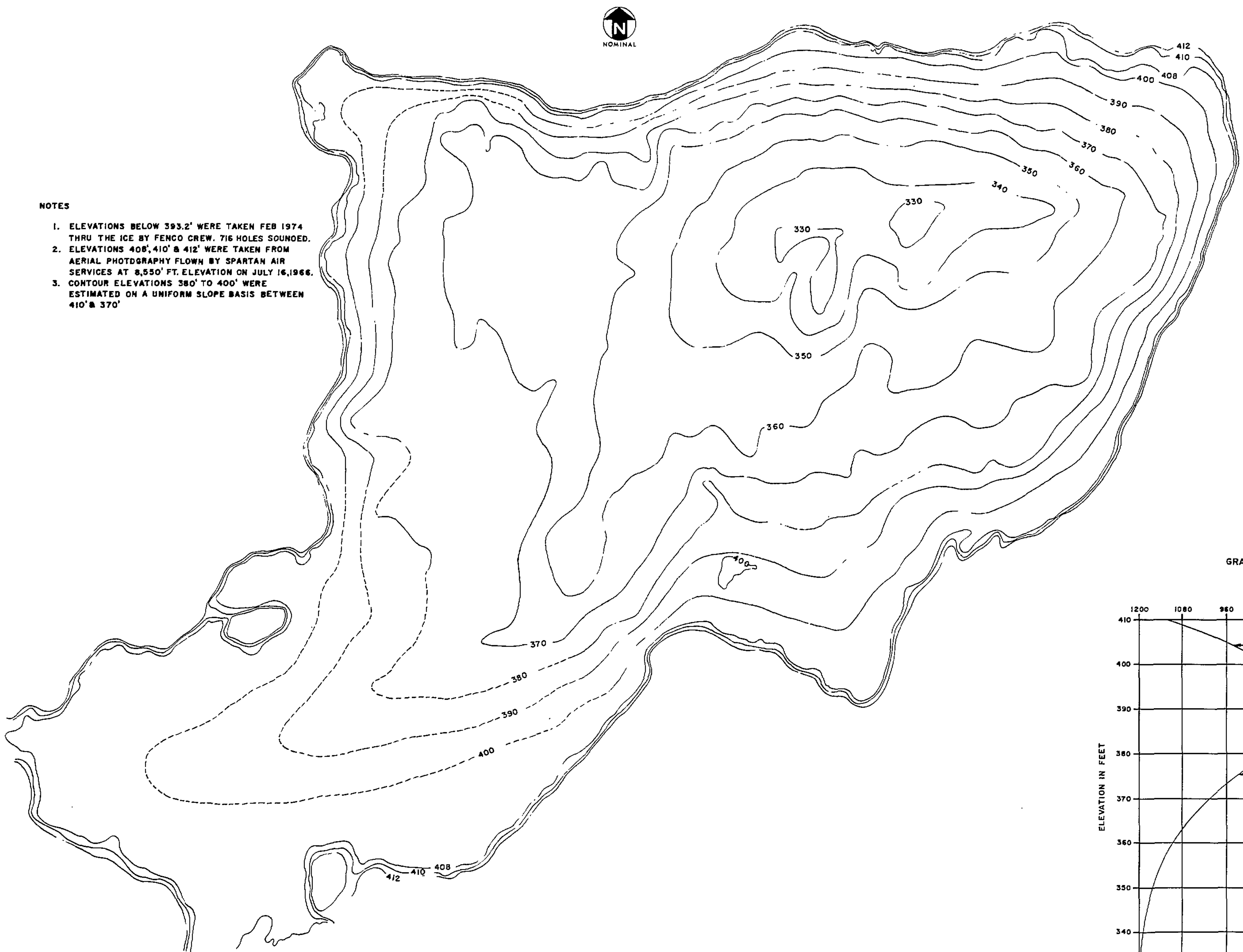
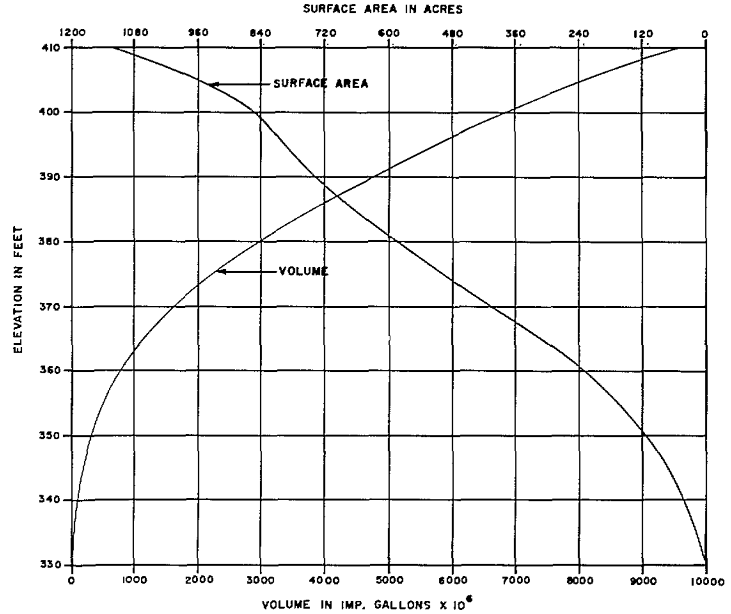


NOTES

1. ELEVATIONS BELOW 393.2' WERE TAKEN FEB 1974 THRU THE ICE BY FENCO CREW. 716 HOLES SOUNDED.
2. ELEVATIONS 408', 410' & 412' WERE TAKEN FROM AERIAL PHOTOGRAPHY FLOWN BY SPARTAN AIR SERVICES AT 8,550' FT. ELEVATION ON JULY 16,1966.
3. CONTOUR ELEVATIONS 380' TO 400' WERE ESTIMATED ON A UNIFORM SLOPE BASIS BETWEEN 410' & 370'

GRAPH OF ELEVATION VS. IMP. GAL. VS. SURFACE AREA



REFERENCES	SCALES	ST. JOHN'S REGIONAL WATER SYSTEM STUDY BATHYMETRIC SURVEY OF BAY BULLS BIG POND	FENCO FENCO Foundation of Canada Engineering Corporation Limited
	AS SHOWN 0 400" 800" 1200" 3"	D.R.E.E. PROJECT NO. 3.1 FENCO PROJECT NO. I-6068-I	DRAWING NO 5·1

Chapter 6

CHAPTER 6

SYSTEMS ECONOMICS

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CHAPTER 6

SYSTEMS ECONOMICS

SYNOPSIS

The rationale behind the development of, and the conclusions resulting from, an econometric analysis of the proposed St. John's Regional Water System are covered in this Chapter. A summary of the findings and recommendations is as follows:

- To supplement the supply from Windsor Lake, three possible sources as outlined below were considered:

  - Direct supply from Bay Bulls Big Pond.

  - Supply from Bay Bulls Big Pond via Petty Harbour Long Pond

  - Direct supply from Thomas Pond.

- The final conclusion from the systems analysis (as derived by the Econometric Model) is that the new source of supply should be Bay Bulls Big Pond, and that it should be directly connected to the proposed supply system.

- The supply at Little Powers Pond could be made use of most efficiently as direct augmentation to Windsor Lake.

- The St. John's Regional Water System should comprise the following communities:

  - Regional Centre — St. John's and expansion zones, Mount Pearl, New Town, Kilbride, Wedgewood Park, Shea Heights (Blackhead Road).

  - Sub-Regional Centre — Conception Bay South Area (Seal Cove, Gullies, Kelligrews, Foxtrap, Long Pond, Manuels, Chamberlains, Topsail).

  - Local Centres "A" — Paradise, Topsail Road, Torbay, Torbay Road Penetanguishene, Goulds, Petty Harbour.

- The North-West communities of St. Phillips, Portugal Cove, Portugal Cove Road and Thorburn Road (identified under "Local Centres "B") require a relatively expensive system and from an economic view point, any solution to supply them with water should receive a low priority. Although, presently excluded from the regional system, this North-West area could be served by any of five alternative schemes evaluated, including the regional scheme, a sub-regional scheme, or a local supply scheme.

I. GENERAL

This Chapter is concerned with the rationale behind the development of, and the conclusions resulting from, an econometric analysis of the proposed regional water supply system. The introductory section describes the nature and purpose of this type of study and the economic concepts used in the cost comparison of alternatives. The central section documents the way in which the mathematical model was designed together with detailed specification of the several cost functions involved. Finally, the results of several series of analyses are presented and discussed, and conclusions and recommendations summarized.

II. INTRODUCTION TO SYSTEMS ANALYSIS

1. Systems Analysis

Systems analysis is a currently popular term for the application of engineering design methodology to complex systems, the latter usually comprising many component parts between which there is a significant degree of intersection or interdependance. It is convenient to consider the sequential design activities of (a) system identification; (b) system definition; and (c) system optimization.

a. System Identification

System identification involves the selection of possible sources of supply, centres of demand which may reasonably be assumed to be included in the regional network, and trunk main links connecting these centres of supply and demand. In the present study the available sources are assumed to be any or all of the four impounding reservoirs at Windsor Lake, Petty Harbour Long Pond, Bay Bulls Big Pond and Thomas Pond. It was envisaged that following the study of the regional network and the decision concerning the geographical limits of the area to be served, other small local sources of supply might be investigated for the purpose of providing a supply to such communities as might be excluded from the regional network for economic reasons.

The selection of demand centres is rather more arbitrary and subject to refinement as preliminary analysis shows which communities may or may not be economically supplied from the regional network. The initial network used in this study excluded remote communities such as Bauline, Pouch Cove and Flatrock to the north and Bay Bulls and Witless Bay to the south. All other communities of significant size - either now or in the future - were included in the preliminary analysis. To facilitate analysis, several communities which in fact are distributed along a ribbon development were assumed to be concentrated at a point, this geographical location usually (but not necessarily) corresponding to the location-proposed or existing - of a service reservoir from which distribution lines issue. Likewise, large areas of demand were represented by two or more nodes representing different pressure zones.

b. System Definition

System definition involves the quantification of those variables necessary for a proper description of the system and its needs. This includes the reliable yield from each of the possible sources and the average daily demand at each of the communities at intervals over the 20 year planning horizon. In addition the geometry of the network must be described by defining the length of each link, the nodes at each extremity of the links and the pressure elevation at each node. Some means must also be found to quantify the extent of treatment required to bring the quality of each of the sources of supply within acceptable limits.

c. System Optimization

System optimization is aimed at meeting the requirements and other constraints outlined by system definition at minimum cost to the community. When the system contains a large number of elements, the comparison of the many alternative design possibilities is most economically done by means of a computer program. The process consists of the successive adjustment of a set of design variables until some objective function, expressed in terms of the design variables and a set of cost co-efficients, is minimized.

In the present study the design variables were chosen to be the average flows in each of the trunk mains in the network. In order to maintain the maximum generality in system definition, the flow rate is assumed to be possible in either direction so that in each link of the network there is assumed to exist two non-negative flow variables one of which must be found to be zero by the solution. Where a community is connected to a number of links it is likely that all but one of the incoming flows will be zero. It is also quite possible that where the incoming flow rate exceeds the demand for the community, one or more of the out-going flows may be finite, representing either a bifurcation or a transmitted supply to an adjacent community.

2. The Objective Function

In order to measure the effectiveness of alternative solutions it is necessary to define a function which provides an objective evaluation of the ratio of benefits to costs. In this case the benefits were taken to be the specified water demands so that effectiveness is measured simply as the reciprocal of system cost.

The costs used in this evaluation are designed to take account of two major components, viz:

(i) Cost of production of potable water and
(ii) Cost of conveying that water from the source(s) to the demand centres.

Since in both these activities, account must be

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taken of both capital and operating costs, it is convenient at this point to discuss the economic concepts which were used in the calculation of system cost.

3. Economic Concepts

In planning new works, it is often necessary to compare schemes involving on the one hand high capital investment but low operating costs and, on the other, a more modest capital outlay with higher running costs.

Typical situations are:-

(i) The choice between a large diameter main with negligible pumping costs or a smaller main with higher pumping costs.

(ii) The decision to abstract water from a source which is conveniently located but which requires immediate or expensive treatment compared with the use of a more remote source which needs little or deferred treatment.

(iii) The alternative of supplying two adjacent regions of differing pressure by separate trunk mains possibly in separate stages, compared with providing a single trunk main to the high pressure region and in turn supplying the lower pressure region from the higher pressure region

Objective comparison is possible only if several cost components - capital and operating, immediate and projected - may be expressed on a common basis. Two general methods are available for this purpose.

a. Annual Cost Method

All capital expenditures are amortized over the design life of the works using a capital recovery factor which is a function of the design life in years and the interest rate on borrowed capital.

The annual repayment cost for borrowed capital is then combined with other annual running costs to yield an "equivalent annual cost" on which basis comparison can be made between alternative schemes.

b. Present Value Method

All expenditures, immediate or projected, capital sums or operating costs, are expressed as an equivalent capital sum - present day dollar values. Once again, this conversion involves the use of an interest rate and the period in years over which operating charges will be incurred. For simple cases involving operating costs, the method is essentially the inverse of the annual cost approach. However, some advantage is gained when describing the cost of a scheme in which capital investment is staged over the planning horizon.

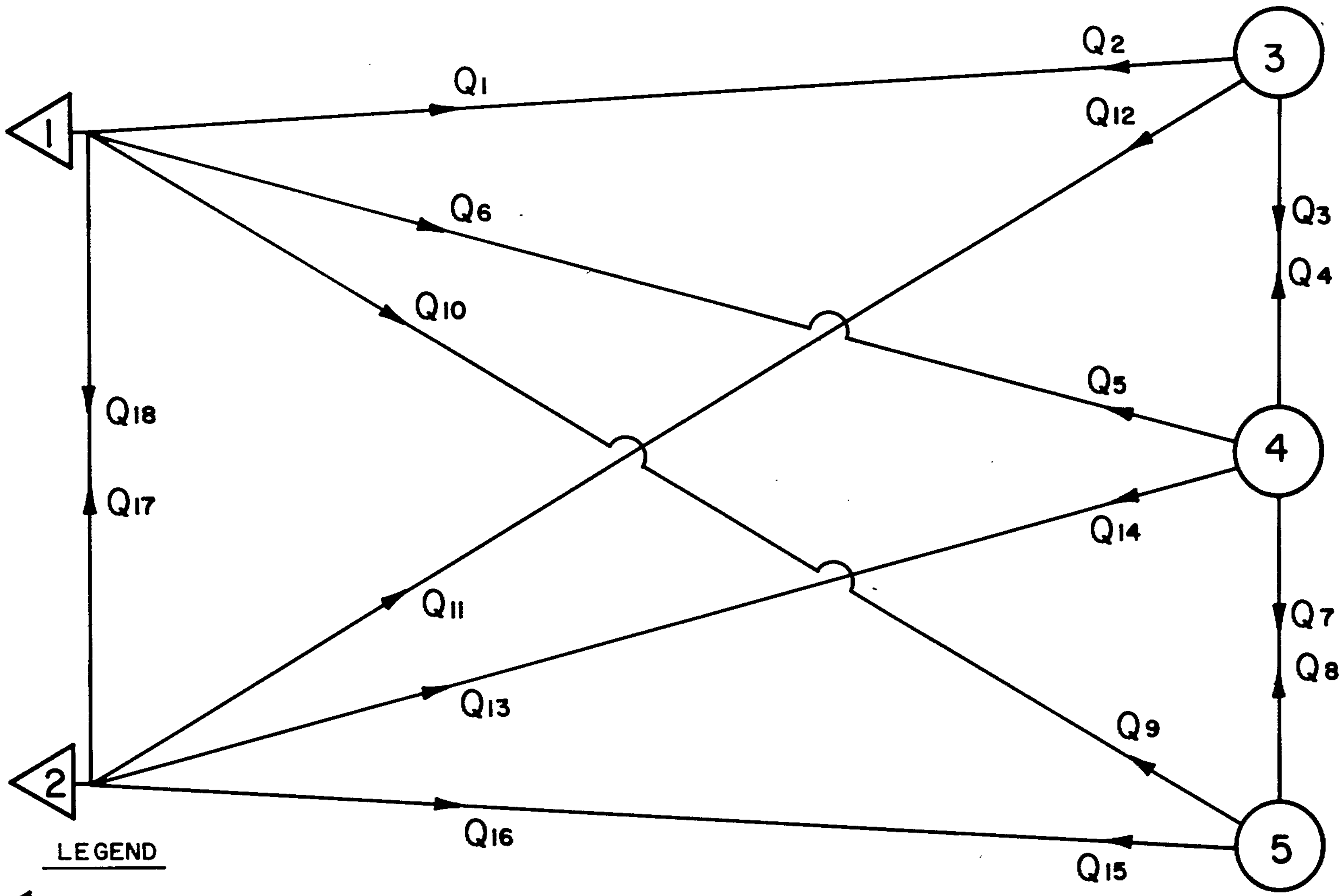
The use of either of the two above methods should lead to the same economic decision in any comparative study. In the present econometric analysis, the annual cost method was employed for some of the preliminary studies. It soon became apparent, however, that the present value method was advantageous for more detailed analysis, particularly where the staging of treatment works appeared to be a crucial factor in the selection of a source.

III. THE MATHEMATICAL MODEL

1. The Network Concept

The econometric analysis is concerned only with cost differences resulting from choice of source and selection of pipe routes whereby these sources and the demand centres are linked. Many practical details such as local distribution mains, the provision of service reservoirs and the like are omitted from consideration since they do not significantly effect the outcome. The regional system may therefore be conveniently viewed as a network of links connecting nodes some of which are sources of supply and others are centres of demand. The concept of the network may be pictured by reference to Figure 6.1 in which a greatly simplified system is depicted. In this system two sources of supply may be used to provide the demand at three locations employing a wide variety of arrangements of links. The possible flow in each link is represented by a pair of flow variables, one in each direction. It will be noted that for complete generality, the possibility is allowed for one source to feed into another; in such a case treatment would occur only at the downstream source.

The cost of providing such a system may be defined



LEGEND

◁ SUPPLY SOURCE

○ DEMAND CENTRE

→← PIPELINE LINK

FIGURE 6·1

NETWORK CONCEPT

by associating with each flow variable a corresponding cost coefficient so that, for example the cost of supplying a quantity Q_1 from Node 1 to Node 3 is given by: C_1Q_1

The total cost is then found by summing all similar products, it being noted that at least half the Q variables will have zero value.

Thus:

$$\text{Cost} = C_1Q_1 + C_2Q_2 + ---- + C_{18}Q_{18}$$

or $$\text{Cost} = \sum_{i=1}^{18} C_iQ_i$$

This then is the objective function which is to be minimized subject to the constraints of supply and demand. These constraints may be defined as:

(i) The algebraic sum of all flows leaving a source (reservoir) must not exceed the reliable yield of that source.

(ii) The algebraic sum of all flows entering a demand node must be equal to the required demand of that node.

These constraints may be easily expressed mathematically.

Thus, for source Node 1, we may write:

$$Q_1 - Q_2 + Q_6 - Q_5 + Q_{10} - Q_9 + Q_{18} - Q_{17} \leq R_1$$

Where R_1 is the reliable yield of source Node 1.

Also, typically, the demand at Node 5 results in the constraint:

$$Q_7 - Q_8 + Q_{10} - Q_9 + Q_{16} - Q_{15} = D_5$$

Where D_5 is the required average demand at Node 5.

Since all of the constraint relationships and also the objective (cost) function involve only simple first power terms in the flow variables Q, and since further, all the flow variables must be non-negative (i.e. have either a zero or positive value), the solution to this optimization problem can be obtained by a method of mathematical programming called Linear Programming. Such problems may be solved very efficiently on a digital computer even in cases involving large numbers of variables and very many constraint relationships.

A complication arises, however, in the fact that the costs must be described in the very simple form of a linear coefficient. The difficulty is due to two main causes, namely:

(i) The cost of supplying water, as mentioned previously, involves both the cost of abstraction and treatment, and also the cost of conveying by pipeline and pumping station where necessary from one node to another.

(ii) The cost of both treatment and conveyance is not directly proportioned to the quantity of water involved as there are substantial economies of scale in operations involving a large volume rate of flow.

In addition, both treatment and conveyance involve both capital and operating costs, which must be expressed in the form of a present value. The methods employed in solving these problems and thus evaluating the necessary cost coefficients are discussed in the following section.

2. Cost Coefficients

The calculations of cost coefficients must commence with the assumption of an arbitrary or approximate value for each of the flow variables. For the purpose of cost coefficient evaluation it is not necessary in the first instance that the assumed flow values represent a feasible solution - i.e. a solution set by the system. Following the evaluation of the cost coefficients a solution may be obtained for the flow variables which is both feasible and optimal with respect to the cost coefficients used. However, these cost coefficients may not be consistent with the flow variables thus calculated, having been computed on the basis of assumed values. Therefore, a new set of cost coefficients must then be computed on the basis of the revised flow rates. This iterative procedure must be repeated until the flow rates from which the cost coefficients have been estimated are acceptably close to the values, found by the optimization process. Convergence is in fact rapid, since the constraints serve to define the flow rates once a particular route has been selected.

3. Conveyance Costs

Once approximate values have been assumed for the flow variables, the cost of conveyance is first determined. These costs include:

(i) Capital cost of pipeline;

(ii) Capital cost of pump station, if required;

(iii) Maintenance cost of pipeline and pump station;

(iv) Operating costs for pumps;

(v) Capital cost for traffic control and reinstatement;

(vi) Annual sinking fund costs for pump replacement.

In the selection of pipe size a sub-optimization problem is encountered, since a wide range of technologically feasible design alternatives may be envisaged depending on the selection of pipe diameter (and thus friction loss) and pump station horsepower (and thus compensating pumping head). A special routine in the computer program selects the pipe diameter and pump size (if any) which will result in the minimum cost to the system. In this calculation pipe diameter and pump are sized to supply the maximum month average daily flow and to allow for various categories of reinstatement costs. The pump operating costs are however calculated on the basis of average annual daily flow rate, and converted to an equivalent present worth capital sum based on an indefinite period of use. It should be noted that for the interest rates currently applicable there is little difference between the uniform series present worth factor computed for 40 years and for an infinite number of years.

For example:

$$\frac{A}{P}\ (8\%,\ 40) = 11.925$$

$$\frac{A}{P}\ (8\%,\ \infty) = 12.500$$

The difference is only 4.825 percent.

In addition to normal maintenance, an annual sum is included for the replacement of machinery at 20 year periods.

In determining the amount of pump capacity required for a particular arrangement, an overall efficiency of 75 percent was assumed and an allowance for standby pump capacity was made by estimating the installed pump horsepower to exceed the design requirements by a factor varying exponentially from 30 percent for large pump stations (in excess of 400 HP) to 100 percent for very small stations.

4. <u>Conveyance Cost Functions and Design Factors</u>

The foregoing section outlines the general approach to calculating conveyance costs. The detailed cost functions, design factors and formulae used in this calculation are summarized as follows:

(i) Friction co-efficient "f" in pipelines computed, using Colebrook formula:

$$\frac{1}{\sqrt{f}} = -2 \log_{10}\left(\frac{k}{3.7D} + \frac{2.51}{R\sqrt{f}}\right)$$

Where: k = equivalent roughness height (0.002 ft)
R = Reynolds Number
D = pipe diameter

(ii) Pipe diameter D determined by Darcy-Weisbach equation:

$$h_f = \frac{fLV^2}{2gD}$$

Where: h_f = Head loss
f = friction co-efficient
V = velocity corresponding to Qmax
Qmax = maximum design flow
g = gravitational acceleration
D = pipe diameter

(iii) Capital Cost of Pipeline C_1:

$$\$C_1 = DL\ (36 - 6.42\ \sqrt{D})$$

Where: D = pipe diameter (ft)
L = pipeline length (ft)

(iv) Design Pump Horsepower HP:

$$HP = 62.4\ Qmax\ (h_f + H_{stat})/(550 \times 0.75)$$

Where:

$Qmax$ = maximum design flow (c.f.s.)

h_f = friction head loss in pipeline (ft)

H_{stat} = static lift in pipeline (ft)

HP = design pump horsepower

(v) Installed Pump Capacity HPI:

$$HPI = HP \left(1.30 + 0.70\ e^{-HP/200}\right)$$

Where: HPI = installed pump horsepower

HP = design pump horsepower

e = 2.7183 (base of natural logarithm)

(vi) Capital Cost of Pump Station C_2:

$$\$C_2 = HPI \times HPCost$$

Where: HPI = installed pump horsepower

$HPCost$ = factor obtained from the function:

$HPCost$ = 3.5 Y, where:

$$\log_{10} Y = 3.1576558 - 0.318448 \log_{10} HPI$$

(vii) Annual Pump Operating Cost C_3:

$$\$C_3 = HP\ (Qave/Qmax)\ 100$$

Where: HP = design pump horsepower

$Qave$ = average design flow

$Qmax$ = maximum design flow

(viii) Annual Maintenance Cost C_4:

$$\$C_4 = 0.01 \left[C_1 + C_2 \right]$$

Where: C_1 = Capital cost of pipeline ($)

C_2 = Capital cost of pump station ($)

(ix) Annual Sinking Fund Cost for Pump Replacement C_5:

$$\$C_5 = 0.3C_2 \left[\frac{r\ (1+i)^n}{(1 + r)^n - 1} \right]$$

Where: C_2 = Capital Cost of pump station ($), 30 percent of which is equipment.

r = interest rate on costs (4.5%)

n = replacement period in years (20)

(x) Total Present Worth Cost C_6:

$$\$C_6 = C_1 \text{ fac.} + C_2 + (C_3 + C_4 + C_5)\frac{1}{r}$$

Where: C_1 = Capital cost of pipeline ($)

fac = reinstatement cost factor,

= 1.0 in rural areas

= 1.2 in semi-urban areas

= 1.4 in urban areas

C_2 = Capital cost of pump station ($)

C_3 = Annual operating cost of pumps ($)

C_4 = Annual maintenance costs ($)

C_5 = Annual sinking fund cost for pump replacement ($)

$\frac{1}{r}$ = Uniform series present month factor **for** ∞ years (r = 8 percent).

It should be noted that special provision was made in the computer program to reduce the calculated cost of certain links by 50 percent or 100 percent in cases where some or all of the required conveyance capacity was already in existence.

5. Production Costs

Conveyance costs may be computed for each link as a function only of the design flow assigned to that link and independent of flows elsewhere in the network. Costs arising from abstraction and treatment of water, however, must reflect the considerable economies of scale and are therefore dependent on the magnitude of the total flows being abstracted from a particular source. Following the assignment of conveyance costs to each flow variable, all of the flow rate variables are scanned to identify those which emanate directly from a reservoir source. It is to these specific flow rate variables alone that the additional production costs are added, and the calculation of these production costs is based on the total amount being abstracted and treated.

Production costs are assumed to include the following components:

(i) Capital cost of intake.
(ii) Capital cost of low-lift pump station.
(iii) Capital cost of treatment works.

(iv) Maintenance cost of pump station and treatment works.

(v) Operating cost for low-lift pumps.

(vi) Operating cost for treatment works.

(vii) Annual sinking fund for plant replacement.

For each potential source in the network, the flow rates issuing from the source are totaled at each stage in the interative process. This annual averages design flow is increased by a factor of 40 percent to allow for maximum flow rates and these values are used in determining the size and hence the capital cost of major items such as the low-lift pump station and the treatment works. The costing of the intake is however, based on the maximum reliable yield of the source being considered, since it is normal practice to provide an intake capable of handling the full potential of the source. Costing of annual operating and maintenance costs are based on the annual average design flow, and an allowance is made for increased unit rate for treatment, when a treatment plant of specific capacity is underutilized in the initial years of operation. Capital costs and operating costs are combined in a present value cost based on the interest rate on borrowed capital and the design life of the treatment works. In addition, an annual sinking fund contribution is allowed for, to cover replacement of plant such as pumps and treatment equipment at intervals of 20 years.

An important consideration in the estimation of production costs is the quality of the source water and the extent to which water treatment may be postponed or curtailed. To allow for this variation, a present worth factor is computed for each treatment works based on an assumed phasing of construction. The computed treatment cost is modified by the present worth factor and then apportioned among the various conveyance links emanating from the source. The total cost of production and conveyance is thus obtained for those flow rate variables issuing from a source. However, treatment costs are stored in order that final cost figures may show a breakdown between capital and annual charges, and between production and conveyance costs.

In order that different costs may be assigned to the differing types of treatment which the new source water may require, provision is made to assign alternate cost calculation factors depending on the physical source being tested. For example, in the calculation of operating costs, a distinction is made between the cost of operating a direct filtration plant at Windsor Lake and a filtration plus clarification plant at Thomas Pond.

6. Production Cost Functions & Design Factors

This section summarizes the detailed cost functions, factors and design formulae used in the calculation of production costs. In the following section, an explanation is given of how a present value factor is determined for a proposed staging of construction, and the sensitivity of this factor to inflation rates.

(i) Capital Cost of Reservoir Intake C_1:

$$\$ C_1 = (44.0 + 45 D)$$

Where: L = assumed length of intake, taken as 1,000 ft. in absence of hydrographic data.

D = intake diameter based on a velocity of 5ft/sec. at a flow equal to the ultimate reliable yield multiplied by a maximum factor of 1.40.

(ii) Capital Cost of Treatment Works C_2:

$$\$ C_2 = K_1 \text{ Qmax. } C_3 \ 10^4$$

Where: Qmax. = total design flow abstracted (m.i.g.d.) multiplied by a maximum factor of 1.40.

C_3 = Cost in cents/g.p.d. obtained from:

$$\log_{10} C_3 = 1.32409 - 0.126895 \ \log_{10}\text{Qmax.}$$

K_1 = factor to relate costs to 1974 St. John's values,

= 2.6 for direct filtration plant

= 3.1 for clarification - filtration plant.

(iii) Annual Treatment Cost C_4:

$$\$ C_4 = K_2 C_5 \text{ Qave. } 365 \times 10$$

Where:

Qave. = toal design flow abstracted (m.i.g.d.)

C_5 = cost in cents per 1,000 gals. obtained from:

$$\log_{10} C_5 = 1.171 - 0.37 \log_{10} \text{Qmax.}$$

K_2 = factor to convert to 1974 St. John's values,

= 2.0 for direct filtration plant.

= 2.2 for clarification - filtration plant.

(iv) Capital Cost of Low-Lift Pump Station C_6:

$$\$ C_6 = \text{HPI} \times \text{HP Cost}$$

Where: HPI = installed pump horsepower.

HP Cost = factor obtained from:

HP Cost = 3.5 Y, where:

$$\log_{10} Y = 3.1576558 - 0.318448 \log_{10} \text{HPI}$$

(note that HP and HPI have been defined in previous sections).

(v) Capital Cost of Wet Well C_7:

For all low-lift pumping stations it was assumed that a wet well would be provided with a capital cost given as follows:

$$\log_{10} (C_7 - 69000) = 3.9228 + 1.01 \log_{10} Qmax.$$

Where: C_7 = capital cost of wet well ($)

Qmax. = maximum daily demand (c.f.s.)

(vi) Annual Operating Cost for Low-Lift Pump Station. C_8:

$$\$ C_8 = HP\ 100/1.40$$

Where: HP = design pump horsepower

(vii) Annual Maintenance Cost C_9:

$$\$ C_9 = 0.01 (C_6 + C_7)$$

Where: C_6 = Capital cost of low-lift pumps station ($)

C_7 = Capital cost of wet well ($)

(viii) Annual Sinking Fund Cost for Plant Replacement C_{10} :

$$\$ C_{10} = 0.30 (C_2 + C_6) \frac{r (1 + i)^n}{(1+r)^n - 1}$$

Where: C_2 = Capital cost of treatment works ($), 30 percent of which is equipment.

C_6 = Capital cost of low-lift pump station ($), 30 percent of which is equipment.

r = Interest rate on borrowed capital (8 percent)

i = Inflation rate on capital costs (4.5 percent)

n = Replacement period in years (20)

(ix) Underutilization Factor:

Reference was made earlier to the fact that the unit operating cost (i.e. cents per 1,000 gallons) is increased when a treatment plant is operated below its designed capacity. To allow for this a typical growth curve was analyzed over a 10 year period and annual operating costs calculated taking into account both an inflationary cost increase of 4.5 percent and an empirically determined factor calculated as a function of the amount of underutilization.

These annual costs were then reduced to an equivalent total present value and compared with the present value which would have resulted from 100 percent utilization over the 10 year period with no allowance for inflation. The calculation was made as follows:

$$\text{Underutilization Factor} = 1.0 + 0.0323\ e^{p/0.2041}$$

Where: p = fraction of capacity not utilized

e = 2.7183 (base of natural logarithm).

Then the annual cost for the n^{th} year discounted to year 1 is given as:

AC_n (discounted) =

$$AC_n\ (1 + r - i)^{-n}\ u_n\ (1 + .0324\ e^{p_n/.2041})$$

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Where: u_n = fraction of capacity used

p_n = $1 - u_n$

r = interest on capital (8 percent)

i = inflation rate (4.5 percent)

$(1+r-i)$ = approximation of discounting rate.

Summing the values of AC_n (discounted) for a 10 year period and comparing with the standard calculation for discounted present worth indicated that the effect of utilization and inflation were compensating and the actual present worth could be approximated by the expression:

Present Worth = 1.014 x USPWF (10, 8 percent) x AC

Where:

USPWF (10,8%) = the uniform series present worth factor for 10 years at 8 percent.

AC = annual operating cost for full design capacity.

(x) Total Present Worth Cost C_{11}:

$$\$ C_{11} = (C_1 + C_2 + C_6) + (C_4 + C_7 + C_8 + C_9)\frac{1}{r}$$

Where: C_1 = Capital cost of intake

C_2 = Capital cost of treatment works

C_6 = Capital cost of low-lift pump station

C_7 = Capital cost of low-lift wet well

C_4 = Annual cost for treatment

C_8 = Annual cost for pump operation

C_9 = Annual cost for maintenance

C_{10} = Annual cost for sinking fund

$(1/r)$ = Uniform series present worth factor for ∞ years (r=8 percent).

7. Present Value of Phased Construction

In any of the proposed solutions, the construction of treatment works represents a considerable fraction of the total capital cost and considerable economization can be effected by postponing or phasing this investment. Deferrment of a major part of the construction may be possible, depending on:

(i) The quality of the source water, and

(ii) The anticipated rate of growth of demand in the region.

Whereas detailed discussion of the qualitative merits of alternate sources and the reliability of projected population and water requirements were presented in

Chapter 4 and 5 of this Volume, it will be appropriate to discuss here the method whereby the cost estimates were modified to reflect an assumed staging of construction. Two typical cases will be considered, one in which construction is completed in a single stage but at a deferred time, the other relating to the development of treatment capacity in two distinct stages.

a. <u>Deferred construction at (say) Windsor Lake</u>

Let capital cost of required treatment works be \$C in terms of 1974 values.

Assuming postponement of construction to (say) 1987, two corrections must be introduced, Firstly, the estimated cost should be increased to reflect an anticipated inflation rate of i percent. Hence the 1987 cost is expressed as:

$$F = C\ (1 + i)^n$$

Where: i = annual inflation rate (say 4.5 percent)
  and n = period of deferrment (say 13 years)

Secondly, the future investment must be expressed in terms of its present value, assuming interest rate on capital to be r percent. Thus the present worth is given by:

$$P = F\ \frac{1}{(1+r)^n}$$

$$P = C\ \frac{(1+i)^n}{(1+r)^n} = C\ \left\{\frac{1+i}{1+r}\right\}^n$$

The present value weighting factor is therefore equivalent to the ration (P/C). For example, for:

Inflation $i = 4.5$ percent,
Interest $r = 8.0$ percent,
Period $n = 13$ years:

$$\frac{P}{C} = \left(\frac{1.045}{1.080}\right)^{13} = \underline{\underline{0.6516}}$$

b. <u>Staged Construction at (say) Bay Bulls Big Pond</u>

For this more complex example, assume that the capital cost of the ultimate treatment works is \$C in 1974 costs, but that construction is to be carried out in two distinct stages as shown in the following schedule:

1974-75	-	40 percent of \$C
1976-77	-	40 percent of \$C
1985	-	20 percent of \$C

Each of these components must be converted to an inflated future cost and then discounted to 1974 values. Using the same figures as previously, the equivalent discounting rate is obtained by:

$$\left(\frac{1 + 0.045}{1 + 0.08}\right) = (1.03349)^{-1}$$

Thus:

$$\begin{aligned}
1974,\ 0.2C \times 1 &= 0.2000C \\
1975,\ 0.2C\ (1.0335)^{-1} &= 0.1935C \\
1976,\ 0.2C\ (1.0335)^{-2} &= 0.1872C \\
1977,\ 0.2C\ (1.0335)^{-3} &= 0.1812C \\
1985,\ 0.2C\ (1.0335)^{-11} &= 0.1392C
\end{aligned}$$

Hence the present value weighting factor is: $\underline{\underline{0.9011}}$

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These typical calculations demonstrate the sign-ficant economic advantage which can resu1t by deferrment of a major capital investment. It is of interest to observe the sensitivity of this trend to changes in the assumed inflation rate. Table 6.1 shows the variation in the present value weighting factors for each of the above examples, calculated for a range of inflation rates.

TABLE 6.1

VARIATION IN PRESENT VALUE WEIGHTING FACTORS

Inflation i%	Interest r%	$\frac{(1+r)}{(1+i)}$	Factor 1	Factor 2	Ratio 2/1
3.5	8.0	1.0435	0.5751	0.8766	1.5243
4.5	8.0	1.0335	0.6516	0.9011	1.3829
5.5	8.0	1.0237	0.7375	0.9272	1.2572

It will be noted that the advantage of the deferred scheme is diminished as the equivalent discounting factor $(1 + r)/(1 + i)$ reduces, irrespective of the absolute values of r and i.

IV. DESCRIPTION OF MODEL TESTS AND RESULTS

1. Summary of Tests

This section describes in outline form the schedule of tests carried out using the econometric model.

a. Location of a New Southern Source

Tests were carried out to compare the relative economics of using alternative sources of supply. Basically the choice lay between Thomas Pond and Bay Bulls Big Pond. The latter source could, in addition, be connected to the network in two alternative ways, either directly or indirectly by pumping raw water from Bay Bulls Big Pond to Petty Harbour Long Pond and thence to the network. The three alternative schemes considered were therefore:

(i) Windsor Lake and Thomas Pond.
(ii) Windsor Lake and Bay Bulls Big Pond
(iii) Windsor Lake and Bay Bulls Big Pond via Petty Harbour Long Pond.

b. Determination of the Economic Limit of the Network

The objective in this series of tests was to determine the incremental change in the total cost of the system as a result of adding or deleting various remote communities from the regional network. By examining each of these remote communities in a systematic way it was possible to determine the marginal cost for each and to rank the communities in order of decreasing marginal cost. This information gives a sound basis for decision making with respect to the size of the network.

c. Consideration of Distributed Sources

As a result of second stage of testing, the marginal communities may be identified and certain specific proposals considered for providing local or sub-regional water supply systems. The areas studied lie in the extreme North-West and South-West of the region and two alternative sources of supply were considered in order to compare the economy of reduced conveyance cost with the diseconomy resulting from an increased number of smaller treatment works.

2. Definition of Input Data

System definition is defined briefly in Section II.1.b above, and in Section III.1 the concept of a network is described. To define this network two basic forms of data are required. Firstly, each of the nodes must be given some form of identification together with data describing the yield or demand, the pressure elevation and other applicable factors. Secondly, it is necessary to describe in a systemetic way the manner in which these nodes are interconnected. This is done by listing, for each interconnecting link, the numbers of the two nodes at the extremeties, the length of the link and other data relating to cost factors and initial approximations to the flows in each link. A typical data file is shown in Appendix I and may be interpreted by means of the detailed notes which follow.

Lines 1 - 2: Heading
Lines 3 - 5: No. of Links, No. of Nodes and No. of Reservoirs.

Lines 6 - 9: Details of Reservoir Nodes comprising Node No; Node Name, Pressure Elevation (feet), Reliable Average Yield (m.g.d.), Present Worth Cost Factor as described in the previous section, and Pump Lift (feet) from draw down reservoir level to treatment works.

Lines 10-45: Details of Demand Nodes comprising Node No., Node Name, Pressure Elevation (feet), Average Demand (m.g.d.) and Factor by which the maximum demand may exceed the average due to fire-fighting requirements, or cyclic monthly variations in water use.

Lines 46-end: Details of Links comprising of "Upstream and "Downstream" Node No., Length of Link (feet), Flows as initial approximations in "downstream" or "upstream" direction, and Cost Factor to reflect variation in reinstatement costs or traffic control overheads.

It will be noted that a number of the nodes are used simply as junctions in the network and do not impose any demand on the system. Also, it should be stressed that the choice of "upstream" and "downstream" directions in the description of the links is entirely arbitrary, the model determining the direction of flow in the course of optimization.

3. Output from the Model

One of the difficulties in using a computer simulation model is the danger of generating more information than can be absorbed by the decision maker, or of presenting the result in a manner which makes practical interpretation by the engineer more difficult than necessary. The program developed for the econometric model was specially designed so that it could be employed either in a time-sharing (interactive) mode or in batch runs. Options were included which allowed the user to modify the network during successive runs and to suppress the detailed output until required.

Appendix II illustrates a typical output file in which details of each non-trivial link in the final solution are displayed together with summaries of the cost. In order to aid practical implementation, details were included of the theoretical pipe diameter and installed horsepower as well as flow quantity and friction loss. This is valuable in adapting a design to commercially available sizes and making other adjustments dictated by practical considerations. The costs are broken down into capital expenditures and annual operating costs and a distruction is made also between costs for production and cost of conveyance. It should be noted that the total costs are in a sense somewhat artificial since the calculation is carried out on the assumption that all of the conveyance links are to be constructed in year 1 but that treatment works will be provided in stages to match growth in demand. This fact does not reduce the value of the data as a basis for economic comparison between alternate ultimate configurations. To enable simple cost comparison to be made between such alternatives, the capital and operating costs for both production and conveyance are combined in an equivalent present worth sum. Once again it must be stressed that this is a basis for comparison and may differ from the actual expenditure depending on the staging of the works.

4. Comparison of Alternative Schemes

Section IV. 1.a lists the three main alternative schemes to be initially examined. To provide for the estimated ultimate demand of 27.47 mgd the following reliable yields were assumed to be available (see discussion on reliable yields in Chapter 5).

Windsor Lake	10.0 mgd
Petty Harbour Long Pond	4.0 mgd
Bay Bulls Big Pond	23.0 mgd
Thomas Pond	18.0 mgd

The figure for Thomas Pond is contingent on augmentation of the storage volume that presently exists. The estimated cost for supplementary storage by means of new works is in the order of $1.0 million. This sum has been added to the total present worth as computed by the model, and is discussed later in this section.

The provision of treatment works at Windsor Lake was assumed in every case to be deferred until 1985. Construction of treatment facilities at any one of the alternative sources was assumed to be approximately as outlined in Section III. 7.b and the present worth factor was kept constant for each of the new southern sources.

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The water quality of Thomas Pond necessitated the inclusion of clarifiers and gravity filters as the treatment facilities at this source, whereas direct filtration would be provided at either Bay Bulls Big Pond or Petty Harbour Long Pond. Cost differentiation for this is built into the model following the cost functions defined in Section III. 6.

In addition to the production and conveyance costs for each of the schemes, additional costs must be included to allow for compensation to the Newfoundland Light and Power Company. As mentioned in our Positional Report No. 2 annual compensation of $50,000 and $25,000 may be charged to the Thomas Pond and Bay Bulls Big Pond schemes, respectively for loss of power potential due to abstraction for water supply. In order to combine this annual charge with the total present worth as computed by the model, the annual cost is compounded at 8 percent for an indefinite period to yield an equivalent present worth sum which is then added to the computed cost.

The result of these analyses are summarized in Table 6.2 in which, for each scheme the capital and operating costs for both production and conveyance are given. These two components are combined in a total present worth cost to which is added the adjustments due to power compensation (applicable in all 3 cases) and the cost of augmenting the storage at Thomas Pond. A schematic layout of the three schemes can be seen on Drawings No. 6.1, 6.2, 6.3. Even without these adjustments it is evident that the "Windsor Lake-Bay Bulls Big Pond" scheme shows substantial economies compared to the others.

TABLE 6.2

SUMMARY OF SYSTEMS ANALYSIS

System (Defined by Sources)	Cost Category	All Costs Expressed in Millions of Dollars					
		Capital Present Value	Annual Cost (1974 Dollars)	Total Present Worth	Compensation Present Cost	Present Worth of Additional Storage	Total Present Worth
Windsor Lake and Bay Bulls Big Pond via Goulds	Conveyance	16.527	0.310	20.016			
	Production	12.627	0.987	23.743			
	Total	29.154	1.297	43.759	0.313	0.0	44.072
Windsor Lake and Bay Bulls Big Pond via Petty Harbour Long Pond	Conveyance	17.999	0.371	21.250			
	Production	12.613	1.039	24.311			
	Total	30.413	1.410	45.562	0.313	0.0	45.875
Windsor Lake and Thomas Pond	Conveyance	18.035	0.323	21.677			
	Production	14.205	1.064	26.189			
	Total	32.241	1.387	47.856	0.625	1.000	49.481

With the adjusted costs, the distinction is reinforced and the conclusion may therefore be made that future additional supplies of water to the region should be provided directly from Bay Bulls Big Pond.

5. Economic Delineation of Network

Having determined which of the alternative water sources should be used, the next step is to determine the contribution made by each community to the total capital and operating cost of the system. This determination need be done only for the more remote communities such as those located in the North-West and extreme South-West ends of the study region. A systematic examination of some 5 or 6 communities gives valuable insight into the economic viability of different extents of the regional system. Table 6.3 shows the contribution of several communities to both demand in mgd and total present value cost. These incremental differences are then combined in a cost per mgd to yield a basis for ranking the communities in order of decreasing marginal cost. At first sight it may seem unreasonable that small communities quite close to Windsor Lake should generate such high marginal or incremental costs. It should be remembered however that as the yield from Windsor Lake is already committed to the large urban areas, addition of a further demand in the North-West requires that additional water be produced at Bay Bulls Big Pond and conveyed northward through pipes of slightly increased size. The phenomenon of increasing marginal costs is graphically illustrated in Figure 6.2 in which the slope of the curve is

TABLE 6.3

MARGINAL COST OF THE SIX MOST EXPENSIVE COMMUNITIES

Community	Total Present Worth of Supplying Community (M$)	Demand of Community (MGD)	Marginal Cost (M$/MGD)
Gullies and Seal Cove	0.947	0.25	3.78
Thorburn Rd.	0.919	0.27	3.40
St. Phillips and Hogans Pond	0.534	0.16	3.34
Portugal Cove	0.388	0.12	3.24
Portugal Cove Rd.	0.969	0.36	2.69
Torbay & Torbay Rd (Partial)	0.985	0.43	2.29
For Comparison: Foxtrap	0.868	0.43	2.02

proportioned to the incremental cost per unit of demand ($million/mgd). The fact that the core area of St. John's including the expansion zones, NewTown, Mount Pearl and Donovans Park incurs an incremental cost of less than $1.5 million/mgd serves to emphasise the higher financial cost of providing properly treated, piped potable water in scattered rural communities, It is from this point that the economic desirability of distributed, local water supplies becomes of interest; this will be discussed in the following section. Before leaving this subject, however, it is important to note that the curve of Figure 6.2 illustrates a rather pronounced change in slope above Portugal Cove Road. Indeed the diagram may be approximately by two straight lines corresponding to communities above and below Portugal Cove Road. This fact suggests that whatever solution is implemented there appears to be good reason to treat the upper five communities as economically homogeneous. Apart from Seal Cove and Gullies, the other communities are also more or less geographically compact and it is recommended that the treatment for all of the communities - St. Phillips, Portugal Cove, Portugal Cove Road and Thorburn Road - be uniform. That is they should all be included in the regional system; they should be left out of the system or they should be linked to a local sub-regional water supply and distribution system. The same arguments are less easily made for Gullies and Seal Cove on account of the relatively small total demand, and the remoteness from other municipal centres.

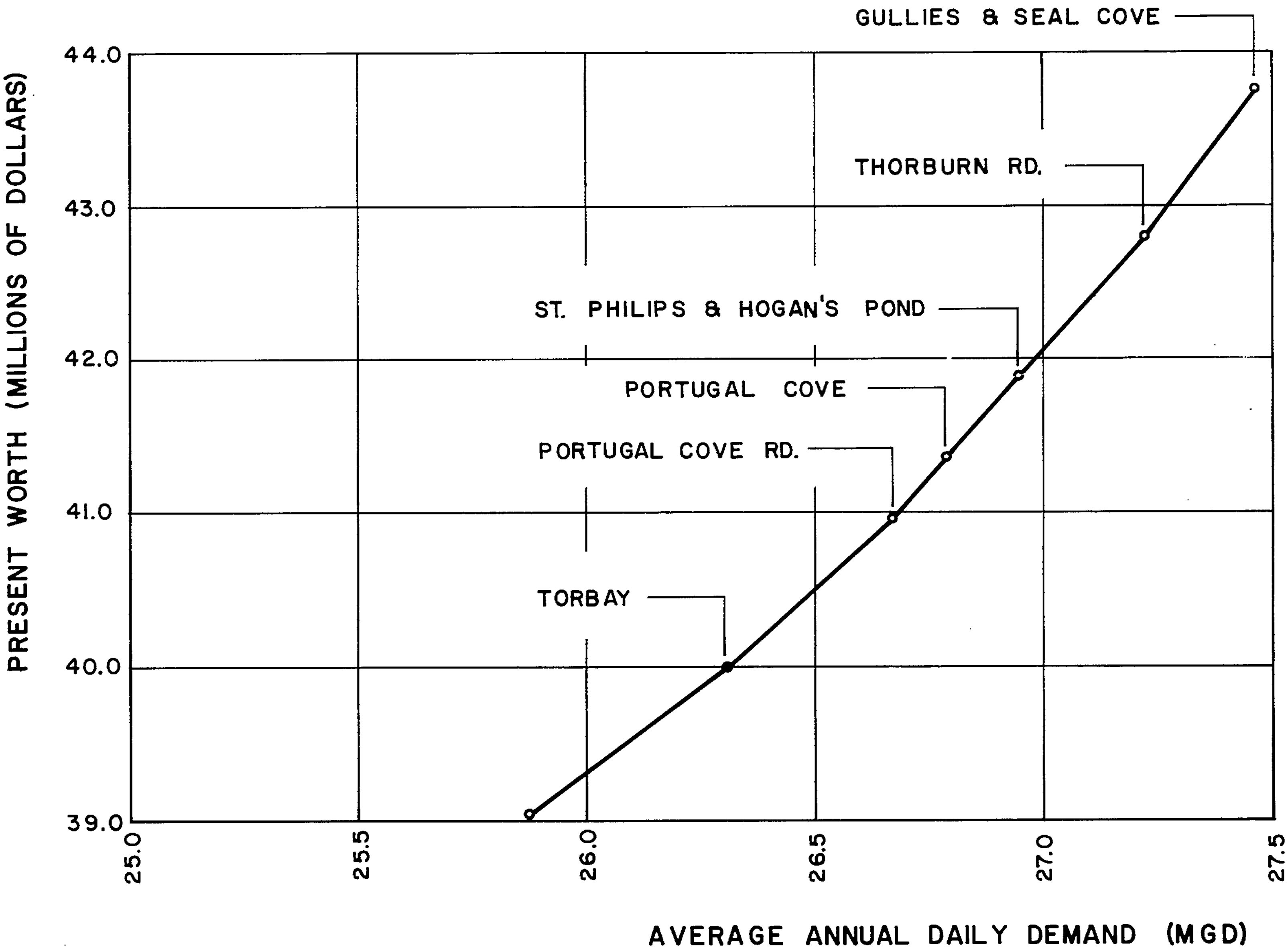

FIGURE 6·2

IDENTIFICATION OF MARGINAL COMMUNITIES

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6. Feasibility of Local Sources

In the previous section evidence was presented in the form of marginal cost rates which suggests that the regional network should perhaps exclude remote communities in the North-West (St. Phillips, Portugal Cove, Portugal Cove Rd., and Thorburn Road), and the South-West (Seal Cove and Gullies). Alternative methods should, therefore, be examined for supplying these areas with an equivalent standard of service.

a. North-West Communities

A possible solution for this area may be found in the development of the source at Little Powers Pond currently used by the City of St. John's for water augmentation in Windsor Lake. Hydrologic study indicates that the reliable yield would certainly be not less than the ultimate demand of 0.91 mgd likely to develop in this area.

Little Powers Pond could be utilised in a number of alternatives, as follows:

(i) As a local sub-regional source; provided with local treatment facilities of the packaged plant type, and conveyance mains to the four North-West Communities.

(ii) As a regional source used to augment Windsor Lake. Treatment facilities will be at Windsor Lake and conveyance mains to

Portugal Cove Road, Portugal Cove, and St. Phillips will extend from Windsor Lake, whereas Thorburn Road will be supplied from an extension to the Kenmount Road conveyance main.

(iii) As a regional source used to augment Windsor Lake. However, treatment facilities and conveyance mains to the North-West Communities will be part of a sub-regional system emanating from Round Pond.

(iv) Abandoning the concept of a regional or sub-regional system and providing a water supply system to each community, or a pair of communities, from a local source. In this regard we have identified Little Powers Pond as the source for St. Phillips and Thorburn Road, whereas Round Pond would be the source for Portugal Cove and Portugal Cove Road. Little Powers Pond would also be utilised to augment Windsor Lake via the existing facilities.

Comparison of the regional scheme without supplementary pumping and of the above four alternative schemes for the North-West Communities is given in Table 6.4.

All of the above five schemes indicate that the supplying of water to the North-West Communities is of significant higher cost than supplying water to the other areas of the region, regardless of which system is chosen.

TABLE 6.4

COMPARISON OF ALTERNATIVE SCHEMES FOR THE NORTHWEST COMMUNITIES

Scheme	System Component	Capital Cost Million Dollars	Annual Cost Million Dollars	Total Present Value Million Dollars
1. Inclusion in Regional - No Supplementary Pumping	Conveyance Production Totals	1.843 0.366 2.209	0.030 0.023 0.053	2.176 0.629 2.805
2. Separate Supply from Little Powers Pond with Local Treatment Works.	Conveyance Production Totals	1.344 0.315 (a) 1.659	0.023 0.070 0.093	1.609 1.120 2.729
3. Inclusion in Regional Network with Supplementary Pumping from Little Powers Pond to Windsor Lake	Conveyance Production Totals	1.847 0.369 2.216	0.034 0.024 0.058	2.221 0.637 2.858
4. Separate Supply from Round Pond with Local Treatment Works and Suplementary Pumping from Little Powers Pond to Windsor Lake	Conveyance Production Totals	1.013 0.315(a) 1.328	0.036 0.070 0.106	1.427 1.120 2.547
5. Separate Supply from Little Powers Pond and Round Pond with Local Treatment Works	Conveyance Production Totals	0.600 0.350 0.950	0.026 0.090 0.116	0.899 1.386 2.285

Notes: (a) 0.90 Present Worth Factor included for 10 year deferrment of 36 percent capital cost of treatment works.

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TABLE 6.5

COMPARISON OF COSTS FOR REGIONAL AND NORTH-WEST WATER SUPPLY

SCHEME	PRESENT WORTH		
	North-West Supply Scheme	Regional Scheme (a)	Regional System (b)
1. Inclusion in Regional. No supplementary pumping.	2.805	41.267	43.072
2. Separate Supply from Little Powers Pond with Local Treatment Works.	2.729	41.267	43.996
3. Inclusion in Regional Network with Supplementary Pumping from Little Powers Pond to Windsor Lake.	2.858	40.900	43.758
4. Separate Supply from Round Pond with Local Treatment Works and Supplementary Pumping from Little Powers Pond to Windsor Lake.	2.547	40.900	43.447
5. Separate Supply from Little Powers Rond with Local Treatment Works.	2.285	40.900	43.185

(a) Excluding North-West Communities
(b) Including North-West Communities

In the final analysis of the above schemes, one should consider the total cost to supply the needs of the region. The results of this analysis are presented in Table 6.5.

From the foregoing it appears that supply of water to the North-West Communities from a local source and treatment works has marginal economic advantages, but the difference in cost is so small that the decision should be based on other reasons such as the regional aspect of water supply. In this regard, augmentation of Windsor Lake from Little Powers Pond with a regional or sub-regional system to the North-West area (alternative schemes 3 or 4 in Table 6.5) appear to be the most favourable. By using the existing facilities at Little Powers Pond to augment Windsor Lake, the large storage capacity of the latter is utilised to enlarge the reliable yield of Little Powers Pond, thereby increasing the yield of the entire proposed system. In addition, it should be noted that any of the regional schemes considered above provides a watermain along the artery of Thorburn Road that could be used, if required, for local supply of water. This watermain is not included in any of the local or sub-regional schemes.

b. South-West Communities

The situation with respect to the Conception Bay south communities of the Gullies and Seal Cove is rather more clear-cut. The marginal costs of water supply to these communities is substantial and significantly different from that for the adjacent communities at Kelligrews and Foxtrap (See Table 6.3). The consideration of Thomas Pond as a local sub-regional source for the Conception Bay South communities shows that the economies of scales resulting from the adoption of a single major new source with treatment works outway the costs of conveyance by $2.95 million.

V. RECOMMENDATIONS AND CONCULSIONS

At the start of this chapter the objectives of the ecometric study were outlined. They may be summarized as follows:

(i) Selection of the new source of supply.
(ii) Determination of the economic limit of the regional network.
(iii) Recommendations relating to communities with high incremental costs for supply.

This section will deal with the first point and the last two points separately.

a. Selection of New Source

To supplement the supply available from Windsor Lake, three possible sources were considered. These are:

(i) Direct supply from Bay Bulls Big Pond
(ii) Supply from Bay Bulls Big Pond via Petty Harbour Long Pond.
(iii) Direct supply from Thomas Pond.

In addition to these alternative schemes, it later became apparent that the small but significant supply at Little Powers Pond could be made use of most efficiently as direct augmentation of Windsor Lake.

In making cost comparisons, account was taken not only of the different qualities of water from the alternative sources and the consequent variation in treatment costs, but also of the economic benefits of staging or deferring large capital intensive components of the system. Allowance was also made for peripheral costs such as storage augmentation and power compensation.

The final conclusion from these studies is that the supply should be obtained from Bay Bulls Big Pond connected directly to the network via a bifurcation point at Ruby line (See Chapter 7) and with treatment works located near that source. In addition, the supply from Little Powers Pond should be used to augment Windsor Lake.

b. Economic Limits of the Network

The decision concerning the scope of a regional supply system must be based on economic, political and subjective arguements. Only the first of these is considered here. To provide an objective basis for decision making, the marginal or implemental costs for inclusion of peripheral communities was obtained

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by successive analyses. Communities in the North-West and South-West were found of significant higher marginal costs as would be expected. However, further analysis showed that provision of an equivalent supply to the North-West area could not be obtained from a local or sub-regional source with any significant saving. The decision must then be social-political in nature and it is pointed out that in making such a decision a scheme based on regional concepts will be advantageous.

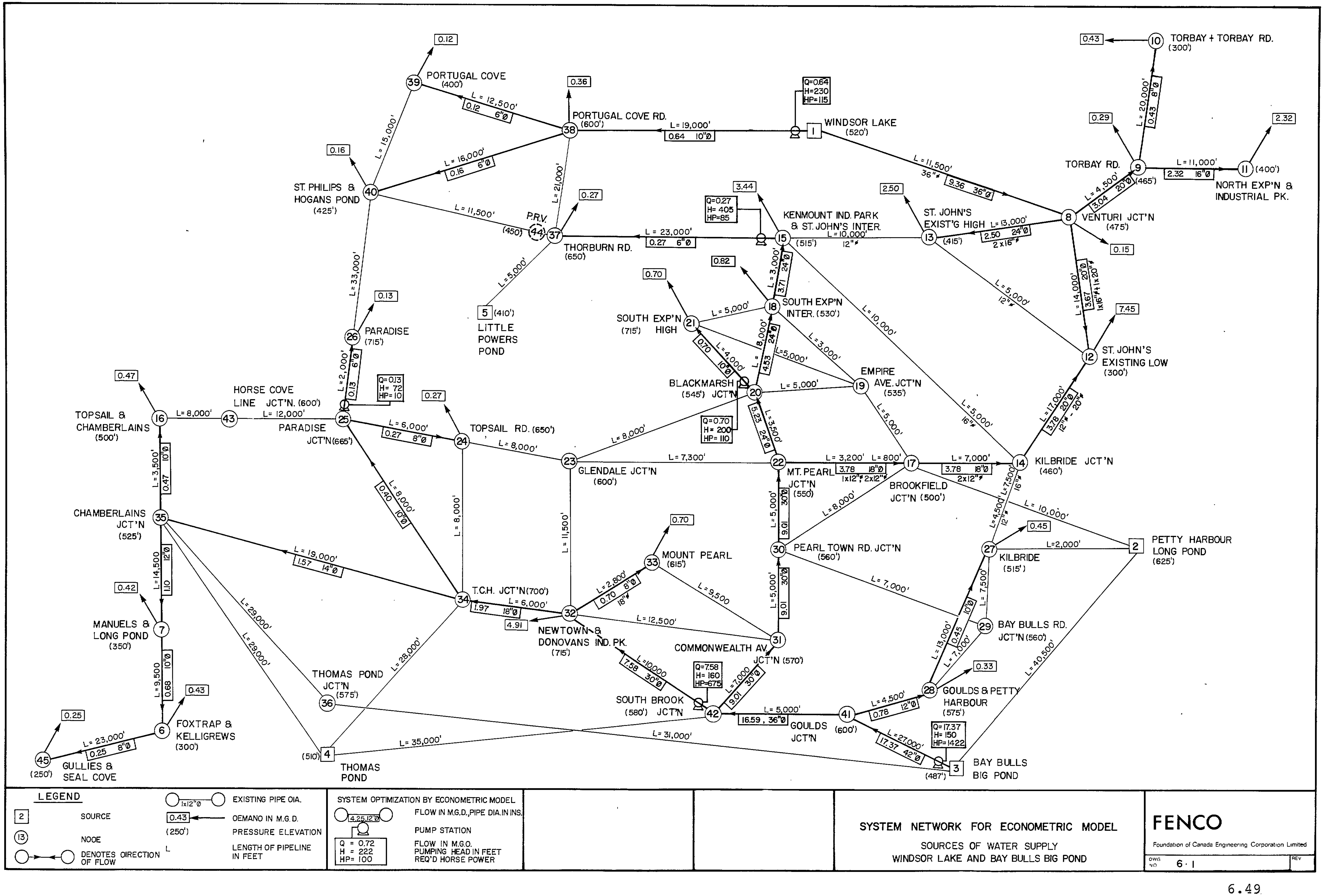
In conclusion, it is recommended that the St. John's Regional Water System comprise the following communities (grouped in the categories used in Chapter 4):

(i) Regional Centre - St. John's and expansion zones, Mount Pearl, New Town, Kilbride, Wedgewood Park, Shea Heights, (Blackhead Road).

(ii) Sub-Regional Centre - Conception Bay South Area (Seal Cove, Gullies, Kelligrews, Foxtrap, Long Pond, Manuels, Chamberlains, Topsail).

(iii) Local Centres "A" - Paradise, Topsail Raod, Torbay, Torbay Road, Penetanguishene, Goulds Petty Harbour.

The North-West communities of St. Phillips, Portugal Cove, Portugal Cove Road and Thorburn Road (grouped under Local Centres "B") require a relatively expensive system and from an economic view point any solution to supply them with water should receive a low priority. The most favourable scheme for these communities was outlined above, however, other factors aside from economics may affect the final decision for a water supply system to these communities. In any event, the system as recommended above (and developed in Chapter 7), will be able to provide water to the North-West area regardless of the scheme that would eventually be selected for this area.


WINDSOR LAKE (520')
Q=0.64 H=230 HP=115
PORTUGAL COVE RD. (600')
L= 19,000' 0.64 10"Ø
PORTUGAL COVE (400')
L = 12,500' 0.12 6"Ø
ST. PHILIPS & HOGANS POND (425')
L= 16,000' 0.16 6"Ø
L= 15,000'
L= 21,000'
L= 11,500'
P.R.V.
THORBURN RD. (650')
L= 23,000' 0.27 6"Ø
Q=0.27 H= 405 HP=85
L=5,000'
LITTLE POWERS POND (410')
KENMOUNT IND. PARK & ST. JOHN'S INTER. (515')
L =10,000' 12"ø
ST. JOHN'S EXIST'G HIGH (415')
L= 13,000' 2.50 24"Ø 2 x 16"ø
L=11,500' 36"ø 9.36 36"Ø
VENTURI JCT'N (475')
TORBAY RD. (465')
L= 4,500' 3.04 20"Ø
L= 20,000' 0.43 8"Ø
TORBAY + TORBAY RD. (300')
L= 11,000' 2.32 16"Ø
NORTH EXP'N & INDUSTRIAL PK. (400')
L= 14,000' 3.67 20"Ø 1x16"ø+1x20"ø
L= 5,000' 12"ø
ST. JOHN'S EXISTING LOW (300')
L= 17,000' 3.78 20"Ø 12"ø - 20"ø
KILBRIDE JCT'N (460')
L= 10,000'
L= 5,000' 16"ø
SOUTH EXP'N INTER. (530')
L= 3,000' 3.71 24"Ø
SOUTH EXP'N HIGH (715')
L= 5,000'
L= 4,000' 0.70 10"Ø
BLACKMARSH JCT'N (545')
L= 8,000' 4.53 24"Ø
L= 5,000'
L= 3,000'
EMPIRE AVE. JCT'N (535')
L= 5,000'
Q=0.70 H= 200 HP= 110
L= 3,500' 5.23 24"Ø
HORSE COVE LINE JCT'N. (600')
PARADISE (715')
L= 33,000'
L= 2,000' 0.13 6"Ø
Q=0.13 H= 72 HP= 10
TOPSAIL & CHAMBERLAINS (500')
L= 8,000'
L= 12,000'
PARADISE JCT'N (665')
L= 6,000' 0.27 8"Ø
TOPSAIL RD. (650')
L= 8,000'
GLENDALE JCT'N (600')
L= 7,300'
MT. PEARL JCT'N (550)
L= 3,200' L= 800' 3.78 18"Ø 1x12"ø+2x12"ø
BROOKFIELD JCT'N (500')
L= 7,000' 3.78 18"Ø 2x12"ø
L= 7,500' 16"ø
L= 10,000'
PETTY HARBOUR LONG POND (625')
L=2,000'
KILBRIDE (515')
L=4,500' 12"ø
L= 8,000'
L= 5,000' 9.01 30"Ø
PEARL TOWN RD. JCT'N (560')
L= 7,000'
L= 3,500' 0.47 10"Ø
CHAMBERLAINS JCT'N (525')
L= 19,000' 1.57 14"Ø
L= 8,000' 0.40 10"Ø
L= 8,000'
L= 11,500'
MOUNT PEARL (615')
L= 2,800' 0.70 8"Ø 18"ø
L= 9,500
L= 5,000' 9.01 30"Ø
T.C.H. JCT'N (700')
L = 6,000' 1.97 18"Ø
NEWTOWN & DONOVANS IND. PK. (715')
L= 12,500'
COMMONWEALTH AV. JCT'N (570')
L= 14,500 1.10 12"Ø
MANUELS & LONG POND (350')
L= 29,000'
L= 29,000'
L= 28,000'
L= 9,500 0.68 10"Ø
FOXTRAP & KELLIGREWS (300')
THOMAS POND JCT'N (575')
L= 10,000 7.58 30"Ø
Q=7.58 H= 160 HP=675
L= 7,000' 9.01 30"Ø
SOUTH BROOK JCT'N (580')
L= 5,000' 16.59, 36"Ø
GOULDS JCT'N (600')
L= 4,500' 0.78 12"Ø
GOULDS & PETTY HARBOUR (575')
L= 13,000' 0.45 10"Ø
L= 7,000'
L= 7,500'
BAY BULLS RD. JCT'N (560')
L= 40,500'
L= 23,000' 0.25 8"Ø
GULLIES & SEAL COVE (250')
L= 35,000'
THOMAS POND (510')
L= 31,000'
L= 27,000' 17.37 42"Ø
Q= 17.37 H= 150 HP=1422
BAY BULLS BIG POND (487')
0.12 0.36 0.16 0.27 0.13 0.47 0.27 3.44 0.82 0.70 2.50 0.15 7.45 0.43 0.29 2.32 0.42 0.43 0.25 0.70 4.91 0.45 0.33
LEGEND
SOURCE
NODE
DENOTES DIRECTION OF FLOW
1x12"Ø EXISTING PIPE DIA.
0.43 DEMAND IN M.G.D.
(250') PRESSURE ELEVATION
L LENGTH OF PIPELINE IN FEET
SYSTEM OPTIMIZATION BY ECONOMETRIC MODEL
4.25,12"Ø FLOW IN M.G.D., PIPE DIA. IN INS.
PUMP STATION
Q = 0.72 FLOW IN M.G.D.
H = 222 PUMPING HEAD IN FEET
HP= 100 REQ'D HORSE POWER
SYSTEM NETWORK FOR ECONOMETRIC MODEL
SOURCES OF WATER SUPPLY
WINDSOR LAKE AND BAY BULLS BIG POND
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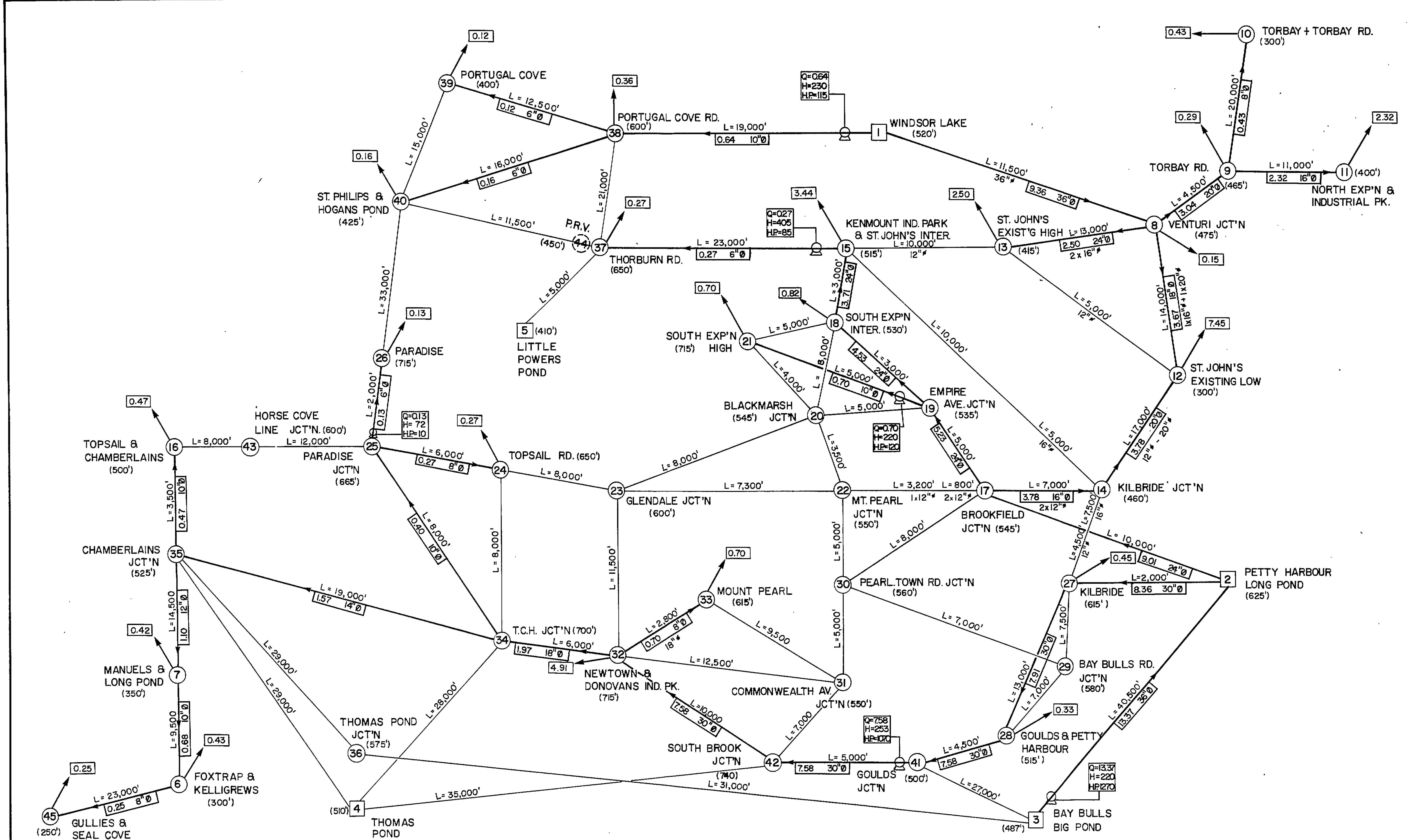
DWG NO 6·1
REV



LEGEND

2	SOURCE	1x12"Ø	EXISTING PIPE DIA.
13	NODE	0.43	DEMAND IN M.G.D.
	DENOTES DIRECTION OF FLOW	(250')	PRESSURE ELEVATION
		L	LENGTH OF PIPELINE IN FEET

SYSTEM OPTIMIZATION BY ECONOMETRIC MODEL

4.25,12"Ø	FLOW IN M.G.D., PIPE DIA. IN INS.
	PUMP STATION
Q = 0.72 H = 222 HP = 100	FLOW IN M.G.D. PUMPING HEAD IN FEET REQ'D HORSE POWER

SYSTEM NETWORK FOR ECONOMETRIC MODEL

SOURCES OF WATER SUPPLY
WINDSOR LAKE, BAY BULLS BIG POND AND PETTY HARBOUR LONG POND

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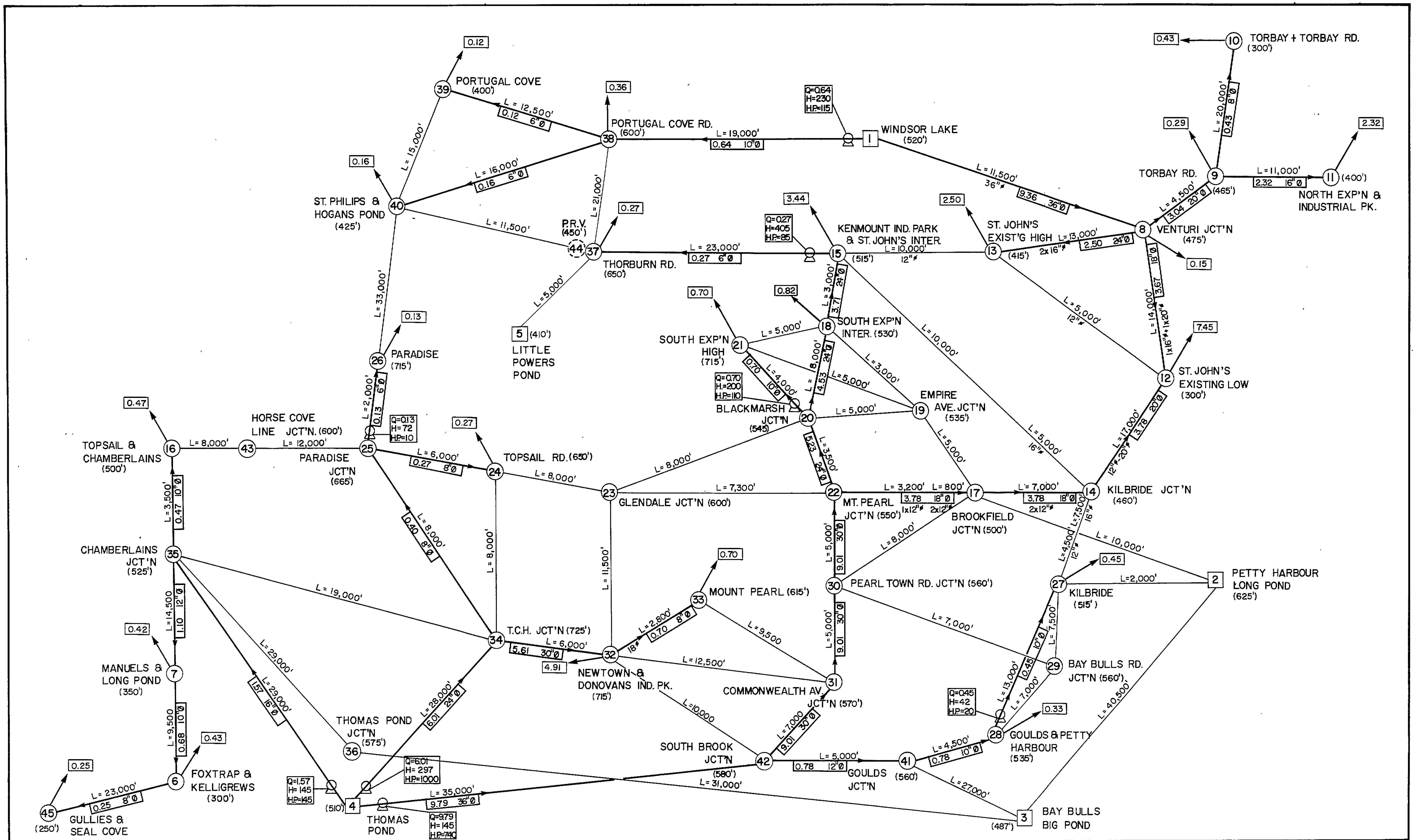

LEGEND

2	SOURCE	1x12"Ø	EXISTING PIPE DIA.
13	NODE	0.43	DEMAND IN M.G.D.
	DENOTES DIRECTION OF FLOW	(250')	PRESSURE ELEVATION
		L	LENGTH OF PIPELINE IN FEET

SYSTEM OPTIMIZATION BY ECONOMETRIC MODEL

4.25, 12"Ø	FLOW IN M.G.D., PIPE DIA. IN INS.
	PUMP STATION
Q = D.72 H = 222 HP = 100	FLOW IN M.G.D. PUMPING HEAD IN FEET REQ'D HORSE POWER

SYSTEM NETWORK FOR ECONOMETRIC MODEL

SOURCES OF WATER SUPPLY

WINDSOR LAKE AND THOMAS POND

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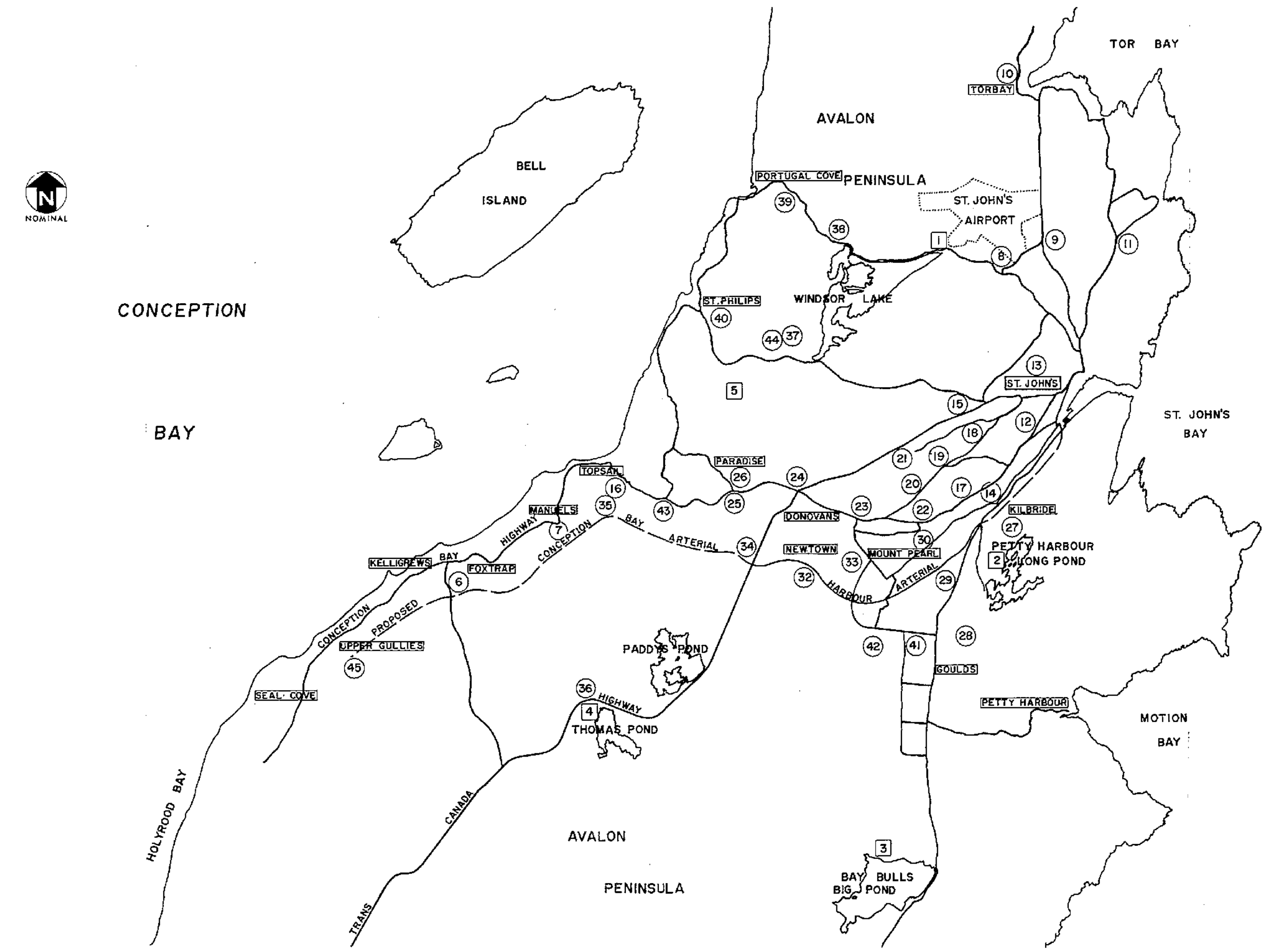

GEOGRAPHICAL LOCATIONS OF ECONOMETRIC MODEL NODES

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APPENDIX I

TYPICAL INPUT DATA

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PROPOSED SCHEME FOR ST JOHNS WATER SUPPLY                                  000100
  BAY BULLS VIA GOULDS                                                     000110
 71 LINKS                                                                  000120
 45 NODES                                                                  000130
  5 RESERVOIRS                                                             000140
 1   WINDSOR LAKE              520.0      9.5   0.6394     40.0            000150
 2   PETTY HARBOUR L.P.        625.0      0.0   0.0       100.0            000160
 3   BAY BULLS B.P.            487.0     23.0   0.8950      0.0            000170
 4   THOMAS POND               510.0      0.0   0.0         0.0            000180
 5   LITTLE POWERS POND        480.0      0.0   0.0         0.0            000190
 6   FOXTRAP/KELLIGREWS        300.0      0.43  1.5                        000200
 7   MANUELS/LONG POND         350.0      0.42  1.5                        000210
 8   VENTURI JNCTN             475.0      0.15  1.5                        000220
 9   TORBAY RD                 465.0      0.29  1.5                        000230
10   TORBAY                    300.0      0.43  1.5                        000240
11   NORTH EXPN/INDUS PK       400.0      2.32  1.5                        000250
12   ST JOHN+S LOW             300.0      7.45  2.05                       000260
13   ST JOHN+S HIGH            415.0      2.50  3.17                       000270
14   KILBRIDE JNCTN            460.0      0.0   1.5                        000280
15   ST JOHN+S INTER           515.0      3.44  2.60                       000290
16   TOPSAIL/CHAMBERLAINS      525.0      0.47  1.50                       000300
17   BROOKFIELD JNCTN          545.0      0.0   1.5                        000310
18   SOUTH EXPN INTER          530.0      0.82  1.50                       000320
19   EMPIRE AVE JNCTN          535.0      0.0   1.50                       000330
20   BLACKMARSH JNCTN          545.0      0.0   1.50                       000340
21   SOUTH EXPN HIGH           715.0      0.70  1.50                       000350
22   MT. PEARL JNCTN           550.0      0.0   1.5                        000360
23   GLENDALE JNCTN            600.0      0.0   1.5                        000370
24   TOPSAIL RD                650.0      0.27  1.5                        000380
25   PARDISE JNCTN             665.0      0.0   1.5                        000390
26   PARADISE                  715.0      0.13  1.5                        000400
27   KILBRIDE                  515.0      0.45  1.5                        000410
28   GOULDS/PETTY HARBOUR      575.0      0.33  1.5                        000420
29   BAY BULLS JNCTN           560.0      0.0   1.5                        000430
30   PEARL TOWN RD JNCTN       560.0      0.0   1.5                        000440
31   COMMONWEALTH RD JNCT      570.0      0.0   1.5                        000450
32   NEWTOWN/DONOVANS          715.0      4.91  1.5                        000460
33   MOUNT PEARL               615.0      0.70  1.5                        000470
34   TRANS CAN HWY JNCTN       700.0      0.0   1.5                        000480
35   CHAMBERLAINS JNCTN        500.0      0.0   1.5                        000490
36   THOMAS POND JNCTN         575.0      0.0   1.5                        000500
37   THORBURN RD               650.0      0.27  1.5                        000510
38   PORTUGAL COVE RD          600.0      0.36  1.5                        000520
39   PORTUGAL COVE             400.0      0.12  1.5                        000530
40   ST PHILIPS/HOGANS PD      425.0      0.16  1.5                        000540
41   GOULDS JNCTN              600.0      0.0   1.5                        000550
42   SOUTH BROOK JNCTN         580.0      0.0   1.5                        000560
43   HORSE COVE LINE JNCT      600.0      0.0   1.5                        000570
44   DUMMY NODE FOR PRV        450.0      0.0   1.5                        000580
45   GULLIES/SEAL COVE         250.0      0.25  1.5                        000590
   1    8   11500.0    9.5      0.0     1.0                                000600
   8    9    4500.0    3.04     0.0     1.0                                000610
   9   10   20000.0    0.43     0.0     1.2                                000620
   9   11   11000.0    2.32     0.0     1.2                                000630
   8   12   14000.0    4.4      0.0     1.0                                000640
   8   13   13000.0    2.7      0.0     1.0                                000650
  12   13    5000.0    0.5      0.0     1.0                                000660
  12   14   17000.0    0.0      2.3     1.4                                000670
  13   15   10000.0    0.3      0.0     1.4                                000680
  14   15   15000.0    0.3      0.3     1.0                                000690
  14   27   12000.0    1.0      1.0     1.2                                000700
   2   17   10000.0    5.0      0.0     1.2                                000710
   2   27    2000.0    1.5      0.0     1.0                                000720
   2    3   40500.0    0.0     12.0     1.2                                000730
  27   29    7500.0    1.0      1.0     1.2                                000740
  28   29    7000.0    0.0      0.33    1.2                                000750
  27   28   13000.0    1.0      1.0     1.2                                000760
  41   28    4500.0   10.0      0.0     1.0                                000770
   3   41   27000.0   10.0      0.0     1.0                                000780
  29   30    7000.0    1.0      1.0     1.2                                000790
  17   14    7000.0    2.3      0.0     1.0                                000800
  17   19    5000.0    1.0      0.0     1.2                                000810
  19   18    3000.0    1.0      0.0     1.2                                000820
  15   18    3000.0    0.0      1.0     1.4                                000830
  18   20    8000.0    1.0      1.0     1.2                                000840
  20   19    5000.0    1.0      0.0     1.2                                000850
  19   21    5000.0    1.0      1.0     1.2                                000860
  20   21    4000.0    0.35     0.35    1.2                                000870
  18   21    5000.0    0.35     0.35    1.2                                000880
  22   20    3500.0    0.50     0.0     1.2                                000890
  22   17    4000.0    0.50     0.0     1.2                                000900
  30   17    8000.     1.0      0.0     1.2                                000910
  30   22    5000.0    1.0      1.0     1.5                                000920
  31   30    5000.0    1.0      1.0     1.2                                000930
  42   31    7000.0    2.0      1.0     1.0                                000940
  41   42    5000.0    3.       0.0     1.0                                000950
  31   33    9500.0    0.7      0.0     1.2                                000960
   3   36   31000.0    1.0      0.0     1.0                                000970
  31   32   12500.0    1.0      1.0     1.0                                000980
  42   32   10000.0    3.0      1.0     1.0                                000990
  32   33    2800.0    1.0      1.0     1.0                                001000
  32   23   11500.0    2.0      4.0     1.2                                001010
  20   23    8000.0    1.0      1.0     1.2                                001020
  22   23    7300.0    4.0      1.0     1.2                                001030
  42    4   35000.0    2.0      5.0     1.0                                001040
   4   34   28000.0    5.0      1.0     1.0                                001050
  32   34    6000.0    2.0      2.0     1.0                                001060
   4   35   29000.0    1.0      0.0     1.0                                001070
  36   35   29000.0    1.0      0.0     1.0                                001080
  35    7   14500.0    1.1      0.0     1.0                                001090
   7    6    9500.0    0.66     0.0     1.2                                001100
   6   45   23000.0    0.25     0.0     1.2                                001110
  34   35   19000.0    1.5      0.5     1.0                                001120
  34   24    8000.0    1.0      0.5     1.2                                001130
  23   24    8000.0    0.5      1.5     1.2                                001140
  34   25    8000.0    1.5      0.0     1.0                                001150
  24   25    6000.0    1.5      0.0     1.2                                001160
  25   43   12000.0    0.5      0.0     1.2                                001170
  43   16    8000.0    0.5      0.0     1.2                                001180
  16   35    3500.0    0.5      0.5     1.0                                001190
  25   26    2000.0    0.4      0.0     1.2                                001200
  26   40   33000.0    0.5      0.0     1.2                                001210
  15   37   23000.0    1.5      0.0     1.2                                001220
   5   37    5000.0    0.9      0.0     1.0                                001230
  37   44     100.0    0.5      0.0     1.2                                001240
  44   40   11400.0    0.5      0.0     1.2                                001250
  37   38   21000.0    0.5      0.0     1.2                                001260
   1   38   19000.0    1.0      0.0     1.2                                001270
  40   38   16000.0    0.5      0.5     1.2                                001280
  39   40   15000.0    0.5      0.5     1.2                                001290
  38   39   12500.0    0.2      0.2     1.2                                001300
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APPENDIX II

COMPUTER OUTPUT

SOURCES WINDSOR LAKE AND BAY BULLS BIG POND

ROUTE	FLOW	HF	DIA	HP	COST..$MILLION	
	MGD	FT	FT		CAPTL.	ANNUAL
WINDSOR LAKE						
...VENTURI JNCTN	9.36	45.	2.41	*	4.413	.318
			(TREATMENT =		3.690	.311)
VENTURI JNCTN						
...TORBAY RD	3.04	10.	1.75	*	.217	.002
TORBAY RD						
...TORBAY	.43	165.	.65	*	.400	.005
TORBAY RD						
...NORTH EXPN/INDUS PK	2.32	65.	1.31	*	.496	.005
VENTURI JNCTN						
...ST JOHN'S LOW	3.67	175.	1.52	*	.600	.006
VENTURI JNCTN						
...ST JOHN'S HIGH	2.50	60.	1.00	*	.665	.007
KILBRIDE JNCTN						
...ST JOHN'S LOW	3.78	160.	1.63	*	1.078	.011
GOULDS/PETTY HARBOUR						
...KILBRIDE	.45	60.	.74	*	.351	.004
GOULDS JNCTN						
...GOULDS/PETTY HARBOUR	.78	25.	.80	*	.118	.001
BAY BULLS B.P.						
...GOULDS JNCTN	17.37	36.	3.75	1422.	11.780	.770
			(TREATMENT =		8.685	.655)
BROOKFIELD JNCTN						
...KILBRIDE JNCTN	3.78	40.	1.59	*	.311	.003
SOUTH EXPN INTER						
...ST JOHN'S INTER	3.71	15.	2.00	*	.226	.002
BLACKMARSH JNCTN						
...SOUTH EXPN INTER	4.53	15.	2.11	*	.540	.005
BLACKMARSH JNCTN						
...SOUTH EXPN HIGH	.70	31.	.70	109.	.237	.008
MT. PEARL JNCTN						
...BLACKMARSH JNCTN	5.23	5.	2.34	*	.258	.003
MT. PEARL JNCTN						
...BROOKFIELD JNCTN	3.78	50.	1.37	*	.180	.002
PEARL TOWN RD JNCTN						
...MT. PEARL JNCTN	9.01	10.	2.70	*	.414	.004
COMMONWEALTH RD JNCT						
...PEARL TOWN RD JNCTN	9.01	10.	2.70	*	.414	.004
SOUTH BROOK JNCTN						
...COMMONWEALTH RD JNCT	9.01	10.	2.88	*	.507	.005
GOULDS JNCTN						
...SOUTH BROOK JNCTN	16.59	20.	2.99	*	.374	.004
SOUTH BROOK JNCTN						
...NEWTOWN/DONOVANS	7.58	26.	2.53	672.	1.078	.050
NEWTOWN/DONOVANS						
...MOUNT PEARL	.70	100.	.59	*	.052	.001
NEWTOWN/DONOVANS						
...TRANS CAN HWY JNCTN	1.97	15.	1.45	*	.246	.002
CHAMBERLAINS JNCTN						
...MANUELS/LONG POND	1.10	150.	.89	*	.386	.004
MANUELS/LONG POND						
...FOXTRAP/KELLIGREWS	.68	50.	.84	*	.289	.003
FOXTRAP/KELLIGREWS						
...GULLIES/SEAL COVE	.25	247.	.50	40.	.499	.007
TRANS CAN HWY JNCTN						
...CHAMBERLAINS JNCTN	1.57	200.	1.01	*	.569	.006
TRANS CAN HWY JNCTN						
...PARDISE JNCTN	.40	35.	.71	*	.174	.002
PARDISE JNCTN						
...TOPSAIL RD	.27	15.	.68	*	.151	.002
CHAMBERLAINS JNCTN						
...TOPSAIL/CHAMBERLAINS	.47	17.	.73	17.	.113	.002
PARDISE JNCTN						
...PARADISE	.13	22.	.40	7.	.051	.001
ST JOHN'S INTER						
...THORBURN RD	.27	270.	.51	85.	.546	.010
WINDSOR LAKE						
...PORTUGAL COVE RD	.64	151.	.76	113.	.905	.034
			(TREATMENT =		.252	.021)
PORTUGAL COVE RD						
...ST PHILIPS/HOGANS PD	.16	175.	.42	*	.259	.003
PORTUGAL COVE RD						
...PORTUGAL COVE	.12	200.	.35	*	.171	.002

TOTAL DEMAND 27.37 M.G.D.

			$ MILLIONS	
CONVEYANCE COSTS	(P.W.=	20.016)	16.527	.310
PRODUCTION COSTS	(P.W.=	23.743)	12.627	.987
TOTALS	(P.W.=	43.759)	29.154	1.297
AVERAGE COST/M.G.D.	(P.W.=	1.599)	1.065	.047

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