ST. JOHN'S

REGIONAL WATER SYSTEM STUDY

Volume III

CANADA
DEPARTMENT OF REGIONAL
ECONOMIC EXPANSION

GOVERNMENT OF NEWFOUNDLAND
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ST. JOHN'S REGIONAL WATER SYSTEM STUDY

St. John's Special Area Project 3.1

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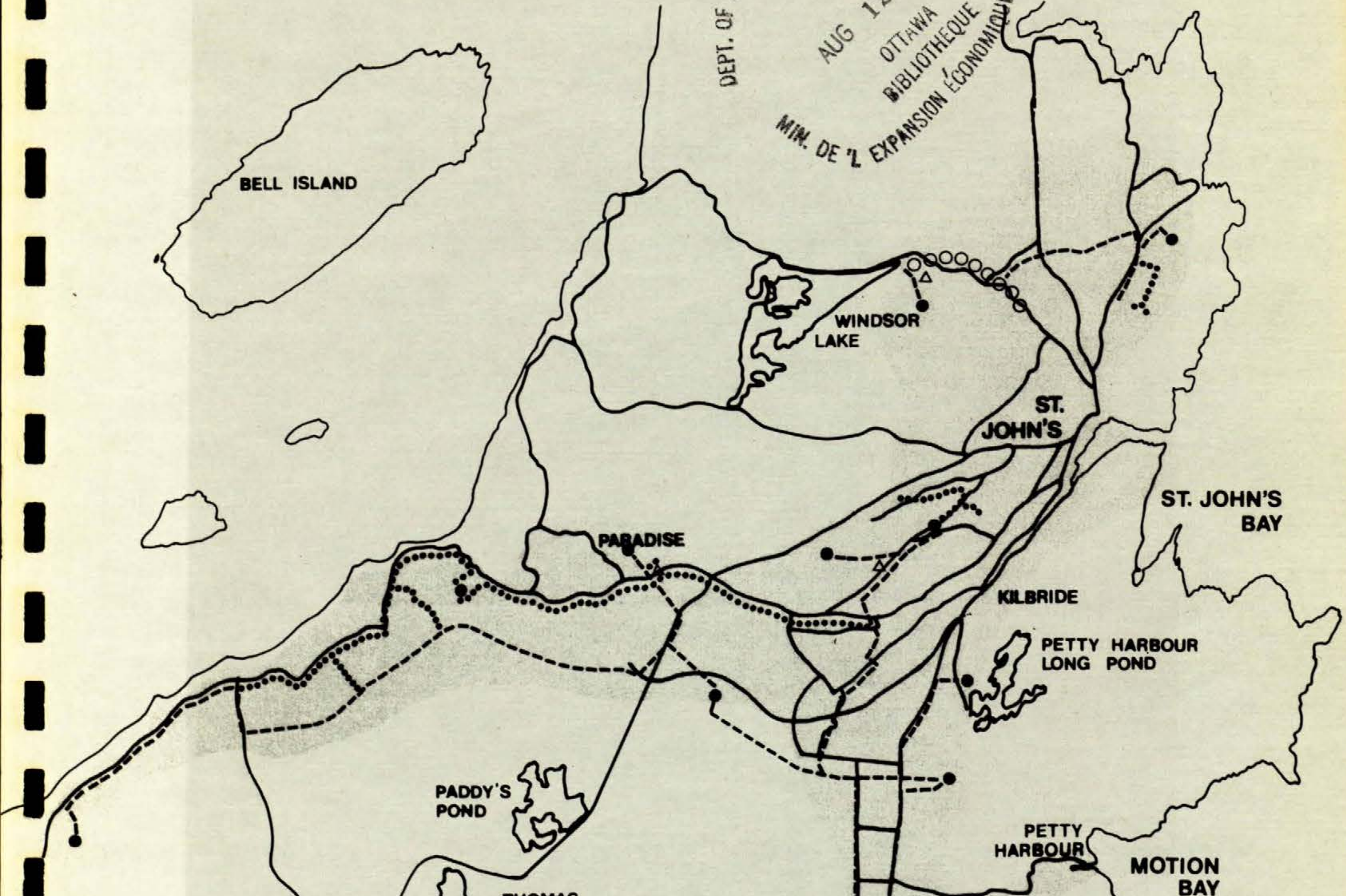

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Abbreviations

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 f
liters l
logarithm (common-base 10) .... log
logarithm (natural-base e) .... ln

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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REGIONAL CONVEYANCE SYSTEM

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CHAPTER 7

REGIONAL CONVEYANCE SYSTEM

SYNOPSIS

The results of the Econometric Model have been used to develop the proposed regional conveyance system. Factors such as reliability of supply, sizing of pipelines and pumps which conform to commercially available components vis-a-vis the life span of these components, and the possible need for increased capacity within this life span which may exceed the design period of 20 years, have been considered and discussed in this Chapter.

A summary of the findings and recommendations is as follows:

- The regional conveyance system includes two divisions, the Bay Bulls Big Pond system and the Windsor Lake system.

- Water from Bay Bulls Big Pond will be supplied to the communities of Conception Bay South area, New Town, Mount Pearl, Paradise, Topsail Road, Seal Cove, Kilbride, Goulds, Petty Harbour, and portions of St. John's lying at the higher elevations.

- Water from Windsor Lake will be supplied to the communities of Torbay, Torbay Road, Penetanguishene, Wedgewood Park, Shea Heights and parts of the City of St. John's that generally lie below elevation 350 feet.

- The Bay Bulls Big Pond conveyance system includes the following major components:

  - A 42-inch diameter main conveyance line from Bay Bulls Big Pond to the junction of Ruby Line and Heavy Tree Road (designated Ruby Line bifurcation).

  - A 30-inch diameter main from Ruby Line bifurcation to New Town service reservoir.

  - An 18-inch diameter main from New Town service reservoir to Trans Canada Highway.

  - A 16-inch diameter main from Trans Canada Highway to Chamberlains Road (in the Conception Bay South area).

  - A 12-inch diameter main from Chamberlains Road, along the proposed Conception Bay Arterial Road and Foxtrap Access Road to Conception Bay Highway.

  - A 10-inch diameter main along Conception Bay Highway, from Foxtrap Access Road to Pellens Road.

  - A 12-inch diameter main from Trans Canada Highway to Paradise.

  - A 30-inch diameter main from Ruby Line bifurcation to Topsail Road.

- A 24-inch diameter main from Topsail Road along Blackmarsh Road and Empire Avenue to Jensen Camp's Lane.

- A 12-inch diameter main from Ruby Line to Goulds Road; south on Goulds Road to supply Goulds (and Petty Harbour), and north on Bay Bulls Road to supply Kilbride.

- The Windsor Lake conveyance system includes the following major components:

  - A 20-inch diameter main from north of the Venturi House to Torbay Road.

  - A 20-18 inch diameter main from Torbay Road to North Expansion Zone.

  - A 12-inch diameter main along Torbay Road to the Town of Torbay.

I. INTRODUCTION

The econometric model, discussed in Chapter 6, provided information regarding the economic boundaries of the regional water system, the economic routes for the regional conveyance mains, and the best choice of a source to supplement Windsor Lake. However, the model was not developed to analyze certain intangible items such as equipment failure (reliability of supply) or storage requirement for fire fighting. In addition, the results of the econometric model necessitate some practical interpretation such as the sizing of pipe lines and pumps which conform to commercially available components vis-a-vis the life span of these components and the possible need for increased capacity within this life span which may exceed the design period of 20 years. These and other similar factors have been considered in developing the proposed regional water system.

The first section of this Chapter presents a general description of the regional conveyance system including considerations for system reliability during emergency conditions. The central section gives a detailed outline of the conveyance main to each community within the regional system. The last section of this Chapter explains some of the basic concepts for local supply systems which were utilized in planning the regional conveyance mains.

II. GENERAL

1. System Description

The regional water conveyance system, as derived from the econometric model, includes two main divisions which are identified by their sources. These divisions are the Bay Bulls Big Pond system and the Windsor Lake system (Drawings 7.1 and 7.2). This section provides a general description of the conveyance components of each of these systems as well as an outline of the manner in which the two systems interact to form a regional supply system.

In principle, the water will be conveyed to service reservoirs which will be connected to the supply mains in a manner which will have the reservoirs "floating on the line". Distribution systems to render the water available to the consumer will extend directly from these reservoirs, and/or from the conveyance mains via primary ring mains. As this project deals only with the conveyance function of the regional system, the construction of conveyance mains will not, therefore, bring water into the homes of the people, except in the case of St. John's, Mount Pearl, and New Town where distribution systems exist and/or are being developed.

The Bay Bulls Big Pond portion of the regional conveyance system will supply water to the communities of Conception Bay South, New Town, Mount Pearl, Paradise, Topsail Road, Seal Cove, Kilbride, Goulds, Petty Harbour, and portions of St. John's lying at the higher elevations.

Generally, the conveyance system from Bay Bulls Big Pond will consist of a main transmission line with a bifurcation at Ruby Line. One branch off this junction will lead to the existing two million gallon service reservoir at New Town. A continuation of this conveyance main from New Town service reservoir will supply the towns of Conception Bay South, Foxtrap, Seal Cove, Paradise and Topsail Road with water. The other main from the bifurcation at Ruby Line will supply water to St. John's. A supply to Kilbride, Goulds and Petty Harbour will be provided via a direct tap into the main transmission line.

Water from Windsor Lake will be supplied to the communities of Torbay, Torbay Road, Penetanguishene, Wedgewood Park, Shea Heights and parts of the City of St. John's that generally lie below elevation 350 feet.

The Windsor Lake portion of the regional conveyance system will utilize the existing 36 inch diameter pipeline as the main transmission line of the system, conveying water to St. John's, Wedgewood Park and Shea Heights. A conveyance main will branch from this existing main transmission to supply water to the North Expansion Zone, Torbay Road and Torbay. A separate pumping station (at Windsor Lake) in conjunction with a forcemain will be used to deliver water to Penetanguishene.

In both of the above sub-systems (which comprise the regional system) metering of water will be provided at each supply station to a local community, to gauge the operation of the conveyance system, to determine the demands of the various communities as well as allowing for the development of an equitable fare structure if and as required.

The conveyance mains will be routed so as to avoid unwarranted traffic disruption and pavement destruction, as well as to facilitate maintenance. Generally, main roads will be avoided in favour of back roads and utility corridors such as the ones used for power lines. Easements secured for the conveyance mains should be permanent and finished in a manner enabling vehicular movement.

2. System Reliability

To provide a water supply which can be depended upon during emergency conditions such as the rupture of a main, the failure of a pump, or a power outage, several measures are recommended.

a. Reserve Storage

Service reservoirs will be installed at strategic locations throughout the region to provide storage to handle peak demands and fire requirements as outlined in Chapter 4. Additionally, a storage capacity equal to one half of a maximum day demand will be provided to maintain a supply of water in the event of an emergency situation as outlined above. Often, the storage provided to handle emergencies of this nature are the prime factor in the sizing of service reservoirs.

b. Stand-by Pumps

To minimize interruptions of service due to the failure of a pump, all pumping stations will be provided with stand-by pumps. Generally, for small pumping stations up to approximately 50 HP, stand-by facilities will be provided to equal the operating capacity of the station. A reduction of the stand-by pump capacity as a ratio of the operating capacity could be made for larger pump stations where stand-by facilities could equal one third of the operating capacity.

c. Stand-by Power

Except for the booster station at Ruby Line, all other stations are of relatively small capacity (see ensuing sections of this Chapter). With the provision of service reservoirs as discussed above, a power failure at one of the small pumping stations would not cause a serious disruptions of service provided that the loss of power did not extend over a long period of time. However, the interruption of electrical service at a large pump station such as the one proposed for Ruby Line, which will handle approximately one half of the flow from Bay Bulls Big Pond, is a more serious matter. Thus, stand-by power is a necessity for this large facility. This reserve power may be provided in the form of a diesel-electric generating unit, or via a power line from a second generating station. The choice between these options will have to be made during the detailed design phase.

d. Twinning of Mains

The installation of twin mains is generally considered for major tranmission lines of large size whose repair, in case of rupture, may take longer than a day. This approach to improving the reliability of the system was considered for the line running from the treatment facilities at Bay Bulls Big Pond to the bifurcation at Ruby Line. An analysis of this possibility was carried out using the following alternatives:

(i) Installation of a single 42 inch diameter main as recommended by the econometric model. This main will convey the design maximum flow of 43 cfs at a velocity of 4.5 fps. At an increased velocity of 7 fps a maximum flow of 67 cfs (equivalent to the normal yield of Bay Bulls Big Pond) could be transported through this main.

(ii) Installation of a single 48 inch diameter main which will transport the maximum flow of 67 cfs at an economical velocity of 5 fps.

(iii) Installation of twin 36 inch diameter mains in stages. This arrangement will provide additional reliability because a single 36 inch diameter main could provide 90 percent of the maximum day demand at the design pressure with all pumps, duty and stand-by, in service.

The results of the economic analysis of the above three schemes are given in Table 7.1. It can be seen that a heavy premium of some 1.5 million dollars would have to be paid for twin transmission main, 36 inch in diameter. Also, there is no distinct advantage in using an oversize main, 48 inch in diameter.

Thus, additional reliability in the form of twin transmission mains is not recommended due to the high cost of such a system. Experience and case histories indicate that such mains when designed and installed properly result in very limited disruptions.

TABLE 7.1

ECONOMIC ANALYSIS

TWINNING OF MAINS

Conveyance System	Pipeline Cost ($M)	Pump Station				Total Cost ($M)	
		Design Capacity		B.B.B.P. Capacity		Design Capacity	B.B.B.P. Capacity
		HP	Cost ($M)	HP	Cost ($M)		
1 x 42" dia. Pipeline	3.2	1500	0.85	3100	1.55	4.05	4.75
1 x 48" dia. Pipeline	3.5	1350	0.80	2450	1.22	4.30	4.72
2 x 36" dia. Pipeline	4.8 (1)	1350	0.80	2550	1.27	5.60	6.07

(1) Present Worth; assuming pipelines will be installed at 10 years interval, and accounting for 8 percent annual interest rate and 4.5 percent annual inflation rate.

3. Hydraulics of the System

In the econometric analysis and the subsequent preliminary design of the regional conveyance system, the Colebrook equation was used to describe flow resistance in the pipes. A roughness dimension of e = 0.002 feet was used in these calculations. The relevant design criteria recommended in Chapter 4 were also incorporated into the selection of component sizes based on the results of the econometric model. Basically, this meant adjusting component sizes to commercially available dimensions and increasing the sizes of some elements to meet minimum practical requirements for fire protection as well as possible greater future water demand beyond the design period of 20 years.

Generally, the system was designed to maintain a pressure between 40 to 130 psi at connection points from which primary distribution rings will extend. The pumping station at Bay Bulls Big Pond treatment plant could provide this pressure to all the areas. except the high level zones for which booster stations will be provided. However, since the pressure at the distribution rings is largely dependent on local topographic elevation, there may be locations where, or times when, the pressure will exceed 130 psi, necessitating the use of pressure regulating valves. Specific cases where this phenomenon may occur will be presented in the ensuing sections of this Chapter.

Generally, control of pumps in pumping (or booster) stations will be from the water level in the corresponding service reservoir by signals transmitted via telephone lines. Such a control system is considered effective, efficient and economic. Also, it facilitates sizing of pumps and the use of constant speed drives on the pumps, which is always a preferred option.

In cases where pumps are discharging into a closed system (primarily due to phasing of construction which delays the installation of service reservoirs), excessive differential pressures may be experienced between periods of low demand and maximum demand. This excessive pressure will not only result in energy wastage, but may well cause damages to pipelines and fixtures. To avoid these detrimental phenomena from happening, controls and pumps must be selected to provide the correct pressure for each range of flows. Two general approaches to solving this problem are:

(a) The use of pumps with variable speed drives controlled as a function of pressure. As the pressure decreases the pump speed will increase developing a higher head and a greater discharge. The opposite will occur when the pressure increases.

(b) The use of pumps with constant speed drives. Two different size pumps can be selected each operating at an optimum head-discharge range. During periods of low demand one small pump will be in service. As demand increases the pressure drops and this will actuate a second small pump to operate in parallel thus widening the head-discharge range. An additional increase in demand and pressure drop may actuate a third small pump, or alternatively, may shut off the small pumps and start the large pump. This kind of pump combination may cover the entire range of the system demand without developing excessive pressure.

As the use of constant speed pumps is usually the most economical solution, this type of arrangement should be preferred. Special attention should be given in the detailed design stage to the selection of pumps with characteristic curves which efficiently fit that of the conveyance system at incremental ranges that will satisfy both demand and pressure. Also, if service reservoirs will be added at any future date, consideration should be given to a control system that could easily be adjusted to the reservoir water level.

A minimum pressure of 20. p.s.i. has been used as the design criterion for fire fighting purposes.

General hydraulic and flow data for the design of the proposed Regional Conveyance System or components thereof can be found on Drawing 7.1.

III. CONVEYANCE MAINS - BAY BULLS BIG POND SYSTEM

The regional conveyance mains from the Bay Bulls Big Pond source will consist of some 36 miles of pipeline ranging in diameter from 10 to 42 inches. These mains as shown on Drawings 7.3, 7.4 and 7.5 have been identified as follows:

1. Main Conveyance Line

This line extends from Bay Bulls Big Pond to Ruby Line bifurcation (Drawing 7.3.) In considering its route (to comply with the basic concepts laid out previously,) two alternatives have been identified - the Backline Goulds Road and an easement to its west. However, since maps indicate the latter route to traverse areas of bog, preference was given, at this stage, to the route along the Backline Road. More in depth investigation is, however, warranted during the detailed design stage before a final selection of this pipeline route is made.

Accordingly, the route of the Main conveyance Line from the treatment plant at Bay Bulls Big Pond has been chosen to parallel the access path from the dam to just north of Raymonds Brook from where it will turn north, crossing Raymonds and Cochrane Pond Brooks, through virgin land to meet the Backline Goulds Road. The pipeline will then be routed in the right-of-way of the Backline Road to a point south of its intersection with Ruby Line. The first section of this right-of-way consists largely of a laneway about twelve feet wide. It is envisioned that the pipeline will be installed along the edge of this laneway.

The municipal road running north from Doyles Road is made up of a short length of pavement but is by and large a gravel road varying in width up to 30 feet. It is proposed to place the pipeline in the travelled portion of the roadway.

It is worth noting here that a recently enacted "Agricultural Land Protection Act" prohibits development along the above right-of-way. The inference is that this particular road will in all probability remain as a municipal roadway under the control of the Town of Goulds.

The last section of the pipeline route from the Backline Road to Ruby Line bifurcation will parallel a Hydro line easement belonging to the Newfoundland Light and Power Company. The concept here is to create a "service corridor," thereby consolidating the grouping of the public services in the area.

This Main Conveyance Line will be required to traverse Raymonds Brook and Cochrane Pond Brook in addition to four smaller creeks. It is intended to install the pipeline under the bed of these watercourses and surround it with concrete. Crossing of larger streams, such as Raymonds Brook, will necessitate the examination of a pipe bridge as an alternative during the detailed design stage.

The Main Conveyance Line has been designed for a maximum day flow of 23.5 MGD. It is recommended that a 42 inch diameter pipe be installed, as selected by the econometric model. As discussed previously in this Chapter, this pipe diameter will comfortably convey a maximum day flow of 36 MGD.

The Main Conveyance Line will be fed by the high lift pumping station located at the treatment plant (see Chapter 8.) From the considerations for phasing of construction, as presented in Chapter 9, this pumping station could be feeding a closed system initially. The basic hydraulic concepts outlined in Section II-3 of this Chapter should be followed in the detailed design stage of this pumping station.

Drawing 7.7 shows the hydraulic gradient of the Main Conveyance Line under maximum day flow regime in 1995, and the design concepts developed herein.

2. New Town and Mount Pearl Conveyance Main

The conveyance main from the Ruby Line bifurcation to the existing 2.0 million gallon service reservoir at New Town - Mount Pearl will be routed parallel to the Newfoundland Light and Power Company's Hydro Line easement. As selected by the econometric model, this main will consist of a 30 inch diameter pipeline.

At the maximum day design flow of 11.4 MGD the velocity will be 4.5 fps. A 50 percent increase in flow could be conveyed in this pipe at a reasonable velocity of 7.5 fps. A local booster pumping station will be required to lift the water to the 715 foot top water level of the service reservoir. This pumping station, designated "Ruby Line," will be located on Ruby Line, south of the Harbour Arterial Road interchange (Drawing 7.3.) The pumps at the booster station will be controlled from the water level in the service reservoir.

Based on our considerations for phasing of construction (as presented in Chapter 9,) and taking into account constraints imposed by the local electric distribution system, it is recommended (at this stage) that pumps equipped with motors of no more than 200 HP be considered. Accordingly, the sizing of the pumps could be a multiple of 3 MGD units.

Floor area and the superstructure of the booster station will be designed to accommodate all of the pumps required for the horizon year. When water demand will increase beyond the design capacity, changes in the presently proposed set-up will have to take into account the then prevailing electric distribution conditions.

Drawing 7.7 shows the hydraulic gradient of the New Town - Mount Pearl conveyance main under maximum day flow regime in 1995.

To meet the criterion set up previously for storage capacity of service reservoirs, an additional 3.0

million gallon reservoir will be required in the future.

3. Goulds and Kilbride Conveyance Main

The conveyance main to Goulds and Kilbride will branch off the Main Conveyance Line at Ruby Line. It will be routed easterly parallel to the Newfoundland Light and Power Hydro line (along an easement which will avoid areas of bog) to the Goulds Road (Drawings 7.3 and 7.4). From this point supply to both Goulds and Kilbride will proceed from the higher lying lands to the lower ones (a preferable situation, see Chapter 4 Section IX of Volume II).

The conveyance main to Kilbride will go in a northern direction along Bay Bulls Road. It will be laid in the shoulder of the road (where possible), and terminate at a service reservoir, 500,000 gallon capacity with a top water level of 515 feet, to be located on the east side of Bay Bulls Road, at the northern boundary of Kilbride.

The conveyance main to Goulds service reservoir will continue easterly from the Goulds Road along an easement parallel to the Hydro line. The service reservoir will be 500,000 gallons in capacity and will have its top water level at elevation 515 feet. A high tract of land on the north side of the town, just inside its boundary, is ideally suited for the location of this reservoir. A separate conveyance/supply main will run from the service reservoir to Goulds. It will be routed along the above easement and then southward along the Goulds Road.

The size of the conveyance main from its connection to the Main Conveyance Line and up to the Goulds Road will be 12 inches in diameter, as selected by the econometric model. Along this section the conveyance main will be transporting the maximum day flow only, and the 12 inch diameter size is considered adequate.

At the Goulds Road the conveyance main will branch north to Kilbride and east to Goulds service reservoir. At this point a pressure regulating valve (to provide a steady lower downstream pressure) is recommended (see Drawing 7.10).

The conveyance/supply main to Kilbride was sized by the econometric model as a 10 inch diameter pipe. However, this main will also fulfill a function of supply. In order for it to provide maximum day plus fire demands, a 12 inch diameter pipe is recommended.

The main feeding Goulds service reservoir will be 10 inch in diameter, as selected by the econometric model. However, the line feeding Goulds from this reservoir will be 12 inch in diameter to accommodate maximum day plus fire demands. This main will have the capacity to extend water supply to Petty Harbour.

4. City of St. John's Conveyance Main

The conveyance main to the City of St. John's will extend from the Ruby Line bifurcation (Drawing 7.4) northward along Heavy Tree Road to Old Placentia Road (Brookfield Road). It will continue in a north-eastern direction along Old Placentia Road to the eastern boundary of the Provincial Forestry Research Station, and then northward in an easement parallel to this boundary. After crossing the CN railroad tracks the main will pass through an easement in private land, cross the Waterford River at Dunnes Road, and continue north on Dunnes Road to Topsail Road. From here the conveyance main will be routed in an easement across private property to Blackmarsh Road; eastward on Blackmarsh Road to its junction with Empire Avenue, along Empire Avenue to its junction with Jensen Camp's Lane, and then northward parallel to this Lane, to the proposed service reservoir for St. John's intermediate pressure zone (Pressure Zones II and III, Drawings 7.1 and 7.12).

In accordance with the econometric model analysis, the size of the St. John's conveyance main will be as follows:

(a) 30 inch diameter pipeline from Ruby Line bifurcation to Topsail Road.

(b) 24 inch diameter pipeline from Topsail Road to the service reservoir.

Drawing 7.7 and 7.8 show the hydraulic gradient of the St. John's conveyance main under maximum day flow regime in 1995.

Tapping into the St. John's conveyance main will be at the following location (see Drawings 7.1 and 7.12):

(i) Topsail Road East to supplement supply to St. John's pressure Zone II.

(ii) Topsail Road west to provide supply to Mount Pearl through the existing system, under emergency conditions only.

(iii) Topsail Road west to provide emergency supply to the community of Topsail Road.

(iv) North of Canada Drive to provide a connection for ring main to South Expansion Zone intermediate pressure area.

(v) Blackmarsh Road south to provide 3 connections for ring mains to South Expansion Zone intermediate pressure area.

(vi) Blackmarsh Road west to provide a connection for a supply main to South Expansion Zone areas lying west of the conveyance main.

(vii) At the junction of Blackmarsh Road and Empire Avenue to provide a connection to the South Expansion Zone high pressure area booster station.

(viii) At the junction of Empire Avenue and Jensen Camp's Lane to supplement supply to St. John's pressure Zone III.

(ix) At the proposed service reservoir (Jensen Camp's east) to provide a connection for a new supply main to Kenmount Road.

A more detailed description of the above connections can be found under Section V - Distribution Systems.

To meet the criterion set up previously for storage capacity of service reservoirs, a 5.0 million gallon reservoir will be required to provide the needs of St. John's intermediate pressure zone (zones I and II.)

It should be noted that detailed design stage analysis may show a necessity to provide a suction well type booster station at Ruby Line bifurcation to feed the St. John's and New Town - Mount Pearl conveyance mains.

5. <u>Conception Bay South Conveyance Main</u>

From the water storage facilities at New Town the conveyance main to Conception Bay South will be routed in a western direction along an easement parallel to the Hydro line, until it reaches the Trans Canada Highway (Drawing 7.4.) In selecting this easement (during the detailed design stage,) precaution should be taken to avoid the crossing of lands lying at elevations above the hydraulic gradient (see Drawing 7.9.)

From the Trans Canada Highway the Conveyance main will be routed southward along an easement, until it meets the proposed Conception Bay Arterial Road. From here the main will run southward parallel to the Arterial Road (and within the latter right-of-way,) up to the Foxtrap Access Road, along this Access Road to Conception Bay Highway, and along the Highway to Pellen's Road (Drawing 7.5.)

Along the above proposed route, several streams will be encountered, such as Manuels River and Conway Brook. If the Arterial Road bridges are constructed prior to the pipeline, these rivers may be traversed by way of the bridges. In other cases the pipeline will be laid in a trench across the river and will be suitably protected. It has been assumed that this section of the Arterial Road will be completed to, at least, the final grading of the earthworks by the time construction of the pipeline is begun. Thus the pipeline could be constructed with the desired degree of permanency and the profile will have suitable hydraulic characteristics.

Utilization of the hydraulic gradient elevation at New Town storage facilities will result in an economized conveyance main system to Conception Bay South. Accordingly, the econometric model sized the conveyance main as follows (Drawings 7.4 and 7.5):

(a) 18 inch diameter pipeline from New Town water storage facilities to Trans Canada Highway.

(b) 14 inch diameter pipeline from Trans Canada Highway to the draw-off junction to Conception Bay South reservoir No. 1.

(c) 12 inch diameter pipeline from the above draw-off junction to Dawes Road.

(d) 10 inch diameter pipeline from Dawes Road to the junction of Foxtrap Access Road with Conception Bay Highway.

(e) 8 inch diameter pipeline along Conception Bay Highway to Pellen's Road.

(f) 10 inch diameter pipeline to Conception Bay South reservoir No. 1.

In planning the conveyance main system to Conception Bay South area, we have taken into consideration the following important factor:

-Water will be conveyed to this area via (relatively) long transmission mains. The life span of these mains exceeds the design period of 20 years adopted for this project. It is, therefore, warranted to plan the system for a capacity in excess of the 1995 maximum day design flow of 2.37 MGD. It would seem that a system capable of providing 150 percent the 1995 design capacity at adequate pressure, by utilizing the available hydraulic gradient pressure at New Town water storage facilities, will be more appropriate. Accordingly, the conveyance main system proposed for Conception Bay South area is as follows:

(i) 18 inch diameter pipeline from New Town water storage facilities to Trans Canada Highway.

(ii) 16 inch diameter pipeline from Trans Canada Highway to the draw-off junction to Conception Bay South reservoir No. 1.

(iii) 12 inch diameter pipeline from the above draw-off junction to the junction of Foxtrap Access Road with Conception Bay Highway.

(iv) 10 inch diameter pipeline along Conception Bay Highway to Pellen's Road.

(v) 12 inch diameter pipeline to Conception Bay South reservoir No. 1.

The additional capacity of this system could be used to supply larger quantities of water to the Town of Conception Bay South, or to developments in adjacent lands, should this become a necessity at any future time.

Drawing 7.9 and 7.10 show the hydraulic gradient of the Conception Bay South conveyance main under maximum day flow regime in 1995.

Since the Conception Bay South supply area extends over a distance of some 10 miles, it was felt that a number of service reservoirs at different locations will provide a more reliable system than a single central reservoir. Accordingly, it is recommended that a 1.0 MG reservoir, designated Conception Bay South reservoir No. 1, be located off Allen's Road at the Metropolitan boundary; a second 1.0 MG reservoir, designated Conception Bay South reservoir NO. 2, will be located off New Talc Road at the power transmission line easement; and a third reservoir, designated Conception Bay South reservoir No. 3 and having a storage capacity of 0.5 MG, will be located off the proposed Conception Bay Arterial Road at Pellen's Road.

Service reservoirs No. 1 and 2 will have a top water level at elevation 350 feet, whereas reservoir No. 3 will have its top water level at elevation 250 feet.

The(relatively) substantial length of the Conception Bay South conveyance main system, and its over-sizing for possible water demands beyonds the design period, result in a significant differential pressure between periods of low demand and maximum demand. This phenomenon necessitates the use of a pressure regulating valve at the terminus of the 16 inch diameter main.

It will be worthwhile emphasizing here that, water to the Conception Bay South area will be supplied via the service reservoir(s) at New Town. An alternative to this arrangement will be by passing of the service reservoir(s). However, such an alternative will make the sizing and selection of the pumps and their controls at Ruby Line booster station more complicated and costly.

6. Paradise and Topsail Road Conveyance Main

The conveyance main to the communities of Paradise and Topsail Road will branch off the 18 inch diameter main to Conception Bay South, at Trans Canada Highway.

The Paradise and Topsail Road conveyance main will be routed north along Trans Canada Highway to a power transmission line right of way. The main will then follow this right of way to Topsail Road as shown on Drawing 7.4. A small booster pumping station will be provided off Topsail Road to lift water to the hill north of Topsail Road where the Paradise - Topsail Road service reservoir will be located. This reservoir will have a capacity of 0.5 MG and its top water level will be at elevation 715 feet.

As analysed by the econometric model, a 10 inch diameter pipe for the conveyance main to the booster station, and a 6 inch diameter pipe for the forcemain to the service reservoir would be adequate. However, in planning the system, we have selected the above respective pipe sizes to be 12 inch and 10 inch in diameter, for the following reasons:

(a) To provide reserve capacity in the transmission capability of the conveyance main. This additional capacity could be used by the communities of Paradise and Topsail or by new consumers via a connection extending west on Topsail Road.

(b) To utilize the main feeding service reservoir as a supply main to Topsail Road (primarily for fire protection.)

Hydraulic data such as pumping heads and horsepowers shown on Drawings 7.2, 7.3 and 7.4, indicate order of magnitude only (based on the econometric model analysis) and should therefore not be construed as being final design figures.

IV. CONVEYANCE MAINS - WINDSOR LAKE SYSTEM

Emanating from Windsor Lake will be approximately 14 miles of regional conveyance mains. Unlike the system which conveys the water from Bay Bulls Big Pond, a substantial portion of the Windsor Lake conveyance system utilizes existing transmission lines; approximately 7.5 miles of proposed conveyance system will be existing mains.

1. City of St. John's Conveyance Main

The conveyance main to the City of St. John's has been described in Chapter 3 of Volume I. Essentially, it consists of a series of 36 inch and 32 inch diameter steel mains that extend to a Venturi and new screening house at Ricketts Bridge. The conveyance system then changes to two parallel 24 inch diameter mains. One of these mains feeds a set of 16 inch and 20 inch transmission mains located along Portugal Cove Road, Rennie's Mill Road to Rawlins Cross. The second 24 inch diameter main feeds two 16 inch diameter transmission mains located along Higgins Line, Bonaventure Avenue, and Mayor Avenue to Merrymeeting Road.

As can be seen from Drawings 7.1 and 7.12, and from the presentation in Section V.1, this conveyance main will supply water primarily to the low pressure zone (pressure zone I.) To meet the criterion set up previously for storage capacity of service reservoirs, a 5.0 million gallon reservoir will be required. A possible location for this reservoir could be the site of George's Pond.

2. <u>North Expansion Zone Conveyance Main</u>

The conveyance main to the North Expansion Zone will branch off the existing conveyance main to the city, in the vicinity of the Venturi house. The new main will be routed along Majors Path to Torbay Road. It will then follow a gravel road to Middle Cove Road, and continue south to and along Logy Bay Road (Drawing 7.6).

The econometric model selected for this conveyance main a 20 inch diameter pipe up to Torbay Road and a 16 inch diameter pipe up from Torbay Road through the South Expansion Zone area.

In planning the system, we have continued the 20 inch diameter pipeline for about 3,500 feet past Torbay Road, along the gravel road, and increased the 16 inch diameter pipe size to 18 inch. At the junction of Logy Bay Road and the northern access road to the White Hills Industrial Park, the pipe diameter will change to 12 inch. The rationale behind this planning is as follows:

(a) The area immediately to the north of the gravel road (connecting Torbay Road and Middle Cove Road), which belongs to the community of Torbay Road, is designated for subdivision development. Provisions should be made to supply water to this area from the conveyance main routed along the gravel road.

(b) To satisfy maximum day and fire demands at adequate pressure, a larger pipe size is warranted.

The service reservoir for the above areas will be located on the slope of a hill off Logy Bay Road. This reservoir will have a capacity of 3.5 MG and its top water level will be at elevation 475 feet.

Drawing 7.8 shows the hydraulic gradient of the North Expansion Zone conveyance main under maximum day flow regime in 1995.

3. Torbay - Torbay Road Conveyance Main

The conveyance main to the communities of Torbay and Torbay Road will branch off the 20 inch North Expansion Zone conveyance main at Torbay Road. It will be routed north along Torbay Road, and across the town of Torbay to north of Watts Pond where a service reservoir will be located.

The econometric model selected an 8 inch diameter pipe for this conveyance main. However, since the main will be fulfilling a function of both conveyance and supply, a pipe size of 12 inch diameter will be required to accommodate maximum day and fire demands at adequate pressure.

The service reservoir will have a storage capacity of 0.5 MG and its top water level will be at elevation 465 feet.

In order to utilize the pressure available in this conveyance main effectively, its routing through the town of Torbay should be on high ground. The lower lying areas around Torbay Bight will be served by a ring main branching off the conveyance main and equipped with a pressure regulating valve.

Drawing 7.11 shows the hydraulic gradient of the Torbay - Torbay Road conveyance main under maximum day flow regime in 1995.

It should be noted here that the pressure prevailing in the Windsor Lake supply system is not adequate for filling of the service reservoirs proposed at the North Expansion Zone and Torbay. However, if improvements to the intake and treatment works are implemented (as discussed and recommended in Chapters 8 and 9 of this Volume,) sufficient pressure will be maintained in the system to fill these reservoirs.

V. DISTRIBUTION SYSTEM

It was mentioned earlier in this Chapter that, while design of distribution systems was not included in the scope of this study, none the less consideration was given to the layout of local supply lines when the main conveyance lines were studied. The presentation in the sections explains some of the concepts which were utilized in developing preliminary schemes for local supply systems.

1. City of St. John's Supply System.

As the central city with an existing water supply network, St. John's forms the core of the Regional Water System. A detailed hydraulic analysis of this network is, therefore, warranted.

a. The Approach

St. John's possesses a diversified water supply network with mains of widely varying ages, many abrupt changes in ground elevation and a variety of pressure reducing valves (and one booster pump) to suit.

The pipes themselves are structurally in excellent condition. During the study, sections of cast-iron main as old as 83 years (pipes laid in 1890) were excavated, and the exterior proved in all cases examined to be corrosion-free with the pipes retaining their full original wall thickness. However, iron bacteria have built up very heavy incrustations in certain pipes. A loss of diameter estimated at as much as 2 inches or more for pipes as old as those mentioned, in the vicinity of Water Street, was allowed for in the analysis to be described.

The total lengths of pipes in different diameter categories are approximately as follows:

6 inches and 4 inches - 1,200,000 feet
8 inches and 12 inches - each 170,000 feet

with upwards of 120,000 feet of pipe in other diameter categories.

The only practical method to establish the hydraulic regime of the water supply network, as described above, is a pressure and flow analysis through a computer for both present conditions and future design conditions.

The number of separate mains as well as of pipe junctions is of course a vital factor in determining how well a distribution network may be fitted on to a computer. This factor necessitated the expansion of the standard HYNAL computer programme.

The computer programme used is an enlarged version of a HYNAL pipe network analysis developed by IBM. It is used to simulate flow rates, flow directions and pressures at every point in a pipe network.

The expanded version of HYNAL used allowed up to:

- 1000 pipe junctions (some 700 were simulated in the case of St. John's)

- 1750 pipes connecting the pipe junctions. In practice it is assumed that most pipes will run between neighbouring junctions; this assumption enables the computer storage requirements and consequent cost per run to be held within economical limits (there were nearly 1100 pipes in the case of St. John's)

- the use of reservoirs, pressure reducing valves, booster pumps, and the like.

The programme gives flows, pressures and hydraulic gradients at all desired points in the network. Accuracy of convergence of the final answer can be adjusted as desired. Pipe characteristics or specified flows can be changed in most cases without the necessity of re-reading the punched cards specifying the entire network; in other words, after the entire set of network cards (nearly 2,000 in all) is read in once, subsequent use of the programme requires only a handful of cards, those specifying changes in network flows or layout and of course a specification of just what printout is needed.

A more elaborate discussion of the computer programme is contained in Appendix I.

Three different situations have been studied to establish the adequacy of the network in terms of quantities of water supplied and pressure, and to determine what type of improvements would be required as the water usage increases.

b. Present Conditions

The water supply network is fed by the two sources of Windsor Lake and Petty Harbour Long Pond. The existing isolating valves and pressure reducing valves divide the network into two pressure zones, as follows:

(i) Low Pressure Zone which basically includes the areas east of Higgins Line, Allandale Road, Bonaventure Avenue, and south of Lemarchant Road up to Shaw Street.

(ii) Intermediate Pressure Zone which includes the areas west and north of the zone described under (i) above.

The major direct connection between the above pressure zones is through the floating pressure reducing valve on the 12 inch diameter main along Saunders Place, at the intersection of Cabot Street and Young Street.

The system was analysed for a peak daily flow of 19.5 MGD. The results show that generally the pressure is adequate, except for the area around Mundy Pond where the pressure is relatively low (25 to 35 psi), and the Kenmount Road area which is pressure-deficient (10-25 psi). It would therefore, be advisable to include in the early construction phases of the Regional Water System works that will improve pressure (and supply) in the above two areas. Recommendations for such works are defined in the ensuing two sections.

Drawing 7.13, 7.14 and 7.15 show the hydraulic gradient of the supply system along 9 typical sections of the City.

c. <u>1982-83 Conditions</u>

The remedial works to improve pressure (and supply) in the Mundy Pond - Kenmount Road areas, and to satisfy future increases in water demand, are recommended to be carried out in two phases, as follows:

- An early construction phase that will coincide with the tapping of Bay Bulls Big Pond and will supply water from this new source to the above areas. Based on the projection of water demand (see Chapter 4, Volume II), this first phase remedial work will be satisfactory to the year 1982-83.

- A second phase construction works that will satisfy the projected demand conditions of 1995.

Following is a description of the proposed first phase construction works and modifications recommended to the existing system.

(i) From the 30 inch diameter conveyance main at Topsail Road an easterly connection will be made, via a combination pressure reducing valve-check valve unit and a water meter, to the existing 12 inch diameter main on Topsail Road.

(ii) From the 24 inch diameter conveyance main at the junction of Empire Avenue and Jensen Camp's Lane, a 12 inch diameter connection will be made, via a shut-off valve, to the existing 8 inch diameter ring main.

(iii) From the 24 inch diameter conveyance main on Jensen Camp's Lane, north of Empire Avenue and south of Pennywell Road, an easterly connection will be made, via a shut-off valve, to a proposed 20 inch diameter main that will extend (at this stage) to the Cross Town Arterial just north of Empire Avenue.

(iv) From the above proposed 20 inch diameter main a connection will be made, via a shut-off valve, to a proposed 12 inch diameter main that will extend southward on the Cross Town Arterial to Empire Avenue and then east on Empire Avenue to join the existing 12 inch diameter main on Murphy's Lane.

(v) A pressure reducing valve will be installed on the existing 12 inch diameter main running down Blacker Avenue, just north of its junction with Blackmarsh Road.

(vi) A closed valve will replace the existing booster station on Blackmarsh Road near Murphy's Avenue.

(vii) The existing valve on the 8 inch diameter main on Mundy Pond Road, east of Murphy's Avenue, will be removed.

(viii) A shut-off valve on the existing 6 inch diameter main on Mundy Pond Road, at its junction with Pierce Avenue, will be kept closed.

(ix) A shut-off valve on the existing 12 inch diameter main on Empire Avenue, just west of its junction with Ropewalk Lane, will be kept closed.

(x) A shut-off valve on the existing 8 inch diameter main on Anderson Avenue, just north of its junction with Empire Avenue, will be kept closed.

(xi) A shut-off valve on the existing 6 inch diameter main on Guy Street, just east of its junction with Anderson Avenue, will be kept closed.

(xii) A shut-off valve on the existing 8 inch diameter main on Elizabeth Avenue, just east of its junction with Anderson Avenue, will be kept closed.

(xiii) A shut-off valve on the existing 12 inch diameter main on Larkhall Street, around Leary's Brook Diversion, will be kept closed.

(xiv) The booster pump on O'Leary Avenue could continue to function until the second phase of the proposed construction works (see next section.)

The above proposed works and modifications all but create three pressure zones, as follows:

(a) Zone 1 includes low pressure areas east of Higgins Line, Allandale Road, Bonaventure Avenue, and south of Lemarchant Road up to Shaw Street.

(b) Zone II - includes intermediate pressure zone west and north of Zone I up to Anderson Avenue, Empire Avenue, and south of Mundy Pond and Canada Drive, plus low pressure areas west of Shaw Street.

(c) Zone III - includes intermediate pressure zone in the area of Mundy Pond and Kenmount Road plus South Expansion Zone north of Canada Drive.

The major direct connections between the above pressure zones are as follows:

- Zone II to Zone I through the existing floating pressure reducing valve on the 12 inch diameter main along Saunders Place, at the intersection of Cabot Street and Young Street.

- Zone III to Zone II through the proposed pressure reducing valve on the existing 12 inch diameter main running down Blacker Avenue, just north of its junction with Blackmarsh Road.

It should be noted here that, the pressure zones as proposed and described above, and shown on Drawings 7.1 and 7.12, do not coincide with those defined in Chapter 4 (Volume II.) In order to adhere to the defined pressure zones, major changes would have been required in the existing distribution system. Our objective has been to bring the new supply in from Bay Bulls Big Pond with the least possible changes and interruptions in the existing system. At the same time consideration was duly given to the developing of a system that will satisfy future water need and pressure.

The water supply network (incorporating the additions and modifications described above) was analysed for a maximum daily flow of 20.5 MGD. The results show satisfactory flow and pressure conditions throughout the system. The pressure at the Mundy Pond area will be between 50 and 80 p.s.i, and in the Kenmount Road area it will be between 35 and 50 p.s.i.

d. 1995 Conditions

From the above discussion, it would appear that around the year 1982-1983 the second construction phase should be implemented to satisfy increases in water need and to further improve supply and pressure conditions, primarily in the Kenmount Road area.

It is recommended that this construction phase include the following works:

(i) Continuation of the 20 inch diameter main northward along the proposed Cross Town Arterial, then taking a circuitous route around high ground in the Pennywell Road - Wishingwell Road loop, it will be continued to the corner of Kenmount Road and Confederation Parkway.

(ii) Construction of the 5.0 MG service reservoir at Jensen Camp's Lane.

The water supply network (as proposed above) was analysed for a maximum daily flow of 24.5 MGD. The results show satisfactory flow and pressure conditions throughout the system. The pressure that would be maintained at the critical area of Kenmount Road will be in the order of magnitude of 50 p.s.i.

This system as proposed was analysed for maximum daily flow plus fire demand, at a fire flow of 1,000 gpm (recommended for sub-urban areas.) The results showed the critical pressure at Kenmount Road to be approximately 25 p.s.i., which is considered adequate.

However, with the above arrangement of maximum daily flow plus a fire flow of 2,500 gpm, distribution system pressures in this critical area were inadequate.

With a fire flow of 2,500 gpm in addition to the maximum daily flow and substitution of a supply main of 24 inch diameter (in lieu of the proposed minimum size of 20 inch) the pressures at the out of town end of the Kenmount Road area would be adequate if minor reinforcements were provided to the local distribution system. Since present requirements for fire flows are in excess of the ones used at the time the distribution system was designed, it is recommended that the entire network be thoroughly analysed to determine the economics of local reinforcements versus larger supply mains. This same concept applies also to the proposed connection at Topsail Road and the water supply from Bay Bulls Big Pond to pressure zone II.

In each of these analysis the gap in the existing system on Topsail Road, between Hamlyn Road and Forbes Street was taken to be closed with a 12 inch diameter main.

Drawing 7.13, 7.14 and 7.15 show the hydraulic gradient of the supply system along 9 typical sections of the City, under 1995 maximum daily flow regime.

The above concepts, utilized in developing a preliminary scheme for St. John's supply system, have not touched on local schemes for South and North Expansion Zones. This will be discussed in the following sections.

e. South Expansion Zone

The South Expansion Zone encompasses two pressure areas - intermediate (south of Blackmarsh Road) and high (north of Blackmarsh Road.) The 24 inch diameter conveyance main (routed along Blackmarsh Road) will feed the intermediate pressure zone, whereas a booster pumping station, at the junction of Blackmarsh Road and Empire Avenue, will supply water to the high pressure zone.

(i) Intermediate Pressure Zone - It is proposed that this zone be serviced by main ring loops as follows:

(a) 12 inch diameter pipeline running in an easterly direction along a road right of way north of Canada Drive. This proposed main will be connected to the 24 inch diameter conveyance main, at its route north of Topsail Road, via a shut-off valve.

(b) Three 12 inch diameter valved connections extending off the 24 inch diameter conveyance main from its route along Blackmarsh Road and Empire Avenue. The location of these connections will be approximately as follows:

- The continuation of Burgeo Street
- The continuation of Cowan Avenue
- The continuation of Hamlyn Road

  Each of the 12 inch diameter mains extending along the above three routes will join the 12 inch diameter main proposed under (a) above to form 3 major ring loops. Secondary distribution mains will extend from these rings.

(c) A 12 inch diameter valved connection at Blackmarsh Road will be provided for future servicing of the area west of the 24 inch diameter conveyance main.

(ii) High Pressure Zone - From the booster pumping station, proposed at the intersection of Blackmarsh Road and Empire Avenue, a 10 inch diameter main ring loop will extend in an easterly and westerly direction to encircle this pressure zone. This ring loop will end at the proposed service reservoir to be located west of the junction of Empire Avenue and Wishingwell Pond.

The westerly leg of the main ring will be routed along Empite Avenue to the service reservoir. The eastern leg of this ring will be routed along Empire Avenue (parallel to the 24 inch conveyance main) to join the existing 8 inch diameter main on Jensen Camp's Lane. This line will be continued with a 10 inch diameter main to join the existing 8 inch diameter main on Pennywell Road, just north of the proposed Cross Town Arterial, a valve will be kept closed to isolate this high pressure zone from the intermediate pressure zone. From here the proposed 10 inch diameter main ring will continue in a northerly direction to Wishingwell Road, and then eastward on Wishingwell Road to the service reservoir.

It is recommended that the area within the above outer main ring be traversed with several 10 inch diameter mains to create two or three ring loops. It is difficult to elaborate at this time on these ring loops since the layout of roads and development in the area has not been finalized.

It should be noted that, the area between Empire Avenue and Blackmarsh Road, east of the intersection of these two streets, forms part of the high pressure zone. To supply this area with water, it is recommended that a 10 inch diameter main branch off the above outer main ring, just east of the proposed booster pumping station. This 10 inch diameter main will be routed southward to Blackmarsh Road and then eastward along Blackmarsh Road to join the existing 8 inch diameter main at the junction of Jensen Camp's Lane and Blackmarsh Road. To isolate this high pressure zone from the adjacent intermediate pressure zone, shut-off valves on Blackmarsh Road and Empire Avenue, just east of Jensen Camp's Lane, will be kept closed. At this stage of development (comtemplated to occur after the year 1982-83), the presently proposed 12 inch connection between the 24 inch diameter conveyance main and the existing 8 inch diameter main at the junction of Jensen Camp's Lane and Empire Avenue will also be kept closed.

The high pressure zone booster pumps will be controlled by the water level in the service reservoir proposed for this zone.

Whereas the above preliminary schemes will satisfy the ultimate supply and pressure needs of the intermediate and high pressure zones, there will be an interim period of time whereby the pressure at the existing 8 inch diameter main along Pennywell Road would be inadequate. This unsatisfactory situation will occur with the connection of the 24 inch diameter conveyance main to the water supply network and the creation of a third pressure zone as described above, thereby making the booster pumping station at Blackmarsh Road and Mercer's Lane obsolete. Once main rings will be developed in the high pressure zone, as proposed, satisfactory supply and pressure conditions will be maintained along Pennywell Road. It is, therefore, recommended that a temporary booster pumping station service Pennywell Road during the above interim period of time. This booster station could be located at a convenient site along the proposed 20 inch diameter main.

f. North Expansion Zone

The proposed conveyance/supply main of 20 - 18 - 12 inch diameter main on Logy Bay Road forms an outer main ring loop for this zone. It is envisaged that the area will be traversed with several mains 10 or 12 inch in diameter resulting in two or three main rings from which secondary distribution mains will extend. The location and pattern of the main rings would be established at the planning stage of the area when road layouts have been finalized.

The proposed location of the service reservoirs, which is at the opposite end from the supply connection, will provide the area with a greater security of water supply.

To facilitate the control of water quantities used in the system, a water meter could be installed on the proposed 20 inch diameter main at its connection to Windsor Lake conveyance main.

2. New Town Supply System

The New Town supply system extends from the existing 2.0 MG service reservoir and is being designed by others.

In accordance with our proposed conveyance system, future supply to New Town will continue to be from this service reservoir (plus an additional 3.0 MG reservoir recommended at the same site). Since the New Town conveyance main and the service reservoir(s) are at the same end of the town, any failure in the main supply line from the reservoir will result in water supply disruption. It is, therefore, recommended that the concept of an outer main ring loop be used to provide maximum security of supply to the town and the adjacent industrial park. The implementation of this concept calls for a main of adequate capacity to encircle the area in one or more ring loops so as to supply the water needs of the area through only one leg of the main ring loop.

At the bifurcation of the supply main from the service reservoir, a water meter will be installed on the branch line feeding New Town.

It is recommended that the additional 3.0 MG service reservoir be installed adjacent to the existing one. The two reservoirs will have the same top water level and they will be interconnected. For all practical purposes they will operate as a two-cell reservoir.

3. Mount Pearl Supply System

The existing water supply network of Mount Pearl is presently being fed from the east via a series of booster pumping stations. With the implementation of the Regional Water System, Mount Pearl will receive water from the west via New Town service reservoir. It is proposed, however, to provide Mount Pearl with a connection from the east for emergency conditions. This connection will be via a normally closed valve to the proposed 30 inch diameter St. John's conveyance main at Topsail Road. Under this arrangement Mount Pearl will be provided with connections at two opposite ends of the town, a much desired situation for security of supply. The existing booster pumping stations will be used as required during emergency situations when water supply will be from the east.

It should be noted here that, under certain emergency situations the proposed eastern connection to Mount Pearl could be used with the existing booster pumping stations to supply water to New Town.

A water meter to control the quantity of water used by Mount Pearl will be installed at the 0.5 MG service reservoir.

4. Conception Bay South Supply System

While planning the conveyance main to Conception Bay South area, we have given consideration to the layout of local supply lines. In essence, we propose to utilize the concept of outer main ring loops in the following manner:

(a) From Conception Bay South reservoir No. 1 a 10 inch diameter supply main will extend westward along Topsail Road and then southward along Conception Bay Highway to join the proposed 12 inch diameter conveyance main at the junction of Conception Bay Highway and Foxtrap Access Road.

(b) From a connection to the proposed 12 inch diameter conveyance main along Conception Bay Arterial Road, a 10 inch diameter supply main will extend, via a pressure reducing valve, down Fowlers Road to join the proposed 10 inch diameter supply main under (a) above.

(c) From a connection to the proposed 12 inch diameter conveyance main along Conception Bay Arterial Road, a 10 inch diameter supply main will extend, via a pressure reducing valve, down Dawes Road to join the proposed 10 inch diameter supply main under (a) above.

The above arrangement will cover the area from Topsail to Kelligrews with three main ring loops from which secondary main rings will extend. The area between Kelligrews and the Gullies will be served by a 10 inch diameter conveyance/supply main. However, at each end of this main will be a service reservoir (Conception Bay South reservoirs No. 2 and 3) which will provide the desired security of supply.

Conception Bay South reservoirs No. 1 and 2 and the two connections via pressure reducing valves on Fowlers Road and Dawes Road will be set to provide a uniform pressure elevation of 350 feet. At these four points water meters will also be installed to facilitate control of water quantities used by the Conception Bay South area.

5. Kilbride Supply System

The proposed 12 inch diameter conveyance main to Kilbride will also serve as a supply main, and secondary main rings will extend from it. Since the supply connection and the proposed service reservoir are on two opposite ends of the town, the desired requirement for security of supply will be satisfied.

At the supply connection a water meter and a check valve will be installed.

6. Goulds Supply System

The proposed 12 inch diameter conveyance main to Goulds will also serve as a supply main, and secondary main rings will extend from it.

Due to hydraulic considerations, both the supply connection and the proposed service reservoir are at the same end (north) of the town. In order to provide security in supply, it is recommended that a connection for emergency situations be made to the 42 inch diameter regional conveyance main at Williams Line. This connection will be in the form of a 12 inch diameter main equipped with a shut-off valve that will be kept closed under normal conditions. During emergency situations, when water would not be supplied from the north, this valve will be throttled to provide water from the south at adequate pressure.

With proper valving arrangement, the above 12 inch diameter emergency supply main could be used in the future to convey water to areas south of Goulds.

Hydraulic conditions of the system and the requirement to have maximum recirculation of water in service reservoirs, lead us to recommend that a 10 inch diameter main be used to feed the reservoir and a separate 12 inch diameter main be used to supply water from the reservoir. A water meter will be installed on this latter main.

7. Paradise Supply System

Water will be supplied to Paradise from a proposed service reservoir which, in turn will be fed by a booster pumping station. The booster pumps will be controlled by the water level in the reservoir.

Ideally, when the supply connection and service reservoir are on the same side of the town, as in this case, the outer main ring loop concept (as previously described) should be used to provide security of supply. However, since Paradise is basically a ribbon type development, this preferred concept may not be attainable and some sacrifices in this respect may be required during the detailed design stage.

8. Topsail Road Supply System

The conveyance/supply main to Topsail Road will extend from the high lying ground at Paradise to the lower lying areas at the boundary of St. John's. Seconday main rings will extend from this conveyance/supply main.

Since Topsail Road along the above route extends over two pressure zones, high from Paradise to Glendale, and intermediate from Glendale to the City of St. John's, a pressure reducing valve will be required on the proposed 12 inch diameter main at the vicinity of Glendale. From here to St. John's the main will continue as a 10 inch pipeline.

As noted, the conveyance/supply main to Topsail Road will be fed from the Regional conveyance main at Paradise junction with peak and fire demands supplied from Paradise service and system hydraulic conditions, the feed line from the reservoir will have to be equipped with a pressure reducing valve.

To facilitate the control of water quantities used by Topsail Road, a water meter will be installed on the proposed conveyance/supply main, just downstream of Paradise junction.

Since the supply connection and service reservoir are on the same end (west) of the community, an eastern connection is proposed to the 30 inch diameter St. John's conveyance main at Topsail Road.

This connection, to be used under emergency conditions when there is a failure in supplying water from the western connection (at Paradise), will be equipped with a shut-off valve that will be kept closed.

9. Torbay Road Supply System

We envisage two supply systems to Torbay Road; one to the subdivision development north of North Expansion Zone; the second to the ribbon-like development along Torbay Road. The former will be supplied via a main ring extending from the 20 inch diameter North Expansion Zone conveyance main and will be serviced by the North Expansion Zone reservoir. A water meter will be installed on the main ring. Conceptual design of this main ring and of secondary distribution mains could be developed only after town planning and road layouts have been finalized.

The ribbon-like development along Torbay Road will be served by the proposed 12 inch diameter Torbay-Torbay Road conveyance/supply main from which secondary distribution mains will extend. A water meter will be installed on this line at its connection to the 20 inch diameter Regional conveyance main. Water supply under emergency conditions, such as a failure at the supply connection, will be from the Torbay service reservoir which is at the opposite end of the community.

10. Torbay Supply System

The 12 inch diameter conveyance/supply main along

Torbay Road will extend through the town of Torbay to the proposed service reservoir at the north end of the town. From this conveyance/supply main will extend secondary ring mains.

Since Torbay extend over two pressure zones, low around Torbay Bight, and intermediate inland of the Bight, pressure reducing valve(s) will be required on the secondary ring main(s) servicing the low pressure zone.

A water meter will be installed on the 12 inch diameter conveyance/supply line at the southern boundary of the Town. Since the supply connection (from the south) and the service reservoir(at the north) are on the two opposite ends of the town, security of supply can be considered safisfactory.

It should be noted here that, Torbay reservoir will also be servicing Torbay Road. Should it be found advantageous to measure the flows in the direction from Torbay to Torbay Road, the following piping arrangement could be considered:

- The conveyance/supply main feeding Torbay will be equipped with a water meter and a check valve allowing water to flow in Torbay's direction only.

- A by-pass to the above installation will be provided, equipped with a water meter and check valve allowing water to flow in Torbay Road's direction only.

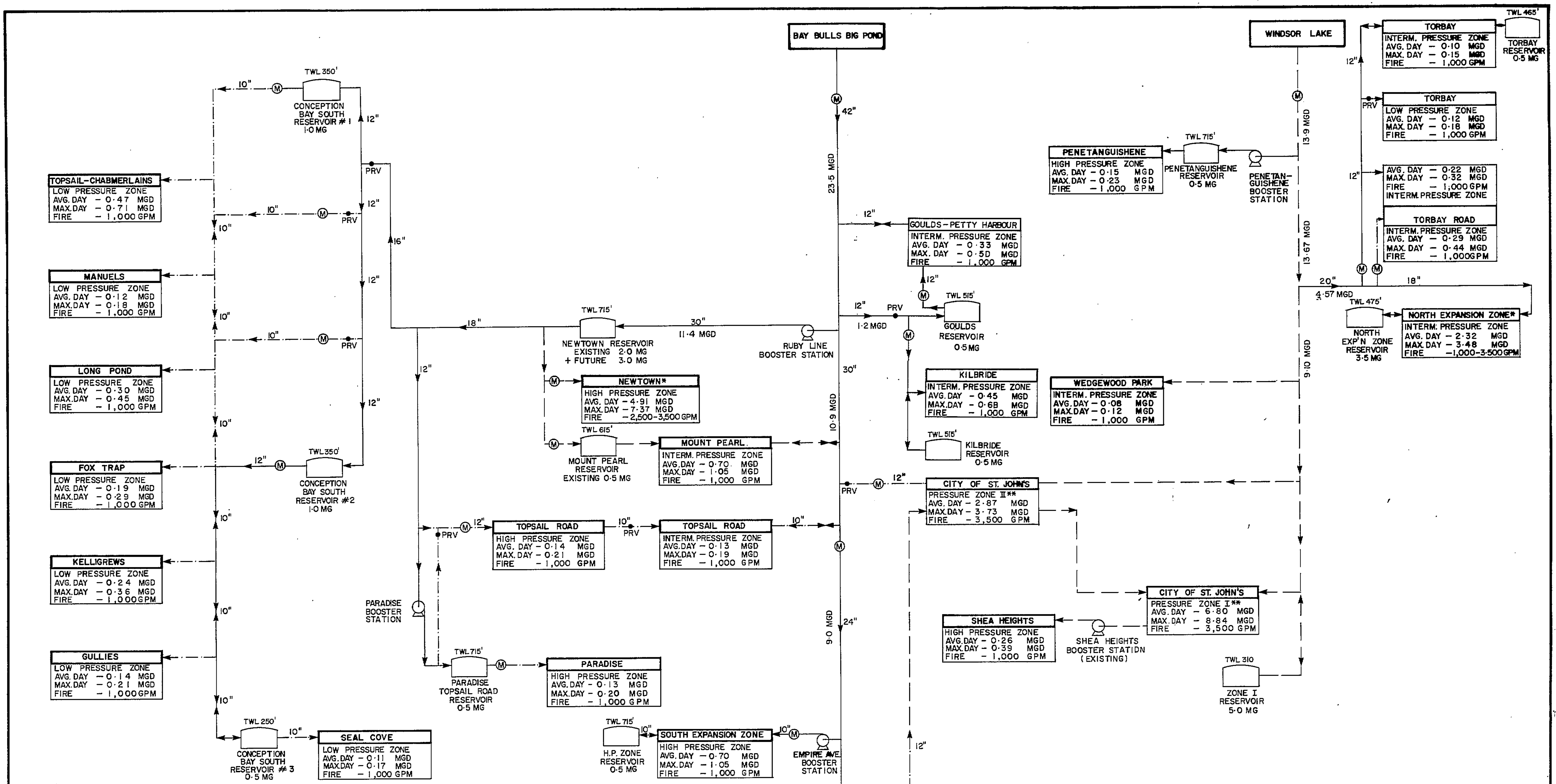
11. Penetanguishene Supply System

Penetanguishene forms a high pressure zone. A booster pumping station is proposed at Windsor Lake to lift the water to the proposed Penetanguishene service reservoir. To provide this community with a reliable supply system, the outer main ring loop concept is recommended.

12. Sundry

Existing water supply networks for the smaller communities, such as Shea Heights and Wedgewood Park have not been evaluated under the scope of this study. They have been assumed to receive their water supply in the same manner as they do presently. Should this be found to be inadequate, a preliminary design study would be required to establish rectifiable measures.

As a general note for all pressure reducing (regulating) valves, it should be realized that, these valves should be set in such a manner as to balance the flow through them to conform with the basic design concepts. The initial setting could be determined during the detailed design stage when all the pertinent system information is available. For the City of St. John's supply network this setting could be established by the computer simulation programme. These initial setting values would receive their final adjustment in the field based on factual experience.



NOTES:

1. LOW PRESSURE ZONE EXTENDS BETWEEN GROUND ELEVATION 0 AND 210 FEET.
2. INTERMEDIATE PRESSURE ZONE EXTENDS BETWEEN GROUND ELEVATION 215 AND 425 FEET.
3. HIGH PRESSURE ZONE EXTENDS BETWEEN GROUND ELEVATION 425 AND 625 FEET.
4. * INCLUDES INDUSTRIAL PARK.
5. ** ZONE I INCLUDES LOW PRESSURE AREAS EAST OF HIGGINS LINE, ALLANDALE ROAD, BONAVENTURE AVENUE AND SOUTH OF LEMARCHANT ROAD UP TO SHAW STREET.
   ZONE II INCLUDES INTERMEDIATE PRESSURE ZONE WEST AND NORTH OF ZONE I UP TO ANDERSON AVENUE, EMPIRE AVENUE AND SOUTH OF MUNDY POND AND CANADA DRIVE + LOW PRESSURE AREAS WEST OF SHAW STREET.
   ZONE III INCLUDES INTERMEDIATE PRESSURE ZONE IN THE AREA OF MUNDY POND AND KENMOUNT ROAD + SOUTH EXPANSION ZONE NORTH OF CANADA DRIVE.

LEGEND

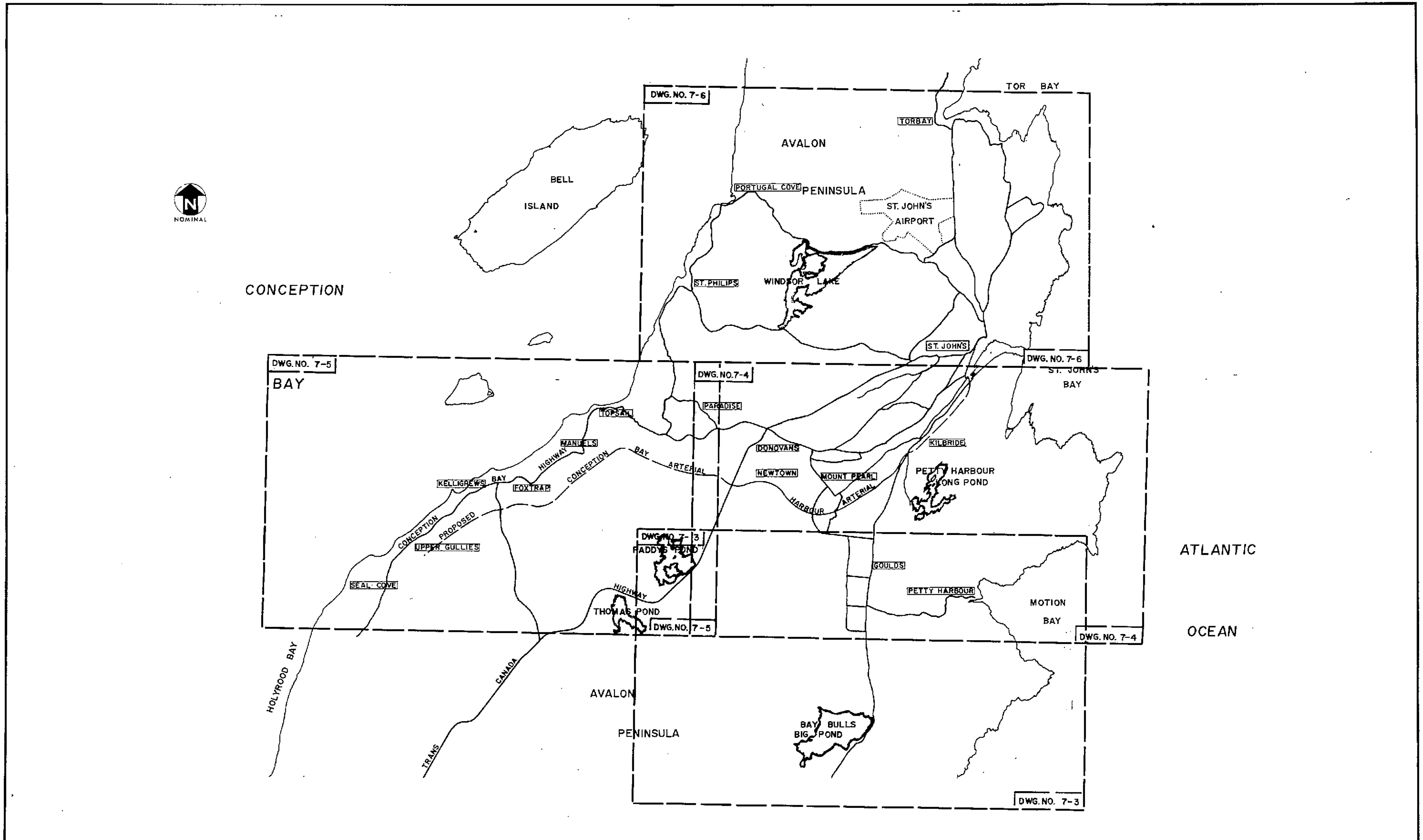
- (M) WATER METER
- PRV PRESSURE REDUCING VALVE
- VALVE NORMALLY CLOSED (FOR EMERGENCY USE ONLY)
- PROPOSED CONVEYANCE MAIN
- RECOMMENDED DISTRIBUTION MAIN
- EXISTING MAIN

SCALES

ST. JOHN'S REGIONAL WATER SYSTEM STUDY
SCHEMATIC LAYOUT
REGIONAL CONVEYANCE SYSTEM

D.R.E.E. PROJECT NO. 3.1 | FENCO PROJECT NO. I-6068-I | DWG. NO. 7-1 | REV

FENCO
Foundation of Canada Engineering Corporation Limited



LEGEND

▬ ▬ ▬	PROPOSED CONVEYANCE PIPELINE	···········	METROPOLITAN AREA BOUNDARY
○○○○○○	EXISTING WATERMAIN	●●●●●●●●	POSSIBLE FUTURE DISTRIBUTION PIPELINE
——·——·——	POWER TRANSMISSION LINE	—·——·—	PLANNED DEVELOPMENT ZONE BOUNDARY
— - — - —	MUNICIPAL BOUNDARY	— — — —	PLANNED INDUSTRIAL PARK BOUNDARY

SCALES

ST. JOHN'S REGIONAL WATER SYSTEM STUDY
REGIONAL KEY PLAN
SHEET NO. 1

D.R.E.E. PROJECT NO. 3.1

FENCO PROJECT NO. 1-6068-1

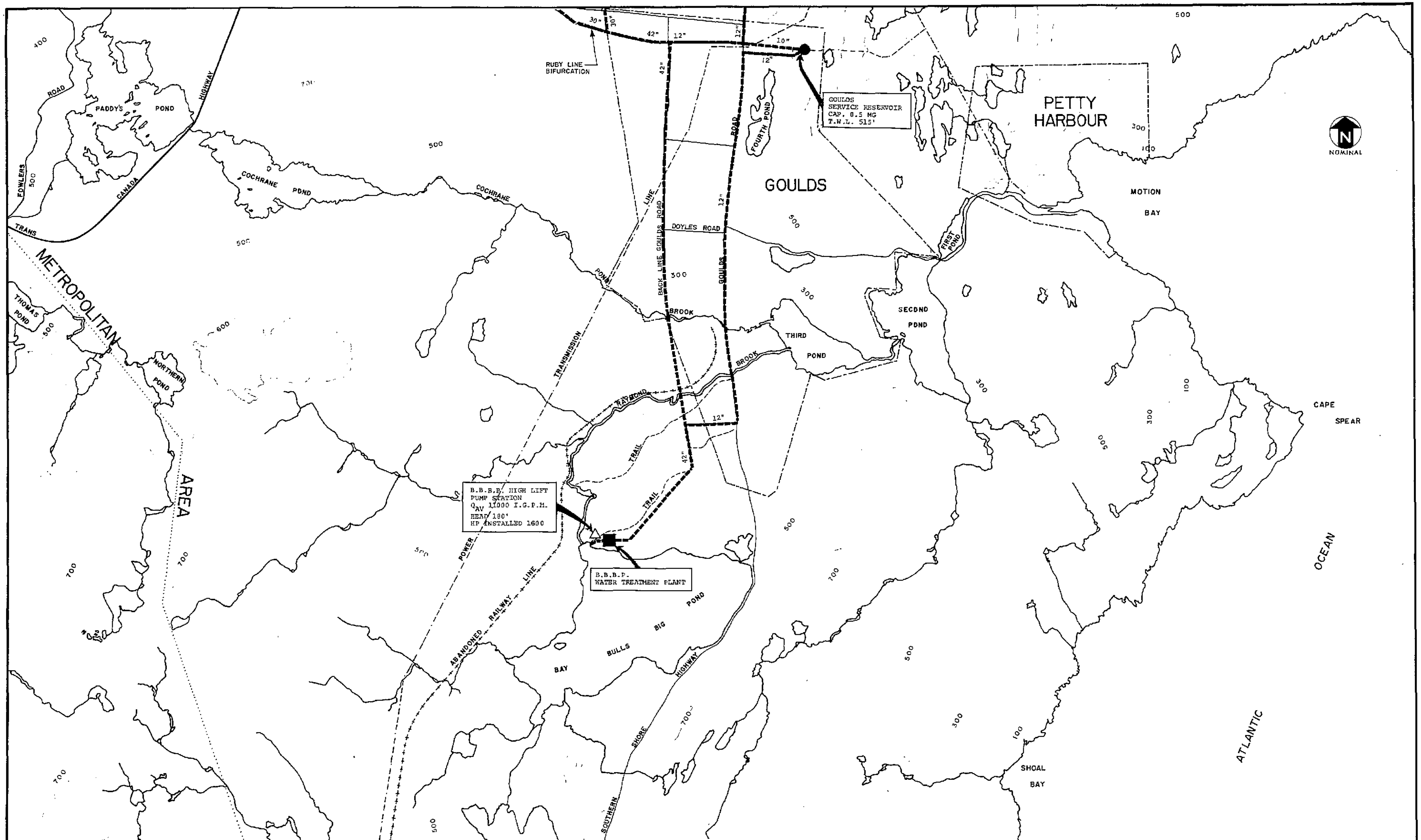
FENCO

FENCO

Foundation of Canada Engineering Corporation Limited

DWG NO. 7-2

REV



NOTE:
FINAL HEAD AND HP OF PUMP STATIONS TO BE ESTABLISHED DURING DETAILED DESIGN STAGE.

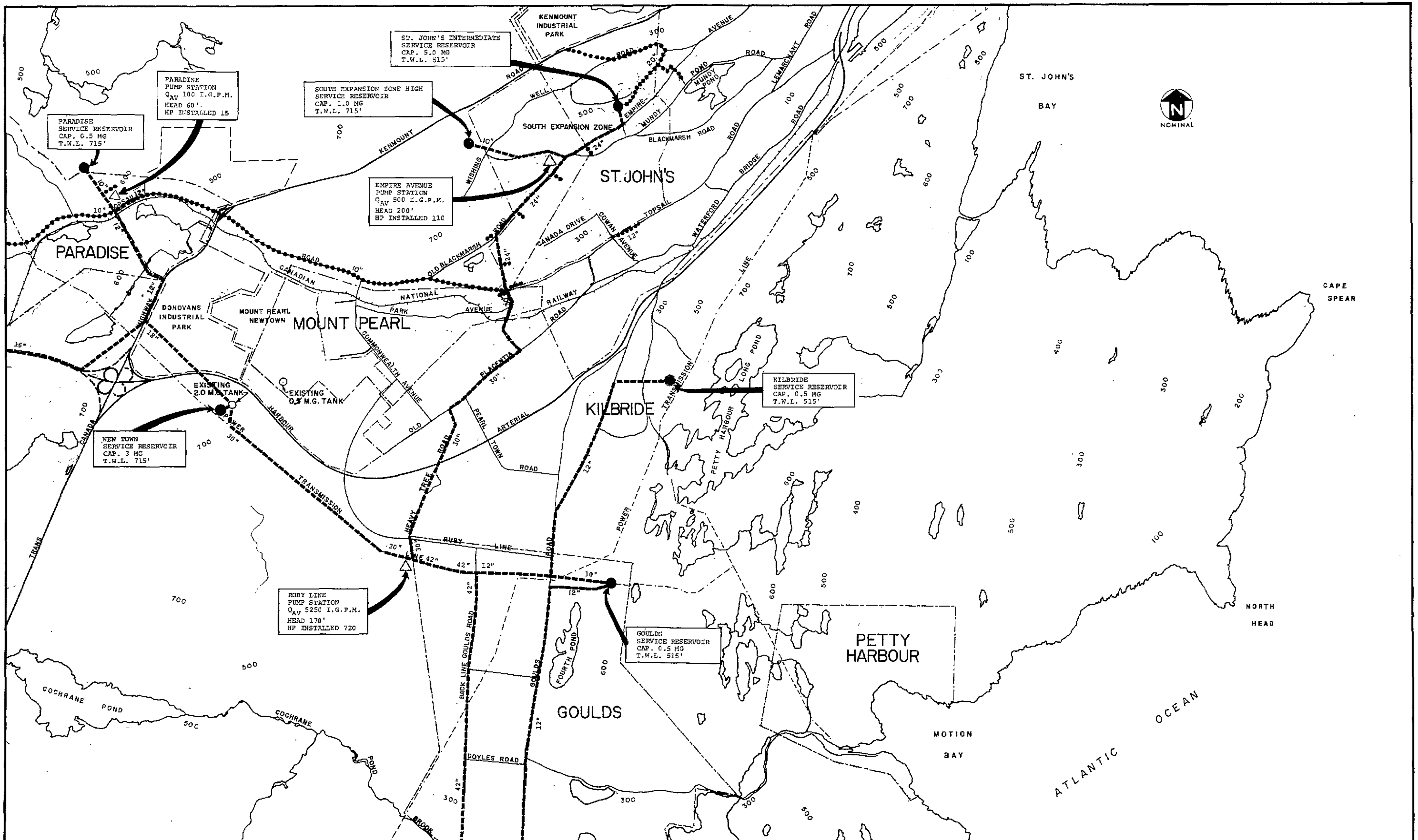
REFERENCES: 7-6, 7-5, 7-4, 7-3, ST. JOHN'S

SCALES: 2000 0 4000

ST. JOHN'S REGIONAL WATER SYSTEM STUDY
GENERAL LAYOUT PLAN
SHEET NO. 2

FENCO
Foundation of Canada Engineering Corporation Limited

D.R.E.E. PROJECT NO. 3.1 | PROJECT NO. I-6068-I | DWG NO 7-3



NOTE:
FINAL HEAD AND HP OF PUMP STATIONS TO BE ESTABLISHED DURING DETAILED DESIGN STAGE.

REFERENCES: 7-6, 7-5, 7-4, 7-3, ST. JOHN'S

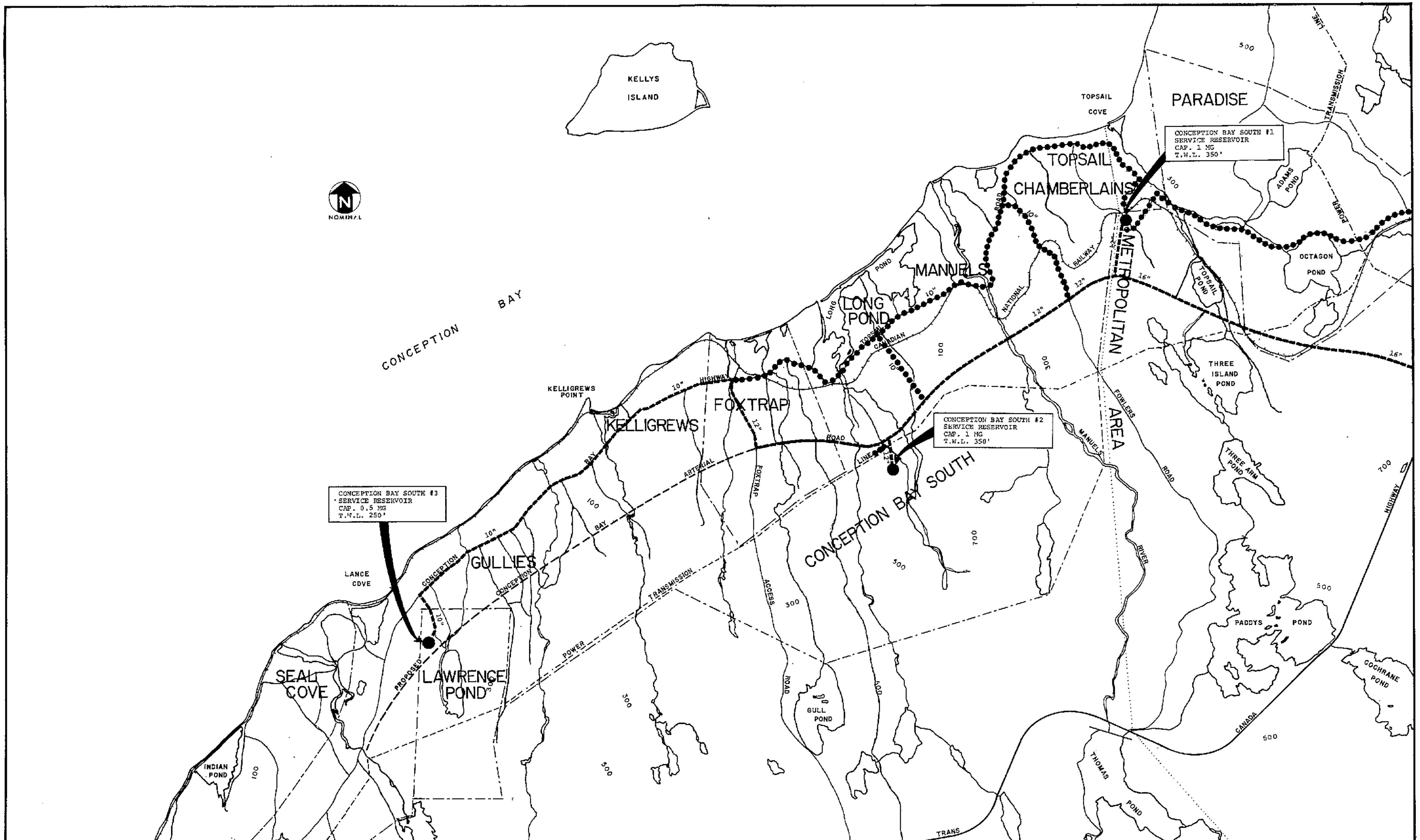
SCALES: 2000 0 4000

ST. JOHN'S REGIONAL WATER SYSTEM STUDY
GENERAL LAYOUT PLAN
SHEET NO. 4

DREE. PROJECT NO. 3.1 | PROJECT NO. I-6068-1

FENCO
Foundation of Canada Engineering Corporation Limited

DWG NO. 7-4



NOTE:
FINAL HEAD AND HP OF PUMP STATIONS TO BE ESTABLISHED DURING DETAILED DESIGN STAGE.

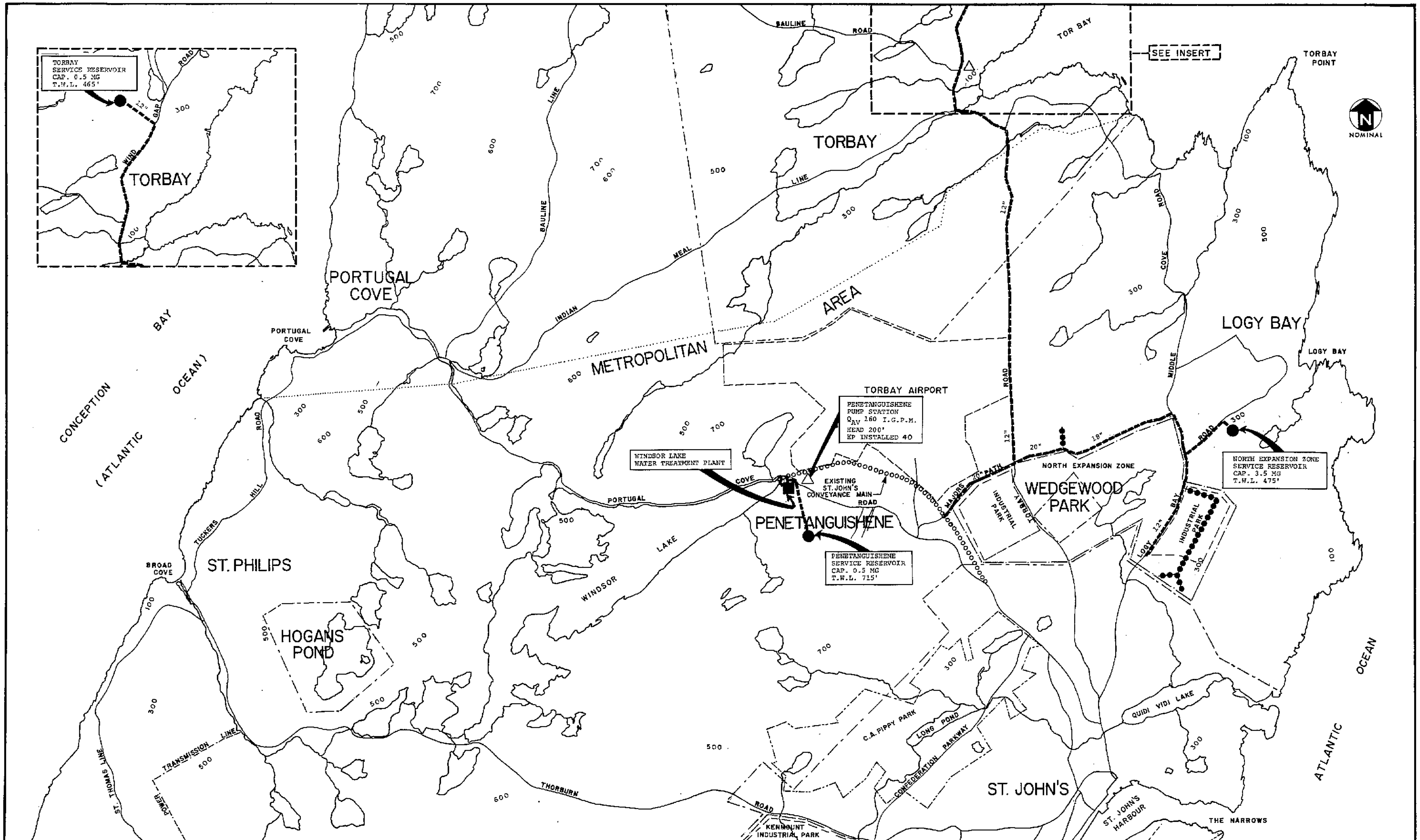
REFERENCES: 7-6, 7-5, 7-4, 7-3, ST. JOHN'S

SCALES: 2000 0 4000

ST. JOHN'S REGIONAL WATER SYSTEM STUDY
GENERAL LAYOUT PLAN
SHEET NO. 3

D.R.E.E. PROJECT NO. 3.1 | FENCO PROJECT NO. I-6068-I | DWG NO 7-5

FENCO
Foundation of Canada Engineering Corporation Limited



NOTE:
FINAL HEAD AND HP OF PUMP STATIONS TO BE ESTABLISHED DURING DETAILED DESIGN STAGE.

REFERENCES: 7-6, 7-5, 7-4, 7-3, ST. JOHN'S

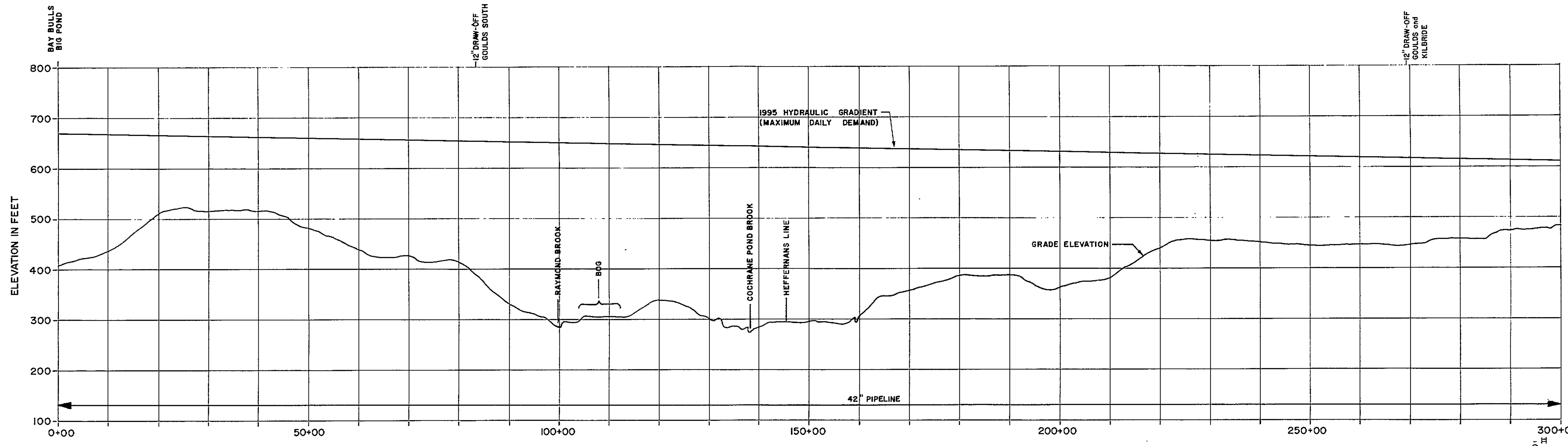
SCALES: 2000 0 4000

ST. JOHN'S REGIONAL WATER SYSTEM STUDY
GENERAL LAYOUT PLAN
SHEET NO. 5

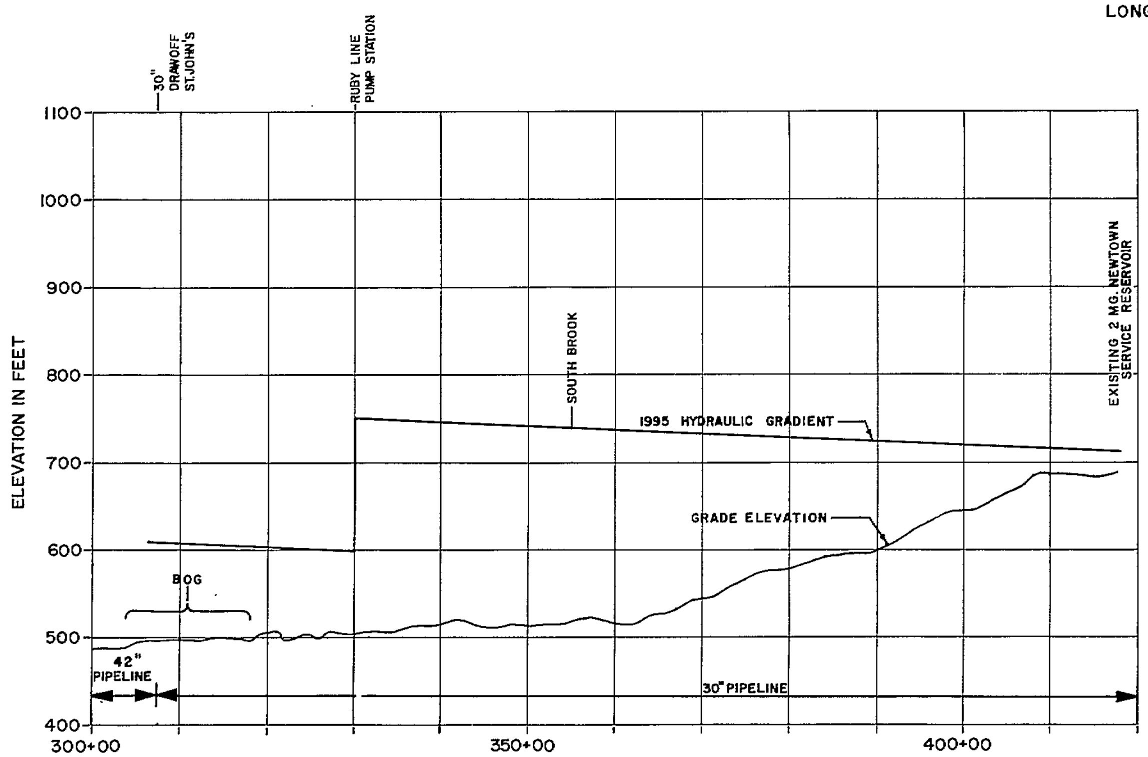
D.R.E.E. PROJECT NO. 3.1 — FENCO PROJECT NO. I-6068-I

FENCO
Foundation of Canada Engineering Corporation Limited

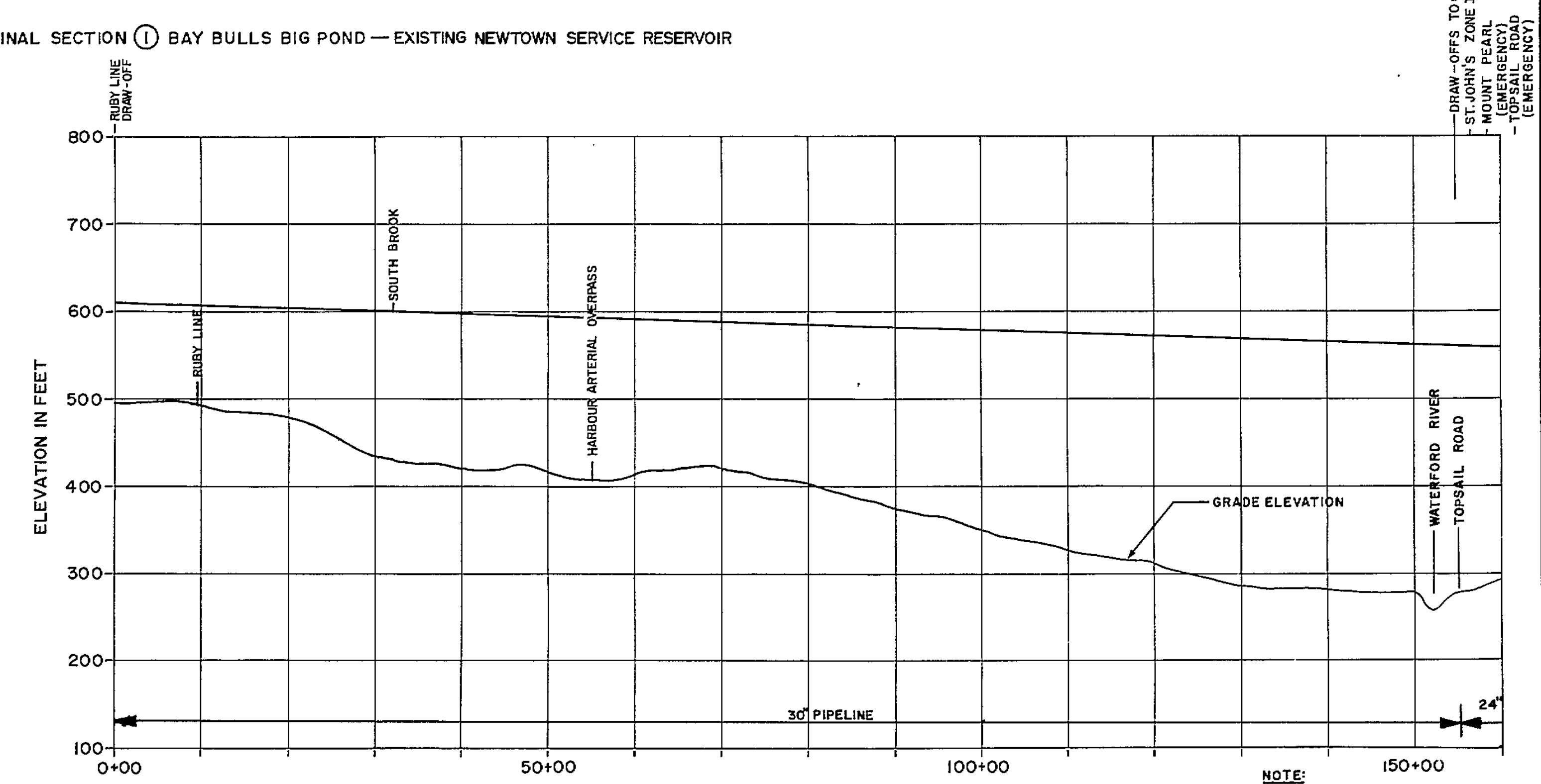
DWG NO. 7-6



LONGITUDINAL SECTION (1) BAY BULLS BIG POND — EXISTING NEWTOWN SERVICE RESERVOIR



SECTION (1) CONTINUED



LONGITUDINAL SECTION (2) RUBY LINE, DRAW OFF — ST. JOHN'S INTERMEDIATE SERVICE RESERVOIR

NOTE: SECTION 2 CONTINUES ON SHEET 7-8

REFERENCES	SCALES	ST. JOHN S REGIONAL WATER SYSTEM STUDY	FENCO
	VERT. 50 0 100	LONGITUDINAL SECTION	Foundation of Canada Engineering Corporation Limited
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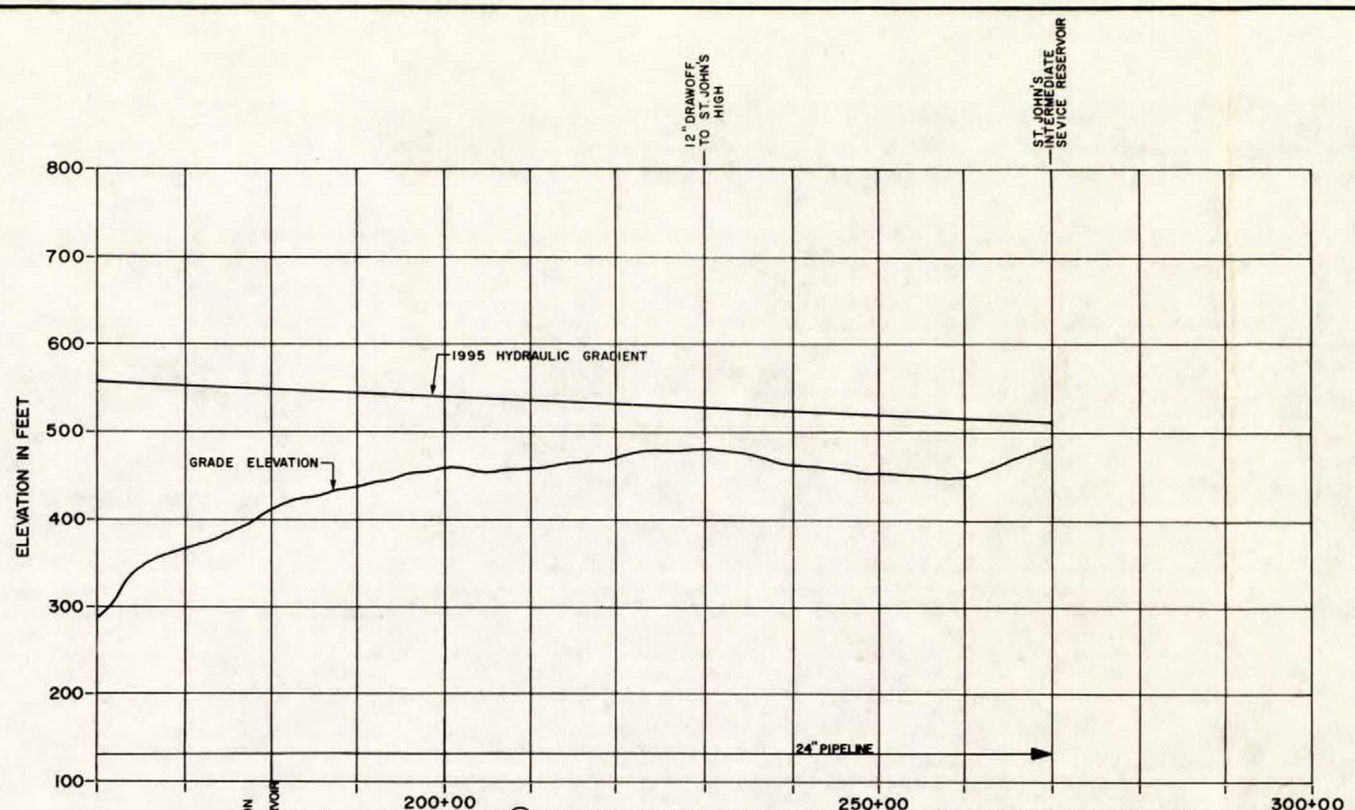
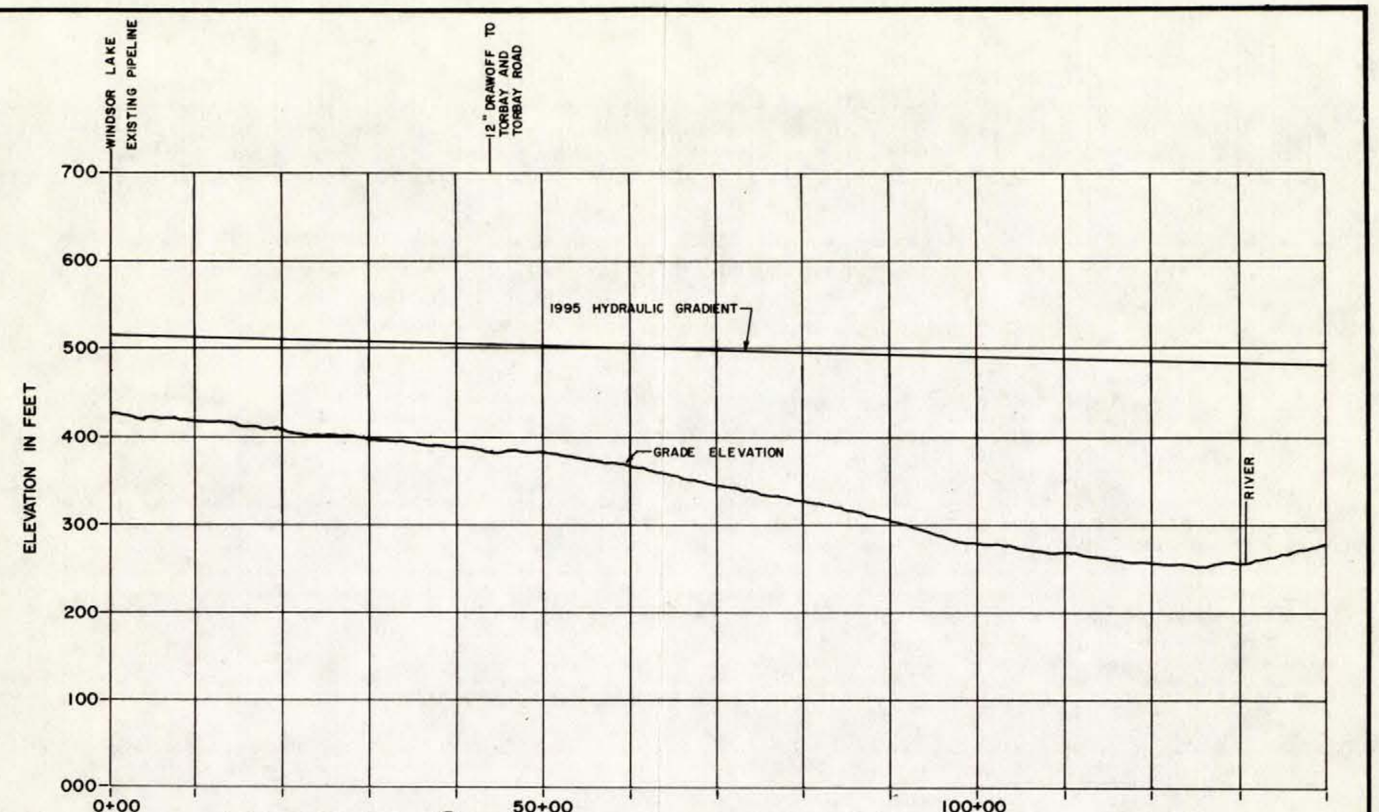
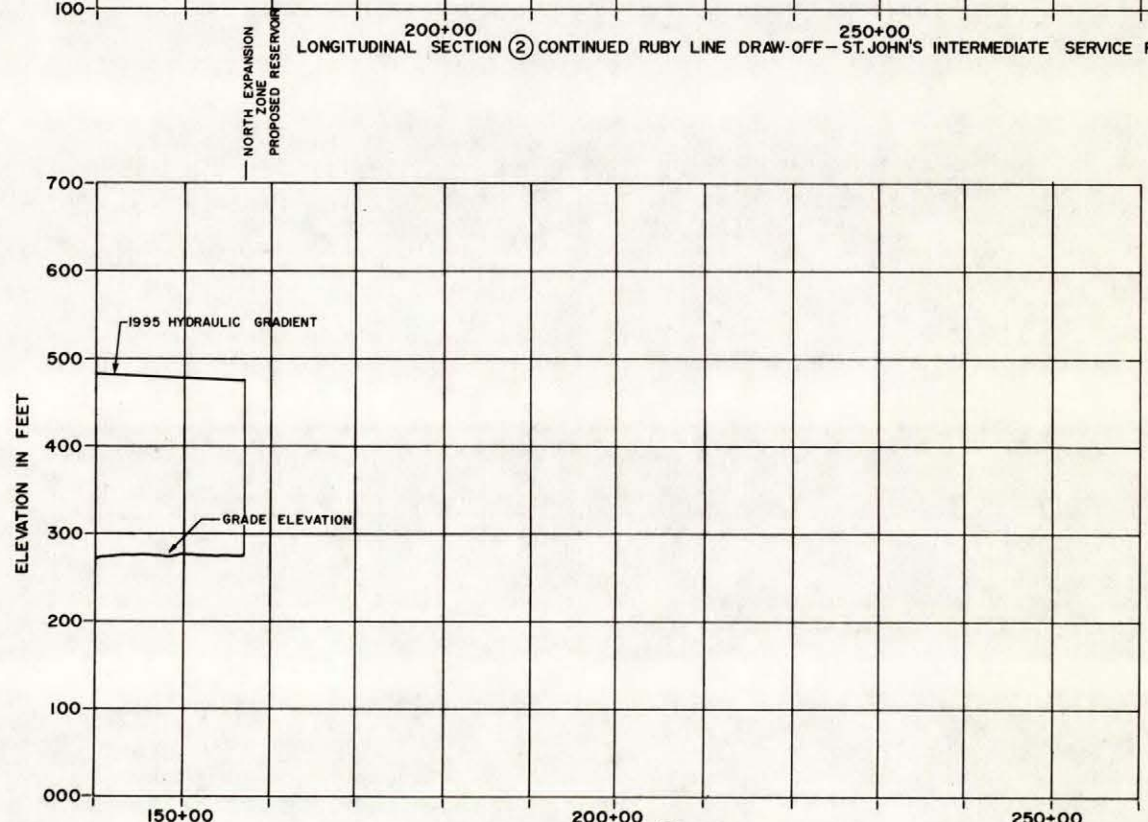
		D.R.E.E. PROJECT NO. 3.1 FENCO PROJECT NO. I-6068-I	DWG NO 7-7



LONGITUDINAL SECTION (2) CONTINUED RUBY LINE DRAW-OFF – ST. JOHN'S INTERMEDIATE SERVICE RESERVOIR



LONGITUDINAL SECTION (3) WINDSOR LAKE EXISTING PIPELINE TO NORTH EXPANSION ZONE SERVICE RESERVOIR



LONGITUDINAL SECTION (3) CONTINUED

REFERENCES

SCALES

VERT. 50 0 100

HORIZ. 500 0 1000

ST. JOHN'S REGIONAL WATER SYSTEM STUDY
LONGITUDINAL SECTION
SHEET NO. 7

D.R.E.E. PROJECT NO. 3.1

FENCO PROJECT NO. I-6068-1

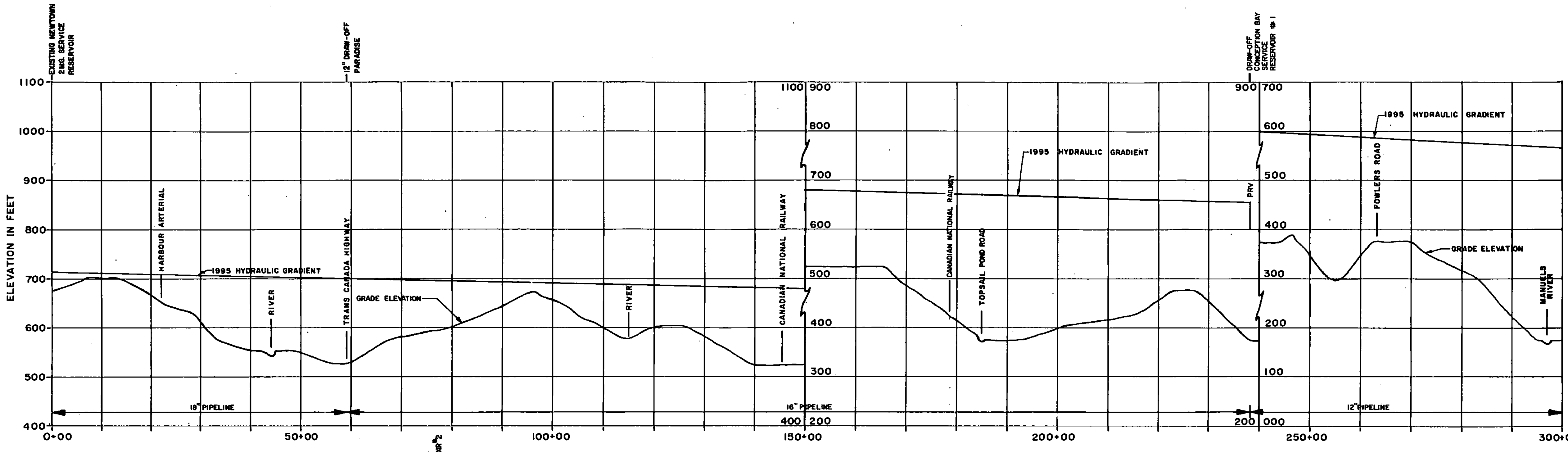
FENCO

FENCO

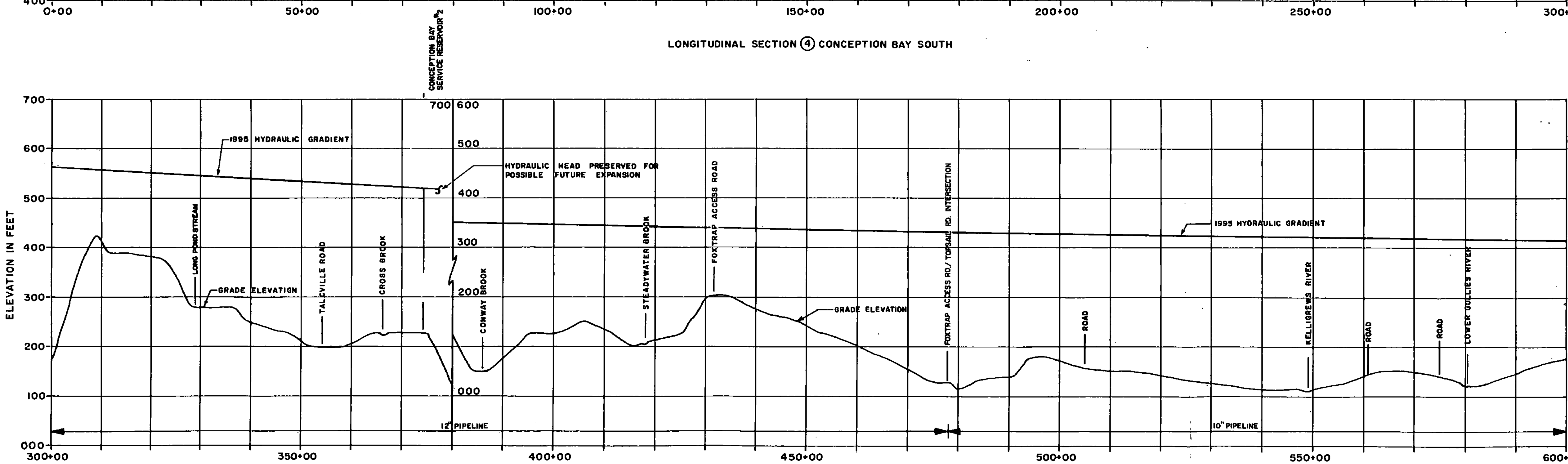
Foundation of Canada Engineering Corporation Limited

DWG NO 7-8

REV



LONGITUDINAL SECTION (4) CONCEPTION BAY SOUTH



NOTE:
GRADE ELEVATIONS SHOWN BETWEEN CHAINAGES 99+49 AND 249+47 ARE EXISTING GROUND ELEVATIONS AND NOT THAT OF THE PROPOSED CONCEPTION BAY ARTERIAL HIGHWAY

SECTION (4) CONTINUED

NOTE: SECTION (4) CONTINUES ON SHEET 7-10

REFERENCES

SCALES
VERT. 50 0 100
HORIZ. 500 0 1000

ST. JOHN'S REGIONAL WATER SYSTEM STUDY
LONGITUDINAL SECTION
SHEET NO. 8

D.R.E.E. PROJECT NO. 3.1
FENCO PROJECT NO. I-6068-I

FENCO
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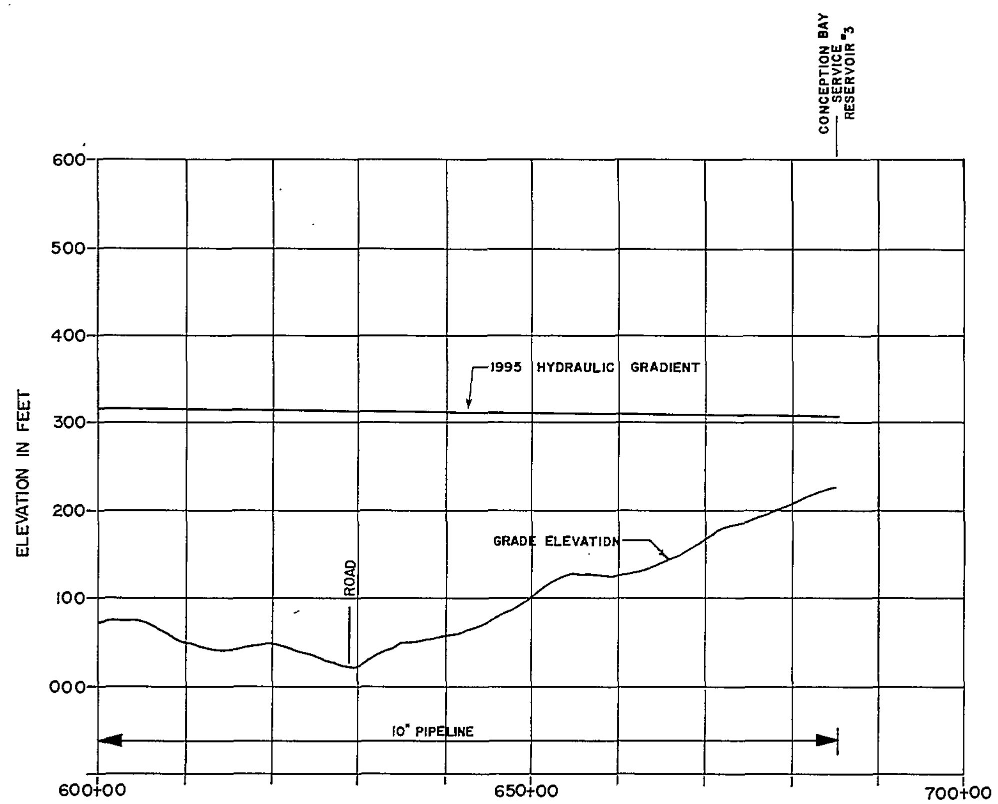
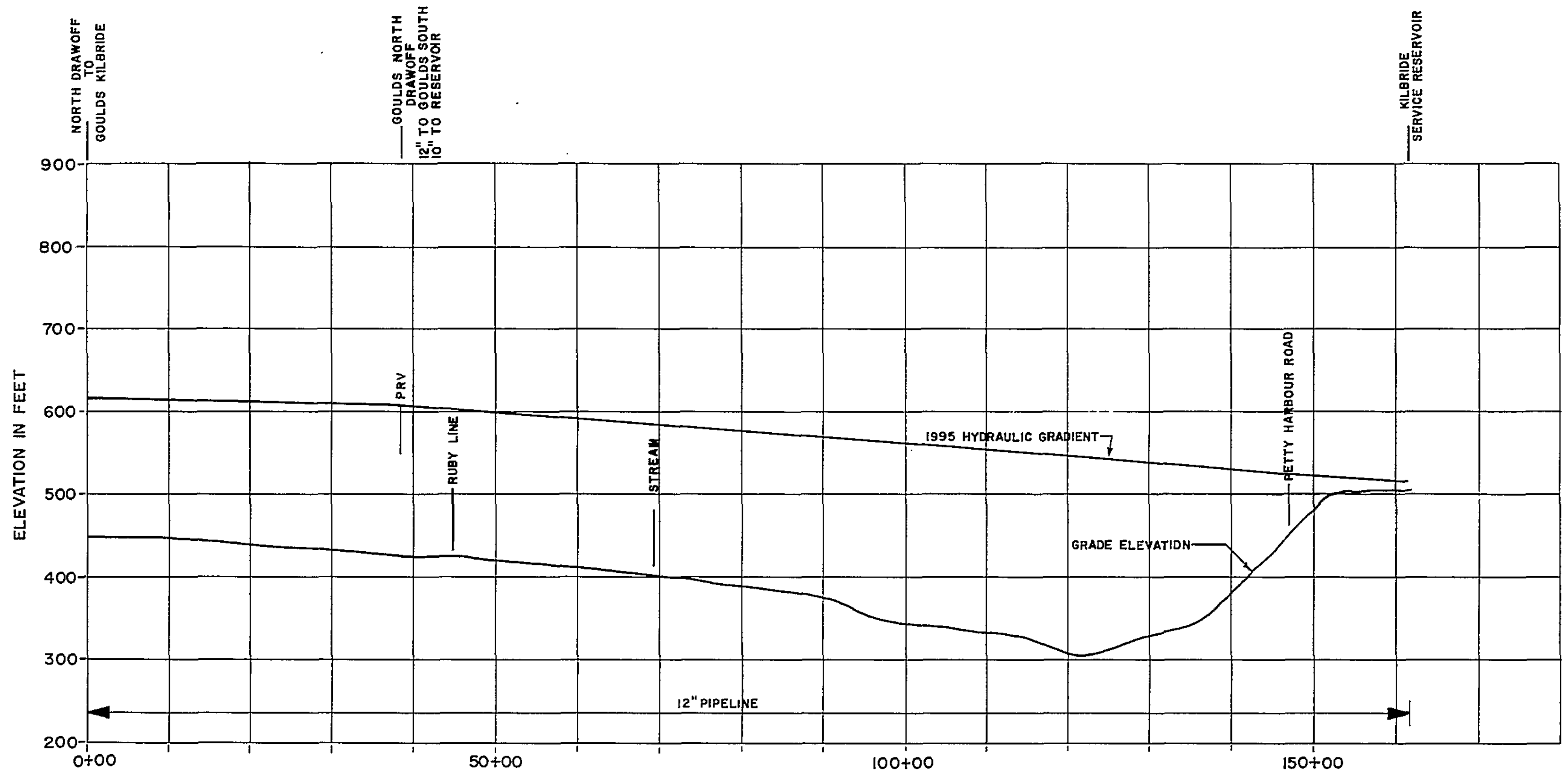
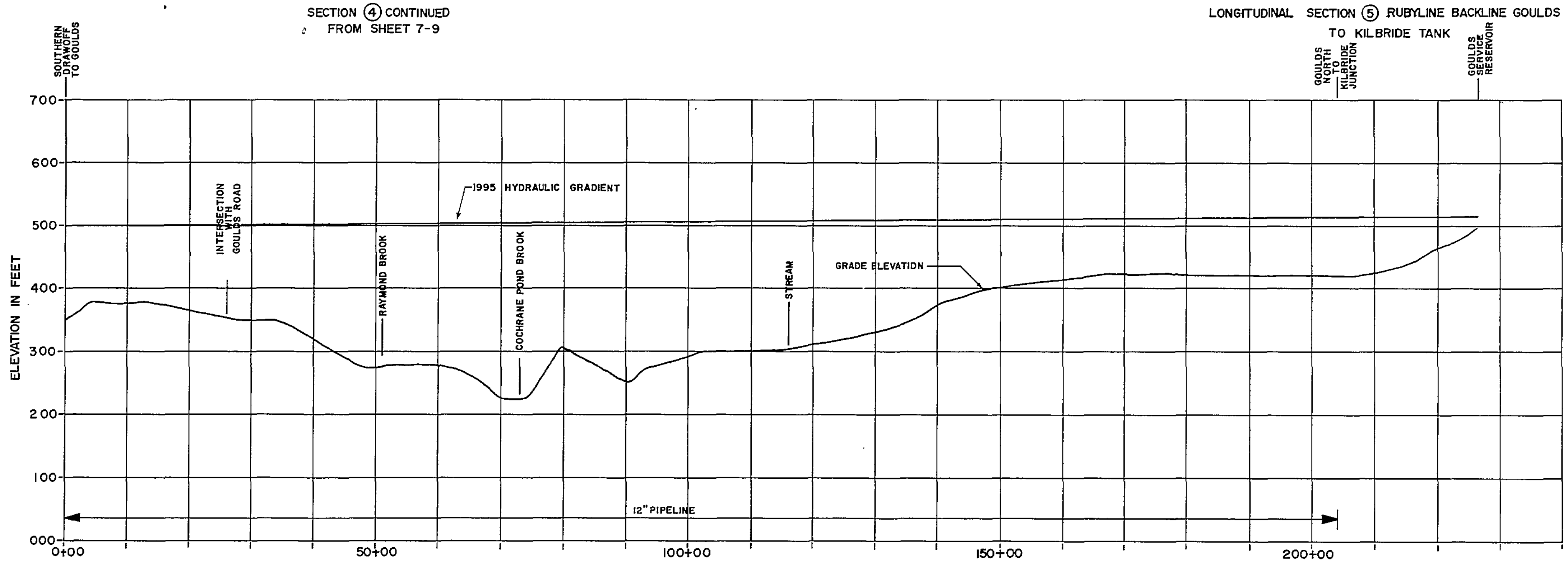
DWG. NO. 7-9



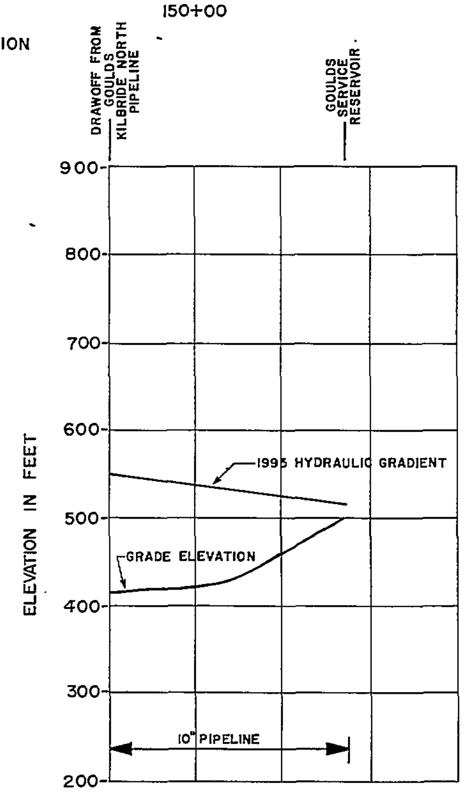
SECTION ④ CONTINUED
FROM SHEET 7-9



LONGITUDINAL SECTION ⑤ RUBYLINE BACKLINE GOULDS JUNCTION
TO KILBRIDE TANK

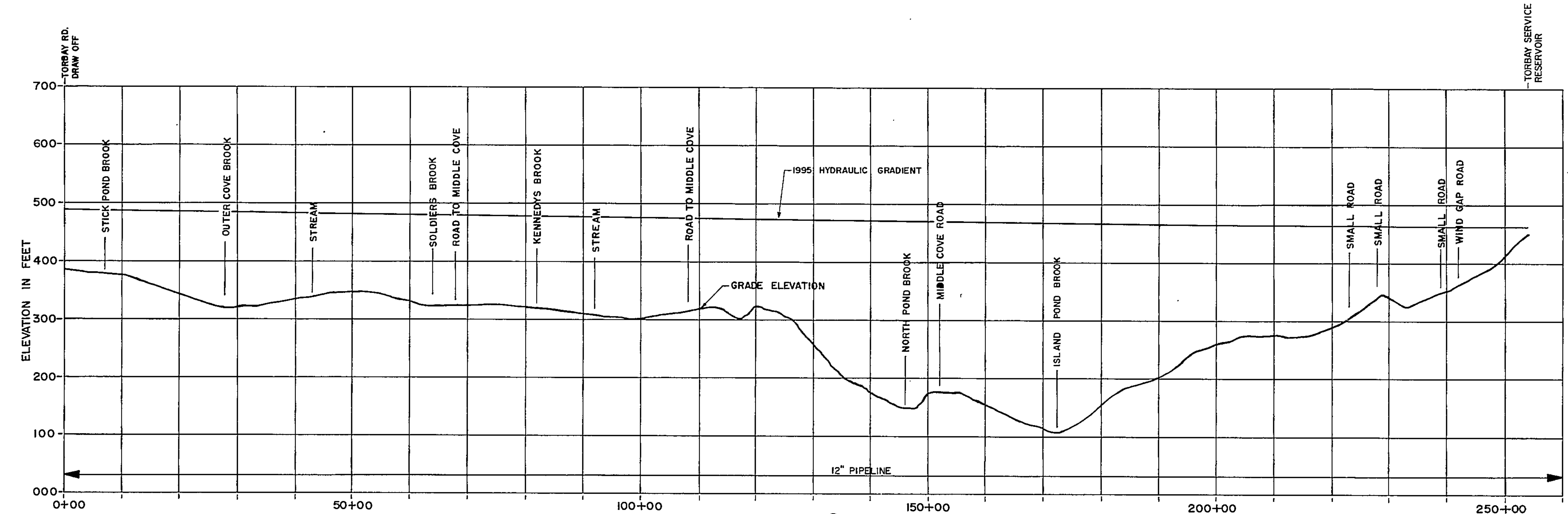


LONGITUDINAL SECTION ⑥ GOULDS DRAWOFF TO GOULDS / KILBRIDE JUNCTION



LONGITUDINAL SECTION ⑦
GOULDS NORTH / KILBRIDE JUNCTION
TO GOULDS SERVICE RESERVOIR

REFERENCES	SCALES	ST. JOHN'S REGIONAL WATER SYSTEM STUDY LONGITUDINAL SECTION SHEET NO. 9	FENCO Foundation of Canada Engineering Corporation Limited
	VERT. 50 0 100 HORIZ. 500 0 1000	D.R.E.E. PROJECT NO. 3.1 FENCO PROJECT NO. I-6068-I	DWG NO 7-10 REV



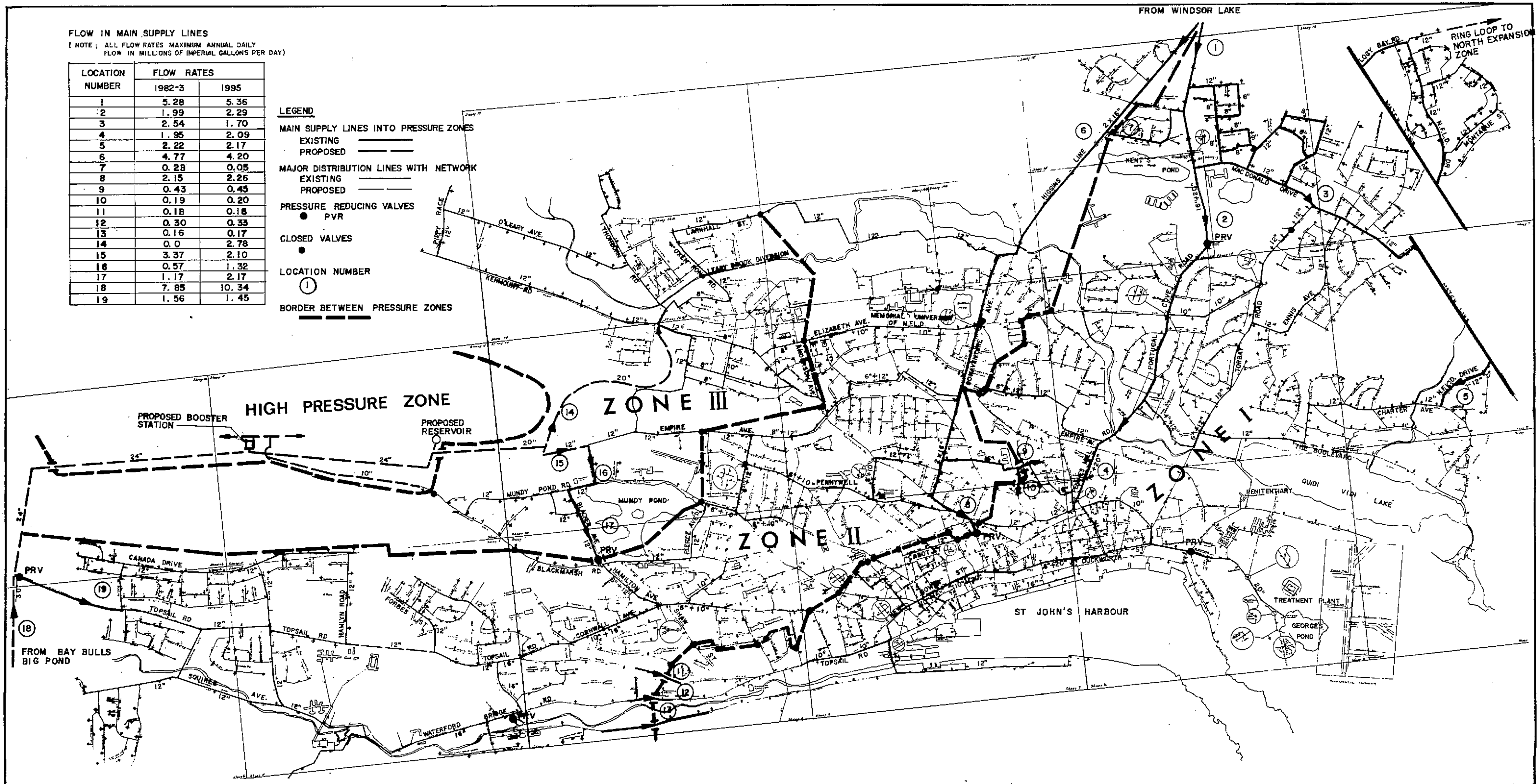
LONGITUDINAL SECTION (8) TORBAY ROAD

REFERENCES	SCALES		
	VERT. 50 0 100 HORIZ. 500 0 1000	ST. JOHN S REGIONAL WATER SYSTEM STUDY LONGITUDINAL SECTION SHEET NO.10	FENCO Foundation of Canada Engineering Corporation Limited
		D.R.E.E. PROJECT NO.3.1 — FENCO PROJECT NO. I-6068-1	DWG NO 7-11

FLOW IN MAIN SUPPLY LINES

(NOTE ; ALL FLOW RATES MAXIMUM ANNUAL DAILY FLOW IN MILLIONS OF IMPERIAL GALLONS PER DAY)

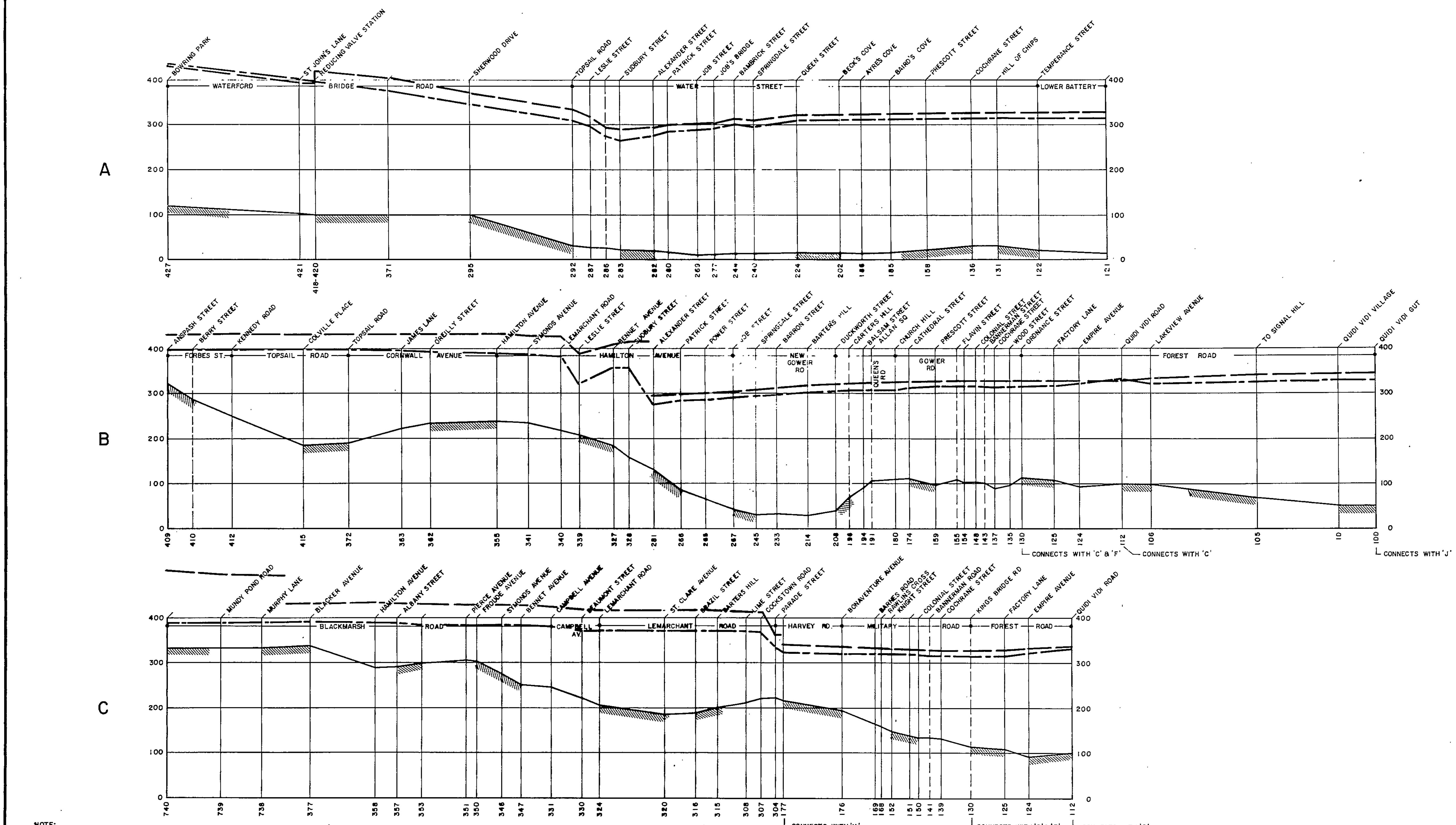
LOCATION NUMBER	FLOW RATES	
	1982-3	1995
1	5.28	5.36
2	1.99	2.29
3	2.54	1.70
4	1.95	2.09
5	2.22	2.17
6	4.77	4.20
7	0.28	0.05
8	2.15	2.26
9	0.43	0.45
10	0.19	0.20
11	0.18	0.18
12	0.30	0.33
13	0.16	0.17
14	0.0	2.78
15	3.37	2.10
16	0.57	1.32
17	1.17	2.17
18	7.85	10.34
19	1.56	1.45



ST. JOHN'S REGIONAL WATER SYSTEM STUDY
CITY OF ST. JOHN'S
WATER SUPPLY NETWORK

FENCO
Foundation of Canada Engineering Corporation Limited

D.R.E.E. PROJECT NO. 3.1 | FENCO PROJECT NO. 1-6068-1 | DWG NO 7-12



NOTE:
DISCONTINUATION OF HYDRAULIC GRADIENT DENOTES A CLOSED VALVE.

LEGEND

— - — HYDRAULIC GRADIENT PRESENT PEAK FLOW CONDITION.

— — — HYDRAULIC GRADIENT 1995 MAXIMUM DAILY FLOW CONDITION.

ST. JOHN'S REGIONAL WATER SYSTEM STUDY
HYDRAULIC GRADIENS
ALONG TYPICAL SECTIONS OF
ST. JOHN'S NETWORK

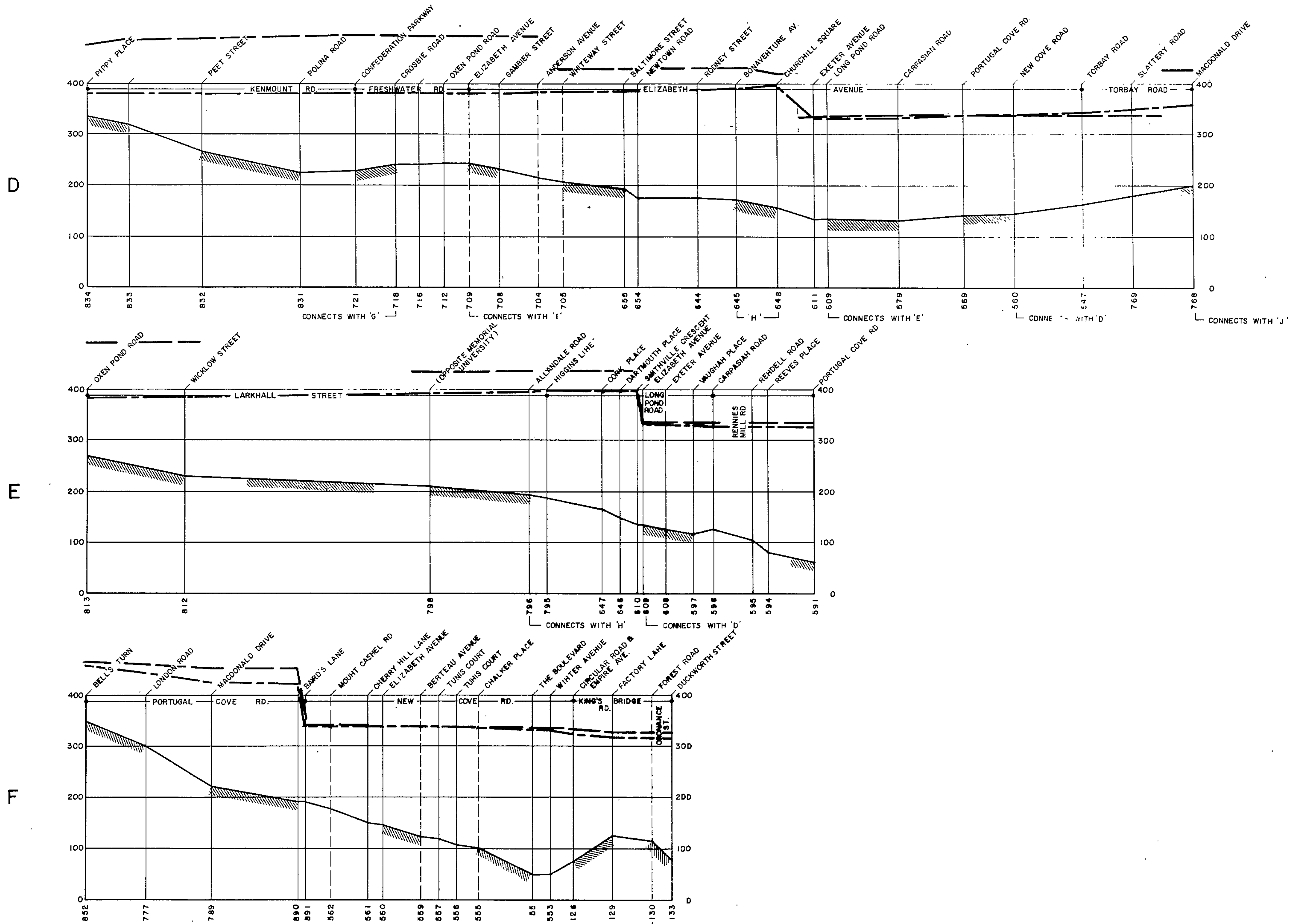
FENCO
Foundation of Canada Engineering Corporation Limited

D.R.E.E. PROJECT NO. 3-1

FENCO PROJECT NO. I - 6068-1

DWG NO 7-13

REV



NOTE:
DISCONTINUATION OF HYDRAULIC GRADIENT DENOTES A CLOSED VALVE.

LEGEND

—·—·— HYDRAULIC GRADIENT PRESENT PEAK FLOW CONDITION.

— — — HYDRAULIC GRADIENT 1995 MAXIMUM DAILY FLOW CONDITION.

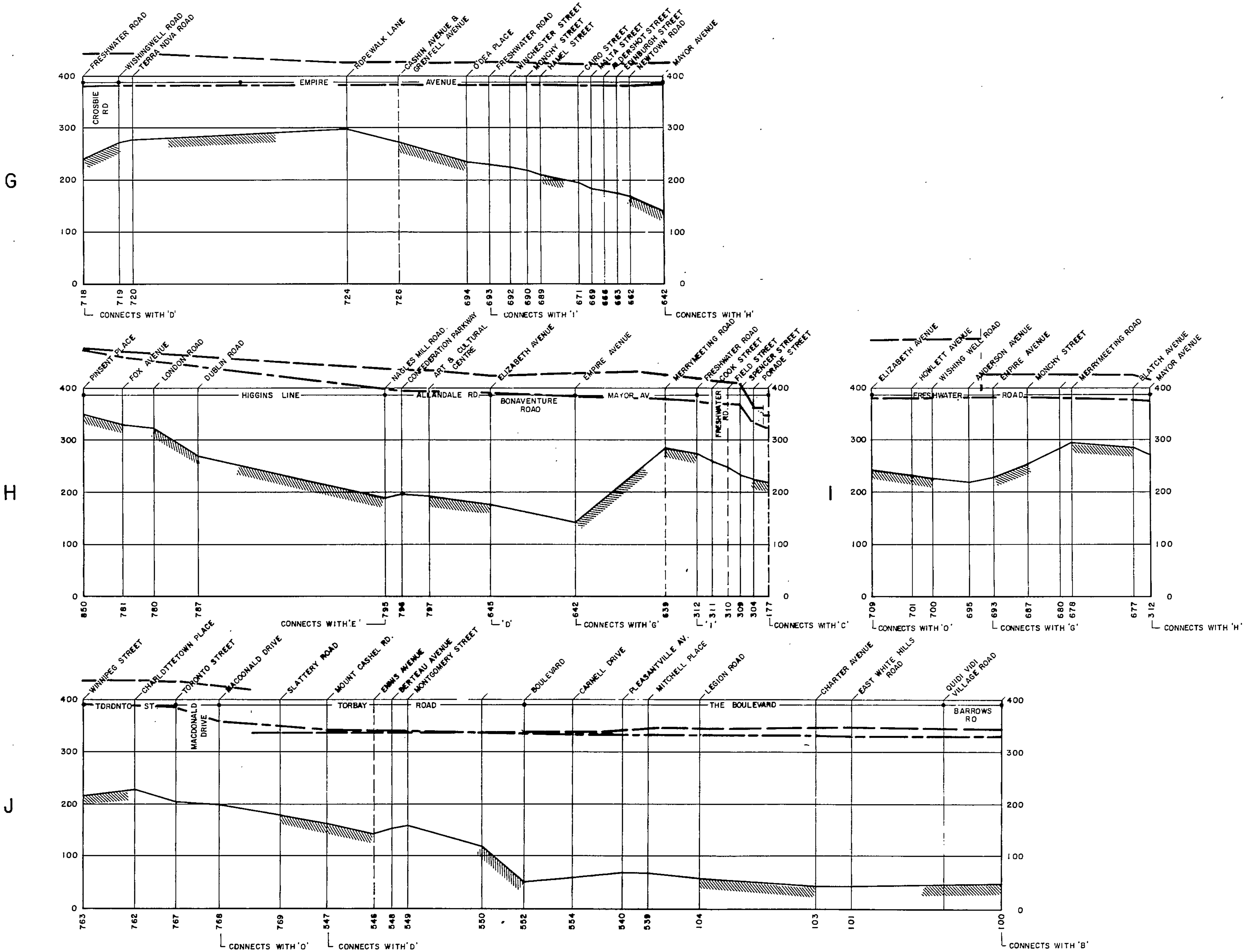
ST. JOHN'S REGIONAL WATER SYSTEM STUDY
HYDRAULIC GRADIENS
ALONG TYPICAL SECTIONS OF
ST. JOHN'S NETWORK

D.R.E.E. PROJECT NO. 3-1

FENCO PROJECT NO. I - 6068-I

FENCO
Foundation of Canada Engineering Corporation Limited

DWG NO 7 - 14



NOTE:
DISCONTINUATION OF HYDRAULIC GRADIENT DENOTES A CLOSED VALVE.

LEGEND

HYDRAULIC GRADIENT PRESENT PEAK FLOW CONDITION.

HYDRAULIC GRADIENT 1995 MAXIMUM DAILY FLOW CONDITION.

ST. JOHN'S REGIONAL WATER SYSTEM STUDY
HYDRAULIC GRADIENS
ALONG TYPICAL SECTIONS OF
ST. JOHN'S NETWORK

FENCO
Foundation of Canada Engineering Corporation Limited

D.R.E.E. PROJECT NO. 3-1 FENCO PROJECT NO. I-6068-I DWG NO 7-15

APPENDIX I

COMPUTER MODEL OF THE WATER SUPPLY NETWORK OF ST. JOHN'S

General Features

The computer model of the water supply system of St. John's enables pipe flows and pressures to be calculated throughout the whole system, given the inflows and outflows at all points and basic physical data (pipe lengths and diameters, etc.). It can be readily modified to examine the effect either of different flow conditions or of changing parts of the system itself, e.g. adding or removing pipes, or closing or reopening valves. More will be said about the composition of the network represented after a discussion of the basic principles involved in the computer solution.

Solution of Flow Problems by Computer

Whatever method of solution is used, it must necessarily be based on an iterative procedure (i.e. successive approximations). The merit of any particular procedure should be judged on the rapidity of convergence to the correct results; this statement raises two further questions which will be commented on subsequently:

(a) Is convergence always ensured?
(b) How to judge when convergence is achieved?

Two iterative methods have been used in the past to solve pipe flow problems:

(i) The well-known Hardy Cross method, which is also in use to solve for forces in redundant structural frames, starts out by making assumptions as to the flows round closed pipe loops in the distribution system, and proceeds to adjust these flows so as to correct for any imbalances in head differences.

(ii) A procedure in which heads, rather than flows, are assumed to start with at all pipe junctions (or at designated inflow and outflow points) within a network, these heads being subsequently adjusted so as to correct for any imbalances in flows, i.e., non-respect of the continuity equation at these pipe junctions.

In the present instance, the latter procedure has been used. The reasons for doing so when treating flow problems on a computer have been outlined in an article by Shamir and Howard (1), which also describes the method chosen in detail (the Newton-Raphson (2) method of solution). It so happens that neither the Hardy Cross method nor the Newton-Raphson one can be guaranteed to converge under all starting assumptions. In practice they do, provided the initial guess as to flow conditions in the network is reasonable. In actual fact, in the 'Super-Hynal' programme (the I.B.M. name for the flow network analysis routine used for St. John's) much care is taken in selecting the pressures around the network in order to reduce to a minimum the number of iterations required for convergence subsequently to the correct solution.

In effect, despite the large number of equations (just under 700) that have to be solved in the case of the St. John's network, covergence has proved to be extremely rapid indeed: 2 iterations only proved adequate for low-flow conditions, 6 or 7 for high flows well distributed throughout the City, and apparently about 5 for high flows plus a concentrated drawoff of water for fire-fighting (convergence criterion: 5 percent --- see below).

This is indeed a remarkable achievement and corresponds to relatively modest computer charges per run: not more than $60 for the largest number of iterations mentioned, plus the cost of printout of the 2,000 odd lines generated each time (once the network is stored in the memory bank of the computer, the cost of reading in the instructions for each run is generally negligible).

Specific Features of the Solution Process

A general knowledge of the principles underlying the solution procedure, as just described, is not enough in itself to achieve satisfactory results on a computer.

The IBM programme 'SUPER-HYNAL' used has been developed specially for treatment of the St. John's network (the predecessor IBM programme still in use is much smaller and does not contain many of the advanced features to be mentioned below). Implementation of the programme required months of work by IBM, the object being to reduce computation costs to their present satisfactory level. In order to achieve this, the 'SUPER-HYNAL' programme does the following:

(a) It develops its own numbering system for the pipe junctions--these numbers are the means of specifying not only the 'nodes' or pipe junctions themselves, but also the connecting pipes between nodes.

Thus the user's own system of node numbers can be perfectly arbitrary, and even if the numbering is not proceeded with in logical order (as for example nodes 18 and 22 had to be inserted on the harbour quay, between surrounding node numbers in the 180 - 220 range).

(b) The equations can be solved starting from any selected pipe junction, specified by the programmer. This option enables the time required for the computer to solve the equations to be sharply reduced, as compared with a situation in which the computer started off from the first pipe junction listed.

The starting node or junction chosen as the most suitable here (after a summary investigation conducted arbitrarily on nodes in the even hundreds) is 500. The computer time was cut by more than 50 percent, as compared with the situation prior to implementation of the option, where the starting node was 18.

(c) The desired degree of accuracy which serves as the criterion for convergence may be chosen by the programmer. In our runs, it is currently fixed at 5 percent, meaning that the residual imbalance in flow at every node must be less than 5 percent of the average of flows entering and flows leaving (or 2.5 percent of the flows at the node summed without regard to sign).

In most of the runs to date, the actual maximum error has been about 0.001 Mgd. (about 1 gpm) and the average error per node an order of magnitude smaller than this. The point is that the 5 percent criterion is made to apply to all nodes irrespective of the total inflow and outflow, so that, in the case of locations where there is little flow going past, the restriction on residual imbalance is actually quite severe.

In order to avoid the pursuit of accuracy which, on account of inevitable rounding-off errors, would be impossible to achieve, the computer programme is instructed to assume any error less than 0.5 gpm as negligible, for the purposes of convergence.

Data on which the Computer Programme Operations

The data required for operating the programme must be such as to specify the water supply completely in a physical sense (physical data) as well as the inflows and outflows from the network (use data).

Physical Data

(A) Mains

(i) Numbers of two nodes or pipe junctions (see below), between which the pipe extends.

(ii) The length of pipe (ft.)

(iii) The internal diameter, in inches

(iv) The Hazen-Williams roughness coefficient

In addition to the above information the original punched cards used as input to the programme contain the following data:

(v) Nominal pipe diameter in inches. If two diameters are indicated, the pipe used in the programme results from the combination of two real mains either in parallel or in series (certain simplifications, without being mandatory, were used to reduce the overall number of pipes and pipe junctions where such changes affected secondary mains only; the object of these simplifications was to lower the cost per computer run, without of course jeopardizing overall accuracy).

(vi) Year when the pipe was laid. This information was used to assess the effect of incrustation on the older, unlined cast iron mains in the system (those laid in pre-war days).

(vii) Geographical location of the main, for reference (name of street, and direction of main going from the low-numbered node to the high-numbered node; in specifying the pipes, the convention adopted was to give the low numbered node first, e.g. 787-788 rather than 788-787).

(B) Nodes (or pipe junctions)

(viii) Number assigned to the node. As already explained, the numbering order chosen could be completely arbitrary.

(ix) Height above sea level. The heights chosen were those of ground level, so that the printed results given the pressure above the ground surface (rather than above the pipe junction itself).

(x) Geographical location of the node, for reference purposes. Additional information was required for the reservoirs, pressure reducing valves, and, when used, the booster pump.

Use Data

The consumptions (water drawn off from the system) were indicated on the node cards in millions of Imperial gallons per day*.

Modifications of Data

Two separate procedures were available for modifying the basic data entered up on the original punched cards and stored in the computer:-

(a) Modifications of the physical data relating to the internal diameters and roughnesses of mains:

After three pipes (the oldest, from Ayre's Cove, dating back to 1900) had been excavated, it became apparent that both roughnesses and internal diameters would have to be adjusted within the older section of the system, to allow for the heavy incrustation occurring there.

---

* Units chosen here. Actually a wide choice is available in the programme US or Imp. Gpm, Cfs, Litres per second.......

This was done by a subsidiary computer programme which in effect punched out new sets of cards prior to their being stored in the main computer peripheral memory.

(b) Modification to use data, and also to the mode of operation of the pressure reducing valves:

The original flow considered was a peak one: 19.5 Mgd. spread around the system, some 4.5 Mgd. being from metered industrial users active during working hours only (i.e. not including hospitals, hotels) and 15 Mgd essentially domestic, but with certain other users as just mentioned included.

To convert this consumption pattern to a night use situation, the 4.5 Mgd mentioned were removed from the appropriate pipe junctions and the remaining flows halved, leaving 7.5 Mgd net. Special instructions in the programme enabled such changes to be carried out without re-punching the entire set of node cards.

The operation of the pressure reducing valves had to be varied according to circumstances, and this indeed is one of the crucial points in the programme

(i) If, as for example during periods of low flow in the St. John's network, the pressure on the outlet side of a reducing valve rises to above the pressure setting for that valve, then complete closure occurs.

This, according to our computations, the two PRV's on Portugal Cove Road (between nodes 790/791 and 820/891) close at night, because the outlet pressure rises above their setting of 150 ft. of water, or 65 psi.

(ii) Under other flow conditions, the outlet pressure from a PRV may drop below the pressure setting until restored to that value by flow through the valve.

For all high or medium flows, the 419/418 PRV located on Waterford Bridge Road behaves in this way for example, the regulated pressure being 300 ft. of water, or 130 psi. This valve supplies the Southside Road area.

(iii) A third mode of operation is one in which a PRV is fully open and unable to regulate the pressure on the outlet side; the valve 'floats' so to speak on the system, and the outlet pressure with the valve supplying as much flow as it can, nevertheless drops below the target figure.

For instance, at the highest flow studied of 19.5 Mgd., three of the total of 5 PRV's in use function in this way; these are located at nodes 306/189 (Carters Hill) and 420/417 and 419/418 (Waterford Bridge Road).

Some assumption has to be made about the head loss incurred by a fully open valve; this was put at ten times the velocity head in the approach pipe.

Modifications to the Data

Within the present size limits, which are more than adequate to treat the existing system plus foreseeable extensions, it is very easy to add or remove mains, change flows, change the operation of valves etc.

The size limits just referred to are as follows:

1000 nodes or pipe junctions (696 in use currently)
1750 pipes (1039 used)
20 reservoirs (2 in use, George's Pond having been severed from the system.
20 pressure reducing valves (5 used)

with similar limitations on the number of check valves and booster or source pumps.

Adding extra components does not necessitate reading in the entire deck of 1800 or so data cards, but only as many cards as the items being changed (or less if the changes made can be grouped). At the time of writing 3 separate versions of the St. John's network have been stored in the computer, corresponding to the two flows of 7.5 Mgd. and 19.5 Mgd. (network as operated now), and, for the latter flow, the network with all closed valves separating the high level and low level service areas opened.

It is clear that any future enlarged network can be accommodated with ease.

References

1. U. Shamir & C. D. D. Howard, "Water Distribution Systems Analysis", Journal of the Hydraulics Division, ASCE, January 1968, pp 219-234.

2. S. H. Crandall, "Engineering Analysis" McGraw-Hill, New York, 1956.

Chapter 8

CHAPTER 8

TREATMENT WORKS

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CHAPTER 8

TREATMENT WORKS

SYNOPSIS

The test programme undertaken to study the treatment process most effective and economical for Bay Bulls Big Pond water is discussed in this Chapter. This is followed by an evaluation of the test findings, and recommendations as to the treatment process and works to be implemented. A summary of the findings and recommendations is as follows:

- The test programme included two stages - jar tests and pilot plant studies.

- Jar tests were carried out with alum and ferric - sulphate as coagulants. The optimum pH for coagulation was found to be at a level of 8.5 $\pm$. In both cases colour was removed, however, the pH of the water after coagulation dropped to between 6.0 and 6.3, necessitating additional carbonation as a corrosion inhibitor.

- Pilot treatment plant studies included chemical precipitation of colour with alum, followed by filtration; and oxidation of the colour with ozone, followed by filtration.

- Alum was used with and without polyelectrolyte. At an alum dose of 15 mg/l colour was reduced from 25 TCU to 8 TCU. Addition of Nalcolyte 607 in dosages of 0.5 mg/l and 1.0 mg/l had only a limited effect on colour removal, down to 7 TCU. However this polyelectrolyte did improve the quality of the floc by increasing filter running time by up to 40 percent; from 10 hours to 14 hours in the case of 15 mg/l alum dose and 1.0 mg/l Nalcolyte 607 addition.

- Two contact units were used for the application of ozone - diffused columns and a contactor utilizing a positive pressure injector. The latter was found to be more effective and efficient. At a contact time of less than 2 minutes and an ozone dose of 1.91 mg/l colour was reduced from 23 TCU to less than 5 TCU. Filter running time was 120 hours.

- Based on the results of the pilot plant studies, as related to such important factors as treatment efficiency, filter running time, filter media, residuals in the treated water, safe and reliable water quality, and construction and operating costs, it has been recommended that ozonation be adopted for colour removal (and disinfection) followed by direct high-rate filtration to remove residues and seasonal high concentrations of coliform organisms, turbidity and algae. A cationic polyelectrolyte (such as Nalcolyte 607) will be used as a filter aid in the treatment for removal of the higher concentrations of turbidity and algae. Post chlorination-ammoniation will provide necessary combined chlorine residual in the long conveyance system. This basic process will be adaptable to Bay Bulls Big Pond water as well as Windsor Lake water.

- Design and construction of the treatment works at Bay Bulls Big Pond is envisaged to be undertaken in three stages, as follows:

STAGE	CONSTRUCTION YEAR	DESIGN YEAR	DESIGN CAPACITY
First	1975	1985	16 MGD
Design	1985	1995	24 MGD
Ultimate	1995	reliable yield of B.B.B.P.	36 MGD

- Major components of the treatment works and their design capacity will be as follows:

COMPONENT	DESIGN CAPACITY
- Intake (inlet and conduit)	36 MGD
- Low Lift Pumping Station	
- Structure	36 MGD
- Pumps	*16 MGD
- Travelling Screens	36 MGD
- Ozone Generators	
- Structure	***24 MGD
- Equipment	*16 MGD
- Ozone Contactors	
- Structure	***24 MGD
- Equipment	**16 MGD
- Filters	
- Structure	24 MGD
- Equipment	**16 MGD
- Chemical Feed Systems (Lime, Chlorine, Ammonia, Nalcolyte 607)	
- Structure	36 MGD
- Equipment	*16 MGD

COMPONENT	DESIGN CAPACITY
- High Lift Pumping Station	
- Structure	36 MGD
- Pumps	*16 MGD

---

* plus stand-by capacity

** Hydraulic design will enable the processing of the noted flow with one unit out of service

*** provisions will be included for further future expansion

- Instrumentation will provide for the measurements of:
  - flow of raw water, filter effluent, washwater, service water.
  - temperature of washwater
  - water level in raw water suction well, filter influent channel, clear-well
  - turbidity of raw water and filter effluent
  - chlorine residual and pH level of treated water
- The "inner and outer" loop concept will be used for process control.
- The treatment works at Windsor Lake (to be constructed at a future date) will have a capacity of 16 MGD if augmentation from Little Powers Pond continues. Should this augmentation be discontinued, then the treatment plant will be of 14 MGD capacity.

I. INTRODUCTION

The assessment of the water quality of the two sources of supply, Windsor Lake and Bay Bulls Big Pond, was presented in Chapter 5, Section 4 (see Volume II). In summary, Windsor Lake water quality is within the "objective" and "acceptable limit" criteria set out in the "Canadian Drinking Water Standards and Objectives, 1968". In terms of coliform organisms, however, faecal coliforms are within the "objective" (for treatment by chlorination only), but total coliforms exceed this "objective" at times. Accordingly, it has been recommended that the present practice of treatment by screening, chlorination and pH adjustment, should continue and tight measures be adopted to control activities in the catchment area. This approach would enable deferment of the construction of more conventional treatment facilities, such as filters, resulting in a more favourable cash flow programme.

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. Accordingly, it has been recommended that conventional treatment be provided at the time this source is tapped for supply. This conventional treatment could be direct filtration or clarification-filtration. The objective of treatment will be to render the water constituents to within the "objectives" of the Canadian Drinking Water Standards which were recommended earlier (Chapter 4) for adoption as the standards of this project.

It has further been recommended (Chapter 5, Section 5) that a test programme be undertaken to establish the treatment process most effective and economical for Bay Bulls Big Pond water.

This Chapter concerns itself with the treatment process and treatment works for Bay Bulls Big Pond water. The first part describes the test programme carried out, reports on the findings and makes recommendations as to the treatment process. The centre part of the Chapter outlines the treatment works recommended for Bay Bulls Big Pond. In the last part suggestions are made for treatment works at Windsor Lake.

II. JAR TEST PROCEDURE

The first stage of the test programme included the jar test procedure.

1. The Approach

This procedure follows a pattern whereby raw water is put in beakers (say 1,000 ml volume), each of which is dosed with a chemical to a different pH level. A coagulant is then added to each beaker to qualitatively determine the best chemical reaction. The procedure is then repeated with different coagulant dosages at the optimum level of pH. The purpose of such a test, which is quick and inexpensive, is to ascertain the treatability of the water and to obtain an indication of the level of pH most suitable for the chemical reaction, and the type and dose of the chemical most effective for coagulation-flocculation.

Since batch treatment in a beaker cannot truly simulate continous flow conditions of a full scale treatment unit, interpretation of jar test results should be done very cautiously and preferably correlated with experience from similar full scale plants. Common practice is to assume that actual full scale efficiency, in terms of at least certain constituents, could be downgraded by as much as 50 percent, whereas chemical dosages could be upgraded by as much as 100 percent. The value of jar tests lies in the quick and inexpensive determination of parameters under which a pilot treatment plant could be tested under continuous flow conditions and more closely related to the"real life" situation. This approach can be justified for this project since the effect of the coagulated water on the performance of filters would be a prime factor in the final selection of the treatment process. Short filter-run times (due to clogging resulting from storage of excessive amounts of flocs), poses a serious problem, as increasingly large quantities of treated water have to be diverted for filter backwashing in lieu of supply to the consumers.

2. Lime Treatment

Lime was added at concentrations of up to 15 mg/l. This step was followed by 30 minute periods of aeration and settling. The colour remained stable.

3. Lime-Alum Treatment

Lime was added to provide the necessary alkalinity for the flocculation of the coagulant.

Alum was used as the coagulant to precipitate colour.

The findings of this test indicate that pH level for optimum chemical reaction should be around 8.5 ($\pm$) and that an alum dose of 7 mg/l precipitated the colour out. The pH of the treated water dropped to 6.3 which would necessitate additional carbonation as a corrosion inhibitor.

4. Lime-Ferric Sulphate Treatment

Lime was added to provide the necessary alkalinity for the flocculation of the coagulant.

Ferric-sulphate was used as the coagulant that will precipitate colour out.

The findings of this test indicate that the pH level for optimum chemical reaction should be at 8.5 ($\pm$), and that a ferric-sulphate dose of 10 mg/l precipitated the colour out. The pH of the treated water dropped to 6.0, necessitating additional carbonation as a corrosion inhibitor.

III. PILOT TREATMENT PLANT STUDIES

1. Purpose

Two processes can be considered for treatment of Bay Bulls Big Pond water, namely:

(a) Chemical coagulation followed by direct filtration or combination of clarification filtration.

Impurities in water (in the form of solids or flocs) can be removed in filters or in a combined system of clarifiers and filters. 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 it from collected impurities. Consequently, the shorter filter-run times may pose a serious problem as described above.

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. In the latter plant clarifiers are provided ahead of the filters to remove the bulk of the impurities (solids and flocs.)

(b) Ozonation with and without filters.

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 to 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.

The purpose of the pilot treatment plant studies is therefore, to test processes and define the treatment plant most economical for Bay Bulls Big Pond water, having regard for capital and operating costs and attainment of water quality standards.

2. Objectives

The primary objectives of the pilot plant is to determine the effect of treatment for colour removal on filter performance. The following basic parameters have been studied:

(a) Chemical dosage vs. filter-run time.
(b) Filter head loss build up vs. chemical and ozone addition.
(c) Relationship of colour to ozone dose.

3. Treatment Processes

Essentially two processes have been tested, as follows:

(a) Direct filtration with:
(i) Alum coagulation
(ii) Alum plus polyelectrolyte coagulation
(iii) Polyelectrolyte as a primary coagulant

(b) Ozonation with:
(i) Diffused contact columns followed by filtration
(ii) Positive pressure injector contactor followed by filtration

The above processes have been tested relative to treated water quality (effectiveness), operational parameters (economics), and by-products (e.g.sludge) that may require additional separate treatment (economics).

The requirement for chlorine to disinfect the water prior to its supply was determined in each case by analytical methods, on a sample of the treated water.

4. Pilot Plant Description

The pilot filter-plant flow schematic with chemical pre-treatment and ozone pre-treatment is shown on Figures 8.1 and 8.2, respectively.

a. Chemical Pre-treatment Pilot Plant

Starting at the head of the pilot plant, it consisted of the following major components:

(a) A 1,000 litre chemical pre-treatment tank equipped with a variable speed drive agitator.

(b) A Flotec variable flow pump with a rotameter on its discharge pipe to measure and adjust flows.

(c) A 10 inch diameter gravity filter. The filter bed above the false bottom strainers included:
- (i) 2 1/2 inch layer of gravel 3/8 inch size
- (ii) 1 1/2 inch layer of gravel No. 4 sieve size
- (iii) 8 inch layer of sand; effective size 0.35 mm, uniformity coefficient 1.5.
- (iv) 14 inch layer of anthracite coal; effective size 0.75 mm, uniformity coefficient 1.5

(d) Chemicals:
- (i) Hydrated lime, $Ca\ (OH)_2$
- (ii) Alum, $Al_2\ (SO_4)_3 . 14H_2O$
- (iii) Polyelectrolyte, Nalcolyte 607.

b. Ozone Pre-treatment Pilot Plant

Starting at the head end of the pilot plant, it consisted of the following major components:

(a) A 1,000 litre holding tank.
(b) A Flotec variable flow pump with a rotameter on its discharge pipe to measure and adjust flows.
(c) Two ozone contact columns each providing a 5 minute retention time at a flow rate of 1.0 I.G.P.M.
(d) A second 1,000 litre holding tank equipped with a Flotec variable flow pump and rotameter.
(e) A 10 inch diameter gravity filter. The filter bed above the false bottom strainers included:

(i) 2 1/2 inch layer of gravel 3/8 inch size
(ii) 1 1/2 inch layer of gravel No. 4 sieve size
(iii) 8 inch layer of sand; effective size 0.35 mm, uniformity coefficient 2.0
(iv) 14 inch layer of anthracite coal; effective size 0.75 mm, uniformity coefficient 1.5.

(f) An ozone generator (GRACE Model LG-2-L2) capable of producing from air 1 lb/day ozone @ 1% concentration by weight.

5. Pilot Plant Set-Up

The pilot plant was set up at the laboratories of the Environment Protection Service in St. John's. Operation of the pilot plant was with fresh water from Bay Bulls Big Pond hauled daily in 1,000 litre batches, and fed continuously through the treatment units.

The pilot plant unit of the positive pressure injector type ozone contactor was set up on site at Bay Bulls Big Pond.

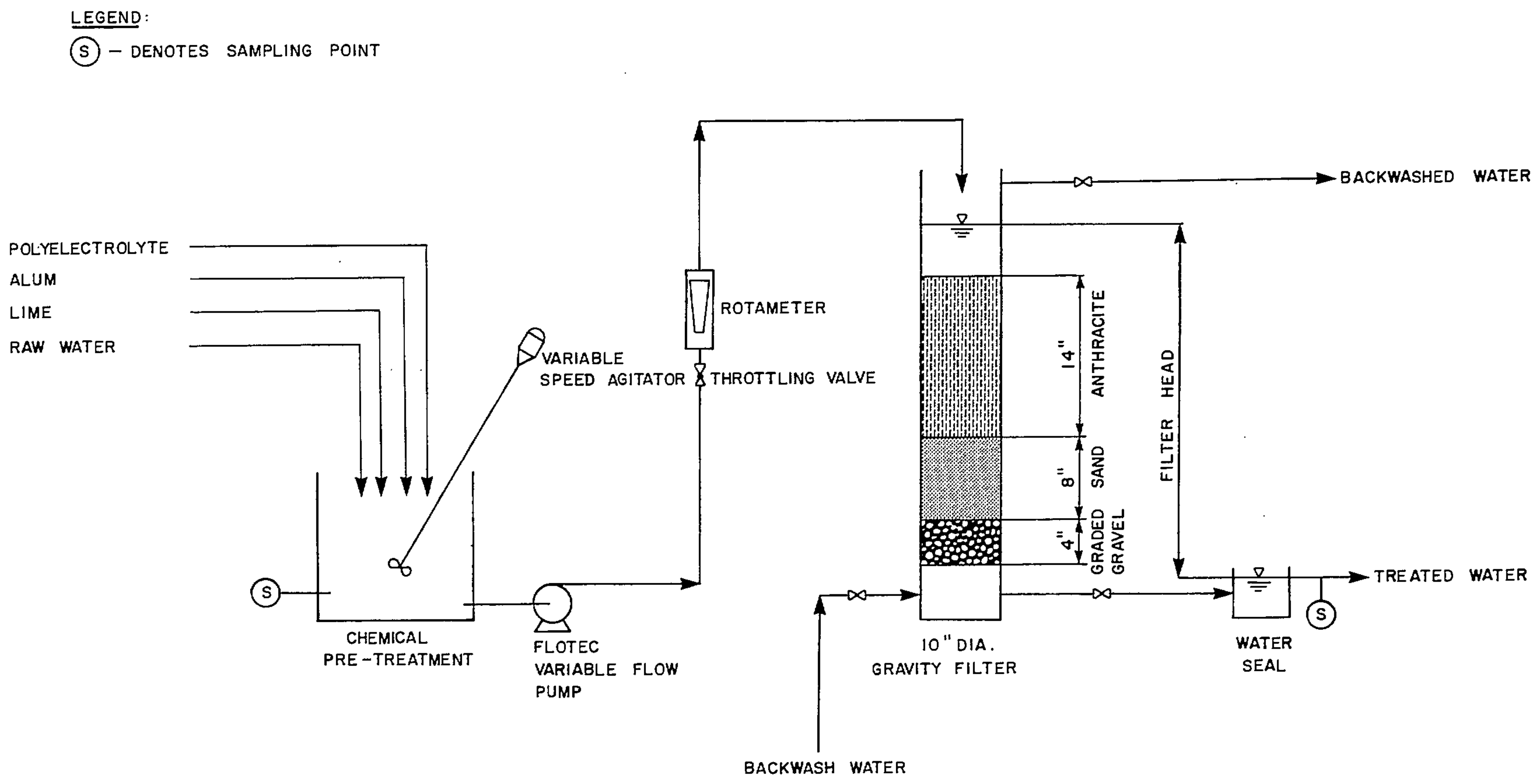

FIGURE 8.1
PILOT FILTER — PLANT FLOW SCHEMATIC
(CHEMICAL PRE — TREATMENT)

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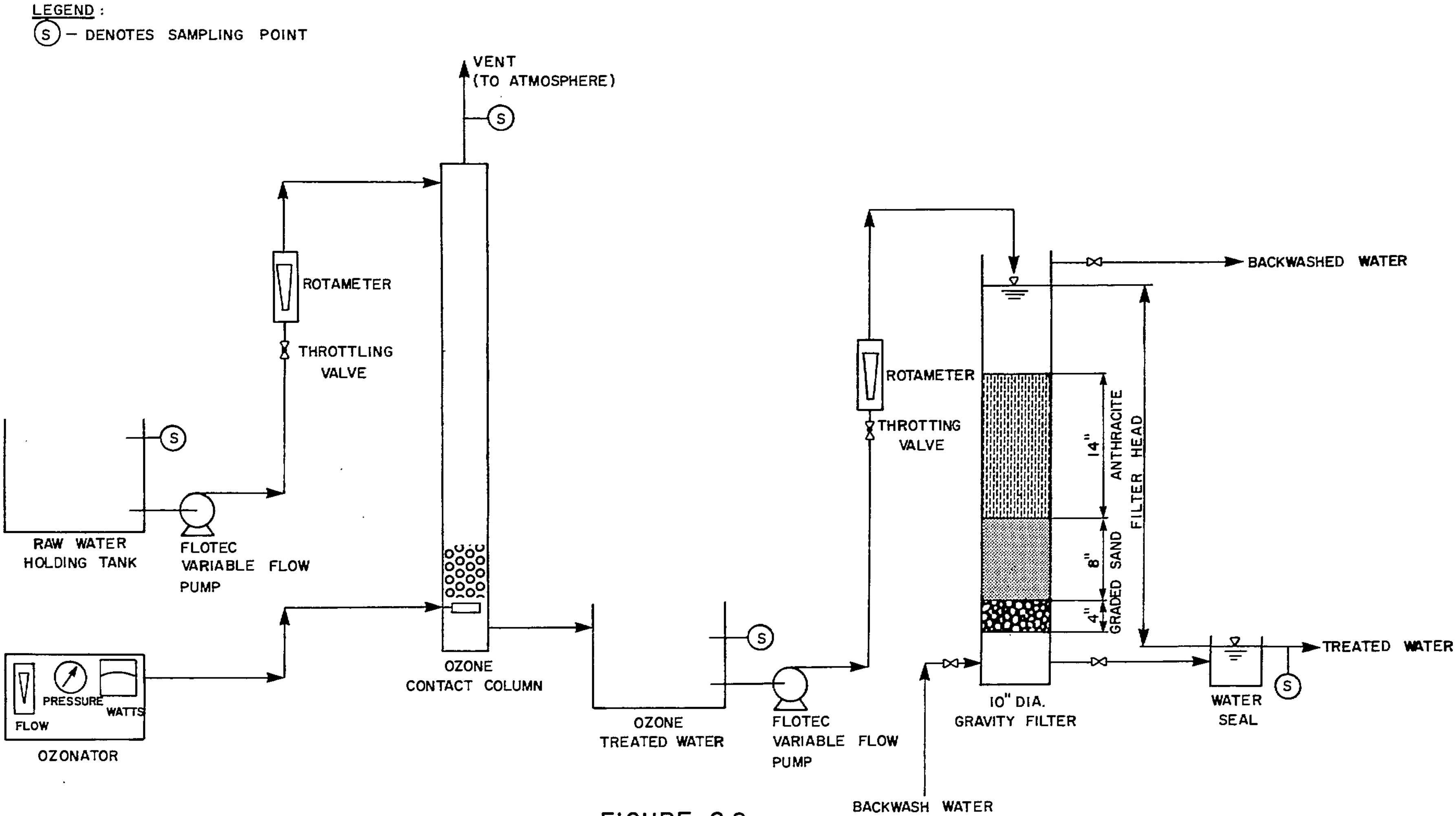

FIGURE 8.2
PILOT FILTER – PLANT FLOW SCHEMATIC
(OZONE PRE – TREATMENT)

6. Analytical Procedure

A scheme as shown in Appendix 'I' has been developed for sample collection and analysis. This scheme was applied to samples collected from points as shown in Table 8.1 and on Figures 8.1 and 8.2. Analysis was conducted in accordance with "Standard Methods", thirteenth edition, on daily composite samples collected from the batches processed during the day.

TABLE 8.1

SAMPLING SCHEDULE

Sampling Point	Treatment Process	Purpose
Holding Tank (at head end of plant)	Chemical and Ozonation	To establish the characteristics of the raw water.
After ozone Contact Column(s)	Ozonation	To establish the effectiveness of ozonation without filtration.
After Filter	Chemical and Ozonation	To establish the effectiveness of direct filtration as well as lengths of filter run (clogging effects)
Ozone Feed & Exhaust	Ozonation	Ozone determination only to establish the amount of ozone used in process.
Coagulated Raw Water and Backwashed Water	Chemical	To assess the characteristics of sludge produced.

7. Pilot Plant Study Programme

Two programmes were carried out, one with chemical pretreatment and the second with ozone.

a. Chemical Pre-Treatment

(i) Chemicals - The following chemicals were used:

(a) Hydrated Lime, Ca $(OH)_2$

(b) Alum, $Al_2 (SO_4)_3 \cdot 14 H_2O$

(c) Polyelectrolyte, Nalcolyte 607

Three procedures were carried out, one using alum only as the coagulant; the second using a combination of alum as the primary coagulant and polyelectrolyte as a coagulant aid; the third using polyelectrolyte as the primary coagulant. The purpose of these procedures has been to determine reasonable filter runs by reducing the amount (and size) of the floc.

Lime was used for pH correction both prior to and after the chemical process.

A series of runs were performed for each procedure to establish the most effective and economic chemical dosage.

(ii) Operation - Water hauled in from Bay Bulls Big Pond was pumped into the chemical pre-treatment tank. After reading the pH of the raw water, lime was added to raise same to a level of 8.0 $\pm$. Operating the mixer at high speed ensured thorough mixing of the lime. At this stage alum was added at the following dosage:

- Series 1, 5 mg/1
- Series 2, 10 mg/1
- Series 3, 15 mg/1

Operating the mixer at high speed induced coagulation; slowing down the mixer to low speed resulted in flocculation and the production of pin-point type flocs.

The flocculated raw water was now ready for application onto the filter. The filtration rate used was 1.9 U.S.G.P.M. which is equivalent to 3.8 U.S.G.P.M. per square foot of filter.

Pressure drop through the filter was recorded as a function of time.

Samples for analysis were collected in accordance with the scheme and schedule previously described.

The filtered water sample was treated with lime for pH adjustment to a value of 7.0 ($\pm$) prior to the analytical determinations.

Procedure 'B' was carried out in the same manner as the above procedure, except that the chemicals used were as follows:

- Series 1, 5 mg/1 alum and 0.5 mg/1 Nalcolyte 607
- Series 2, 10 mg/1 alum and 0.5 mg/1 Nalcolyte 607
- Series 3, 15 mg/1 alum and 0.5 mg/1 Nalcolyte 607.

Procedure 'C' was carried out in a similar manner as the above procedures except that the coagulants used were as follows:

- Series 1, 1 mg/1 Nalcolyte 607 and 5 mg/1 alum
- Series 2, 1 mg/1 Nalcolyte 607 and 10 mg/1 alum
- Series 3, 1 mg/1 Nalcolyte 607 and 15 mg/1 alum

b. Ozonation

(i) Ozone Columns - The initial intent was to use 2 diffused columns and to test the efficiency of single vs. split ozone treatment. However, early experience showed that the diffusivity of ozone in water, and subsequently its reaction rate with colour, is greatly dependent on the size of the bubbles. The finer the bubble is the more efficiently the system operates. After the diffusers had been adjusted and the system tested, it was decided that, for achievement of the best possible results, the two columns should be run in series. The entire dose of ozone was applied to the first column. Free ozone then passed to the second column for further contact with the water. Only the second column was equipped with a vent to exhaust excess ozone to the atmosphere. This exhausted ozone was sampled and analysed to determine the percent of ozone generated that actually reacted with the colour (and other oxidizable materials) in the water. Since the ozone transfer rate (from its gas state to the water) varied considerably with the dose, it was felt warranted to relate all ozone parameters to the dose that was actually transferred to the water. Furthermore, since this diffused column system proved to be inefficient, a second series of tests was carried out in a more sophisticated ozone contactor.

The ozone contactor used was a W.R. Grace & Co. unit. This contacting unit introduces ozone and water at the top of a 10 foot column through a positive pressure injector. The gas/water mixture flows co-currently from the injector down a central pipe. The mixture reverses direction at the base of the central pipe and rises up a concentric circular tank where the gas and water separate. The gas (and any foam formed) flow to a receiving tank for foam separation. Inlet gas and liquid pressure is maintained at approximately 3 psig. By varying different operating parameters of the system dosage rates can be adjusted to optimum levels.

(ii) Operation - Water hauled in from Bay Bulls Big Pond was stored in the raw water holding tank. From here it was pumped at a rate of 1.1 I.G.P.M. into the first ozone column, into which ozone was diffused in a counter flow pattern. Overflows from this column passed into the second ozone column and were then collected in a holding tank. At a rate of 1.6 I.G.P.M. (1.9 US.G.P.M.) the ozone treated water was applied onto the filter. The reasons for double pumping are the unavailability of head to utilize gravity flow and the different rates of application.

Pressure drop through the filter was recorded as a function of time.

Samples for analysis were collected in accordance with the scheme and schedule previously described.

The ozonated and filtered water samples were treated with lime for pH adjustment to a value of 7.0 $\pm$ prior to the analytical determinations.

These samples were also analysed for chlorine demand (expected to be low due to the ozonation of the water), to determine the requirements for disinfection.

Procedure 'B' utilizing the positive pressure injector type contactor for ozonation was carried out at Bay Bulls Big Pond site. Raw water was pumped directly from the pond into the contactor. After ozonation the water was collected in a holding tank for application onto the filter. Filter operation, sample collection and analysis followed the pattern described above for diffused ozonation.

IV. PILOT PLANT FINDINGS

The intensity of the raw water colour was 25 TCU (True Colour Units) during the pilot plant study.

1. Colour Removal by Chemical Pre-Treatment

Preliminary tests were carried out at a pH range of 8.0 to 8.2 and 8.5 to 8.7. Results obtained at the former pH range were somewhat superior, and subsequent tests were carried out at the pH range of 8.0 to 8.2.

A dose of 5 mg/l alum proved to be ineffective. However, when the dose was increased to 10 mg/l a marked effect on colour removal was noticed. The results, as can be seen in Figure 8.3, show a colour intensity drop from 25 to about 10 units. Further increase in alum dose, from 10 to 15 mg/l

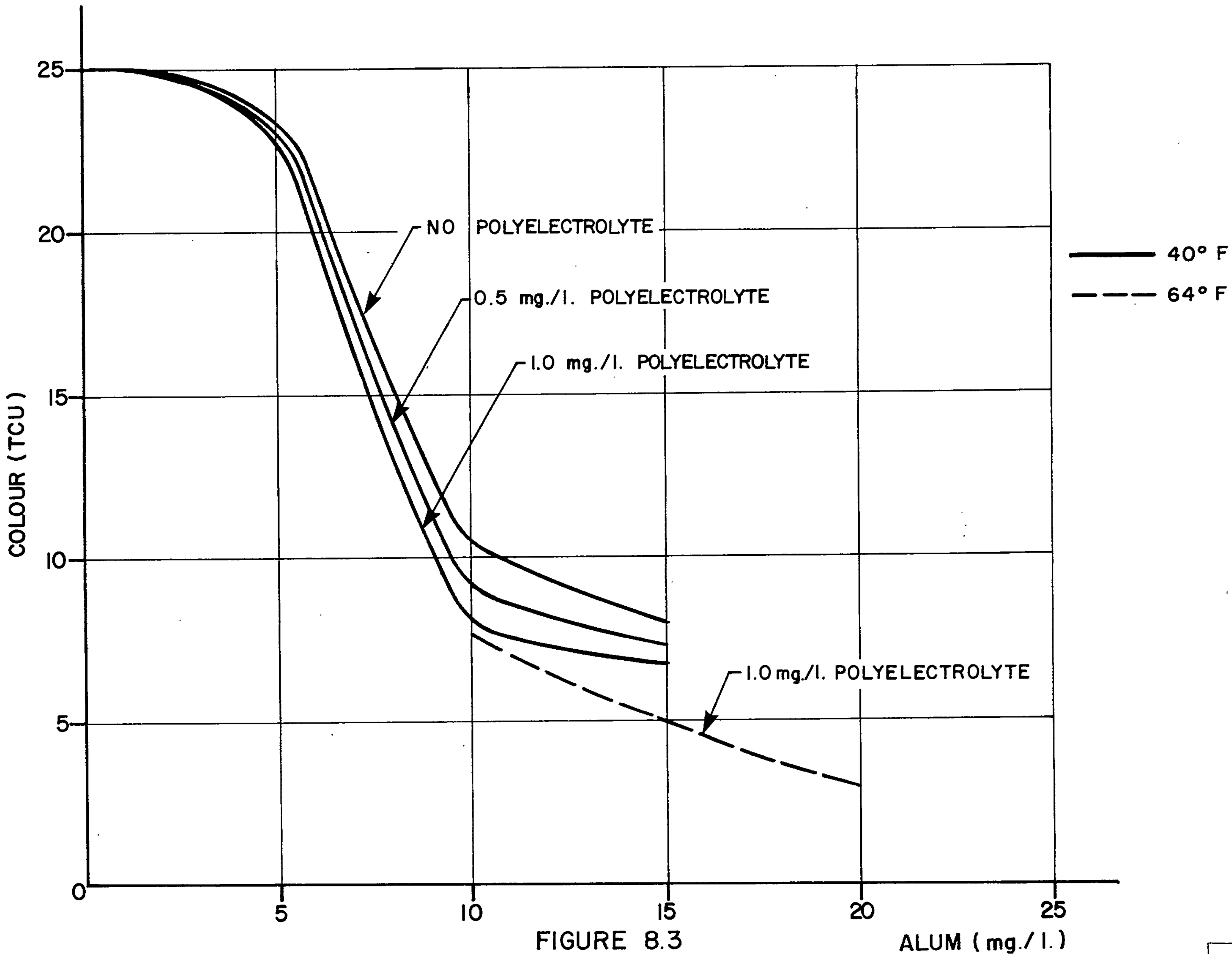

FIGURE 8.3
COLOUR vs. ALUM DOSE

has a small effect on colour removal as its intensity decreased to 8 units only.

At a head loss of about 6.5 feet breakthrough occurred resulting in a deteriorated water quality.

The addition of a polyelectrolyte (Nalcolyte 607) to the above dosages of alum, to improve colour removal and prevent breakthrough gave the following results:

(a) Colour removal was only slightly affected by the polyelectrolyte. At dosages of 0.5 mg/l and 1.0 mg/l the improvement to the alum treatment was in each case by only 1 colour unit.
(b) The polyelectrolyte was effective in preventing breakthrough.

2. Colour Removal by Ozonation

Figures 8.4 and 8.5 show the relationship of colour to ozone dose for the systems of diffused ozonation and the positive pressure injector ozonation, respectively.

In order to obtain a water quality of less than 5 colour units (to conform to the "objective" of the Canadian Drinking Water Standards and Objectives, 1968), it can be seen from Figure 8.4 that an ozone transfer dose of 4 mg/l would be required for the diffused column system. This value would be equivalent to a production dose of 7 mg/l. The comparable values in the positve pressure injector system would be 1.91 mg/l production dose, as can be seen in Figure 8.5, and 1.45 mg/l transfer dose. The contact time in the former system was 8 minutes whereas the

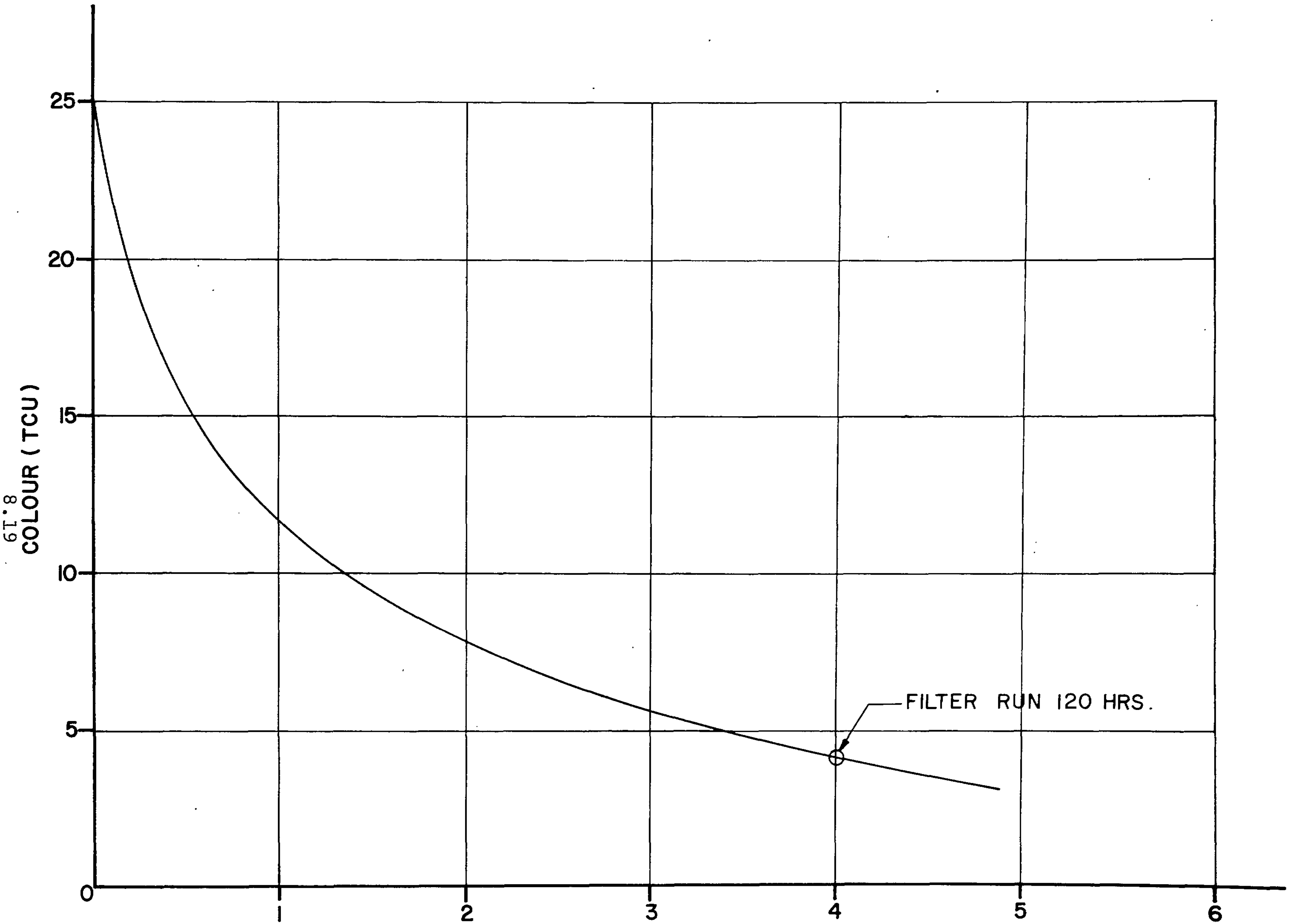

FIGURE 8.4
COLOUR vs. OZONE DOSE

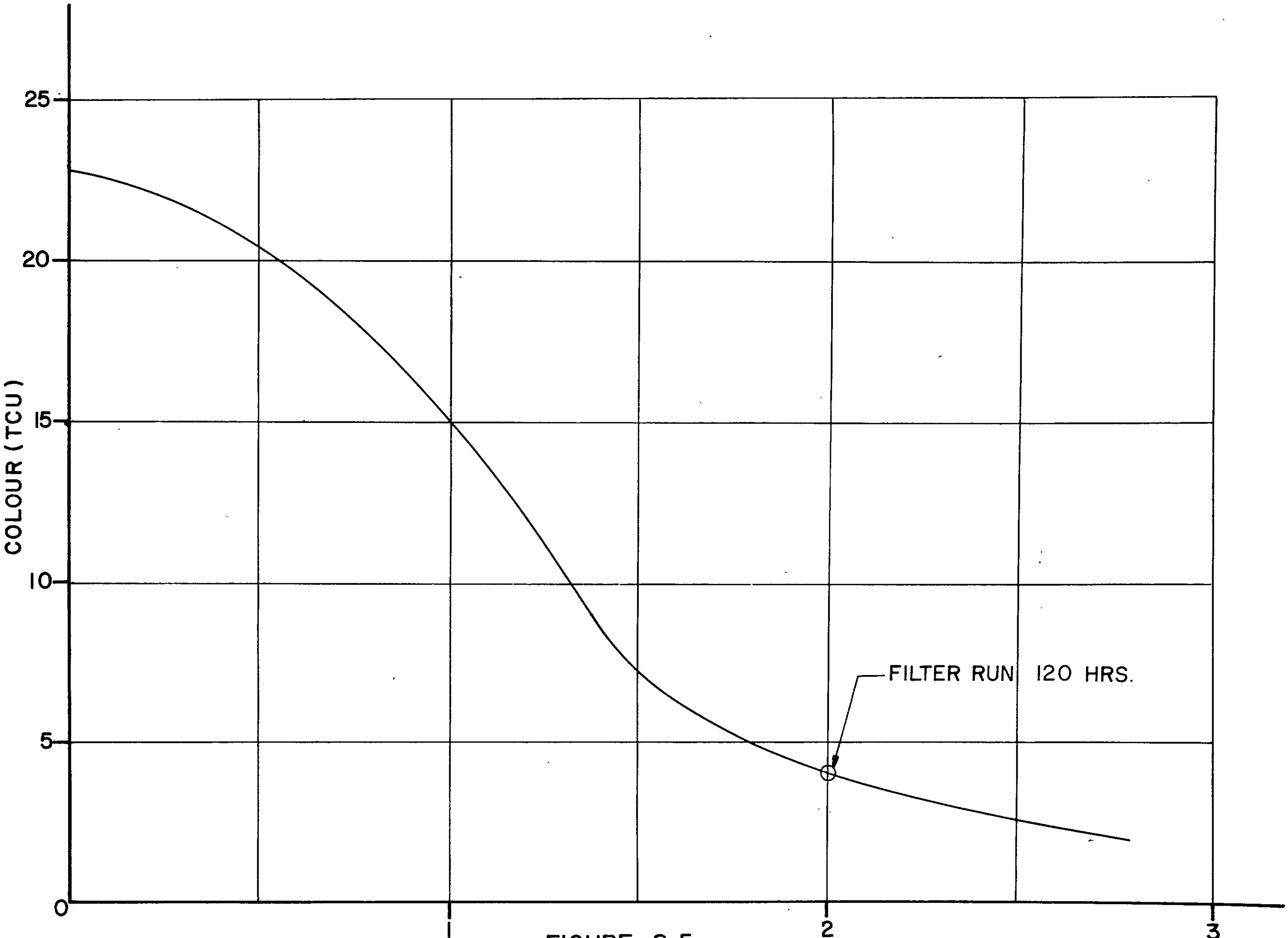

FIGURE 8.5
COLOUR vs. OZONE DOSE

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latter system provided a contact time of less than 2 minutes. The effectiveness of the positive pressure injector system is very obvious.

Using the above doses, the ozone treated water was processed through the filter. It will be of interest to note here that on a continuous basis ozonation, in both systems, did not produce consistently water of a quality less than 5 colour units. Colour intensities of 8 units were occasionally observed. In the case of the diffused column system limited improvement was noticed after filtration. However, with the positive pressure injector system, the colour intensity of the filtered water was consistently below 5 units.

3. Filter Running Time

Figure 8.6 indicates the effect of alum addition on the length of filter runs. This result agrees with most findings on floc volume being related to the coagulant dosage used and the subsequent effect on the overall length of filter runs. Figure 8.6 also shows the marked effect that polyelectrolyte addition had on the length of filter runs.

At an alum dose of 5 mg/l colour removal was negligible. Consequently, the filter run was long at 40 hours. Although the addition of polyelectrolyte did not improve colour removal, it appears that this chemical, at a dose of 0.5 mg/l, did aid reaction with other water constituents to form a floc which shortened the filter run to about 20 hours. When the polyelectrolyte dose was increased to 1.0 mg/l it probably acted to

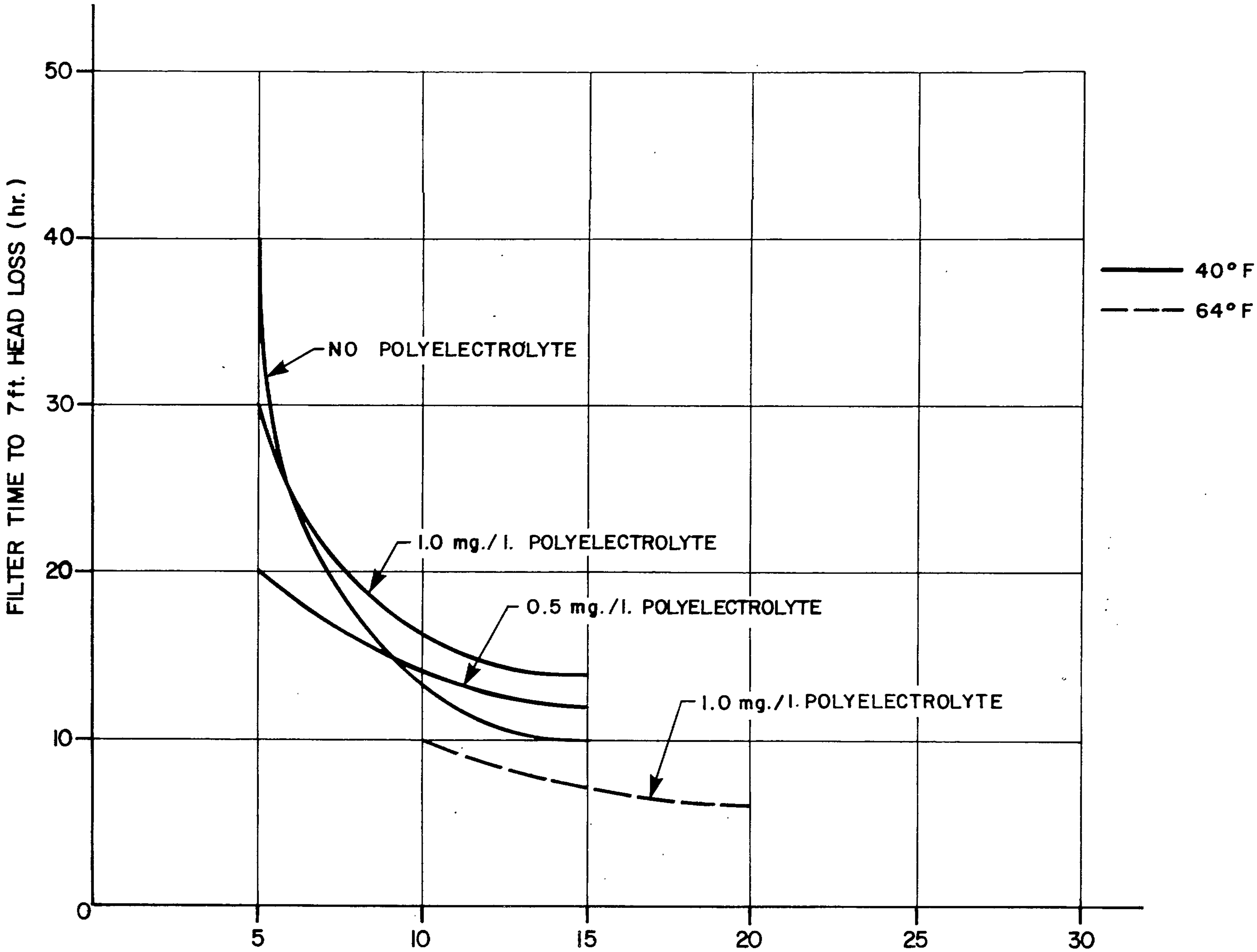

FIGURE 8.6

RELATIONSHIP OF FILTER TIME TO ALUM DOSE

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improve the floc quality and consequently lengthened the filter run to 30 hours.

An alum dose of 10 mg/l was effective in removing colour. This is clearly noted in terms of the filter run which was reduced to 12 hours. However, the addition of polyelectrolyte served to condition the floc thereby increasing the filter run, up to over 30 percent at a dose of 1.0 mg/l.

The increased alum dose to 15 mg/l further reduced the filter run to 10 hours. Again, polyelectrolyte addition served to improve the floc quality and lengthen the filter run by 20 and 40 percent with dosages of 0.5 and 1.0 mg/l, respectively.

4. Filter Media

Filter media size is normally expressed in terms of the "effective size" - the size particle which is exceeded in fineness by 10 percent of the sample, and the "uniformity coefficient" - the distribution of sizes as the ratio of the 60 percent size to the effective size. In contemporary filters using dual-medium beds, the underlying layer of sand usually has an effective size range of 0.4 to 0.5 mm. The effective size range of anthracite is normally between 0.8 and 1.4 mm.

In the case of floc that lacks the strength or toughness to resist the shear forces that occur in a filter bed, medium size, bed-thickness, filtration rate, and other similar variables exert a profound effect on the performance of the filter and the water quality. We consider this to be the case in the chemical coagulation - flocculation of colour.

Economic considerations will promote the use of high-rate filters. Therefore, the bed thickness and medium size are the two major variables that have to be tested and optimized.

Under the pilot plant conditions a fine medium size was tested. Whereas such a medium will decrease floc penetration and possible break-through, it may reduce the filter running time since it does not provide enough storage area for floc deposits. An alternative of a medium of a somewhat larger effective size with an increased bed thickness will allow deeper penetration of the floc, but, because of the deep bed may still prevent breakthrough. At the same time, the additional storage area for floc deposits would result in longer filter runs. All of these variables will be evaluated for the optimization of the filter bed medium.

5. Overall Treatment Effect

During the pilot plant study continuous analysis were carried out to determine the overall treatment effect on the natural characteristics of the water. Typical analysis results of Bay Bulls Big Pond water before and after treatment by filtration with alum-polyelectrolyte dosage and ozone dosage are given in Table 8.2.

TABLE 8.2

SUMMARY OF TYPICAL ANALYSES

Water Constituent	Raw Water	Chemically Treated Water	Ozone Treated Water
Alkalinity, as $CaCO_3$ (mg/l)	1.5	3.5	5.0
Hardness, as $CaCO_3$ (mg/l)	4.0	12.0	8.0
Iron (mg/l)	0.18	0.09	
	0.16		0.10
Manganese (mg/l)	0.04	0.04	
	0.025		0.025
pH	6.0	5.6	5.7
Coliform Organisms		Negative	Negative

Turbidity of water is often associated with disease carrying organisms. Analyses to determine the turbidity of the water showed its levels to fluctuate. At the start of the studies, during January and February, the turbidity of the raw water was equal to or less than 1.0 JTU. Since chemical treatment (carried out at that time) resulted in a low pH of the treated water (see Table 8.2 above), analyses for turbidity after pH adjustment of the water with lime to a level of 7.0± did not show a significant improvement in the level of turbidity.

During the months of April and May the turbidity of the raw water varied between 1 and 2 JTU. The turbidity recorded in June (the last month of the study) was again equal to or less than 1.0 JTU. Treatment with ozone alone usually resulted in a turbidity level of about 0.5 JTU.

V. DISCUSSION AND RECOMMENDATIONS

1. Treatment Efficiency

The two processes tested included chemical pre-treatment and ozonation, primarily for colour removal, followed by filtration for removal of residues, turbidity, algae, and coliform organisms.

The chemicals used were alum and a combination of alum and a cationic polyelectrolyte (Nalcolyte 607).

Essentially, the results of the pilot-plant study show that the two processes will produce water low in colour, turbidity, iron, manganese, algae and coliform organisms. However, whereas pre-treatment with ozone will produce water of a quality that will meet the "objective" levels of the Canadian Drinking Water Standards some 90 - 95 percent of the time (and the "acceptable" levels at all times), a plant providing for colour removal with alum will be more susceptible to fluctuation in the quality of the treated water.

Temperature is known to be one of the major factors that affect the chemical pre-treatment of water. At lower temperatures, the chemical coagulation process becomes more difficult, resulting in lower efficiencies for the same chemical dosage. This phenomenon was experienced in the study. At a water temperature of 40°F and a dosage of 15 mg/l of alum and 1.0 mg/l of polyelectrolyte (Nalcolyte 607), the colour intensity was reduced by some 70 percent to 7 units (True Colour Units). These same dosages to water at a temperature of 64°F produced a water quality of 5 (or less) units.It, therefore, could be expected that a plant providing for alum pre-treatment will produce water of a colour intensity within the "acceptable" levels during cold weather conditions, and within the "objective" levels during warm weather conditions. We consider this phenomenon important under St. John's weather conditions.

Another important factor in chemical coagulation is the pH level of the medium and its available alkalinity.

Both of these parameters, as found in Bay Bulls Big Pond water, are not favourable for chemical coagulation. There is, therefore, a definite need for additional carbonation in the form of lime application. This requirement will have to be employed twice, in two successive stages of the treatment, as follows:

(a) Prior to chemical pre-treatment.
(b) After filtration to add carbonation as a corrosion inhibitor, and to render chlorination more effective and efficient.

We consider the addition of lime after filtration (the last stage in treatment) a disadvantage as possible residues may find their way into the distribution system.

The use of a polyelectrolyte as a primary coagulant could conceivably reduce the problems encountered with alum. However, the dose used in the study did not prove to be effective, and higher dosages would make this concept of treatment too costly.

Compared to alum and chemical treatment, ozonation is less sensitive to such factors as the temperature and pH of the raw water. Direct application of ozone to the water, without any pre-conditioning of the latter, will produce good quality water. Furthermore, any addition of chemicals, such as for adjustment of pH and improvement of filter performance (during seasons of high algae concentration and turbidity) could (and would) be added ahead of the filters (and after ozonation) so that possible residues will be caught in the filter.

A summary of the requirements for alum pre-treatment and ozone pre-treatment is presented in Table 8.3

TABLE 8.3

REQUIREMENTS FOR PRE-TREATMENT

REQUIREMENT	ALUM PRE-TREATMENT	OZONE PRE-TREATMENT
Pre-Chlorination	Yes	No
Post-Chlorination	Yes	Yes (low dose)
Pre-pH Adjustment	Yes	No
Post-pH Adjustment	Yes	Yes
Temperature Effect	Yes	Limited
Polyelectrolyte Addition	Yes	Intermittently
Waste Disposal	Yes	Limited

2. Filter Running Time

As indicated previously in this Chapter, filter running time would be considered a prime factor in the final selection of the treatment process. This consideration is warranted (in our opinion) since short filter-run times (due to clogging resulting from storage of excessive amounts of residues) poses an economic burden on a treatment plant, as increasingly large quantities of treated water have to be diverted for filter backwashing in lieu of supply to the consumer.

Experience in direct filtration plants utilizing alum for coagulation - flocculation, clearly correlates the dosage of alum to the length of filter runs. Most data confirm the findings of this study that alum dosage has a very significant effect on the overall length of filter runs[1]. The length of filter runs is almost inversely proportional to the alum dose. This phenomenon is probably due to the proportional relationship between floc volume and alum dose, and to the lack of floc compressibility within the filter bed[2]. Furthermore, the phenomenon of breakthrough should also be considered at higher dosages of alum, especially in the precipitation of colour. All of the above, coupled with the results of the pilot plant studies lead to the following suggestions:

a. Pre-treatment with alum should be combined with a cationic polyelectrolyte. The advantage of using such a polyelectrolyte (as Nalcolyte 607) is two-fold:

   (i) Floc strengthening thus preventing breakthrough and ascertaining a better quality water, and

(ii) Prolonging filter-running time thus producing more water per filter run, and reducing the total quantity of water used for filter backwashing.

b. Even with the pre-treatment process described under (a) above, short filter runs could be experienced under two situations:

(i) During warm weather conditions when the chemcial pre-treatment process of coagulation - flocculation is more effective thus producing larger quantities of flocs which will clog the filter at an accelerated rate. One way to overcome this disadvantage could be by sacrificing water quality, that is, reduce the amount of chemicals and precipitate less colour.

(ii) During the algae blooming season when the concentration of algae will tend to clog the filter fast. The presence of coagulated alum flocs certainly will not help this situation. The use of a coarser filter medium and a deeper filter bed could prolong filter runs under these conditions.

c. High rate filtration plants (with chemical pre-treatment) are presently operating at a filtration rate of 4 gpm/sq. ft. However, such plants are designed for a filtration rate of 7 to 8 gpm/sq.ft. since it is conceivable that these higher rates are operationally feasible. However, shorter filter runs occur for the higher filtration rates. Again, a coarser filter medium and a deeper filter bed may help prolong filter runs. A reasonable alternative will be to use a standard filter medium (see discussion in next section), and adopt the clarification - filtration process.

Contrary to colour precipitation by chemical pre-treatment, ozone oxidizes (bleaches) colour, and any by-production of residue is kept at minimum. This explains the long filter runs, of 120 hours, experienced during the pilot plant study. Filter runs of this order of magnitude will be amenable to higher filtration rates. Shorter filter runs could be expected due to high algae concentrations, for about two months a year during the Fall.

3. Filter Media

Whereas the pilot plant studies were carried out with one type of filter media (reported on earlier in this Chapter), it was felt that experience elsewhere could be utilized to optimize the filter media for this project. Also, the intent was to consult the specialist firm of Camp Dresser and McKee (of Boston, Mass.) on the aspect of filter media as well as the treatment process as developed from the pilot plant studies.

A dual media of coal anthracite supported with a sand layer is considered for this project. Whereas, the specification for the sand layer is quite standard at an effective size of 0.45 - 0.50 mm., and a uniformity co-efficient of 1.5, the gradation of the coal anthracite layer and the thickness of the layers warranted additional investigation.

Anthracite is a carefully selected and graded coal material, classified hydraulically to contain a minimum inherent ash and mineral matter. It is then meticulously screened to yield sizes suitable for filter purposes. The standard specification for the coarsest anthracite calls for an effective size of 1.0 - 1.1 mm. and a uniformity co-efficient of 1.5. Because of the advantages of coarser filter media (as discussed earlier), it has become common practice to select this size of anthracite and use it in a layer of about 20 inches. Under special conditions, such as short filter runs and high filtration rates, a specially selected coarser anthracite would be considered in a thicker layer (because of deeper floc penetration).

It is our opinion (endorsed by Camp Dresser McKee) that for ozone as pre-treatment the above standard gradation of coarse anthracite will be satisfactory. However, for chemical (alum) pre-treatment, serious consideration should be given to the use of specially graded coarser anthracite in a thicker layer. As previously discussed, this approach is warranted in view of seasonal algae concentrations (on top of the alum floc), and the basic concept of designing the filters to enable a filtration rate higher than 4 gpm/sq.ft.

4. Residuals in the Treated Water

Ozone residual is not persistent and, therefore, could not be considered as a disinfectant in the distribution system. Accordingly, it has been decided that the treated water (in an ozonation system) will have to be chlorinated to provide a chlorine residual in the distribution system.

It should be noted here that, in the treatment of coloured water with ozone, the organic matter which causes the colour could be converted to compounds which are suitable food for bacteria and slime. Maintaining a chlorine residual in the treated water will prevent the growth of nuisance organisms in the distribution system.

Since ozone is a strong oxidant, the chlorine demand of the treated water will be for the maintenance of a chlorine residual only. In the case of chemical (alum) treatment, the chlorine demand will be for bacteria kill as well as maintaining a chlorine residual in the distribution system. Furthermore, chemical treatment will necessitate the use of pre-chlorination for algae kill during periods of high algae concentration.

The use of a polyelectrolyte (Nalcolyte 607) warrants a look into the toxicological effects of this chemical. To this end we have been advised that a dose up to 40 mg/l is considered safe for use in potable water. A document to this effect issued by the U.S. Environmental Protection Agency is contained in Appendix II.

5. Safe and Reliable Water Quality

It is self evident that the treatment process should primarily produce water of a safe and reliable quality in as much as public health is concerned.

In accordance with the requirements of the"Canadian Drinking Water Standards and Objectives, 1968" (see Chapter 4 in Volume II), filtration and disinfection are reliable methods to render the water safe in terms of total and faecal coliform organisms.

Turbidity, when kept below the level of 1 JT units is usually associated with water free from disease organisms. In this respect we have identified the water of Bay Bulls Big Pond to be very low in turbidity. With a deep intake inlet, it is to be expected that the turbidity of the raw water will be equal to or less than 1 JT units most of the time. Chemical pre-treatment will remove higher seasonal turbidity which is not expected to exceed 3 JT units. Ozone pre-treatment will have a limited effect on the removal of turbidity, and such a system will have to include a cationic polyelectrolyte (such as Nalcolyte 607) to handle excess seasonal turbidity. Experience elsewhere has shown Nalcolyte 607 to be effective in the removal of turbidity when added directly on filters.

Colour has been identified as the major "unacceptable" constituent in the raw water. Its intensity exceeds the "acceptable limit" of 15 units (Platinum-Cobalt Scale) for 75 percent of the time. Whereas there is no evidence to support any claims that colour is injurious to man, there are several disadvantages to having colour in water[3]. These are as follows:

- Aesthetics : most people prefer a clear, colourless water for home use.
- Taste : there have been claims that colour imparts a taste to water.

- Chlorine demand : the presence of colour increases the chlorine demand of a water.
- Nutrient : colour may act as a nutrient for bacteria and algae.
- Industrial requirements : many industries require low colour in process water.
- Analyses : colour interferes with colourmetric methods of analysis.
- Chelation : colour may increase the concentration of soluble iron, manganese, and lead in waters and may stabilize their presence by chelation.

The reliability and effects of colour removal by chemical or ozone pre-treatment, followed by filtration, have been discussed extensively in the preceding sections.

Iron and manganese are objectionable elements in water. Because of their avid appetite for ozone their presence can compete and interfere with the procedure for reducing colour by ozonation. However, experience elsewhere has shown the limiting concentrations of these metals which can be tolerated in a water subjected to ozonation treatment for colour reduction to be in the range from 0.2 to 0.3 mg/l for iron, and in the range from 0.05 to 0.10 mg/l for manganese. The amounts actually present in the waters of Bay Bulls Big Pond (and Windsor Lake) are below these critical levels. This is no doubt a factor that contributed to the high efficiency colour reduction, and relatively low percentage iron removal, achieved in the pilot plant studies.

It should be noted here that, although the sanitary survey of Bay Bulls Big Pond water started in July 1973 (see Chapter 5 in Volume II), the analytical data collected during the pilot plant study extended to June 1974, bringing the sampling period to one full year.

6. Cost Comparison

Assuming all other components of the treatment plant to be equal, a cost comparison between a chemical pre-treatment system and an ozone pre-treatment system has been prepared on the basis of the following assumptions:

a. Chemical Pre-Treatment System

Major components of this system include the following:

- In-line flash-mixing for chemical coagulation	$50,000
- Three parallel flocculation basins each providing a retention time of 5 minutes, including mechanical equipment	$500,000
- Pre and post lime feed facilities, complete with storage and housing	$300,000
- Pre and post chlorine feed facilities, complete with storage and housing	$150,000
- Alum feed facilities, complete with storage and housing	$100,000
- Contingencies 20%±	$200,000
Total estimated cost	$1,300,000

b. Ozone Pre-Treatment System

Major components of this system include the following:

- Ozone generators, complete including housing	$450,000
- Air preparation units, complete including housing	$250,000
- Ozone contact columns, complete including mechanical assemblies and housing	$300,000
- Post lime feed facilities, complete with storage and housing	$150,000
- Post chlorine feed facilities complete with storage and housing	$ 50,000
- Contingencies 20%±	$250,000
Total estimated cost	$1,450,000

The cost comparison presented above is slightly in favour of the chemical pre-treatment system; however, facilities for handling of waste sludge from such a system (which eventually would be required), as well as additional costs to the filters (as discussed above), or possibly the inclusion of a solids contact tank in the system (ahead of the filters, thus facilitating to a large extent operation) have not been accounted for.

When comparing the annual operation costs (of Chemicals only) of both systems, a advantage can be seen in favour of the ozone pre-treatment system.

c. Annual Operating Cost

The annual operating cost presented in Table 8.4 has been based on the following assumptions:

i) Average daily water production of 11 and 16 MGD (corresponding to First and Design Stage flows).

(ii) Alum feed rate of 15 mg/l @ $100 per ton.

(iii) Ozone dose of 2 mg/l; power consumption of 11.5 kwhr per lb. of ozone produced @ 2 cents per kwhr.

(iv) Lime feed rate for the chemical treatment system and the ozone treatment system is 15 mg/l and 9 mg/l, respectively @ $60 per ton.

(v) Chlorine feed rate for the chemical treatment system is 3 mg/l @ 8 cents per lb. Chlorine feed rate for the ozone treatment system is 0.5 mg/l @ 17 cents per lb.

(vi) Polyelectrolyte feed rate of 1 mg/l @ $.80 per lb. Continuous feed (365 treatment days per year) for the chemical treatment system; 200 days per year feed in the ozone treatment system.

TABLE 8.4

SUMMARY OF ANNUAL OPERATING COST

EXPENDITURE	CHEMICAL PRE-TREATMENT		OZONE PRE-TREATMENT	
	11 MGD	16 MGD	11 MGD	16 MGD
- Alum	$31,000	$45,000	-	-
- Ozone	-	-	$20,500	$30,000
- Lime	$17,000	$25,000	$10,000	$15,000
- Chlorine	$10,000	$15,000	$ 3,500	$ 5,000
- Polyelectrolyte	$31,000	$45,000	$17,000	$25,000
Total annual operating cost (chemicals)	$89,000	$130,000	$51,000	$75,000

In the cost comparison summarized in Table 8.4 the cost of water for more frequent backwashing of filters in the chemical pre-treatment system had been neglected. Assuming a non-conservative frequency of backwashing equalling to twice that of the ozone pre-treatment system, at 0.5 MGD and a cost of 5 cent per 1,000 gallons, the extra cost for filter backwashing in the chemical treatment system will amount to $10,000 per year. A more realistic figure will be twice this amount of money. Also, in a chemical pre-treatment system, allowance should be made for the annual handling of sludge.

7. Recommendations

Based on the discussion presented in the previous six sections, and consultation with the specialist firm of Camp Dresser & McKee (of Boston, Mass. see Appendix III), the following water treatment works are recommended for Bay Bulls Big Pond:

a. Intake Works

The intake works will comprise an intake crib, a conduit 48 inch in diameter and a low-lift pumping station equipped with travelling screens.

The intake crib will have a minimum submergence of 10 feet at a pond minimum water elevation of 380 feet. Water collected in the crib will flow by gravity via the 48 inch diameter conduit into the suction well of the low-lift pumping station. After receiving preliminary treatment through the travelling screens, the water will be pumped into the treatment plant complex.

b. Treatment Plant Complex

(i) Ozone Pre-Treatment

Raw water conveyed from the low-lift pumping station will be discharged directly into the ozone contact columns. Ozone applied to these columns will react with the raw water to remove colour, iron, manganese, taste and odour. Ozone will also produce disinfection.

(ii) Filter Influent Channel

Ozonated water will discharge into a filter influent channel where chemicals will be added as required by the quality of the water.

Lime will be applied continuously to add carbonation to the water. A cationic polyelectrolyte (Nalcolyte 607) will be added intermittently, as a filter aid, to improve filter performance in the removal of algae and turbidity during those periods of time when the concentration of these constituents will be found to be high.

(iii) High Rate Filters

Final removal of residues, algae, turbidity, coliform organisms will take place in a bank of high-rate, dual-media filters. The filter media will comprise a 10 inch layer of sand with an effective size of 0.45 - 0.5 mm., and a uniformity co-efficient of 1.5, underlying an anthracite layer 22 inch thick with an effective size of 1.0 - 1.1 mm., and a uniformity co-efficient of 1.5. Initial filtration rate will be 4 gpm/sq.ft. The hydraulic design will allow for an increase in this rate by 50% to 6 gpm/sq.ft.

(iv) Chlorination

Chlorine (and probably ammonia) will be added to the filtered water to maintain a combined available chlorine residual in the distribution system.

(v) Chemical Feed System

As indicated above, the chemical feed system will comprise lime and Nalcolyte 607).

c. High-Lift Pumping Station

The high-lift pumping station, which will supply water to the regional system, will be incorporated in the treatment plant complex. It will contain sufficient capacity to satisfy the water needs of the regional system plus pumps for backwashing of the filters.

VI. BAY BULLS BIG POND TREATMENT WORKS

1. Intake Works

a. Design Considerations

Newfoundland Light and Power Company operates a gravity intake at Bay Bulls Big Pond. This intake comprises a coarse bar screen, a concrete conduit through the dam, and gates. The utilization of these intake works (with any required modification) for the St. John's Regional Water System was discussed with the Power Company and found to be feasible. Subsequently, careful and extensive observations were carried out to determine the physical condition of the intake conduit. These showed the downstream section of the conduit, in particular, to be in poor condition. Extensive remedial work under difficult and uncertain conditions would be required to restore the conduit to a satisfactory condition. In addition, concentrated leakage observed through the dam in at least two places indicate the possibility of restoration works to the dam, regardless of improvements to the conduit.

A second alternative for an intake will be new works comprising a crib, an intake pipe, and a low-lift pumping station. A conceptual evaluation of the two systems including a comparison of their relative merits and disadvantages was, therefore, carried out.

b. Gravity Intake System

The Gravity Intake System (utilizing the conduit through the dam as part of the intake) requires the downstream portion of the conduit to be re-lined and an outlet regulating valve or gate to be installed for the control of water flow to the stream, to satisfy power generation and fish life requirements. Extending from this outlet would be a 48 inch diameter gravity pipeline, some 7,000 feet long to the proposed treatment plant site. This pipe would be buried in the flood plain and may very likely involve considerable excavation in shale (the flood channel has cut its way into shale in places.) Underwater construction will certainly be involved for the first 500 feet or so. At the treatment plant end, a combination surge tank and travelling screen well is visualized. A superstructure on top of this tank would house the travelling screen drive and electrical. The access road to the treatment plant would be approximately one half mile long from the existing access road, which needs to be upgraded all the way to the existing dam. An access road to the downstream side of the dam will also be required.

c. Pumped Intake System

The Pumped Intake System will consist of a new intake in a deeper section of the pond, a suction for vertical turbine pumps and travelling screens located inshore of the Low Water Level shoreline, and the treatment plant located off the shoreline on high ground. There would be a 50 foot wide causeway above the High Water Level from the shore to the pumping station. A superstructure above the suction well will house the motors and electrical. The water will be pumped through a 42 inch diameter buried forcemain to the proposed treatment plant site. The length of the 42 inch diameter forcemain from the treatment plant to St. John's would be increased by about 7,000 feet so that the total pipe length in the two systems will remain the same.

d. Relative Advantages and Disadvantages of a Gravity Intake System and a Pumped Intake System

i. Gravity Intake System:

(a) Advantages:
- Does not require pumping.
- Utilizes existing equipment.

(b) Disadvantages:
- Construction would be in existing dam which is owned by others.
- Leaks past the curtain wall (observed in two places) shed suspicion as to the stability of the dam.
- In case of dam washout, the intake would disappear also. The resulting floodwave might also cause damage to the treatment plant, which would be located downstream on or near the flood plain. In such an eventuality the water supply would be interrupted for a considerable time, until reconstruction is completed.

- Because of the relatively shallow inlet to the dam conduit, the water supply for the most time would be surface water, which is not the recommended practice for municipal water supply.

- Since the system would have to be designed to operate at the lowest reservoir level, the potential head at higher water levels would be wasted.

- Because of the relatively shallow intake, the drawdown would be limited to some 20 feet.

- The dam is presently owned by others. Any problems occurring during or subsequent to construction would automatically be blamed on the contractor.

- One half mile of new access road would be required to a treatment plant site in a relatively poor location.

- Close and frequent monitoring of the condition of the dam and appurtenances would be required.

- Because of the nature of the works estimates of construction cost would be uncertain.

ii. <u>Pumped Intake System</u>

(a) Advantages:

- Accurate cost estimate is possible due to the nature of the construction.

- Water, taken from 30 to 40 feet below the surface would ensure the withdrawal of better quality water.

- The dam would not be touched during construction, therefore no unforeseen liabilities should arise.

- The water level in the Pond via the existing dam conduit could be lowered to a low level to permit construction of the intake suction well (and probably the causeway) in the dry, thus saving in construction costs.

- In the case of dam washout, the water level would be lowered by about 20 feet from normal. An additional drawdown of some 10 feet would still be possible, resulting in the water supply not being interrupted. In addition, the security of the treatment plant would not be threatened.

(b) Disadvantages:

- Operation and maintenance costs of the pumps.

e. Recommendations

Based on the above presentation it has been recommended that a pumped intake system be adopted.

It is also strongly recommended that an early investigation be carried out to establish the stability of the Bay Bulls Big Pond dam. This investigation should comprise the following works (which we consider to be the minimum required):

(i) Three boreholes at the middle cross section of the dam.

(ii) Two boreholes one on each side of the above cross section.

(iii) One borehole beyond the toe of the dam (to establish the phreatic line).

Of the above six boreholes, one should be to 25 feet into rock, one to 50 feet into rock, and the remaining four to rock surface. Packer testing should be included to establish seepage to rock.

In concept, the proposed inlet works will be as follows:

i. <u>Intake Conduit</u>

To minimize the length of the intake conduit and, at the same time, to achieve a depth of water over the inlet in excess of 15 feet at times of minimum pond level conditions during drought periods, it is recommended that the intake be located some 1,000 feet east of the dam. A causeway, some 100 feet along will extend from the shoreline into the pond where the low lift pumping station will be located. The length of the intake conduit extending into the pond from the low lift pumping station will be approximately 300 feet.

In sizing the intake conduit consideration was given to the fact that the major expenditure in providing a new intake is the cost of the underwater construction rather than the cost of the pipe itself. It has therefore been recommended that the intake conduit be sized for the reliable yield of Bay Bulls Big Pond, estimated at 23 MGD (see Chapter 5, Volume II). The equivalent maximum design flow will be 36 MGD. On the basis of the foregoing and preliminary hydraulic analysis, a 48 inch diameter intake conduit is recommended. For this size of intake,either steel pipe lined with coal-tar enamel or epoxy; reinforced concrete pipe; or polyethylene pipe could be considered. On the basis of economics of pipe joints and pipe laying methods under water, the polyethylene pipe is recommended.

ii. Intake Inlet

It is recommended that the intake inlet (at the down-stream end of the intake conduit) be designed to limit inlet velocities to 0.50 feet per second. The inlet structure will have clear openings of approximately 3 inches. It can be fabricated from steel plate. In this case, the use of a durable taste and odour free coating, such as coal-tar epoxy, is recommended. A cathodic protection system should also be considered in the final design stage.

The course (bar) screen at the inlet should preferably be fabricated of a material having a low Thermal con-ductivity (to discourage the formation of frazil ice). Fibreglass reinforced plastic has this property together with light colour and a smooth surface texture which also tend to inhibit the formation of frazil ice. Additionally, this material is chemically inert and has good structural properties.

iii. Low Lift Pumping Station

Water from the intake conduit (48 inch diameter) will flow via travelling screens to the suction well section of the low lift pumping station.

The general design of the travelling screens and low lift pumping station will be such that no major construction will be required to accommodate all flows up to and including 36 MGD which is the maximum daily demand flow equivalent to the reliable yield of Bay Bulls Big Pond.

The design minimum low water level for the pond will be elevation 380 feet.

(a) Travelling Screens

Two dual flow travelling band screens sized for a flow of 36 MGD (18 MGD each) are recommended. They will be built into two separate channels that can be isolated from each other for the purpose of bypassing any one screen.

To minimize the chances of small debris (and the larger stringy algae) being forced through the screens, the following design criteria should be considered during the detailed design stage:

- Screening media to have a square aperture size of 0.25 inch.
- Basket design to be based on a maximum permissible velocity of 2 feet per second through the media apertures when 30 percent clogged and at a flow of 18 MGD and a water level in the pond at elevation 385 feet.

Washing of the screens will be automatically actuated by the time switch with provisions for high differential head loss alarm. Screen washwater will be disposed of via a local sump pump to the treatment plant main yard drain.

The power requirement for each screen will be approximately 2 HP.

(b) Suction Well

Particular attention must be given to the suction well geometry to prevent the formation of vortices above the pumps suction bellmouths. Implicit in the geometric design is the provision of adequate submergence of the pumps to satisfy vortex prevention and N.P.S.H. requirements. The suction well design should therefore be in accordance with the latest "Standards of the Hydraulic Institute" and recommendations of pump manufacturers.

The suction well would preferably be divided into two cells with a gated interconnection. Each cell will receive the effluent from a single screen (18 MGD), and have space for pumps to accommodate this ultimate maximum flow. The two cell construction will greatly facilitate inspection and maintenance of the suction well.

(c) Pump Room

The operating floor elevation above the pump suction well will be set by the allowance for up-surge due to rejection of the full pumping capacity at times of maximum pond level. The minimum water level in the pump suction well will be equal to the design minimum low pond level, less the total losses between the inlet and the suction well, at times of maximum discharge into the plant.

The pumps selected will be of the vertical turbine type.

The initial installed low lift pump capacity will be equal to the recommended first stage treatment plant capacity of 16 MGD plus one stand-by pump. Space will be provided, however, for the total number of pumps to allow for future expansion to the intake capacity of 36 MGD.

In concept, the installation of constant speed vertical turbine pumps will have the capacities as shown in Table 8.5.

TABLE 8.5

PROPOSED LOW LIFT PUMP ARRANGEMENT

Plant Capacity	Duty-Pumps No.	Stand-by Pumps No.	Pump Capacity MGD	Pump HP (Approximates)
16 MGD	3	1	6	200
24 MGD	4	1	6	200
36 MGD	4	1	9	300

The output of the pump will be set-up manually and controlled by the water level in the upper 12 inch band of the clearwell.

2. Treatment Works - Process and Flows

The treatment process recommended for Bay Bulls Big Pond water is direct filtration preceded by ozonation for colour removal, and a polyelectrolyte (Nalcolyte 607) feed system for seasonal application to remove algae and turbidity on the filters. Lime will be used to correct the pH of the water and to add carbonation to the water. Disinfection will be provided by the ozone. Post chlorination and ammoniation will be added to maintain a combined chlorine residual in the distribution system, and to inhibit the growth of any slimes. Provisions will be included for pre-chlorination and the addition of essential chemicals to by-pass lines in all cases of emergency situations.

The process flow and controls diagram is shown on Drawing 8.1.

Design and construction of the treatment works at Bay Bulls Big Pond is envisaged to be undertaken in three stages.

The projected stage construction and design years are as shown in Table 8.6 below.

TABLE 8.6

PROJECTED STAGED CONSTRUCTION

Stage	Construction Year	Design Year	Design Capacity
First	1975	1985	16 MGD
Design	1985	1995	24 MGD
Ultimate	1995	reliable yield of B.B.B.P.	36 MGD

The projected flows for the various construction stages are as shown in Table 8.7 below.

TABLE 8.7

PROJECTED FLOWS - B.B.B.P. SUPPLY SYSTEM

Stage	Min. Hourly	Avg. Daily	Max. Daily	Peak Hourly
First	8	11	16	20
Design	11	16.5	24	30.5
Ultimate	-	24	36	45
Initial	3.5	5	7.5	9.2

3. Treatment Plant Components

(a) Exterior to Plant - Low lift pumping station complete with intake, causeway and interconnecting pipelines.
- Yard piping, drainage, grading and access road.

(b) Area A - Filters and pipe gallery
- Clearwell
- High lift and filter backwash pumping station
- Workshop

(c) Area B - Ammoniation room
- Chlorination room
- Lime feed and storage room
- Polyelectrolyte room
- Mechanical room
- Electrical room
- Space for fluoridation

(d) Area C - Ozonator and contact tanks area

(e) Area D - Administration and Control - Offices
- Laboratory
- Main control panel
- Washrooms, lockers and lunch room

A preliminary general site plan, floor plans and sections, and elevations of the proposed treatment plant are shown on Drawings 8.2, 8.3, and 8.4, respectively.

4. Ozonation

Ozonation equipment includes three major components:

- Air preparation units
- Ozone generation units
- Ozone contact units

a. Air Preparation Units

This equipment includes air compressors, dryers and coolers which provide clean, cool and dry air (-40^{o} dewpoint) for feed to the ozone generators.

A minimum of two duty units should be considered for the First Stage with a third unit providing stand-by service. Floor space should be provided to accommodate duty and stand-by units for the Design Stage. The air preparation unit should be sized to supply the ozone dosages as outlined in sub-section 4.b below. The use of heavy duty compressors, such as Nash (which is not standard with all suppliers) is recommended.

b. Ozone Generators

Ozone is generated by passing clean and very dry feed-gas containing oxygen through the corona of a relatively high voltage discharge.

In selecting the ozone generators special attention should be given to the following three factors:

(i) Voltage and Frequency -
The efficiency of ozone generating is directly proportioned to voltage and frequency. Some suppliers offer high voltage-low frequency generators, while others provide low voltage-high frequency units. Of the two the latter generator is recommended for consideration. High voltage is considered detrimental to the dialectric glass.

(ii) Power Input -
Since electric power is used to produce ozone, special attention should be given to the criterion of kwhr. requirement for the production of 1 lb. of ozone per day.

(iii) Cooling -
A substantial portion of the power input is converted to heat. Generators that provide cooling of both sides of the dialectric elements should, therefore, receive preference. In this regard the following should be added:

- Dialectric elements could be glass-tubes or glass-plates. Both of these elements would be acceptable as long as the cooling requirement is met.

- Cooling could be provided by air or water. Whereas we consider air cooling to have a slight advantage, this factor should not be the sole criterion in the final selection of the ozonators.

The ozone generators, for both First Stage and Design Stage should be sized to provide a dosage of 3 mg/l with the duty units in service and 4 mg/l with all units (including stand-by) in service. The number of units will depend on the proven experience with the largest unit of the make selected.

Floor space should be provided to accommodate duty and stand-by units for the Design Stage.

c. Ozone Contact Units

Ozone is applied to the water in specially designed contact units. The effective transfer and reaction of ozone with the water has been a process parameter requiring improvement. Essentially, three contact methods are available for consideration. These methods are:

(i) Diffusers - This is probably the most conventional method. The gas bubbles produced by the finest diffusers are still to be considered coarse for optimum efficiency. Consequently, this method is dependent on residence time. It is carried out in a gravity counter-flow contact tank with residence time of about 10 minutes.

(ii) Non-Pressure Injectors - in this system water and ozone are applied into contact columns, the former by gravity and the latter by vacuum. The gas bubbles produced are very fine resulting in a reasonably efficient contact system. Residence time is usually about 5 minutes.

(iii) Positive Pressure Injectors - this is probably the latest development in contacting systems. Both the water and ozone are applied into contact columns under pressure and a controlled ratio of gas to water flow. The mixing and contacting is so intense that in lieu of bubbles froth is produced. The system is very efficient for time non-dependent reactions (such as disinfection and colour reduction). Residence time is usually 2 minutes or less.

It is recommended that the positive pressure contact columns be considered. These columns are available in 1 USmgd nominal capacity modules and they have a reasonably wide flow tolerance. Utilizing the foregoing characteristics of the system, it is envisaged that four banks each containing 5 column modules could be sized and designed to handle both the First Stage and Design Stage flows. Since in the contacting process oxygen is saturated in the water, attention should be given in the detailed design stage to the possible release of this saturated oxygen in a gas form downstream of the contact unit.

The First Stage power requirement will be about 360 HP with an additional 130 HP for stand-by. The total power for the Design Stage will be about 490 HP and 130 HP, respectively.

5. Filters

a. Filter Design

For both the First Stage and Design Stage flows the filters will be designed for a filtration rate of 4 Igpm per square foot. It is envisaged that beyond the Design Stage the filters would operate at a filtration rate of 6 Igpm per square foot. All conduits, pipes and other components of the system will, therefore, be designed hydraulically for flow equivalent to a filtration rate of 6 Igpm per square foot.

The filter size considered at this time is about 700 square feet each. At 700 square feet, each filter will handle a daily flow of 4 million gallons. The dimension of a filter bed will be 20 feet by 35 feet. A total of 4 beds will be used for the First Stage, and 6 beds for the Design Stage. Thereafter, it is envisaged that each of the 6 filters will handle a daily flow of 6 million gallons.

The filters will be arranged in a single bank with a common inlet channel running adjacent to them. Provisions will be included for the complete bypass of the filters.

The filter bed will consist of 10 inches of silica sand with an effective size of between 0.45 and 0.50 mm and a uniformity co-efficient not greater than 1.5, underlying a layer of a minimum thickness of 20 inches of anthracite coal with an effective size between 1.0 and 1.1 mm and a uniformity co-efficient not greater than 1.5. This bed will be supported by a graded gravel layer, overlying a Leopold type or equal underdrain system.

It will probably be economical to provide the concrete works for all 6 filter beds in the First Stage construction but fully equip only 4 of these beds.

Chemically treated water will be applied through the filter inlet gate to a transverse gullet from which it will be distributed to the filter media through 4 or 5 longitudinal washwater troughs. Filtered water will be collected in a front end collection conduit (underlying the Leopold bottom) and directed through the filtered water piping and rate controller to the water seal and into the clearwell.

b. <u>Filter Washing</u>

The removal from the filter bed of all the foreign material collected in it during the preceding filter run will be accomplished by surface washing and backwashing. Source of backwash water will be through a backwash pump located in the High Lift Pumping Station.

The backwashing system will be designed to provide a maximum rate of flow of 20 Igpm per square foot. Backwash water will be applied to the filter's front end conduit and will upflow through the filter bed to the longitudinal troughs. These troughs will discharge to the transverse gullet and through a valve will drain into the washwater conduit. Valves controlling the filter and backwash operation will be motorized and automatically operated.

Surface washing will be accomplished by a rotary sweep of the Palmer or Leopold type. The system will be designed to provide a flow of 0.5 Igpm per square foot at a minimum pressure of 60 psi. Source of surface washwater could be the high lift pumps via a pressure regulator, a siphon beaker and check valve.

6. Chemical Feed System

a. Lime

Lime is to be added to the ozonated water prior to filtration in order to adjust the pH. The lime feed area will extend over two floor levels and will include the following equipment:

(i) Upper Floor Level - a loading dock, a lime bag storage area, a bag loading hopper with a dust collector.

(ii) Lower Floor Level - a 100 cubic foot lime storage hopper, a feeder, a 500 gallon lime slurry tank with an agitator, 2 lime metering pumps.

The lime system components will be sized for a dosage of 10 mg/l and the Design Stage flows. Storage area should accommodate the Ultimate Stage lime requirements.

Power requirements will be about 1 1/2 HP.

b. Polyelectrolyte

A small storage area and full strength polyelectrolyte feed system will be designed to handle the seasonal loads of high turbidity and algae. The polyelectrolyte will be Nalcolyte 607 or equal, fed directly as a filter aid after the lime feed. Equipment sizing will be for a dose of 1 mg/l. Power requirement will be 1/4 HP.

c. Fluoride

Floor space will be provided for the inclusion of a fluoride feed system at any future date.

The chemical to be used will be Hydrofluosilicic Acid which is a very corrosive liquid. Construction material of equipment to handle the liquid will have to be resistant to this acid. In concept the equipment will include a 6,000 gallon storage tank, a transfer pump, a day tank, and a metering pump. Sizing of equipment will be for a dosage of 2 mg/l free fluoride and the Design Stage flow.

7. Disinfection

a. Disinfection Process

Strong oxidants are potential agents for disinfection of water, control of taste and odour, and kill of algae. Facilities for disinfection should be sufficient not only to kill pathogens but to kill inactive enteric viruses.

Treatment with ozone, a very strong oxidant, yields a zero chlorine demand so that only post chlorination is required to leave a combined chlorine residual of 0.3 - 0.5 mg/l in the distribution system. As previously mentioned, the kinetics of disinfection with ozone is basically not a time dependent reaction (as compared with chlorine).

Provisions will be included for pre-chlorination in emergency situations when ozonation is out of service.

b. Ammoniation

In order to produce a sustained chlorine residual in the long mains, chloramine residual should be considered. From tests on the water, it was concluded that ammonia will have to be fed to the water prior to chlorination, at a dosage equal to about 1/3 of the chlorination rate (0.5 mg/l).

The ammoniation area will include one duty ammoniator, one stand-by ammoniator, a 2 cylinder scale (suitable for 150 lbs. cylinders), a manifold suitable for a connection of 10 cylinders and storage space for one month. The ammoniator will be a 240 lbs. per 24 hours unit. The ammonia will be fed to the water after filtration as it enters the clearwell.

c. Chlorination

The chlorination equipment will basically be similar to the ammoniation equipment, except for the chlorinators.

The chlorinators (one duty and one stand-by) will have a maximum feed rate of 2,000 lbs. per 24 hours, which would be adequate to meet pre and post chlorination requirements for ultimate conditions. The injector and rotameter to be installed initially will have a capacity of 500 lbs. per 24 hours.

The chlorinators and ammoniators will be of the solution feed vacuum type, suitable for automatic proportioning in relation to the influent flow rate, and with manual control of dosage.

The scales, for the ammonia and chlorine cylinders, will be equipped with a loss of weight recorder with low alarm contact.

Both areas will be equipped with gas detection and alarm equipment.

The clearwell will have a capacity in the order of magnitude of 2 million gallons. At full capacity and the Design Stage flow of 24 MGD, the retention time provided will be 2 hours.

8. High Lift Pumping Station

As mentioned earlier in Chapter 7, a detailed hydraulic analysis of the pumping system will be required during the detailed design stage. The analysis may indicate the necessity and/or advantages to be gained in simplifying the system by opening it up at Ruby Line and providing a pump station to deliver the water to St. John's and New Town.

a. Wet Well

The high lift pumps wet well will be an extension of the filter clearwell. It will be built for the Design Stage when the maximum pumping capacity will be 24 MIgpd.

b. Pump Type

The pumps will be of the vertical turbine type. They will be installed according to the various construction stages, but at any stage, sized for the maximum daily flow and include stand-by capacity. Pump units will probably correspond to the ones selected for the Low Lift Pumping Station.

Approximate power requirements for the Design Stage will be 2,000 HP.

c. Washwater Pumping Capacity

There will be two filter washwater pumps, each having a capacity of 20 MIgpd. One pump will be used for applying washwater at a rate of 20 Igpm per square foot of filter, while the other serves as a stand-by. The pumps will be of the mixed-flow turbine type.

In order to prevent air being pumped to the filter bed, it will be necessary to ensure the removal of air accumulated in the vertical pump column between backwashing cycles. This can be achieved by bleeding off air through an air release valve and the inclusion of a vent pipe on the discharge header. The backwash pump will draw about 300 HP.

9. Electrical

The entire electrical installation must comply with the requirements of local Electrical Codes.

All electrical equipment must have C.S.A. approval label or be approved by Newfoundland Light and Power Company special inspector.

a. Power System

Newfoundland Light and Power Company will supply a 12.5 KV, 3 phase, 60 cycle overhead line to the treatment plant site. The power system will be obtained from this line via 2.3 KV and 600 volt transformers to be installed in the electrical room.

In concept, all motors 75 HP and smaller will be fed by a 600 volt current. Motors 75 HP and larger, up to 200 HP, could be connected to the 600 volt system provided they will be equipped with voltage reducing switches. Motors larger than 200 HP will have to be connected to the 2.3 KV system.

Power for emergency conditions will be supplied from 2, 800 KVA diesel generators to be installed in the mechanical room.

b. Lighting

(i) Interior Lighting

Generally, high bay areas will be provided with deluxe white mercury lighting, low bay areas and general areas with cool white fluorescent. Incandescent lighting will be used in storage areas, feature areas.

Average illumination levels will be as follows:

High lift and low lift pump areas	-	50 fc
Work shops	-	50 fc
Offices, control room, laboratory	-	100 fc
Corridors	-	30 fc
Lobby	-	40 fc
Washrooms, lockers, lunch rooms	-	30 fc
Storage areas and garages	-	20 fc
Mechanical and electrical rooms	-	30 fc
Conference room	-	70 fc
Ozonation and filter areas	-	50 fc
Filter gallery	-	50 fc

Emergency lighting will be required throughout the buildings and grounds.

(ii) Exterior Lighting

Mercury lamps, deluxe white type will be used for all outdoor lighting.

Post-top mounted luminaries could be used to illuminate the parking lot, access road and landscaping. Outdoor lighting could be controlled by a photocell and time switch. The photocell will turn the lights on at dusk and the time switch will turn them off at a preset (adjustable) time at night.

Security lighting could be left on all night; controlled by a photocell only.

(iii) Communication Systems

Conduit systems will be provided for a telephone system with outlets in all offices, the lobby, the control room, and the like.

A fire alarm system should be provided with automatic detectors in electrical rooms, mechanical rooms, offices, storage areas, and the like. Alarm stations should be located at all exits and bells provided to cover the entire building.

Areas for installation of wall clocks will be the control room, maintenance shop, laboratory and lunch room.

An intercom system will be provided between the common room and various plant areas such as the high lift pump room, low lift pump room and the work shops.

10. Heating and Ventilation

a. Air Conditioning and Heating

The following design conditions will be considered:

- Outside	winter	0 F.	
	summer	79 F.	dry bulb
		69 F.	wet bulb
- Office areas	winter	72 F.	MIN. 20% RH MIN.
	summer	75 F.	50% RH MAX

- Chlorine & Ammonia Rooms :	60 F. MIN.
- Chemical Storage Room :	55 F. MIN.
- Low Lift Pump Room :	65 F. MIN.
- High Lift Pump Room:	65 F. MIN.
- Filter Pipe Gallery:	50 F. MIN.
- Filter Operating Room:	65 F. MIN.
- Ozonation Area:	50 F. MIN.
- Workshop:	68 F. MIN.

The above temperatures assume that minimum process equipment is operating.

Only the conference room and lunch room areas will be considered for air conditioning. For this service a water cooler condensing unit can be used, with the cooling water being run to drain.

Two packaged oil fired boilers will be considered for the provision of heat for the building and ventilation loads.

Office areas will be provided with a hot water perimeter heating system, in addition to a tempered air ventilation system. All other areas will be furnished with a hot water recirculating unit heater system.

Where winter fresh air loads are involved, a water/glycol system will be used.

Electric heating or an indirect oil fired air heating system should be considered for the low lift pumping station.

b. Dehumidification

A solid or liquid type dehumidification system will be considered for the filter pipe gallery and the lime handling areas. The air to be conditioned will pass through a chamber, where it is dried by intimate contact with a desiccant. A cooling coil will then cool the dried air. The dew point of the conditioned air will be controlled below the temperature of the intake water. The cooling water will be plant service water run through the cooling coils and then to drain.

c. Ventilation

(i) Washrooms
Washrooms will be exhausted as required by the building codes.

(ii) Pump Rooms
The high lift pump room and the low lift pump room will each have a recirculating heating system. Additional ventilation will be provided during the summer months. This ventilation will not normally operate during the winter.

(iii) Chlorine and Ammonia Rooms
Continuous mechanical ventilation at a rate of 4 air changes per hour will be provided. In addition, provision will be made for emergency mechanical ventilation at the rate of 30 air changes per hour. The suction inlets will be located at a maximum of 3 feet above floor level. This installation must meet the requirements of the Department of Labour.

(iv) Electrical Rooms
Ventilation systems will be designed to meet the requirements of Newfoundland Light and Power Company.

d. Plumbing

Plumbing services will supply water and drainage to the administration area, such as the washrooms, lunch room, laboratory. Sanitary and laboratory wastewater drainage will be kept separate from all other wastewaters. After treatment this wastewater will eventually be disposed of with filter backwash water to Goose Pond off Bay Bulls Big Pond.

Other water and drainage requirements are for equipment cooling water, pump water seal units, utility use, and general floor drainage.

The laboratory, aside from the acid drainage piping for the sinks, will have an emergency eye wash and shower.

A sampling sink will be provided for sampling pipes coming from the low lift pumps discharge, the ozonation effluent, the filter influent, the filter effluent, the clearwell and the high lift pumps discharge.

11. Instrumentation and Control

a. General

This section presents a broad outline of the proposed measurement and control of the water treatment process.

Acceptable types of instrumentation and control systems are pneumatic, hydro-pneumatic, electronic.

The main control panel in the control room will include a large scale graphic panel.

b. Process Measurements

(i) Flow Measurement

The major areas of flow measurement are as follows:

- Raw Water

  This is a major variable in plant operation. Flow measurement will permit the determination of plant input rates, and will generate the signal by which the chemicals will be proportioned to this flow.

- Filter Effluent

  The filter effluent flow measurement could be used in conjunction with a final control element and controller to form a control loop which will maintain a filtering rate equal to a variable set point.

- Washwater

  Washwater flow must be effective by controlled efficient washing of filters.

- Service Water

  High accuracy measurement must be provided for the service water supplied to the customers, for billing purposes.

(ii) Temperature Measurement

- Backwash Water

  This temperature is important for efficient and effective control of filter backwashing.

(iii) Level Measurement

- Intake Forebay

  A level indicator will provide records of the water levels in the pond.

- Raw Water Suction Well

  Well level will indicate the suction conditions of the pumps and provide a differential level alarm to protect the travelling screens.

- Filter Influent Channel

  Channel level will be used to establish automatically the set point for the filter effluent rate controllers.

- Clear Well

  The clear well will be used essentially for low lift pump control. Minimum low water level will shut down the high lift pumps.

(iv) Turbidity Measurement

- Raw Water

  Turbidity readings will serve as an indication to the operator for the use of chemical treatment.

- Filter Effluent

  Knowledge of the turbidity of filter effluent will enable the operator to quickly check the performance of the filters and the need for chemical addition.

(v) Miscellaneous

- Chlorine residuals will be measured, recorded and used as control levels for chlorine application rates.

- Loss of head through filters will be measured and controlled.

- pH levels will be recorded and used as a control for chemical and chlorine application rates.

c. Process Control

(i) Inner and Outer Loop Concept

The level in the filter influent channel will be used as a cascade control automatically setting the rate through the filter by setting the set point of the effluent controllers.

The clearwell will absorb the fluctuations of the distribution system. Rising water level (say over the top 12 inch band) in the well, will progressively throttle the flow controller on the low lift pumps discharge header until eventual shut down of the pumps. Thus, for any change in the outer loop (the raw water flow and clearwell level) the inner loop (channel level and filter effluent flow) will automatically follow. In this manner, pretreatment will not be disturbed even during the washing of a filter. In this situation, the other filters would pick up the load for the out of service filter.

(ii) Filter Backwashing

A filter will be washed when, or before, a preset loss of head occurs. The loss of head and rate of flow through each filter will be recorded on the main control panel.

The filter will backwash automatically. However, the operator could initiate manually the automatic backwashing of a filter at will, or whenever an alarm signal indicates the need. During the backwashing of a filter, all other filters will be locked out to prevent the inadvertent backwashing of a second filter at the same time.

The system will be of the sequential type. The functioning through the various backwashing steps will be as follows:

On backwash initiation, the filter influent valve will close. The filter level will lower through normal filtering action until a preset low level is reached, at which time a level sensor will cause the filter effluent valve to close. This closure will initiate the opening of the drain valve, followed by opening of the washwater valve. Surface washing will then start, proceeded by a low rate of backwash water. Following the low rate of wash, a high backwash rate will be applied for a preset rate. At the expiration of the high rate, a second, low backwash rate will be applied to assist in re-classifying the bed. The surface wash valve will remain open until after the backwash is terminated. The filter will then be returned to operation by the automatic opening and closing of valves in the reverse order in which they were operated in applying the backwash.

After backwashing, the filter will be brought back into service gradually over an adjustable timed period.

Local controls at each filter bed will allow the operator to stop the programmed sequence or prolong the high backwash rate if visual inspection indicates that this is desirable. In addition, any filter could be taken off the automatic operation and washed by manual operation of the valves.

(iii) Chemical Feed

The chemical dosages of ozone, chlorine, ammonia, polyelectrolyte and lime will be set by the plant operator, and the amount of chemical fed to satisfy the set dosages will be automatically proportioned to the influent flow by a signal from the raw water flow transmitter.

(iv) High Lift Pumps

The high lift pumps will be controlled by levels in a storage reservoir. The system will operate over rented telephone lines.

12. Grading, Drainage and Roadways

A 5,000 foot long access road will be built from Goulds Road to the treatment plant site. It will be a paved approach road.

The filter backwash and plant wastewater pipe to Goose Pond (off Bay Bulls Big Pond) will be routed along this access road.

There will be considerable regrading of the site. The entire treatment plant site will be finish graded, drained, loamed and seeded, landscaped and paved where necessary.

VII. WINDSOR LAKE WATER TREATMENT WORKS

Thw water treatment plant envisaged for installation (at a future date, see Chapter 9 of this Volume) at Windsor Lake is, in many respects, similar to that recommended for Bay Bulls Big Pond. The treatment works should be located near the lake and at an elevation which will ensure a gravity supply to the proposed service reservoirs at the North Expansion Zone and Torbay (see Chapter 7 of this volume).

Construction of a new intake and low lift pumping station to deliver raw water to the plant will also be required.

The treatment process presently indicated as being the most suitable is direct filtration and ozonation, essentially similar to that described previously for Bay Bulls Big Pond.

The sizing of the filter beds will depend upon whether or not Windsor Lake will continue to.be augmented from Little Powers Pond. If augmentation continues then 4, 4 MGD units will be required, and, if not, then the 4 filter beds proposed will be of 3.5 MGD capacity.

All descriptive material related to Bay Bulls Big Pond treatment works as included in this Chapter will basically be applicable to Windsor Lake.

REFERENCES

(1) Hutchison W, Foley, P.D. "Operational and Experimental Results of Direct Filtration", Journal Americal Water Works Association, 66:2:79 (Feburary 1974)

(2) Hudson H. E. Jr., "Physical Aspects of Flocculation", Journal American Water Works Association, 57:7:885 (July 1965).

(3) "Research Committee on Color Problems", Report for 1966, Journal American Water Works Association, 1923 (August 1967)

APPENDIX "I"

ANALYTICAL SCHEME

ANALYTICAL DETERMINATION	HOLDING TANK	OZONE CONTRACT COLUMN (S)	FILTER
Colour	Yes	Yes	Yes
Turbidity	Yes	Yes	Yes
Iron	Yes	Yes	Yes
Manganese	Yes	Yes	Yes
Total Dissolved Solids	Yes	Yes	Yes
Organic*	Yes	Yes	Yes
Alkalinity (as $CaCo_3$)	Yes	Yes	Yes
pH	Yes	Yes	Yes
Coliform Organisms	Yes	Yes	Yes
Chlorine Demand	Yes	Yes	Yes
Ozone Residual	No	Yes**	No

* As total carbon chloroform and carbon alcohol extractibility

** Of feed and exhausted ozone

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
NATIONAL ENVIRONMENTAL RESEARCH CENTER
CINCINNATI, OHIO 45268

January 4, 1973

APPENDIX "II"

Mr. Claude H. Wolf
Supervisor, Regulatory
Activities
Nalco Chemical Company
1810 Frontage Road
Northbrook, Illinois 60062

Dear Mr. Wolf:

This is in reference to your request for approval of your Coagulant Aid Nalcolyte 607 for use in the treatment of potable water.

It has been concluded that this product may be safely used at the recommended application rate not exceeding 40 milligrams per liter, provided the material continues to meet the quality specifications previously furnished.

We have not inquired into the effectiveness of this product for the proposed use and this action should not imply that it is necessarily superior, equivalent, or inferior in performance to other products intended for similar use. The toxicological advice on the safety of this coagulant aid does not constitute endorsement or recommendation of the product by the U. S. Environmental Protection Agency.

Nalcolyte 607 will be listed on the next issuance of the Report on Coagulant Aids for Water Treatment.

I am requesting that if this letter is to be used in any way, that it be quoted in its entirety.

Sincerely yours,

Robert G. Tardiff

Robert G. Tardiff, Ph.D.
Criteria Development Branch
Water Supply Research Laboratory

CAMP DRESSER & McKEE

Inc.

CONSULTING ENGINEERS

ONE CENTER PLAZA BOSTON, MASS. 02108 TEL. 617 742-5151 CABLE: CAMDRES

July 18, 1974

FENCO
1 Yonge Street
Toronto
Canada M5E 1E7

Attention: Mr. M. Schwartz

Dear Sir:

I have reviewed the data on water quality in the Bay Bull Big Pond near St. Johns, Newfoundland and the results of your pilot plant work on treatment of that water. Based on these data and our discussions of July 16, I feel certain that unless there is a substantial change in the quality of the pond water the treatment with ozone and rapid filtration should produce a treated water of good quality, low in color, turbidity, algae, iron, manganese, and coliform organisms. It should meet the "objective" levels of your drinking water standards some 90-95 percent of the time and the "acceptable" level at all times. This assumes that the plant is properly operated with competent personnel.

Following ozonation the water should flow to a detention basin for removal of the foam or scum which is commonly produced in ozonation of colored waters. The ozone contact chamber should be covered and vented in a manner which will destroy any residual ozone. This is most commonly done with heat as ozone decomposes at 270°C but other methods can also be used.

From this basin the water should flow to rapid dual media filters. We recommend that a 10 inch depth of sand with an effective size of 0.45 - 0.50 mm and a uniformity coefficient no greater than 1.5 be used with a 20 inch depth of anthracite coal with an effective size of 1.0 to 1.1 mm and a uniformity coefficient no greater than 1.5. Water backwash should provide a rise rate of about 36 inches per minute. A filtration rate of 4 gallons per minute per square foot can be used with such filters. They should remove the iron and manganese, and particulate matter including algae.

Provision should be made to add a cationic polyelectrolyte such as Nalco 607 to the water if it should be needed to improve filtration efficiency.

Turbidity of the filtered water should be monitored by a sensitive light scattering turbidimeter such as the Hach low range turbidimeter and backwashing of the filters done when turbidity exceeds one unit or when the head loss reaches its limiting value.

Post chlorination facilities should be provided to maintain a chlorine residual as the water enters the distribution system. It has frequently been found that in treating colored waters with ozone the organic matter which caused the color is converted to compounds which are suitable food for bacteria and unless a chlorine residual is maintained there may be growths of nuisance organisms in the distribution system.

Adjustment of pH for corrosion control with lime should be done. It will be better to do this prior to filtration in order to be able to remove any insoluble material that may be present in the lime.

Your pilot plant results also indicate that a plant providing for color removal with alum could also produce a satisfactory water but probably with a slightly higher color than would be produced by ozonation. There would also be more of a waste disposal problem if alum is used.

It has been a pleasure to work with you on this project. I trust that your plans will be carried out expeditiously.

Yours truly,

CAMP DRESSER & McKEE Inc.

Richard L. Woodward

Richard L. Woodward
Vice President

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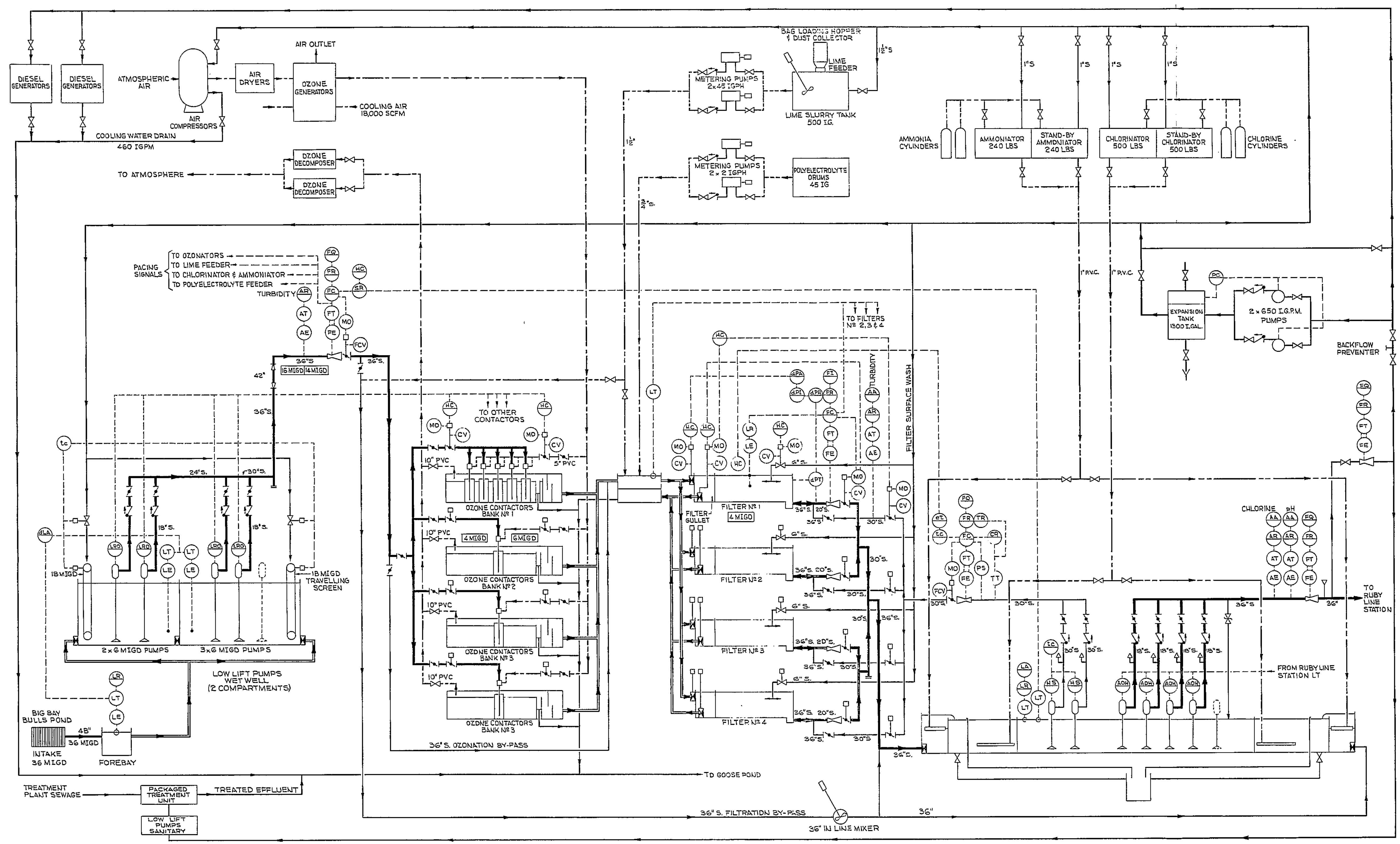


NOTES AND REFERENCES

SYMBOLS:-

- VERTICAL TURBINE PUMP
- METERING PUMP
- CENTRIFUGAL PUMP
- MIXER
- SHUT OFF VALVE
- CHECK VALVE.
- BUTTERFLY VALVE
- MOTOR OPERATED VALVE
- SURGE RELIEF VALVE
- SLUICE GATE
- AIR RELEASE VALVE
- VACUUM BREAKER
- VENTURI METER
- FLOW RATE CONTROLLER

ABBREVIATIONS:-

S.	STEEL PIPE
G.S.	GALVAINISED STEEL PIPE
S.S.	FABRICATED STAINLESS STEEL PIPE
R.C.P.	REINFORCED CONCRETE PIPE
P.V.C.	RIGID POLYVINYL CHLORIDE PIPE
S.C.F.M.	STANDARD AIR, CUBIC FEET PER MINUTE
I.G.P.M.	IMPERIAL GALLONS PER MINUTE
M.I.G.D.	MILLIONS OF IMPERIAL GALLONS PER DAY

INSTRUMENTATION:-

AA	ANALYTICAL ALARM
AE	ANALYTICAL ELEMENT
ADH	AUTOMATIC-OFF HAND SWITCH
AR	ANALYTICAL RECORDER
AT	ANALYTICAL TRANSMITTER
CC	CHEMICAL CONTROLLER
CR	CONTROL RELAY
CV	CONTROL VALVE
dLA	DIFFERENTIAL LEVEL ALARM
dPA	DIFFERENTIAL PRESSURE ALARM
dPC	DIFFERENTIAL PRESSURE CONTROLLER
dPI	DIFFERENTIAL PRESSURE INDICATOR
dPR	DIFFERENTIAL PRESSURE RECORDER
dPT	DIFFERENTIAL PRESSURE TRANSMITTER
et	EXTENDED TIMER
FC	FLOW CONTROLLER
FCV	FLOW CONTROL VALVE
FE	FLOW ELEMENT
FI	FLOW INDICATOR
FQ	FLOW TOTALIZER
FR	FLOW RECORDER
FT	FLOW TRANSMITTER
HC	HAND CONTROL
HS	HAND SWITCH
LA	LEVEL ALARM
LC	LEVEL CONTROL
LE	LEVEL ELEMENT
LR	LEVEL RECORDER
LRO	LOCAL-REMOTE-OFF SWITCH
LT	LEVEL TRANSMITTER
MO	MOTOR OPERATOR
SR	SELECTOR RELAY
TI	TEMPERATURE INDICATOR
TR	TEMPERATURE RECORDER
TT	TEMPERATURE TRANSMITTER
tc	TIMER CONTROL
PC	PRESSURE CONTROL

NOTE:-
PROCESS CONTROLS AS SHOWN FOR ONE BANK OF UNITS APPLY TO ALL OTHER BANKS.

1MIGD 2MIGD — DESIGN STAGE FLOW
— 1ST STAGE FLOW

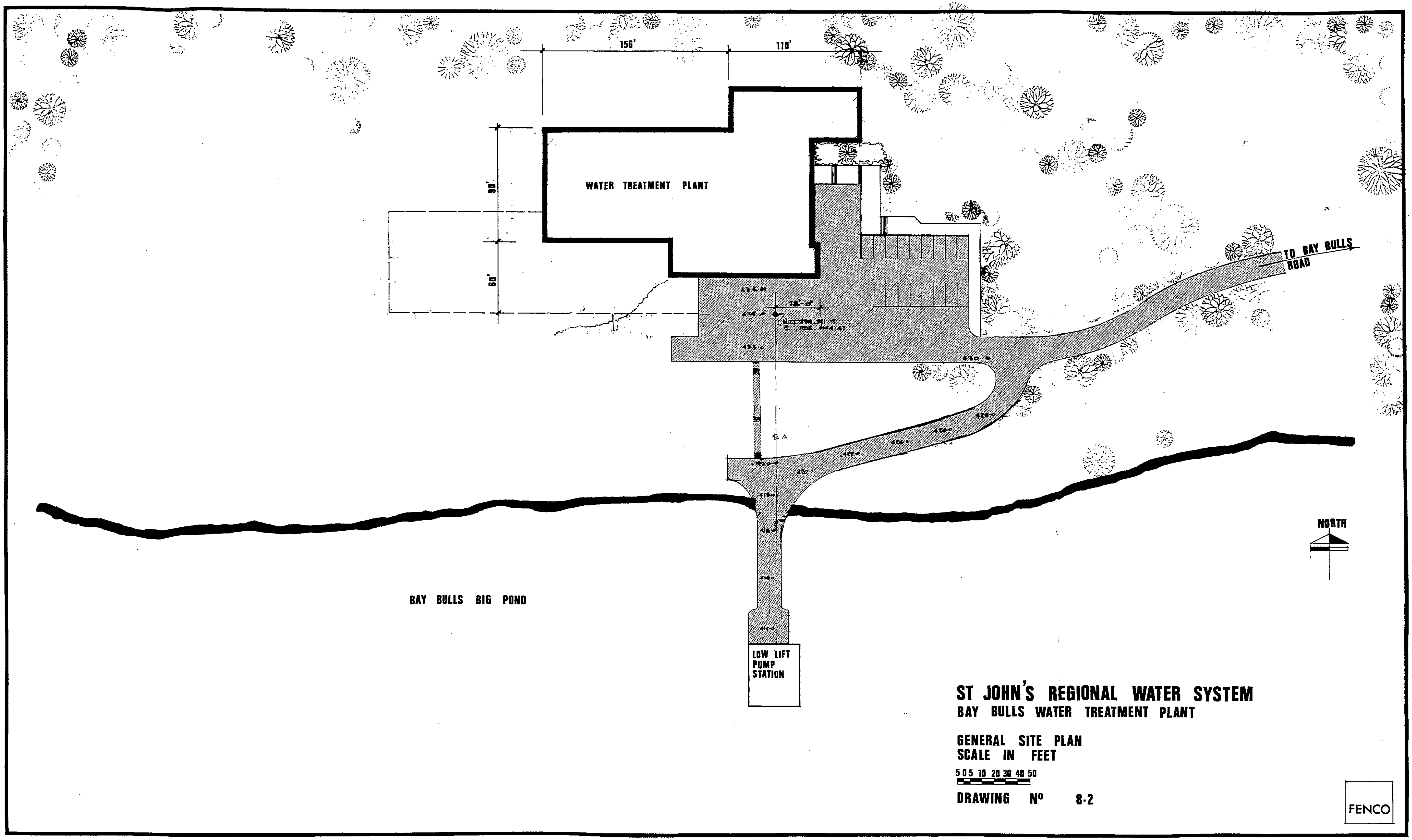
ST JOHN'S REGIONAL WATER SYSTEM STUDY

BAY BULLS WATER TREATMENT PLANT

PROCESS FLOW & CONTROLS DIAGRAM

DRAWING Nº 8·1

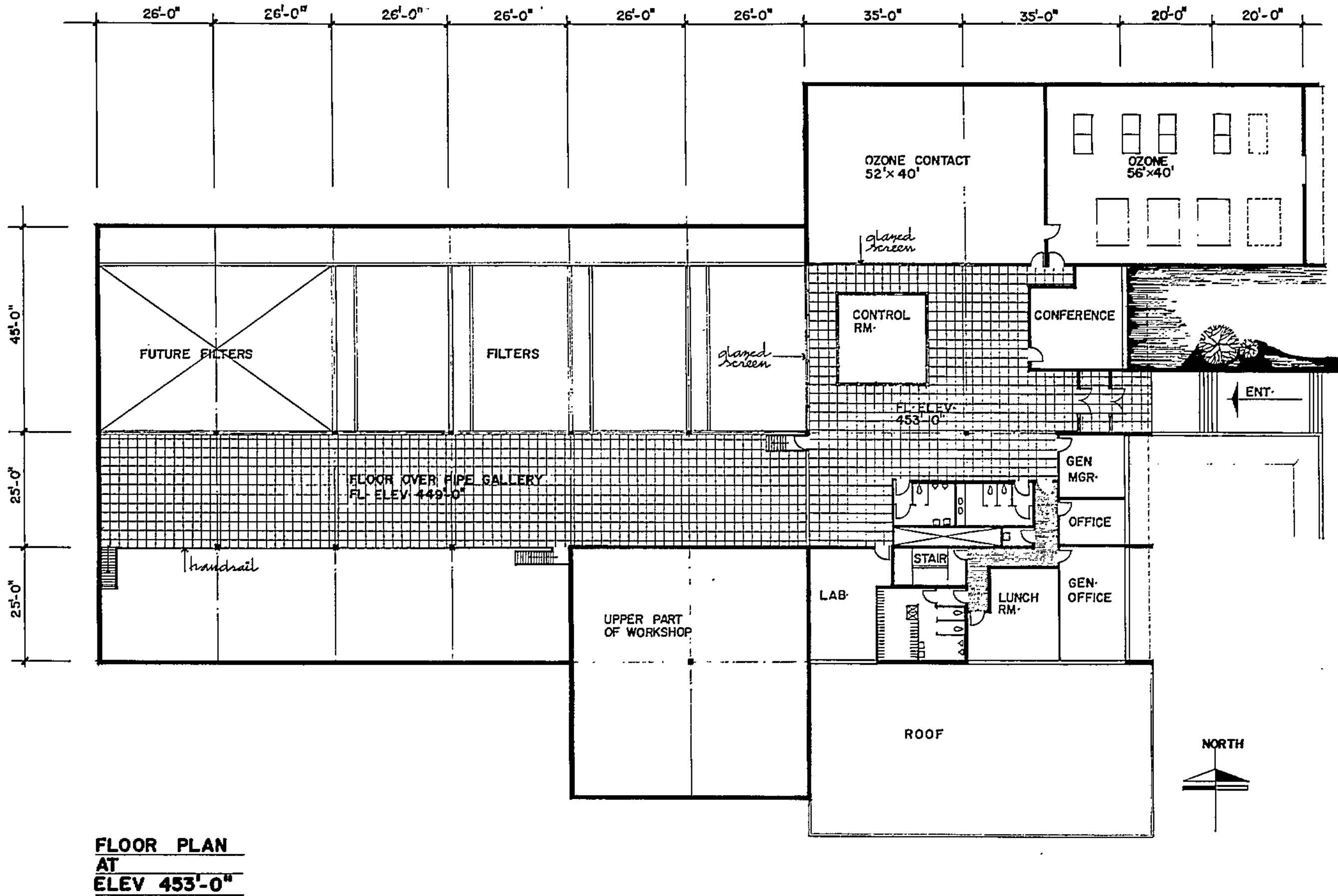
FENCO

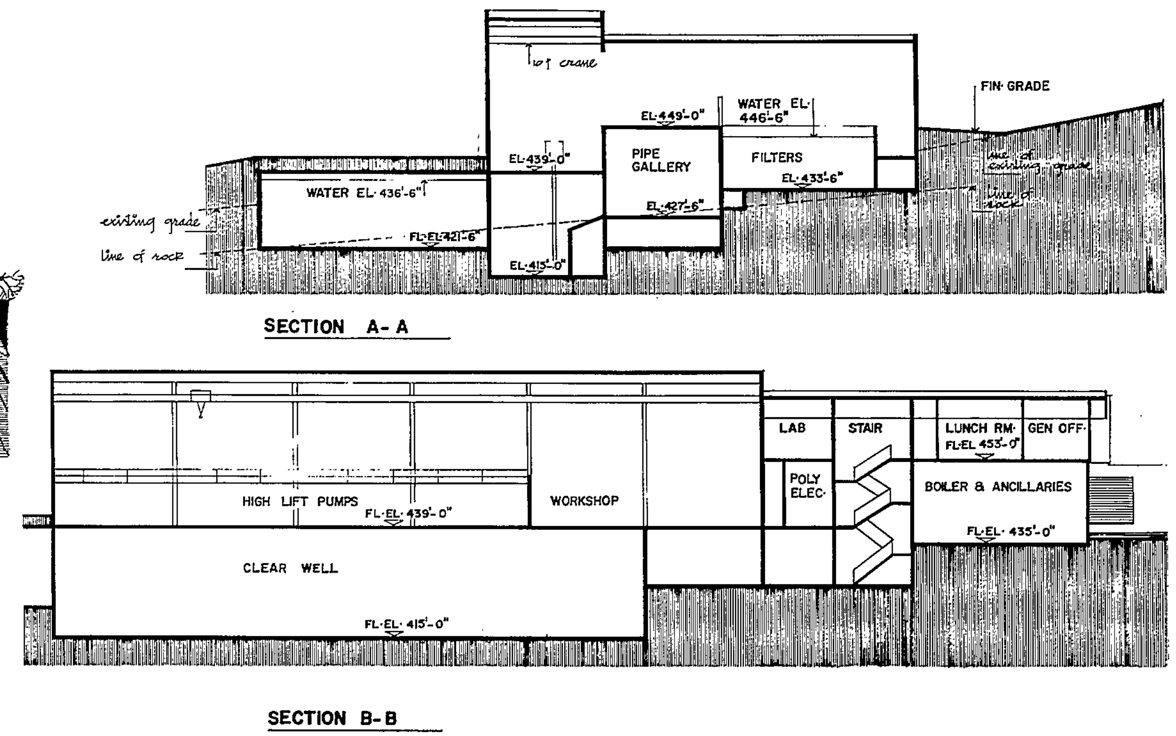


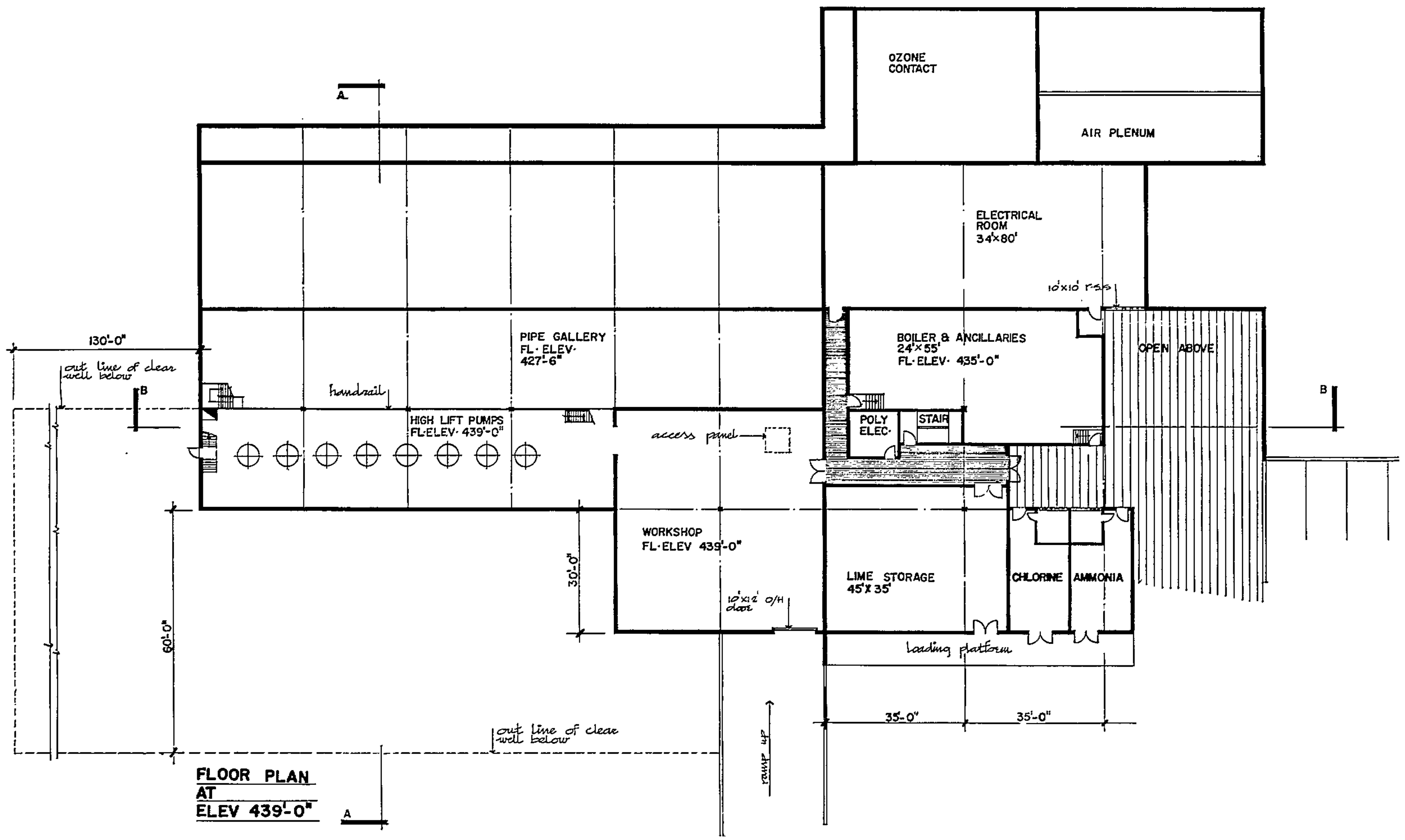
ST JOHN'S REGIONAL WATER SYSTEM
BAY BULLS WATER TREATMENT PLANT

GENERAL SITE PLAN
SCALE IN FEET

DRAWING Nº 8·2





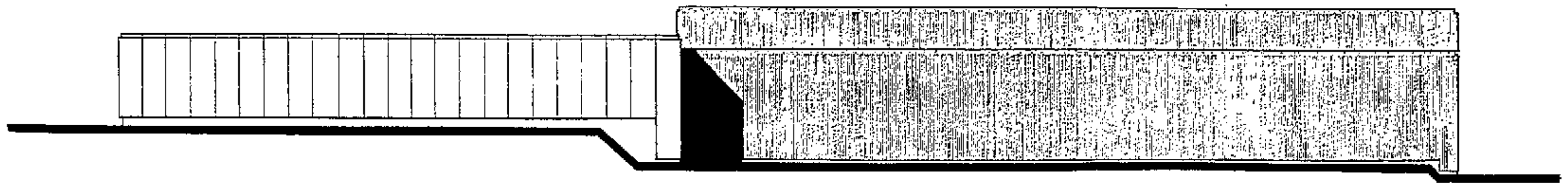


ST JOHN'S REGIONAL WATER SYSTEM STUDY
BAY BULLS WATER TREATMENT PLANT

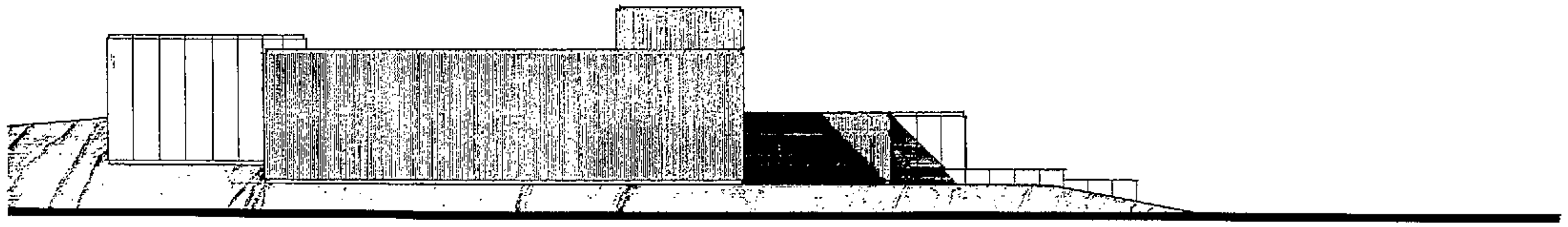
PLANS & SECTIONS
SCALE IN FEET
8 4 0 8 16 24 32

DRAWING Nº 8·3

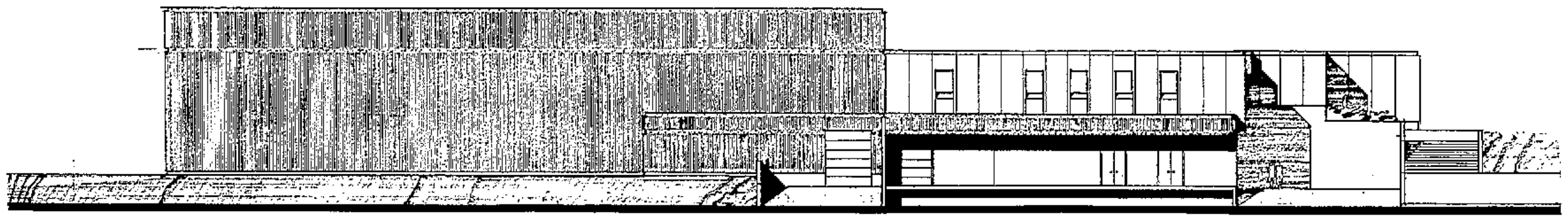
FENCO



NORTH ELEVATION



WEST ELEVATION



SOUTH ELEVATION

EAST ELEVATION

ST JOHN'S REGIONAL WATER SYSTEM STUDY
BAY BULLS WATER TREATMENT PLANT

ELEVATIONS
SCALE IN FEET
8 4 0 8 16 24 32

DRAWING Nº 8-4

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Chapter 9

CHAPTER 9

CONSTRUCTION CONSIDERATIONS

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CHAPTER 9

CONSTRUCTION CONSIDERATIONS

SYNOPSIS

Construction aspects such as phased construction, cost estimates, and factors to be considered in construction, have been presented and discussed in this Chapter.

A summary of the findings and recommendations is as follows:

- The proposed regional waterworks have been divided into 13 packages, each of which could be considered separately for (design and construction) contractual purposes. A description of these packages and the capital cost estimate for their construction is as follows:

Package No.	Package Description	Cost Estimate x $1,000
1	Treatment Plant at Bay Bulls Big Pond, including intake and high lift pumping station.	8,900
2	Conveyance main from Bay Bulls Big Pond to 2MG service reservoir at New Town including Ruby Line booster pumping station at 70% of its ultimate capacity.	4,320

Package No.	Package Description	Cost Estimate x $1,000
2A	Additional storage capacity at New Town service reservoir.	750
3	Conveyance main from Ruby Line bifurcation via Old Placentia Road to Topsail Road and along Blackmarsh and Empire Avenue to proposed service reservoir off Jensen Camp's Lane.	2,240
4	Conveyance main from 2MG service reservoir at New Town to Conception Bay South area, including service reservoirs.	3,300
5	Tapping into the existing conveyance main from Windsor Lake and construction of a ring main at North Expansion Zone.	830
5A	Providing service reservoir for North Expansion Zone - Torbay area.	1,100
6	Extension of conveyance main under Package 3 to high pressure zone, including local booster pumping station and service reservoirs.	2,250
7	Strengthening of trunk mains in St. John's downtown area, including service reservoirs.	1,500
8	Tapping the conveyance main of Ruby Line and construction of mains to Kilbride and Goulds, including service reservoirs.	1,740

Package No.	Package Description	Cost Estimate x $1,000
9	Tapping the main to Conception Bay South at T.C.H. to supply water to Paradise and Topsail Road.	470
10	Conveyance main from North Expansion Zone through Torbay Road to Torbay, including service reservoir.	1,300
11	Expansion of Bay Bulls Big Pond water treatment plant, low and high lift pumping stations and booster pumping station at Ruby Lake.	2,150
12	Treatment Plant at Windsor Lake.	7,000
13	Tapping the Windsor Lake treatment plant to supply water to Penetanguishene, including booster pumps and service reservoir.	310
Total		38,160

- Since pipes (and their associated appurtenances) represent a large proportion of the capital invested in water works undertakings, some factors to be considered in the construction of these system components have been examined and reported on.

I. INTRODUCTION

This Chapter is concerned with construction aspects of the Regional Water System as developed and described in the preceding two Chapters. The first part discusses the aspect of phased construction. The central part of the Chapter presents a cost estimate for the proposed Regional water works. The final part offers some factors to be considered in the construction of the Regional Water System.

II. PHASED CONSTRUCTION

1. The Approach

In the developing of a schedule for phased construction, consideration should be given to such factors as technical, economical, social, political, and other similar subjective priorities. Within the scope of this project, it was not for the consultant to determine these priorities, save where they are demonstrably obvious. However, it was incumbent upon him to point out the water needs of the region and the consequences of not satisfying these needs. In this regard we have taken the following approach:

(a) Classified the communities in the region into three categories of water need.

(b) Divided the proposed regional water works into 13 packages, each of which could be considered separately for (design and construction) contractual purposes.

It is felt that the above approach will enable decision makers to identify the water needs of the different communities, correlate them to the capital works required to bring water to these communities, and accordingly formulate a policy of priorities in phased construction.

2. Categories of Water Need

The three categories of water need and the communities classified into each of these categories are as follows:

(a) Category 1 - Extensive and Immediate Development - Included in this category are areas and communities that require early supply of water to accommodate immediate growth and extensive development. They have been identified as follows:

- North Expansion Zone
- South Expansion Zone
- New Town
- Torbay Road
- Donovan's Industrial Park
- Kenmount Industrial Park
- White Hills Industrial Park

(b) Category 2 - Health Conditions - Areas and communities that require improved supply of water at an early stage, due to deteriorating sanitary conditions, are classified under this category. They have been identified as follows:

- Seal Cove
- Gullies

- Foxtrap
- Kelligrews
- Long Pond
- Manuels
- Chamberlains
- Topsail

(c) Category 3 - Social and Regular Developments - Areas and communities whose present supply of water is considered to be adequate, but which would require additonal or improved supply with the 20 year design period, due to social and orderly developments over the years, have been included in this category. They have been identified as follows:

- City of St. John's*
- Wedgewood Park
- Donovans
- Shea Heights
- Kilbride
- Mount Pearl
- Paradise
- Torbay
- Goulds
- Petty Harbour
- Penetanguishene

The consequences from failure to augment supply in the near future and satisfy the water needs of the region may be assessed as follows:

---

* Except for the Mundy Pond and Kenmount Road areas whose needs are more immediate, but they can be coped with by the South Expansion Zone.

Category of Water Need	Consequence
1	Retardation of development, including that of construction industry, and loss of return on committed investments.
2	The hazards to health will increase.
3	No effect unless augmentation of a regional scheme is very seriously delayed (in this event consequence may fall into those of Category 2).

3. Water Works Packages

The 13 packages recommended for (design and construction) contractual purposes are summarized in Table 9.1.

No attempt has been made to identify priorities and the order in which these packages appear should not be interpreted as an order of priorities, save for packages 1, 2 and 3 which are explained below.

Package No. 1 represents the production of water from the new source of Bay Bulls Big Pond. It includes intake works and treatment and pumping facilities. Once these development works have been implemented additional quantities of water will be available for supply to the region (via the proposed Bay Bulls Big Pond conveyance system).

Package No. 2 is for the Main Conveyance Line from Bay Bulls Big Pond treatment plant to Ruby Line bifurcation, and for the New Town - Mount Pearl conveyance main from Ruby Line bifurcation to the existing 2.0 MG service reservoir at New Town. The former main must be built first under any program of priorities as it brings the water from the source to a central point close to the centres of demand. The latter main is required to supply water to New Town, Donovan's Industrial Park, and Mount Pearl. Upon the implementation of this latter main, not only will the extensive developments in New Town and Donovan's Industrial Park receive adequate quantities of water at satisfactory pressure, but also, the City of St. John's would be relieved of its present obligation to supply the western environs of the City.

There are several factors that, in our opinion, justify the inclusion of package No. 3 - City of St. John's conveyance main - in the first contractual (design and construction) phase, together with packages 1 and 2. These factors can be enumerated as follows:

(a) The Mundy Pond and Kenmount Road areas are basically pressure-deficient. This situation would be more severe and unsatisfactory during periods of fire fighting.

(b) Developments in the Kenmount Road area and South Expansion Zone require the provision of distribution mains. Extension of the existing supply network to these areas will all but be a piecemeal solution which eventually would have to be rectified by water supply from the new source via the City of St. John's conveyance main (package No. 3).

(c) Based on the arguments under (a) and (b) above, and to avoid a situation in the immediate future whereby the City's demand for water would equal the reliable supply from the present day sources, package No. 3 has been identified as having the next priority (after packages 1 and 2).

(d) The cost benefit aspect of first phase construction supports the inclusion of package 3 in this phase.

Except for the above cited packages, the others described in Table 9.1 have not been assigned priorities.

III. COST ESTIMATES

1. Unit Prices

Unit prices used for the preparation of capital and annual running estimates were based on cost data and statistics reported for other similar installations in Canada and the United States.

All capital construction costs were updated to reflect 1974 prices using the Engineering News-Record "Construction Cost Index in 22 Cities, 1949-1973"[1]. These unit prices were then adjusted to suit St. John's conditions. The same approach was taken in developing unit prices for operation and maintenance except that in this case the Skilled Labour and Common Labour Indices[1] were used.

It should be pointed out that at this stage the above approach forms the basis for a preliminary cost estimate. The final cost estimate should be prepared during the detailed design stage of each of the packages proposed.

TABLE 9.1

SUMMARY OF WATER WORKS PACKAGES

PACKAGE NO.	PACKAGE DESCRIPTION	PURPOSE OR WORKS	PRELIMINARY WORK REQUIRED (pre-design)
1.	Treatment Plant at Bay Bulls Big Pond, including intake and high-lift pumping station.	To render Bay Bulls Big Pond water quality acceptable for supply to the region. These works would be carried out in 2 phases; first phase will accommodate water needs projected for 1985, second phase will include expansion to meet 1995 projected needs.	a) Topographic survey of site. b) Soil investigation of site. c) Pilot Plant studies.
2.	Conveyance main from Bay Bulls Big Pond to 2MG service reservoir at NewTown including Ruby Line booster pumping station at 70% of its ultimate capacity.	The installation of this pipe will provide immediate water to NewTown, Donovan's Industrial Park and Mount Pearl. Tapping the 2MG service reservoir will supply water to Conception Bay South areas, Topsail Road and Paradise. Also, tapping this main at Ruby Line will supply water to Goulds and Kilbride.	a) Surveying of pipeline route. b) Soil investigation. c) Negotiations with various agencies.
3.	Conveyance main from Ruby Line bifuracation via Old Placentia Road to Topsail Road and along Blackmarsh and Empire Avenue to proposed service reservoir off Jensen Camp's Lane	To supply water to South Expansion Zone and improve the supply conditions at Mundy Pond and Kenmount Road areas.	a) Survey of Pipeline route. b) Soil investigation. c) Collecting data on existing underground utilities.
4.	Conveyance main from 2MG service reservoir at NewTown to Conception Bay South area, including service reservoirs.	To supply water to Conception Bay South area and improve the local health (sanitary) conditions. Extension of this pipeline could also serve Topsail Road west of Paradise. Tapping this main at T.C.H. will supply water to Paradise and Topsail Road.	a) Survey of pipeline route. b) Soil investigation. c) Negotiations with various agencies.
5.	Tapping into the existing conveyance main from Windsor Lake and construction of a ring main at North Expansion Zone.	To supply water to the development areas at North Expansion Zone and Torbay Road.	a) Survey of pipeline route. b) Soil investigation c) Collecting data on existing underground utilities.

PACKAGE NO.	PACKAGE DESCRIPTION	PURPOSE OF WORKS	PRELIMINARY WORK REQUIRED (pre-design)
6.	Extension of conveyance main under Package 3 to high pressure zone, including local booster pumping station and service reservoirs.	To complete the supply system to South Expansion Zone, and Mundy Pond and Kenmount Road areas.	a) Survey of pipeline route. b) Soil investigation. c) Collecting data on existing underground utilities.
7.	Strengthening of trunk mains in St. John's downtown area, including service reservoir.	To receive water from Bay Bulls Big Pond system to meet expansion requirements.	a) Survey of pipeline route. b) Soil investigation c) Collecting data on existing underground utilities.
8.	Tapping the conveyance main of Ruby Line and construction of mains to Kilbride and Goulds, including service reservoirs.	To add Kilbride and Goulds to the Bay Bulls Big Pond Supply System.	a) Survey of pipeline route. b) Soil investigation. c) Collecting data on existing underground utilities.
9.	Tapping the main to Conception Bay South at T.C.H. to supply water to Paradise and Topsail Road.	To supply water to Paradise and Topsail Road from Bay Bulls Big Pond system.	a) Surveying of pipeline route. b) Soil investigation. c) Negotiations with various agencies.
10.	Conveyance main from North Expansion Zone through Torbay Road to Torbay, including service reservoir.	To supply water to northern part of Torbay Road and Torbay from Windsor Lake system.	a) Surveying of pipeline route. b) Soil investigation.
11.	Expansion of Bay Bulls Big Pond water treatment plant, low and high lift pumping stations and booster pumping station at Ruby Lake	To meet water needs projected for 1995	a) Review of projected water needs relative to actual development that will have occurred in the region at that time.

PACKAGE NO.	PACKAGE DESCRIPTION	PURPOSE OF WORKS	PRELIMINARY WORK REQUIRED (pre-design)
12.	Treatment Plant at Windsor Lake.	To render the quality of the water compatible with the objectives of the Canadian Drinking Water Standards.	a) Topographic survey of site. b) Soil investigation of site.
13.	Tapping the Windsor Lake treatment plant to supply water to Penetanguishene, including booster pumps and service reservoir.	To add Penetanguishene to Windsor Lake supply system.	a) Surveying of pipeline route. b) Soil investigation.

The above cost data were formulated into cost functions to enable general use. These cost functions have been presented in Chapter 6 of Volume II.

2. Capital Costs

All capital construction costs include a 10 percent allowance for contingencies. Allowances for engineering and land acquisition are given separately later. All costs are in 1974 dollar value, estimated on the basis of 4.5 percent per annum inflation rate.

(a)	Package No. 1 - Total	$8,900,000
(i)	Intake Works - (Bay Bulls Big Pond) 48 inch diameter intake; Low lift pumping station, structure for 24 MGD equipment for 16 MGD	$1,000,000
(ii)	Treatment Works - (Bay Bulls Big Pond) High rate gravity filters with pretreatment for 16 MGD, including provision for expansion to 24 MGD; High lift pumping station, structure for 24 MGD equipment for 16 MGD	$7,900,000
(b)	Package No. 2 - Total	$4,320,000
(i)	42 inch diameter main, Bay Bulls Big Pond to Ruby Line bifurcation	$3,120,000
(ii)	30 inch diameter main, Ruby Line bifurcation to New Town service reservoir	$ 650,000
(iii)	Booster pumping station at Ruby Line, structure for 12 MGD equipment for 8 MGD	$ 550,000

(iv)	Package No. 2A - Total	$ 750,000
	(to be implemented separately probably at a later stage)	
	Additional 3.0 MG service reservoir at New Town	$ 750,000
(c)	<u>Package No. 3</u> - Total	$2,240,000
(i)	30 inch diameter main, Ruby Line bifurcation to Topsail Road	$1,400,000
(ii)	24 inch diameter main, Topsail Road to Jensen Camp's Lane	$ 840,000
(d)	<u>Package No. 4</u> - Total	$3,300,000
(i)	18 inch diameter main, New Town service reservoir to T.C.H.	$ 270,000
(ii)	16 inch diameter main, T.C.H. to Chamberlains Road	$ 790,000
(iii)	12 inch diameter main, from Chamberlains Road to Foxtrap Access Road and Conception Bay Highway	$ 780,000
(iv)	10 inch diameter main, along Conception Bay Highway to Pellen's Road	$ 760,000
(v)	Service Reservoir -	$ 700,000
	Two x 1.0 MG capacity, One x 0.5 MG capacity	
(e)	<u>Package No. 5</u> - Total	$ 830,000
(i)	20 inch diameter main (7,500 ft.) and 18 inch diameter main (8,000 ft.), from north of Venturi House to North Expansion Zone	$ 830,000

(ii)	Package No. 5A - Total	$1,100,000
	(to be implemented separately probably at a later stage)	
	3.5 MG service reservoir including connecting piping	$1,100,000
(f)	Package No. 6 - Total	$2,250,000
(i)	Booster pumping station to South Expansion Zone High Pressure Zone, 1.05 MGD capacity	$ 150,000
(ii)	10 inch diameter main from booster pumping station to service reservoir	$ 200,000
(iii)	1.0 MG capacity service reservoir, High Pressure Zone	$ 300,000
(iv)	5.0 MG capacity service reservoir, St. John's Intermediate Pressure Zone, including completion of piping	$1,600,000
(g)	Package No. 7 - Total	$1,500,000
(i)	5.0 MG capacity service reservoir, St. John's Low Pressure Zone	$1,350,000
(ii)	Allowance for strengthening of existing pipelines	$ 150,000
(h)	Package No. 8 - Total	$1,740,000
(i)	12 inch diameter main, Ruby Line to Goulds Road	$ 135,000
(ii)	12 inch diameter main through Goulds	$ 825,000
(iii)	12 inch diameter main through Kilbride	$ 480,000
(iv)	Service Reservoirs -	$ 300,000
	0.5 MG capacity for Kilbride 0.5 MG capacity for Goulds	

(i)	Package No. 9 - Total	$ 470,000
(i)	12 inch diameter main, T.C.H. to junction of Topsail Road and Paradise	$ 235,000
(ii)	Booster pumping station 0.2 MGD capacity, and 10 inch diameter main to Paradise service reservoir	$ 85,000
(iii)	0.5 MG capacity service reservoir at Paradise	$ 150,000
(j)	Package No. 10 - Total	$1,300,000
(i)	12 inch diameter main along Torbay Road to Torbay service reservoir	$1,150,000
(ii)	0.5 MG capacity service reservoir at Torbay	$ 150,000
(k)	Package No. 11 - Total	$2,150,000
(i)	Treatment Plant Expansion - (Bay Bulls Big Pond) Expansion of treatment works to 24 MGD capacity; Increase capacity of low-lift and high-lift pumping stations to 24 MGD	$2,050,000
(ii)	Ruby Line Booster Station - Increase capacity to 24 MGD	$ 100,000
(l)	Package No. 12 - Total	$7,000,000
(i)	Inlet Works - (Windsor Lake) Improvements to intake; Low-lift pumping station for 15 MGD capacity	$ 750,000

(ii)	Treatment Works - (Windsor Lake) High rate gravity filters with pretreatment, 15 MGD capacity	$6,250,000
(m)	Package No. 13 - Total	$ 310,000
(i)	Booster pumping station at Windsor Lake, 0.25 MGD capacity; 8 inch diameter main to Penetanguishene service reservoir	$ 160,000
(ii)	Penetanguishene service reservoir 0.5 MG capacity	$ 150,000

A summary of the cost estimate for capital expenditure, as outlined above, showing also allowances for engineering and land acquisition, is given in Table 9.2.

3. Operation and Maintenance Costs

Cost estimates for operation and maintenance were derived from the cost functions as developed in Chapter 6, Volume II. A summary of this cost for the 13 packages appears in Table 9.3.

The magnitude of operation and maintenance costs associated with pipelines and appurtenances is dependant primarily on the age of the pipe (assuming well practised installation procedures have been followed during construction). The comparable costs given in Table 9.3 could, therefore, be regarded as average for the design period of 20 years.

In the case of water treatment plants, and somewhat to a less extent pumping stations, operation and maintenance costs are dependent not only on the age of the facility (primarily equipment), but also on its size and its degree of utilization. The operation and maintenance costs for these facilities, as given in Table 9.3, are, therefore, related to the horizon year of 1995.

All costs are based on 1974 dollar value.

IV. MATERIALS FOR THE MAJOR SYSTEM COMPONENTS

This section presents some of the factors to be considered in the construction of the Regional Water System, and it covers the aspects of identification and suitability assessment of available materials.

Since pipes represent a large proportion of the capital invested in water works undertakings, system components such as pressure pipes and appurtenances have been examined.

1. Pressure Pipes

a. Criteria for Selection of Pipe Material

The choice of pipe material is generally based upon technical considerations tempered by one's personal preference and experience. Most of the materials available for use today - including but not limited to, cast and ductile iron,prestressed concrete, asbestos - cement, steel and polyethylene - have unique characteristics that may influence their use under different conditions.

TABLE 9.2

SUMMARY OF CAPITAL COST ESTIMATES

PACKAGE NO.	PACKAGE DESCRIPTION	CONSTRUCTION	PRE-DESIGN	DESIGN (1)	LAND (3)	TOTAL
		COST ESTIMATE (Thousands of Dollars)				
1	Treatment Plant at Bay Bulls Big Pond, including intake and high lift pumping station.	8,900	60	320 (2)	40	9,320
2	Conveyance main from Bay Bulls Big Pond to 2MG service reservoir at NewTown including Ruby Line booster pumping station at 70% of its ultimate capacity	4,320	25	175 (2)	40	4,560
2A	Additional storage capacity at Newtown service reservoir	750	10	40	5	805
3	Conveyance main from Ruby Line bifurcation via Old Placentia Road to Topsail Road and along Blackmarsh and Empire Avenue to proposed service reservoir off Jensen Camp's Lane	2,240	15	95 (2)	20	2,370
4	Conveyance main from 2MG service reservoir at NewTown to Conception Bay South area, including service reservoirs	3,300	30	150	30	3,510
5	Tapping into the existing conveyance main from Windsor Lake and construction of a ring main at North Expansion Zone	830	15	40	10	895
5A	Providing service reservoir for North Expansion Zone- Torbay area	1,100	10	55	5	1,170
6	Extension of conveyance main under Package 3 to high pressure zone, including local booster pumping station and service reservoirs	2,250	15	105	20	2,390
7	Strengthening of trunk mains in St. John's downtown area, including service reservoirs	1,500	10	70	5	1,585

TABLE 9.2 CONTINUED

SUMMARY OF CAPITAL COST ESTIMATES

		COST ESTIMATE (Thousands of Dollars)				
PACKAGE NO.	PACKAGE DESCRIPTION	CONSTRUCTION	PRE-DESIGN	DESIGN (1)	LAND (3)	TOTAL
8	Tapping the conveyance main of Ruby Line and construction of mains to Kilbride and Goulds, including service reservoirs	1,740	15	85	35	1,875
9	Tapping the main to Conception Bay South to T.C.H. to supply water to Paradise and Topsail Road	470	10	25	10	515
10	Conveyance main from North Expansion Zone through Torbay Road to Torbay, including service reservoir	1,300	15	60	10	1,385
11	Expansion of Bay Bulls Big Pond water treatment plant, low and high lift pumping stations and booster pumping station at Ruby Line.	2,150	50	100	10	2,310
12	Treatment Plant at Windsor Lake	7,000	40	300	30	7,370
13	Tapping the Windsor Lake treatment plant to supply water to Penetanguishene, including booster pumps and service reservoir	310	5	15	10	340
TOTAL		38,160	325	1,635	280	40,400

(1) Based on the Schedule of Fees for Consulting Professional Engineering Services, as recommended by the Association of Professional Engineers of the Province of Newfoundland (1969).

(2) Assumed to be designed as one parcel.

(3) For packages 1,2,2A assumed purchase price is $1,000 per acre. For packages 3-11 assumed purchase price is $1,500 per acre. For packages 12,13 assumed purchase price is $2,000 per acre. Crown land assumed to be transferred to the project without charge.

TABLE 9.3

SUMMARY OF OPERATION AND MAINTENANCE

COST ESTIMATES

COST ESTIMATE - $ Per Annum

PACKAGE NO.	PACKAGE DESCRIPTION	MAINTENANCE & REPAIR (1)	POWER	TREATMENT (2)	TOTAL
1	Treatment Plant at Bay Bulls Big Pond, including intake and high-lift pumping station		168,000	545,000	713,000
2	Conveyance main from Bay Bulls Big Pond to 2MG service reservoir at NewTown including Ruby Line booster pumping station at 70% of its ultimate capacity	42,000	55,000		97,000
2A	Additional storage capacity at NewTown service reservoir	10,000			10,000
3	Conveyance main from Ruby Line bifuracation via Old Placentia Road to Topsail Road and along Blackmarsh and Empire Avenue to proposed service reservoir off Jensen Camp's Lane	23,000			23,000
4	Conveyance main from 2MG service reservoir at NewTown to Conception Bay South area, including service reservoirs	30,000			30,000
5	Tapping into the existing conveyance main from Windsor Lake and construction of a ring main at North Expansion Zone	10,000			10,000
5A	Providing service reservoir for North Expansion Zone Torbay area	10,000			10,000
6	Extension of conveyance main under package 3 to high pressure zone, including local booster pumping station and service reservoirs	22,000	6,000		28,000
7	Strengthening of trunk mains in St. John's downtown area, including service reservoirs	15,000			15,000

TABLE 9.3 CONTINUED

SUMMARY OF OPERATION AND MAINTENANCE COST ESTIMATES

COST ESTIMATE – $ Per Annum

PACKAGE NO.	PACKAGE DESCRIPTION	MAINTENANCE & REPAIR (1)	POWER	TREATMENT (2)	TOTAL
8	Tapping the conveyance main of Ruby Line and construction of mains to Kilbride and Goulds including service reservoirs	18,000			18,000
9	Tapping the main to Conception Bay South at T.C.H. to supply water to Paradise and Top-sail Road	5,000	1,000		6,000
10	Conveyance main from North Expansion Zone through Torbay Road to Torbay, including service reservoir	10,000			10,000
11	Expansion of Bay Bulls Big Pond water treatment plant low and high lift pumping stations and booster pumping station at Ruby Line.	INCLUDED IN PACKAGE 1			
12	Treatment Plant at Windsor Lake		23,000	430,000	453,000
13	Tapping the Windsor Lake treatment Plant to supply water to Penetanguishene, including booster pumps and service reservoir				
TOTAL		200,000	253,000	975,000	1,428,000

(1) Includes labour

(2) Includes labour, power, fuel, chemicals, water, maintenance and repair.

A number of factors must be weighed in selecting the best type of pipe for installation in any system or even in different parts of the system. Relative costs, availability of both material and skilled labour, strength of pipe, durability, corrosion resistance, initial and sustained carrying capacity, ease and economy of installation and maintenance, and manufacturers after sales service should all be considered.

One of the most critical factors affecting pipe selection is the condition of the ground which might be encountered along the pipeline route. For practical purposes engineers divide geological deposits into two major groups, "soils" and "rocks"; the term "soil" embracing the comparatively soft, loose and uncemented deposits while the term "rock" refers to the hard, rigid and strongly cemented deposits*. The term bedrock generally defines any hard rock bed underlying soft deposits as soil in the engineering sense.

Although soil investigations have not as yet been carried out, one might expect that the soil overburden along the undeveloped sections of the pipeline route will consist of either a dense glacial till (boulder clay) or peat bog. The depth to bedrock might be expected to vary considerably over the route from close to, or on the surface, to depth up to and exceeding 10 to 12 feet. One section of the route crosses through a bog which was surveyed by the Provincial Department of Mines, Agriculture, and Resources in 1968. This bog which is located south of Ruby Line, in the Goulds vicinity, has an area of 5.6 acres and an average depth of 10 feet. The

* BS.C P 2001 Soils Investigations

Cochrane Pond Commercial Bog, which is located immediately west of Cochrane Pond, about 12 miles southwest of St. John's on the Trans Canada Highway, has an average depth of 5 to 7 feet and four selected analysed samples have shown the average moisture content to be in excess of 90 percent, with an average pH value ranging from 4.0 to 5.0. Thus the exterior of a metal pipe may be expected to show some corrosion under these acidic conditions.

Another factor affecting the selection of a pipe material is its strength. This can be defined as its ability to withstand stresses created by internal and external pressures, changes in momentum of the flowing water, external loads, temperature changes, and to satisfy the hydraulic requirements of the project. Internal pressures, tending to burst the pipe are caused by static pressure and water hammer, and produce circumferential tension and longitudinal tension at bends, dead ends and joints. When a liquid flowing in a pipeline is abruptly stopped, say by the closing of a valve, dynamic energy is converted to elastic energy and a series of positive and negative pressure waves travel back and forth in the pipe until they are damped out by friction. This phenomenon is known as water hammer, and the excessive pressures created by it can be greatly reduced by the use of such appurtenances as slow closing valves and automatic relief valves.

Longitudinal stresses of considerable magnitude may develop in pipes exposed to large changes in temperature if the ends are fixed. However, with some flexible joints and operating under most normal conditions these stresses are dissipated due to the changes in length being absorbed at the joint. An unsupported pipe acts as a beam with loads resulting from the weight of the pipe, weight of water in the pipe and any superimposed loads, and flexural stresses are set up. The stresses resulting from beam action may be determined by the usual methods of analysis applied to beams. A pipe is a fairly efficient beam section, and stresses resulting from beam action alone are usually negligible except for long spans or where there are large superimposed loads.

Pipes are normally laid in an excavated trench which is backfilled, or they are laid on the ground surface and covered with earth. In either case a vertical load is imposed on the pipe. If a load is superimposed on the fill, a portion of it will be transferred to the buried pipe. The magnitude of the load thus produced depends on the rigidity of the pipe, the width of the trench, the bedding, and the character of the fill material. These are important factors contributing to the strength characteristics the selected pipe must exhibit.

Normal practice, especially in the case of the more flexible pipes, is to keep trench excavation to minimum width thus enabling the backfilled pipe to transmit load to the undisturbed soil more effectively. Backfill for most rigid pipes is compacted by tamping or consolidated by flooding up to the springline. For the more flexible pipes, the backfill is normally consolidated or compacted to at least 1 foot over the top of the pipe. The remainder of the backfill can be completed with less attention given to compaction, unless it is required in places such as road crossings.

The bedding of a pipe is related to the preparation of the trench bottom (prior to pipe installation) and the type of backfill and its placement. The bedding has an appreciable effect on the pipe carrying capacity of external load. It has been experienced that angular bedding material provides better support than rounded particles, and that optimum support is provided by 1/4 to 3/4 inch crushed stone.

Most pipes and pipeline appurtenances in general use today conform to specifically selected standards covering the material used in fabrication and the actual method of fabrication, including acceptable product tolerances. The most widely accepted standards for pressure conduits and pipeline appurtenances (e.g. valves) in Canada today are the American Standards sponsored by the American Water Works Association (AWWA) and the American Society for Testing and Materials (ASTM). It is recommended that pipeline materials and appurtenances, their methods of selection and installation, should conform, wherever possible and practicable, to these standards.

The Canadian Standard Association (C.S.A.) is now issuing standards for certain types of pipes and it is worth noting that they are generally the same as American Standards.

The importance of ensuring that proper installation methods and procedures are carried out in respect of pipelines is dealt with later in this Chapter.

b. Criteria for Selection of Pipe Joint

Some of the factors that affect the choice of a pipe joint include the material of the pipe, the actual situation of the pipe to be joined, ground movement, and the nature of any adjoining appurtenances. Joints may be classified into three main categories, depending upon their capacity for movement, namely, rigid, semi-rigid, and flexible.

(i) Rigid joints are those which admit no movement at all and comprise flanged and welded joints. Flanged joints require perfect alignment and close fitting, and are frequently used where a longitudinal thrust must be taken, such as at valves and meters. Welded joints produce a continuous line of pipe, with the advantages that interior and exterior coatings when made good are not subsequently disrupted by movement of the joint.

(ii) The semi-rigid joint is represented by the spigot-and-socket caulked lead joint, which is now largely obsolescent, but is still necessary in certain repair work. Proprietary jointing compounds may be used for spigot-and-socket joints where deemed to be suitable.

(iii) Flexible joints are used where rigidity is undesirable, and are also useful where two sections cannot be welded together. They comprise mainly mechanical and rubber joints which permit some degree of deflection at each joint, and are, therefore, able to withstand vibration and earth movement. When only a percentage of the joints in a pipeline is required to be flexible, the correct arrangement is for the flexible joints to be provided in pairs with a pipe length between them.

c. Cast Iron Pipe

Cast iron pipe is a rigid conduit having good lasting qualities. There are many examples of cast iron mains which continue to give satisfactory service after a century of use. The cast iron pipe is referred to in places by as varying names as gray cast iron and spun iron, the latter name refers to the method of manufacture of the pipe by spinning or centrifugal action. Today the cast iron pipe is centrifugally cast in metal moulds. Cast iron pipe is manufactured in Canada in sizes 2 inch through 24 inch and in eleven classes of thickness. Internal pressures of up to 350 psi can be handled by this pipe.

As a rigid conduit, the cast iron pipe is designed in accordance with AWWA Standard C101-67. This requires the establishing of the following criteria:

(i) Internal pressure, including working pressure and water hammer.

(ii) External load, including earth load, superloads (truck loads), and impact of moving trucks.

(iii) Trench and bedding conditions and their contribution to soil support.

Having established the above criteria, the engineer then calculates wall thickness for two situations:

(a) Earth load plus superload and impact in combination with working pressure, and a safety factor applied;

(b) Earth load in combination with working pressure plus water hammer, and a safety factor applied.

The greatest resulting wall thickness will be selected. To this wall thickness are added a corrosion allowance of 0.08 inches and a foundry tolerance, which varies from 0.05 inches to 0.10 inches depending on diameter. It should be noted that there is no minus tolerance from the designed pipe wall thickness.

Cast iron pipe was originally used without special coating or lining. Later the cast pipe was dipped into a vat to coat and line it with a coal-tar pitch varnish or enamel. Such linings did not protect the interior surface for long from the action of certain waters and subsequent pitting and the formation of growths, or tubercles substantially increased friction and reduced carrying capacity. A method of lining with cement mortar was introduced in about 1920. This lining together with thicker, centrifugally applied bituminous linings, protects the interior of the pipe from tuberculation and preserves its carrying capacity. Most cast iron pipe now being installed is of the cement mortar or bituminous enamel lining type.

The Cast Iron Pipe Association in co-operation with the National Bureau of Standards have conducted tests over the past 10 years to examine methods of protection for metal pipes where laying takes place in severe soil conditions, such as through soils with a high acid or high alkali concentration or through garbage dumps, under fills, or other materials which are corrosive to metal pipe. The results have shown that cast iron can usually be protected from corrosion with a thin polyethylene tube drawn over the pipe. The tubing is slightly larger in diameter than the diameter of the pipe and is made from natural untreated polyethylene 8 mils in thickness. Each length of tubing is also slightly longer than the pipe it covers which allows for some overlapping of the polyethylene tubing at the joints where it is joined and sealed with adhesive tape. AWWA Standard C105-72 sets out standards for Polyethylene Encasement for Gray and Ductile Cast Iron Piping for water and other liquids.

Many types of joints are available ranging from rigid flanged joints to ball and socket joints designed for subaqueous crossings. In addition there are several types of joints that provide restraint against longitudinal separation. Push on joints allow deflections from 1 deg. 30 min. for 54 inch diameter pipe to 8 deg. 18 min. for 4 inch diameter and smaller. Greater deflection angles require special joints such as the ball and socket subaqueous pipe joints which offer deflections up to 15 deg.

Water hammer, allowances for which are included in the pressure aspect of wall thickness design, can have damaging effects on thrust-restraint structures such as concrete thrust blocks if proper design has not been carried out. Some respite can be obtained due to the fact that cast iron pipes mechanical or push-on joints have allowances for movement. For example, push-on joints in diameters 12 to 36 inch can move from 0.7 to 1.7 inches without disengagement and mechanical joints in the same diameters can similarly move 1.44 to 2.62 inches.

Proper job site storage of material should include protection of the pipe from contamination as well as from structural damage. The manufacturers recommend that 4 by 4 inch timbers should be placed between tiers with chocks anchored at either end to prevent movement. Specific stocking heights can be obtained from the manufacturer and these should be enforced by the construction supervisor.

d. Ductile Iron Pipe

Ductile Iron and Cast Iron pipes are two different materials with respect to engineering and construction. Whereas the latter is a rigid pipe structure, Ductile Iron is a flexible conduit. The centrifugally cast ductile iron pipes, which were developed only about 10 to 20 years ago, are made from a material produced by treating a relatively pure molten grey iron with magnesium. "Ductile Iron" is the name applied to spheriodal graphite or metal iron when it is centrifugally cast into pipes. It is manufactured in Canada in sizes 4 inch through 24 inch in accordance with C.S.A. Standard B131.13 (AWWA Standard C151-71) and has rated working pressures of up to 350 psi.

The design of ductile iron pipe follows the procedures for flexible conduits set in AWWA Standard H3-71. Bedding conditions, including bedding angle and sidefill support, become increasingly important as the diameter-thickness ratios increase.

After the establishment of the internal pressure and external load criteria, as in the case of cast iron pipe, the engineer calculates wall thickness for the following situations:

(a) Earth load plus superloads and impact with no consideration to internal pressure, and a safety factor applied;

(b) Internal pressure plus a surge or water-hammer allowance of 100 psi, and a safety factor applied.

The greater resulting wall thickness will be selected wall thickness.

Because of its more recent origins, much less historic data exists about the durability and long term qualities of ductile iron pipe than of cast iron. Results of tests on ductile iron pipe of early manufacture claim to have clearly established that the corrosion resistance of ductile iron is at least as good as that of cast iron and is possibly better. More recent work claims in fact that ductile cast iron has an advantage over cast iron in this respect. From an extensive study of available information from British, European, and U.S. tests, A.G. Fuller[2] of the British Cast Iron Association concluded that the ratio of attack ductile to cast iron is of the order of 0.65:1 when the two materials are exposed to the same environments.

The standard outside coating on ductile iron pipe for general use under all normal conditions is usually a bituminous coating of coal tar or asphalt base of approximately 1 mil thickness. Polyethylene encasements similar to that described for cast iron pipe would appear to offer the pipe adequate protection in those exposures known or suspected of being highly corrosive. The standard lining usually supplied is a 1 mil thick bituminous coating similar to the outside surface coating. It is also normal practice now-a-days to specify a cement mortar lining with a bituminous seal if it is suspected excessive tuberculation will take place, and thus preserve the carrying capacity of the pipe.

Aspects of joints and job site storage requirements for ductile iron pipe are not significantly different from those for cast iron pipe.

e. Concrete Pipe

The last 20 years, of the 50 year history of the concrete pipe industry, have brought about a rapid increase in the use of the concrete pressure pipe in the water supply field. This trend has been primarily due to new developments in design and manufacture as well as by the introduction of new types of pipe.

Five distinct types of concrete pressure pipes are now in use. Three of these contain a steel cylinder and either reinforced or prestressed concrete whilst the other two are characterized by the absence of the cylinder and are composed solely ofeither reinforced or prestressed concrete.

The five different types may be listed as:

(i) Non prestressed concrete cylinder pipe (AWWA StandardC-300);
(ii) Prestressed concrete cylinder pipe (AWWA Standard C-301);
(iii) Pretensioned concrete cylinder pipe (AWWA Standard C-303);
(iv) Non cylinder concrete pipe, non prestressed (AWWA Standard C-302);
(v) Non cylinder concrete pipe prestressed (No Standard designation).

The non cylinder concrete pipe prestressed ((v) above) is not in general manufacture in Canada and in fact the use in the United States is not yet great enough to justify the preparation of a standard. AWWA Standard C-300 cage and cylinder pipe can be manufactured but this type is being used less frequently now with the introduction of the prestressed cylinder pipe (AWWA Standard C-301). The reinforced concrete pressure pipe(AWWA Standard C-302), although manufactured nationally in diameters 24 inch to 144 inch, is rated for only a working pressure of up to 45 psi, and is intended for use in low head transmission lines in irrigation, industrial and domestic water supply schemes, sanitary and storm sewers, and culverts. As such, it is not appropriate for use in the regional conveyance mains.

This leaves the choice to be made from the prestressed concrete cylinder pipe (AWWA Standard C-301), and the pretensioned concrete cylinder pipe (AWWA Standard C-303). The former is made in two general types; pipe having a steel cylinder lined with a concrete core, and pipe having a steel cylinder embedded in a concrete core.

The lined - cylinder type is made in diameters of 24 inch, 30 inch, 36 inch, 42 inch, and 48 inch, and has a working pressure range of up to 250 psi. The pipe consists of a welded steel cylinder with sized joint rings welded to the cylinder, a centrifugally cast concrete lining prestressed by high tension wire wrapped directly over the steel cylinder, and a dense cement mortar encasement over the steel cylinder prestressed. The pipe is manufactured in standard classes, each corresponding to a wide variety of possible combinations of internal pressure and external loading. An allowance for water hammer is accounted for in the design. The pipe normally comes in 20 feet and 24 feet length for 24 inch, 30 inch, and 36 inch diameter pipes, and in 20 feet for the 42 inch and 48 inch diameter.

The embedded - cylinder type consists of a welded steel cylinder with sized joint rings welded to the cylinder, embedded in a cast concrete core, which is then prestressed with high tension wire. A dense mortar encasement is applied over the prestressing wire. The pipe is manufactured with two types of core, a heavy core and a standard core. The standard core is made in diameters 48 inch and upwards and has a working pressure range up to 250 psi. The heavy core on the other hand comes in diameters 24 inch to 72 inch with incremental sizes of 6 inches. The manufactured length is 16 feet. Design alternatives are the same as for the cylinder type.

The prestressed concrete cylinder pipe is a rigid conduit. As such, its design is based on combined loadings of internal pressure and external load for the situations previously described under Section IV.I.C.

The pretensioned concrete cylinder pipe is made in diameters of 14 inch to 30 inch with a rated working pressure of up to 400 psi, and is manufactured in 16 feet lengths. It is a semi-rigid pipe and care needs to be extended in providing proper bedding and backfill particularly with the larger pipe sizes. These are important factors in developing the full external load carrying capacities of this type of pipe. The pipe consists of a welded steel cylinder with sized joint rings welded to the cylinder, a centrifugally cast concrete lining , circumferential reinforcing rods wound under measured tension around the steel cylinder, and a dense cement mortar encasement over the steel cylinder and rod reinforcement.

In the cylinder type pipe the normal joint used is the bell and spigot. The spigot ring is a hot rolled section integral with the cylinder. The metal bell plate is also of steel forming an extension of the cylinder. A rubber gasket seal completes the joint. The annular ring spacing on the inside and on the outside is filled with 1:1 mortar in the field after the joint is made, to give added long term protection.

The manufacturers claim that the useful life span is indefinite due to the fact that the properties of buried concrete improve with the passage of time. They also state that the original carrying capacity is maintained over time since the concrete lining is not subject to corrosion or tuberculation. The pipe is stated to be not subject to corrosion, electrolysis or structural hazards, nor is cathodic protection required. In spite of this it is felt that in highly corrosive or acidic environments some form of bituminous protection should be given to the exterior pipe walls. Subaqueous joints, and the normal range of special pipes and fittings are manufactured in concrete pipe. Joint deflections achieveable are considerably less than for cast or ductile iron pipes.

In the case of concrete pipe it is normal practice for the engineer to furnish the manufacturer with plans and profiles, showing alignment and grade of the area along the pipe route; the location of all outlets, connections and special appurtenances; the design head or the required cross-sectional area of effective circumferential reinforcement per foot of pipe wall for each portion of the pipeline; and the specific special details that pertain to the job. The manufacturer will then provide a detailed layout drawing and schedule of all pipes and fittings to be checked and approved by the engineer before commencing manufacture. By indicating the fittings or outlets that may be located a few feet on either side of the position shown on the profile, one may effect considerable economy of installation.

Wherever possible, outlets for future connections to a concrete pipeline should be incorporated in the pipe during manufacture. The cost of doing this is 40 to 70 percent less than the cost of pressure tapping the pipeline when the connection is made in the field.

Concrete pipe is commonly used on long large diameter conduits, but is rarely used for relatively small distribution systems.

f. Asbestos-Cement Pipe

The asbestos-cement pipe is made of a mixture of asbestos and Portland cement, compressed by steel rollers to form a laminated material of great strength and density. The asbestos fibre is thoroughly mixed with the cement and serves as reinforcement.

Pipe of this character has been used for many years for carrying water in Europe. The pipe is manufactured in Canada in diameter 4 inch through 36 inch and in three classes, 100, 150 and 200. These class numbers also refer to the respective operating pressures. The pipe is considered a rigid conduit, and thus, in selecting the pipe class the combination of internal pressure and external load as outlined previously in Section IV-1-C should be considered. AWWA Standard C401 presents selection curves for this purpose.

Some advantages of asbestos-cement pipe include its high resistance to most corrosive conditions, and its freedom of electrolysis. Among the disadvantages of asbestos-cement may be included its relatively low structural resistance to flexural stresses that can cause breakages when the pipe is improperly bedded, moved or undermined under pressure, and the ease with which the pipe can be punctured by excavating tools.

Asbestos cement pipe is assembled by means of a special coupling which consists of a pipe sleeve and two rubber rings which are compressed between the pipe and the interior of the sleeve. The joint is as resistant to corrosion as the pipe itself and is flexible enough to permit as much as 5 deg. per joint deflection in laying pipe around curves.

In terms of construction ease its relative lightness in weight is a considerable advantage. Field cutting is extremely easy and it can be drilled and tapped for connections, but has not the same strength or suitability for threading as iron, and any leakage at the thread will become worse as time passes. In view of this difficulty it is fairly common practice to fix malleable iron or phosphor saddles at the point of connection and to screw the ferrules into them. The sounds of leakage do not readily travel along asbestos cement pipe and the usual methods of leak detection are not, therefore, so effective.

g. Steel Pipe

Steel water pipe is one the of oldest continuously available water utility materials, and the development of welding techniques has increased its usage, particularly in the larger sizes. The material is manufactured in Canada in sizes up to at least 90 inch diameter.

The modern steel pipe for underground transmission and distribution service is a composite pipe, made of three components - the structural steel cylinder; the internal protective lining; the external coating and protection system.

Steel pipe is considered to be a flexible conduit. As such, its design will basically follow the procedure outlined previously under section IV-1-d.

Steel pipe is specified to conform to AWWA Standards (AWWA C201) or ASTM Standards (ASTM A139). The latter provides more latitude in manufacturing and, therefore, does not limit competition.

The internal protective lining, which is required for corrosion protection and the prevention of tuberculation growth and subsequent hydraulic efficiency declination normally consists of a coal-tar-enamel or epoxy lining, or a cement mortar lining. The former has been in use for some 60 years and current standards reflect the improvements made in both application procedures and material. Cement mortar lining can be carried out in either the manufacturer's plant or in the field by way of recognised patent processes.

The external coating and protection system consists of a coal-tar enamel coating wrapped with an asbestos felt shield to prevent soil stresses on the enamel. The coal-tar enamel has an extremely high resistance to the flow of direct electric current present in a corrosive environment, and acts to insulate the pipe from the environment. A cathodic protection system should be incorporated with external coating and wrapping practices. An alternative to this external protection system is the field application of a plain concrete surround to the pipe.

Joints can be either a bell and spigot type joint using an O-ring rubber gasket, or a field welded joint. The former is designed to provide deflections on the order of 3 to 5 deg. and is designed to be pressure tight under all conditions of underground service.

Two types of field welded joints are in common use - the slip-bell or lap-weld joint, and the butt weld joint. In the former joint a fillet (either single or double) is placed in the annular space provided by the bell and plain spigot.

The handling and installation of steel pipe demands no special procedures beyond those required by what would be considered good construction practice. Fittings and specials are available in standard dimensions, however, they are often tailor-made to suite site conditions.

h. Polyethylene Pipe

Polyethylene pressure pipe was first used in England some 25 years ago. Today it is marketed by DuPont of Canada, under the trade name "Sclair pipe". The pipe is manufactured of high strength polyethylene resin in diameter ranging up to 40 inches O.D. and in working pressures up to 160 psi. In the larger diameter, however, the working pressure is much less than 160 psi (45 psi. in the 40 inch diameter pipe).

Polyethelene pipe is considered to be a flexible conduit. As such, its design will basically follow the procedure outlined previously in Section IV-1-d.

Canadian Standard 41-GP-25 deals with polyethylene pipe, 8 inch diameter and larger.

Jointing of the pipe is by the butt-fusion method. This method is mainly used for connecting straight lengths of pipe, for attaching adapters such as stub ends, for mechanical fittings and for terminating. No extra filling is involved in connecting straight lengths. The preferred method of terminating "Sclairpipe" to fittings, valves or other pipe is by flanging. Flanges are also used to connect lengths of pipe when butt fusion is impractical. Polyethylene pipe in all sizes is sufficiently flexible that elbows of up to 45 deg. are not needed; bends of this magnitude are made cold during installation.

i. Failure of Mains

Failures may be caused by vibration, drop in temperature of the water, earth movements due to frost, or subsidence of soft soil (e.g. bog), insufficient ramming of the backfilling under the haunches of the pipe, excessive ramming with a power rammer, and unequal support of the bed. To prevent mains from failure, the following measures should be considered.

(i) Pipe Bed and Support - Care in preparation of the pipe bed, and the provision of adequate support, is absolutely essential so as to ensure even bearing on the ground, especially in the case of rock.

Precautionary measures should be taken to prevent water from running down the bottom of the trench, as it is liable to wash away part of the bed or packing material (especially on steep gradients).

(ii) Thrust Blocks and Anchorages - Thrust is exerted at bends, tapers, tees, and dead ends when a pipe is under pressure. It is usual to add an allowance for surge to the thrust due to the static head of water. A horizontal bend requires a thrust block, conveniently formed by placing a sufficient mass of concrete between the pipe on the outside of the bend and the side of the trench, which may have to be enlarged to enable this to be done.

The vertical face of the thrust block is made of sufficient area for safe bearing pressure, utilizing the passive resistance of the ground. Similarly, at a vertical "looking-up" bend it may be necessary to carry the downward thrust to a foundation of concrete; or conversely, at a bend "looking-down", it may be necessary to surround the pipe by a block of concrete, if the combined weight of the pipe and water in it is insufficient to overcome the upward thrust. It is advisable if possible to avoid sharp bends, and in soft ground it is better not to put two bends together, but to separate them by a length of straight pipe.

Where welded joints are used, full anchorage is not generally necessary, since the longitudinal continuity of the pipe is capable of distributing the forces into the ground.

Pipes laid on steep inclines should be anchored by transverse blocks.

(iii) Flexibility - One of the best ways of preventing pipe bursts due to movement of the ground is by the use of flexible joints.

Whether the movement is caused by frost, traffic, or soft soil subsidence, flexibility is of great advantage. Similarly, where difficult foundation conditions arise, such as at a patch of soft ground or the termination of hard ground, an increase of flexibility by using joints made with rubber, and even by cutting pipes and inserting additional flexible joints, is a form of treatment worthy of comparison with alternative schemes such as concrete or hardcore filling.

2. Appurtenances for Pressure Pipes

a. General

A large number of different types of valves are required for the proper functioning of a pipeline.

Included here is a description of their function within the pipeline system, their operation principles and a guide to selecting locations for them. The valves described are all manufactured in Canada and the criteria for the selection of the individual manufacturer should be made on cost of item, proven reliability of operation and after sales service from the manufacturer, provided of course they all meet the requirements considered to be necessary for their purpose.

It would be highly desirable if within the Regional Water Agency some measure of standardization of make of component could be achieved. This would be reflected in substantial economies in terms of the number of spare parts requiring to be kept on hand by the agency and familiarity of the maintenance aspects of the equipment by the staff of the agency, thus saving many man hours over a long period of time.

In the selection of a valve some of the characteristics to be considered are:

(i) Purpose;
(ii) Operation, whether manual, power or automatic;
(iii) Capacity;
(iv) Characteristics of the fluid to be passed, its corrosiveness, temperature and viscosity;
(v) Pressures to be controlled;
(vi) Head loss and rate of flow at various percentages of opening;
(vii) Sensitivity in control of rate of flow;
(viii) Cost and operational economies;

(ix) Availability;

(x) Ruggedness;

(xi) Simplicity of construction and repair;

(xii) Suitability for water hammer, noise vibrations, and the like.

b. Isolating Valves

Isolating valves are used to regulate the flow in the system. Two basic types are commonly used - gate valves and butterfly valves. Gate valves are designed to be either the rising stem type or the double disc type. Valves used on distribution systems are usually cast iron, with brass mountings, and the rising stem type is normally preferred where the valve is accessible for inspection and the threads can be lubricated. The non-rising stem is suitable for underground conditions and others where the stem is protected within the bonnet of the valve. Butterfly valves are now being more widely used because of their ease of operation, low cost, compact size, reduced size of valve chamber, improved closing and retarding characteristics compared with the gate valve and the absence of sliding parts.

Large valves should be preferably housed in a well ventilated dry environment chamber to preserve them and to facilitate maintenance. This also applies to all other large expensive valves such as pressure reducing valves, and air or pressure relief valves. Valves on large mains are usually of a smaller size than the main itself. The saving on valve cost must be balanced against the increased head loss through the valve. Common practice is either to make the valve one size smaller than the main line or to make it three quarter of that of the main. Valves should always be provided at the point of connection of one main to another and, if that other main can be fed in either direction, a valve on each side of the branch may on occasions make the difference between interruption and maintenance of service to the consumers. The installation of valves is expensive, and judgement is required in selecting their number and position, but many schemes have been carried out with too few valves in order to cut down on the original cost to the detriment of efficient operation.

c. <u>Air Valves</u>

Air valves are fitted to release the air automatically when a pipeline is being filled, to permit air to enter the pipeline when it is being emptied, and to release any entrained air which might accumulate at high points in the pipeline during normal operations. Depending on their function they can be called air-relief or air-inlet valves. Air valves require care in selection and even more care in siting, and it is good practice to plan the pipeline with a view to avoiding air troubles altogether.

It is not sufficient just to position air valves at pipeline peaks, and a special study to deal with air problems is necessary at the design stage of pipelines.

An alternative method sometimes used is to tap the main with a service connection and bring the service pipe to the surface and locate a stop cock in a small chamber. This is of course manually operated and is more applicable to small diameter mains. Air valves are necessary on conveyance mains and large distribution pipes but it is unusual to fix them on service mains as entrapped air is released through statutory fire hydrants and service pipes.

Air valves are mainly available in two forms, either single-ball or double-ball. The single-ball type can have either a large orifice or a small orifice, the former being only suitable for emptying and filling of pipelines, and the latter for discharging small quantities of entrained air. Double air valves are available, which can be classified as dual purpose with a large orifice and a small orifice in one unit, with a common connection to the main.

d. Blow-off Valves

Drain or blow off valves (wash out valves) are necessary at low points in the pipeline to allow the pipe to be drained for repair or inspection, or to allow the line to be flushed out. Normally a gate or butterfly valve teed off the main line and a discharge pipe to a watercourse will serve as a blow off unit.

e. Check Valves

Check valve is the standard term to cover the various types of valves which automatically prevent reversal of flow in a pipeline.

The main consideration in choosing a check valve is to ensure that its closure will not set up excessive shock conditions within the system. To achieve this the valve should be so designed as to allow the door to close as quickly as possible thus limiting its inertia effect.

f. Pressure Reducing Valves

Pressure reducing valves are for the purpose of automatically maintaining a reduced pressure within reasonable limits in the downstream side of the pipeline. This type of valve is always in movement and like all mechanical equipment it will require scheduled maintenance on a regular basis. This work is facilitated if the valve is fitted on a by-pass, with isolating valves to permit work to proceed without taking the main out of service. Alternatively if the pressure reducing valve is fitted on the main-line, a by-pass can be installed to serve the same purpose.

g. Pressure Sustaining Valves

Pressure sustaining valves are similar in design and construction to pressure reducing valves, and are used to maintain automatically the pressure on the upstream side of the pipeline.

h. Pressure Relief Valves

Pressure relief valves are automatic devices arranged to discharge water in order to relieve the pressure in a mains system.

The pressure control valves described above are used to control pressures into and out of pressure zones, and various combinations of them will be required in the regional water system conveyance network.

V. CONSTRUCTION METHODS AND THE IMPORTANCE OF CAREFUL CONSTRUCTION SUPERVISION OF THE WORKS

Two main areas which could cause a water distributing agency concern, with respect to the level of service to the consumer, pertain to the production of a lower than acceptable water quality, and to the interruption in the delivery of the product to the customer.

Water quality control can be successfully taken care of by careful engineering design and construction supervision of a water treatment facility, specifically engineered to handle the raw water constituents encountered, and to produce treated water of the quality standards desired. Thereafter, through the establishing and operation of daily plant control and maintenance procedures, as well as periodic watershed surveys, the water quality would be preserved.

The distribution of a potable water supply to the customer requires the successful operation of system delivery components, that is - pumps, supply pipe and appurtenances. In the situation where operational failure of main delivery pumps occurs, stand-by pumping capacity is used to maintain supply. Proper design selection of the pumping units and their regular maintenance can reduce this cause of delivery interruption to a minimum. Major appurtenances, likely to require routine mantenance or parts replacement, are normally housed to facilitate these operations.

The pipeline, which is the major system component, is entirely inaccessible after installation. Therefore, it follows that this component in particular should receive more than adequate consideration in terms of selection, and also in respect of careful construction procedures. It is also a fact that the pipeline usually forms the major capital investment in water works undertakings. The interior walls of the pipe itself can be maintained in good condition through initial design selection, maintenance of good quality water standards, and the operation of a regular flushing out program.

Since most interruptions in supply delivery usually are caused by pipe breakages, it would seem appropriate to examine the results of studies carried out in this respect. We should note here that, service reservoirs are located in the regional community to take care of emergencies such as the pipeline break, but it is nevertheless economically sound to try to restrict the number of conveyance main breaks to a minimum.

One such study was carried out in 1968 by Quinton and Kulak of the Atlantic Industrial Research Institute, Halifax, for the City of Dartmouth and the Public Service Commission of Halifax under the title "Water Main Failures in the Halifax - Dartmouth Area". Quinton and Kulak's procedural steps were:

(a) To examine all available records of water main failures in the area with regard to type of failure, length of service, pipe size and foundation conditions.

(b) To examine all available literature on water pipes and pipe failures.

(c) To examine all new breaks which might occur during the period of the investigation.

(d) To conduct performance tests on samples of the water pipe being used.

(e) To inspect current installations of water mains.

(f) To investigate supply procedures in shipping, handling and storage of the pipe.

Based on the findings of this study, conclusions of significant importance were as follows:

(i) The major cause of main breaks in the Halifax - Dartmouth area is the result of high bending stresses produced by a combination of bad laying conditions and poor laying practices.

(ii) There should be a tightening of control over watermain installations to ensure that specifications are being followed.

(iii) Care should be taken to see that the fill disturbed between sewer and watermains, when a service connection is made, is properly compacted during backfilling.

In view of the significance of the need to prevent future watermain breaks taking place in the Regional Water System, the conclusions of this study only serve to demonstrate that perhaps the most important aspect, and incidentally the most difficult to control aspect of the construction, is the need to ensure that proper pipe line installation procedures are strictly enforced.

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