ACKNOWLEDGEMENT

I wish to express my deep appreciation and gratitude to my supervisor, Dr. M.S.S. Nasser, whose critical but always encouraging guidance made this study possible. I can sincerely say that I have never known an educator so completely devoted to his students.

I am greatly indebted to the offices of the consulting engineers, "Lalonde, Girouard, Letendre and Associates", for putting at my disposal the expertise of the engineers and the technicians from their computer division.

C. Kochar

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CHAPTER 1

INTRODUCTION

In affluent countries like Canada, an adequate, dependable water supply has always been taken for granted. Indeed, fresh sources of water supply have been relatively easy to tap and hence the population has become used to a rather generous supply of good quality water.

However, during the last decade or so, with the explosion of the industrial sector and the consequent pollution of water sources, the municipalities have been forced to take a second look at their existing water supply networks. Instead of spending enormous sums of tax dollars into tapping new sources, municipal engineers have attempted various methods to maximize the output from an existing source. These endeavours generally recognize the need for a better comprehension of the hydraulics of a water distribution system and the necessity of a thorough planning of future extensions and/or improvements. (2)

The various companies marketing computer programmes have introduced different packages for the analysis of water distribution networks.(8) Most of these programmes require tedious procedures for constructing the initial models, e.g., any change such as the addition of a pipe or a booster pump, would warrant reestablishing the loops, nodes and circuits. Furthermore, any change in the input conditions (water discharge, etc.) necessitates a new simulation. While these drawbacks represent definite problems in the available programmes, the main difficulty lies in the preparation of the input date. This is due to the fact that these packages require the construction of certain loops from the existing water supply networks; the water demands are assigned and the analysis is based on assumed pipe criteria. Thus, a few complications arise, e.g. which pipes to retain for the loops; how to calculate the water discharges at the nodes; how to arrive at the proper pipe constants; how to analyse the results from the computer simulation. The present study tackles these problems and introduces a comprehensive, systematic methodology for the model construction and analysis. To ensure the effective elimination of some of the deficiences encountered in the existing packages and to optimize computer time, the expertise of the computer specialists of "Lalonde, Girouard, Letendre and Associates" was sought. The principal objective was to determine the capacity of the various elements of an integrated water supply system, sufficient to feed, qualitatively (pressure) and quantitatively (discharge), the present and future population of the region for which it is designed.

The preparation of a master plan for a water supply network for a small municipality requires the compilation of the existing data and a hydraulic analysis of the projected extentions. In order to facilitate these operations, the preparation of the master plan was based on the following sequence:

1. Study of the physical features of the region.
2. Inventory of the existing equipment.
3. Population distribution and demographic studies.
4. Study of water demands.
5. Construction and verification of the model network.
6. Hydraulic analysis.
7. Master plan.

The small town of Mercier, located on the St. Lawrence river, just south of Chateauguay in the Province of Quebec was utilized as a test district to check the model developed. For the past two years, this town has been experiencing water supply problems especially during fires. Its water distribution network is interlinked with that of Chateauguay with the common source of water supply being located in the town of Chateauguay.

CHAPTER 2

PRELIMINARY STUDIES

Several factors have to be studied in order to formulate an optimal master plan for the water distribution network. To develop a comprehensive but not superfluous methodology, the most important of these factors were considered in the present study and are outlined below.

1. Study of the physical features

   The aim of this study was to understand the physical characteristics and the topography of the region and their possible direct or indirect effects on the master plan (pumping stations, pressure reducing valves, etc.).

2. Inventory of the existing equipment

   This is an important phase of the methodology as it is both a qualitative and quantitative evaluation of the existing network that will eventually influence the recommendations on the future constructions to be executed.

This phase consisted in tracing the water distribution network on the same scale as the topographic plan. The diameter, material and year of construction were marked on each pipe. In addition, a table was prepared giving the characteristics of the existing pumping stations, reservoirs, etc. (see table 1).

3. Population distribution and demographic studies

a) Existing distribution

With the help of the zonage plan and visual site inspections, a population distribution plan was prepared (see fig. 1).

On this plan, the various types of distribution are classified as follows;

Residential (1 to 10 pers/acre)
Residential (11 to 20 pers/acre)
Commercial
Green space and agricultural

These different types of distribution were separated into developed and undeveloped areas (see table 2).

b) Demographic studies for future distribution

Before establishing the future population that serves as a base for the balancing of the distribution network, it was necessary to study the evolution patterns of population in the test district over the past few years. Projections for future population were made based on these evolution patterns. Numerous methods are available for predicting future population including the arithmetic progression, logistic growth, uniform percentage growth, curvilinear growth, ratio method, etc. (13). The ideal method depends on the region being studied. The town of Mercier is still in its infantile stages of development and the population increase is still predictable. Thus, the population growth was assumed to follow a set pattern and hence the method of modified curvilinear growth projection by extrapolation was used (see figure 2 and tables 3 and 4).

Having determined the future population figures, the next step was to distribute them over their relative areas on the zonage plan. This process consisted of two stages. First, the density within the zones actually developed was increased proportionately and secondly, new territories were assigned in compliance with the requirement of the municipality of Mercier. This is shown in table 5.

4. Study of water demands

One of the most important aspects to be considered in the planification of a water distribution network is the quantity of water required to satisfy the needs of the population to be served over the years covered by the study.

The different types of needs considered in this study are:

- Residential
- Lawn watering
- Commercial
- Wastage (leaks)
- Public usage
- Fire protection

a) Establishing the average consumption

The present and future "annual average daily flows" were established with the help of municipal water statistics. The authorities at the town of Mercier (the test district) regularly record the water consumption at the meter station between the towns of

Chateauguay and Mercier (see fig. 3). These readings were totalled over the years and averaged over 365 days. This figure represents the "annual, average daily flow" in MGD. (13)

The annual average flow for 1976 was found to be 437400 gallons/day (see table 6). As there are presently no industries in Mercier, this figure represents the residential and business consumptions.

Table 7 shows the unit consumptions established for the test district based on a study of population distribution and the average flow. The total consumption breakdown is shown in table no. 8 for 1976 and in table 9 for 2000.

b) <u>Future average flow</u>

It is now generally accepted that with a rise in the standard of living, the consumption of water per capita increases for almost all communities (5). For the present study an annual increase of 0.5 IG/day/person was assumed. The other types of consumptions were considered constant.

The future average flows were then determined based on population projections and the future per capita consumptions.

c) Fluctuations in the water demand

The fluctuations in water demand are caused by variations in the daily or hourly demand with respect to the average demand. These were established with the help of certain coefficients representing factors for the average, peak and low consumptions of the maximum day (see table 10).

These factors are considered generally constant for a certain region (2, 5 and 13).

Aside from fire conditions, the three main critical periods for a water network are:

- the flow for the maximum day
- the flow for the maximum hour of the maximum d
- the flow for the minimum hour of the maximum d

The flow for the maximum day for the town of Mercier for the year 1976 was 699000 IGD. The factor as compared to the average flow was found to be 1.6 and is, therefore, within the general limits of 1.2 and 4.0. (2)

Table 11 gives the unit consumptions for each type of consumer in the test district along with the factors considered to get the flow for the maximum day. Table 12 gives the breakdown for the total flow for the maximum day for 1976 and table 13 shows the projected figures for the year 2000.

d) Peak demand during the maximum day

The flow for the peak demand during the maximum day for the test district, for the year 1976, was 1136000 IGD. Hence the factor from the average flow is 2.6, which is not far from the common limits of 2.7 to 18.0. (2)

Table 14 shows the possible variation for the various consumers during this period and table 15 shows the actual flows for the same period. Table 16 shows the projected figures for the year 2000.

e) Minimum consumption of the maximum day

This condition also represents an important criterion for balancing the water network because if there is a floating reservoir on the system which has to be filled, this is the period during which the network is critically strained. For Mercier the minimum flow for maximum day in 1976 was 481000 IGD. The factor as compared to the average flow is this 1.1, within the general limits.(2)

The variations by the different consumers are shown in table 17 for this period. Table 18 shows the actual consumptions during this period and table 19 shows the projected figures for the year 2000.

f) Study for the reserve

The reserve is defined as the volume of water instantaneously available in case of emergencies (fires, breakdown at source, etc.).

Equilibrium reserve

This reserve should be enough to balance the network during those periods when the consumption exceeds the supply of the maximum day, i.e. the reserve supplies the difference. This reserve was calculated from the consumption curve for the maximum day. The difference between the areas above and below the average consumption line gives the percentage that when multiplied by the average consumption of the maximum day, yields the equilibrium reserve.(2, 13) Table 20 gives this reserve for the years 1976 and 2000.

Fire reserve

This reserve was determined to adequately meet the requirements of the "Groupement Technique des Assureurs". Hence the fire reserve calculations were done according to the "Guide for determination of required fire flow". (9)

With these norms the fire flow for the test district was calculated as 1000 IGM for a duration of 2 hours. The total fire reserve is, therefore, 120,000 imperial gallons and was assumed to be the same for the year 2000 (see table 20).

Emergency reserve

This reserve is necessary during unforeseen circumstances such as breakdown of a feeder, interruptions in the pump operation, etc. The emergency reserve recommended for such situations is 6 hours out of 24 hours of the average flow for the year.(10)

In other words, this is calculated to feed the town for 6 hours of the average consumption period. Table 20 gives this reserve for the years 1976 to 2000.

6. Hydraulic analysis

This is dealt with in detail in the following chapters.

7. Master plan

The master plan resulting from this study is shown in figure 4. It shows the additional pipes to be constructed in order to balance the existing water distribution network (fig. 3) to the year 2000.

CHAPTER 3

FORMULATION OF THE MODEL

The ultimate goal of this study is to determine, within reasonable accuracy, the discharge passing through each pipe of the network. Based on discharge figures, head losses under different water demand conditions could be calculated for each pipe line. The resulting pressures at various nodes yield the approximate pressures available to consumers in each distribution centre and demonstrate the degree of functioning of the equilizing reservoir. The computer model was based on the Hardy Cross method. (2, 13)

1. Construction of the model

The various studies required for the development of the model (demographic studies, water demand, etc.) have already been explained in the preceding chapter. The model itself was constructed from an existing detailed water distribution plan of the sector (Fig.3).

All the major feeders were retained for each developed sector of the town. Some secondary feeders were also retained where a distribution loop had to be closed. The pumping stations and the floating reservoir were superimposed at their respective positions. The details (nodes, diameters, lengths, elevations, etc.) were then included for each distribution centre.

2. Coefficient of friction

The coefficient of friction is a particularly important parameter in the hydraulic equations used to balance the network model. In this study, the Hazen Williams equation (2, 13) was utilized in the analysis. The coefficient of friction can vary from 150 for a new concrete finished pipe to 40 for a very old iron pipe. A poor estimate of this cooefficient may lead to a significant error in the calculations.(3)

For the model used in the present study, municipal statistics were employed to assess the age and material of the different components. The local conditions such as bends, etc. had to be taken into account. A preliminary choice was made with this information and the resulting model was analysed. Thereafter, a field study was conducted on the test district and actual pressures were recorded. The model was subsequently modified to represent a more realistic picture. Table 21 shows the coefficients initially selected.

3. Theoretical considerations

The Hardy Cross method was used in the computer model to balance the existing network and subsequently the projected network.

This method consists in assuming a distribution of flow in the network and balancing the resulting head losses or vice-versa. (13) Pipe-flow formulae are used to determine the head losses and successive corrections are made in the flows until the heads are practically balanced.

Generally, the head loss, h, in any pipe is

$$h = KQ^x \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where Q is the discharge, K is a constant depending upon the size of the pipe, its internal condition and the units used; x is an exponent depending on the flow formula used.

Following the Hardy Cross analysis, it can be said of any pipe in a circuit that

$$Q = Q_1 + \Delta \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which Q is the actual amount of water flowing, Q_1 is the assumed amount and Δ is the required flow correction. Then substituting equation (2) in equation (1),

$$KQ^x = K(Q_1 + \Delta)^x = K(Q_1{}^x + xQ_1{}^{(x-1)} \cdot \Delta + \quad) \ldots (3)$$

The remaining terms in the expression may be neglected if Δ is small compared to Q.

For a pipe circuit, $\sum KQ^x = 0 \quad \ldots\ldots\ldots\ldots\ldots (4)$

Hence equation (3) becomes,

$$\sum KQ^x = \sum KQ_1{}^x + \sum xKQ_1{}^{(x-1)}\Delta = 0 \quad \ldots\ldots\ldots (5)$$

Simplifying,

$$\Delta = - \frac{\sum KQ_1{}^x}{\sum xKQ_1{}^{(x-1)}} \quad \ldots\ldots\ldots\ldots\ldots (6)$$

The steps followed in the computer program are as follows:

a) An initial distribution of flow as to amount and direction is assumed.

b) The head loss in each pipe is computed by means of the Hazen Williams equation.

c) With due attention to sign, the total head loss around each circuit is computed as $\sum h = \sum KQ_1^{x}$

d) For the same circuit, the sum $\sum K_x Q_1^{(x-1)}$ is computed without regard to sign.

e) To balance the head loss in each circuit, the terms obtained in (c) and (d) are substituted in the following formula:

$$\Delta = \frac{-\sum KQ_1^{x}}{\sum xKQ_1^{(x-1)}}$$

4. Analysis of the proposed network

An ideal water distribution network system should be able to feed water to each consumer between predetermined maximum and minimum pressures. Of course, these two limiting pressures depend on the type of consumer. For simplification, the limits were assumed as being 20 psi and 110 psi as the test district ~~is not~~ expected to have special consumers or even industrial consumers during the period of the present study.

Several trial pipes are introduced in the model with different diameters and the process of network balancing continued until the desired pressures at all points in the model were obtained. A flow chart describing the model is illustrated in figure 5.

5. Highlights of the proposed computer model and the network analysis:

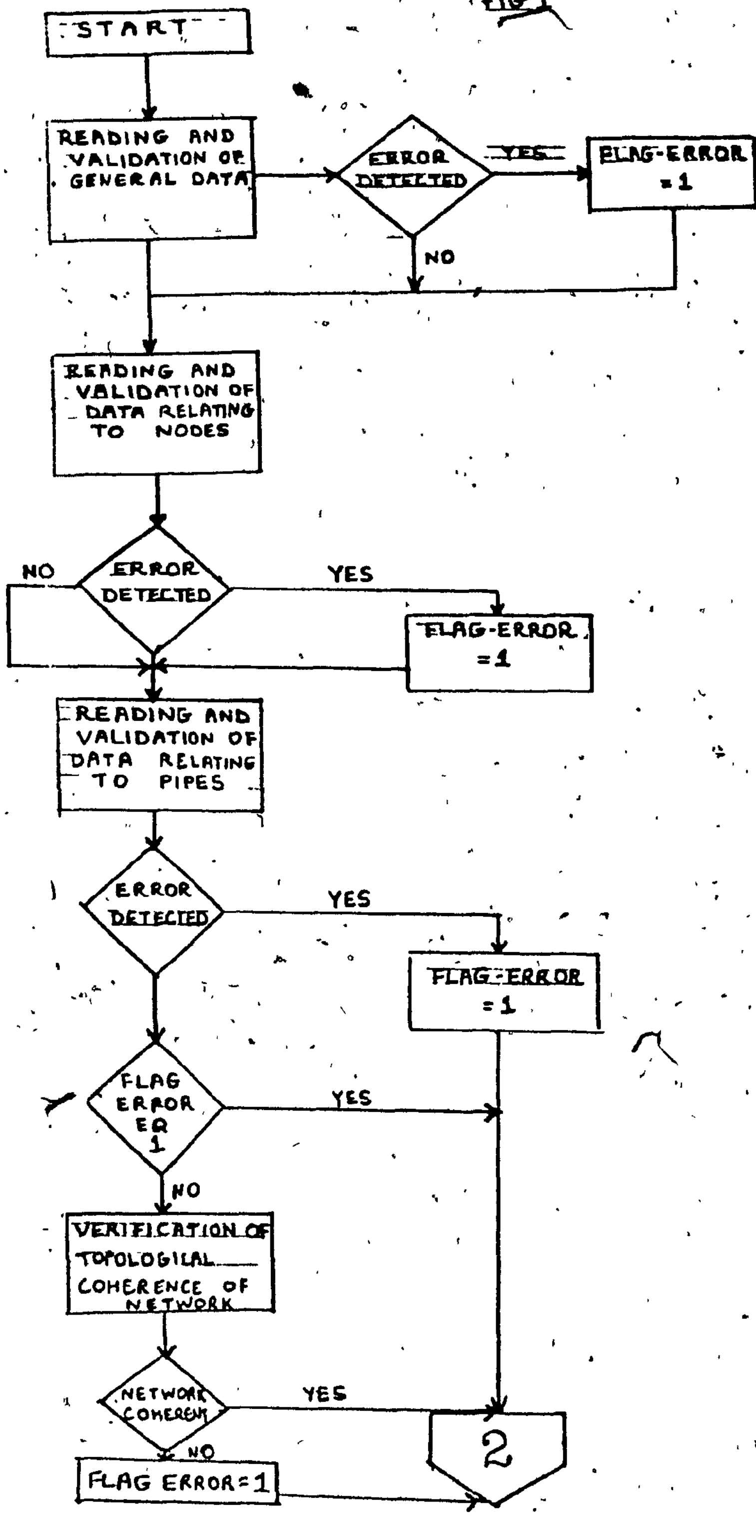
As explained earlier, one of the reasons for developing the present model was to eliminate the drawbacks of the existing packages like "HYNAL" - I.B.M. etc. The following are some of the special aspects of the present model not available on the packages marketed (refer to flow chart).

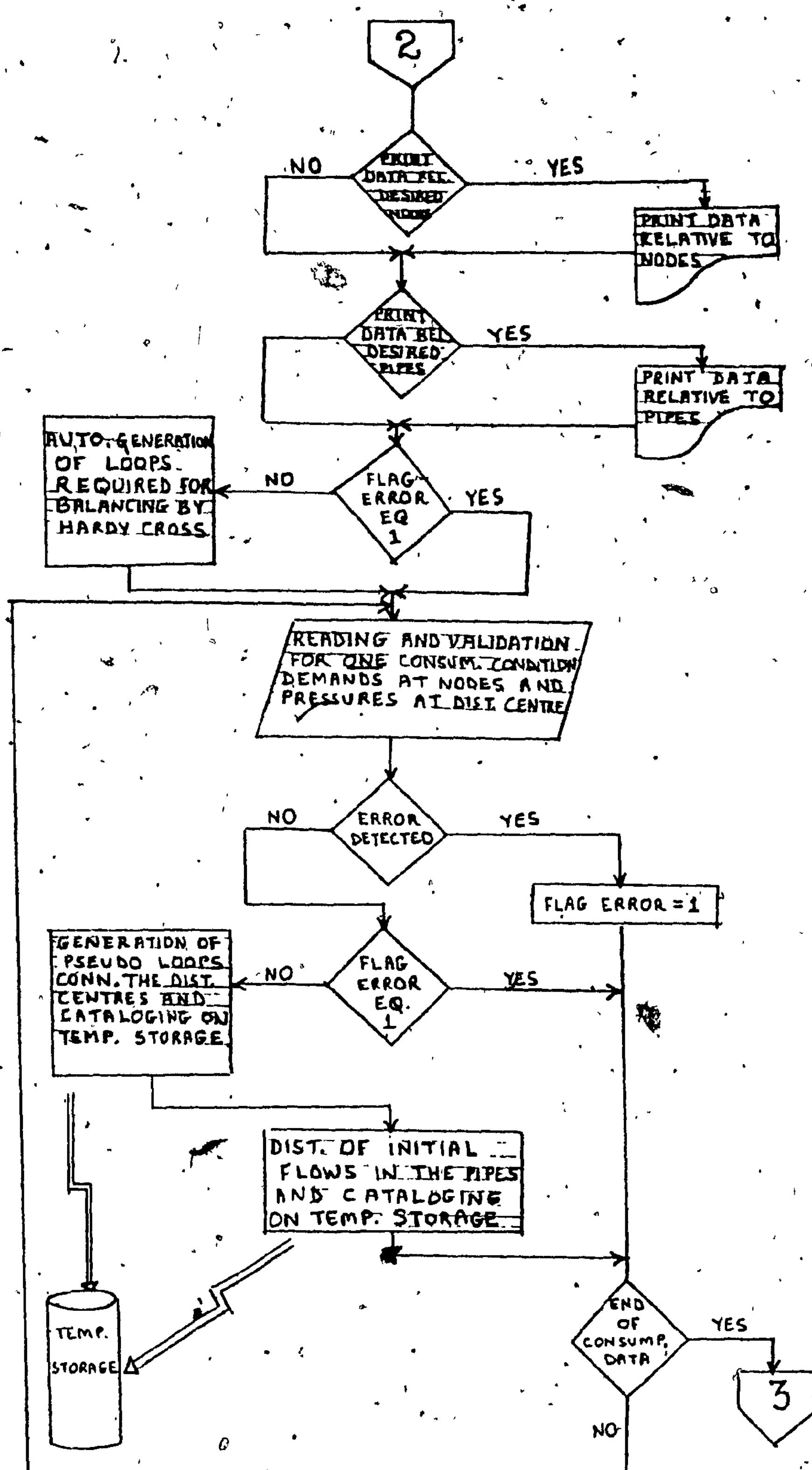
a) The pipes in the network can be assigned numbers at random. The loops will be generated automatically by the computer. If any pipes are added, the loops, pipes and nodes do not have to be renumbered. The new pipe can be given any new number and the change to that particular loop recorded. The computer automatically makes the adjustment.

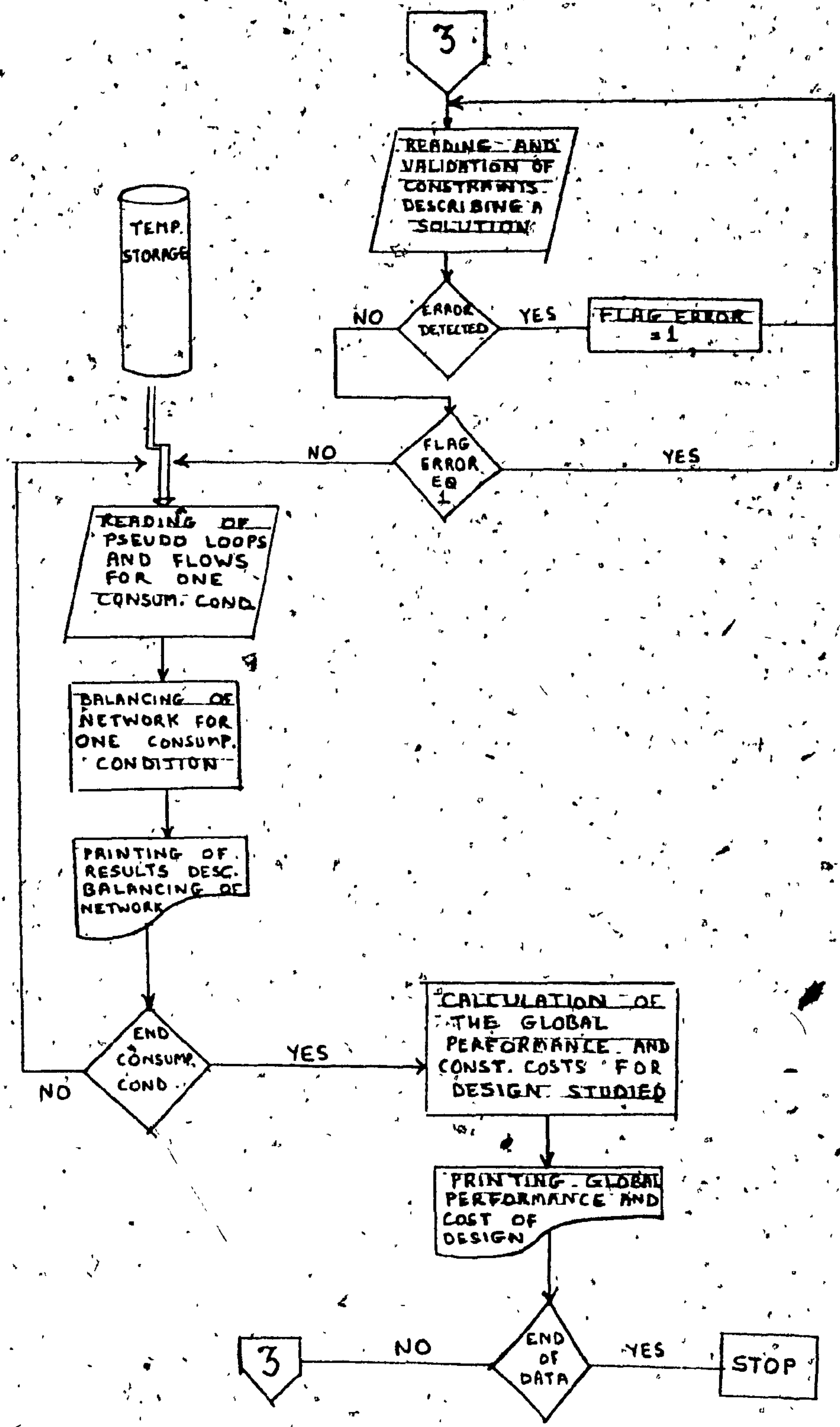
b) Demand hydrographs can be assigned to each node, i.e. an industrial sector may have a constant demand from

FLOW CHART OF THE COMPUTER SIMULATION.

FIG 1




2
PRINT DATA REL. DESIRED NODES
NO
YES
PRINT DATA RELATIVE TO NODES
PRINT DATA REL. DESIRED PIPES
YES
PRINT DATA RELATIVE TO PIPES
AUTO. GENERATION OF LOOPS REQUIRED FOR BALANCING BY HARDY CROSS
NO
FLAG ERROR EQ 1
YES
READING AND VALIDATION FOR ONE CONSUM. CONDITION DEMANDS AT NODES AND PRESSURES AT DIST. CENTRE
NO
ERROR DETECTED
YES
FLAG ERROR = 1
GENERATION OF PSEUDO LOOPS CONN. THE DIST. CENTRES AND CATALOGING ON TEMP. STORAGE
NO
FLAG ERROR EQ. 1
YES
DIST. OF INITIAL FLOWS IN THE PIPES AND CATALOGING ON TEMP. STORAGE
TEMP. STORAGE
END OF CONSUMP. DATA
YES
3
NO


3
READING AND VALIDATION OF CONSTRAINTS DESCRIBING A SOLUTION
TEMP. STORAGE
ERROR DETECTED
NO
YES
FLAG ERROR =1
FLAG ERROR EQ 1
NO
YES
READING OF PSEUDO LOOPS AND FLOWS FOR ONE CONSUM. COND.
BALANCING OF NETWORK FOR ONE CONSUMP. CONDITION
PRINTING OF RESULTS DESC. BALANCING OF NETWORK
END CONSUMP. COND
NO
YES
CALCULATION OF THE GLOBAL PERFORMANCE AND CONST. COSTS FOR DESIGN STUDIED
PRINTING GLOBAL PERFORMANCE AND COST OF DESIGN
END OF DATA
NO
YES
3
STOP

8.00 a.m to 5.00 p.m., after which the demand may diminish to zero, while residential sectors could have another kind of varying demand. In available packages there is no provision for different hydrograms at each node.

c) Various demand conditions can be combined into one output. In most existing packages, each demand condition requires a separate simulation.

d) The computer calculates for each node, its index of satisfaction depending on the resulting pressures. For example, considering 20 psi as zero satisfaction and 80 psi as 100% satisfaction, the index of satisfaction for the global network is calculated as:

$$\frac{x\% \times v_1 + y\% \times v_2}{v_1 + v_2} \quad \ldots\ldots\ldots(7)$$

where x and y are the satisfaction percentages for the nodes n_1, n_2, etc. v_1 and v_2 are the corresponding populations served by nodes n_1, n_2, etc. This index is an indicator of the network's efficiency.

e) Reservoirs and booster stations can be easily introduced into the network for subsequent simulations without renumbering of the loops.

CHAPTER 4

EVALUATION OF THE PROPOSED MODEL

The computer model was tested for accuracy by comparing its predictions with the results of a field study. The preliminary model was then adjusted accordingly before proceeding to the simulation phase.

As discussed earlier, the preliminary test model was first designed with the help of the existing water network plan for the town of Mercier. The nodes were chosen at random but the entire area under study was covered (fig.6).

1. The Field Study

The locations corresponding to the chosen nodes were marked with a wooden picket next to the nearest fire hydrant on the site. The pressures on these fire hydrants were recorded at 7.00 a.m. and 9.00 p.m. on Tuesday and Thursday when they were assumed to be average (13).

A pressure gauge was mounted directly on the outlet of each fire hydrant with the valve closed. As soon as the valve was opened, the gauge showed the water pressure in the main line to which the hydrant was connected.

2. The simulation

The test model was fed average consumptions at the nodes and a computer run was made. The resultant pressures were then compared (for the pre-selected nodes) with the pressures obtained from the field. All the predicted pressures were within ± 5 psi of those measured, except at two nodes (see table 22).

The coefficients of friction in the pipes leading to these nodes were changed, in stages, and successive computer runs were made until the resultant pressures were within ± 5 psi of the field results. Having achieved a tolerance of ± 5 psi at all nodes, the adjusted model was then assumed to be a true representative of the actual existing network. The rest of the computer runs and the consequent recommendations for the additions and modifications were based on the model.

3. Possible errors

Once the predicted and measured pressures were within a difference of ± 5 psi, the model was assumed to be an acceptable replica of the actual network. However, the following could have been possible sources of error:

a) When the difference between the predicted and measured pressures exceeded 5 psi at a node, it was automatically assumed that the coefficient of roughness had incorrectly been estimated. Nevertheless, a leak in the water pipe or a defective valve would affect the pressure reading.(6) These two possibilities were not investigated in this study.

b) The field nodes that were actually found to match those assigned in the computer model (i.e. within 5 psi) could have slightly been mislocated. This may be attributed to an overestimation of friction in one leading pipe and an underestimation in another. The two estimates could have been partially compensating in some cases.

c) The test model was constructed from the principal feeders only and the smaller pipes were neglected. No effort was made to evaluate or adjust the model to account for this omission.

d) The pressure gauges used could be read within an accuracy of ± 1 psi.

4. Limitations of the model

The choice of the pipes for the model was not an obvious one. For practical purposes, larger pipes, where the head losses are comparatively lower, are used to form the major loops for the sector; the smaller pipes in which the head losses are dramatically higher are neglected.

In this analysis, the effect of omitting the smaller pipes on the quality of the model was not thoroughly investigated. It appears, however, from the comparison of the model and field study that that did not have pronounced influence on the results. It is noted that for a bigger, more elaborate sector, such omission may significantly affect the computer output. (1,7,11 and 12)

The floating reservoir was another baffling problem. For the simulation, a certaindischarge for the filling process had to be assumed during minimum demand conditions. This had no basis except for the town superintendant's experience. In bigger towns, this would have to be properly evaluated under different fire conditions.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

A computer model for analyzing water distribution networks has been developed. It has been shown that the pressures predicted by the model are in reasonable conformity with site pressures measured in the town of Mercier, Quebec. Thus, the model may conveniently be applied to other towns with comparable size and similar characteristics, i.e. well planned and zoned locations on relatively plain topography. The master plan developed for the test district forms the platform for future network improvements.

The model has its unique features as compared to available packages. It automatically generates the loops and is capable of accommodating additional pipes, reservoirs and pumps without complications. Furthermore, the model has a provision for simulating different hydrograph conditions at each node.

The findings of this study indicate that the estimate of the roughness coefficient is the most sensitive input component. The programming procedure adopted allows the adjustment of this coefficient in the model to suit observed pressures. This technique has displayed a reasonable degree of reliability. However, field detection of possible leaks and other defects is an additional study that could prove useful in refining the preliminary verification phase of the model.

The accuracy of the model in simulating considerably larger areas should be checked by further studies. Inclusion of small diameter pipes, at least in the vicinity of the major sources of supply, would enhance the simulation. This would particularly be important in simulating the piezometric surface under a large concentrated load such as a fire flow.

It is recommended for future studies that the elevation difference between nodes be determined from the field rather than using aerial contour maps. In addition, the head loss between the pressure gauge location (hydrant) and the nearest reference node should be taken into account, particularly for the small diameter pipes (diameters less than 8 inches).

Finally, several representative network systems can be used to study the simulation expedients and the sensitivity of the model to various assumptions. After adequate data have been analyzed it will be possible to generalize the applicability of the model to treat a wide range of different systems with confidence.

APPENDIX 1 : FIGURES

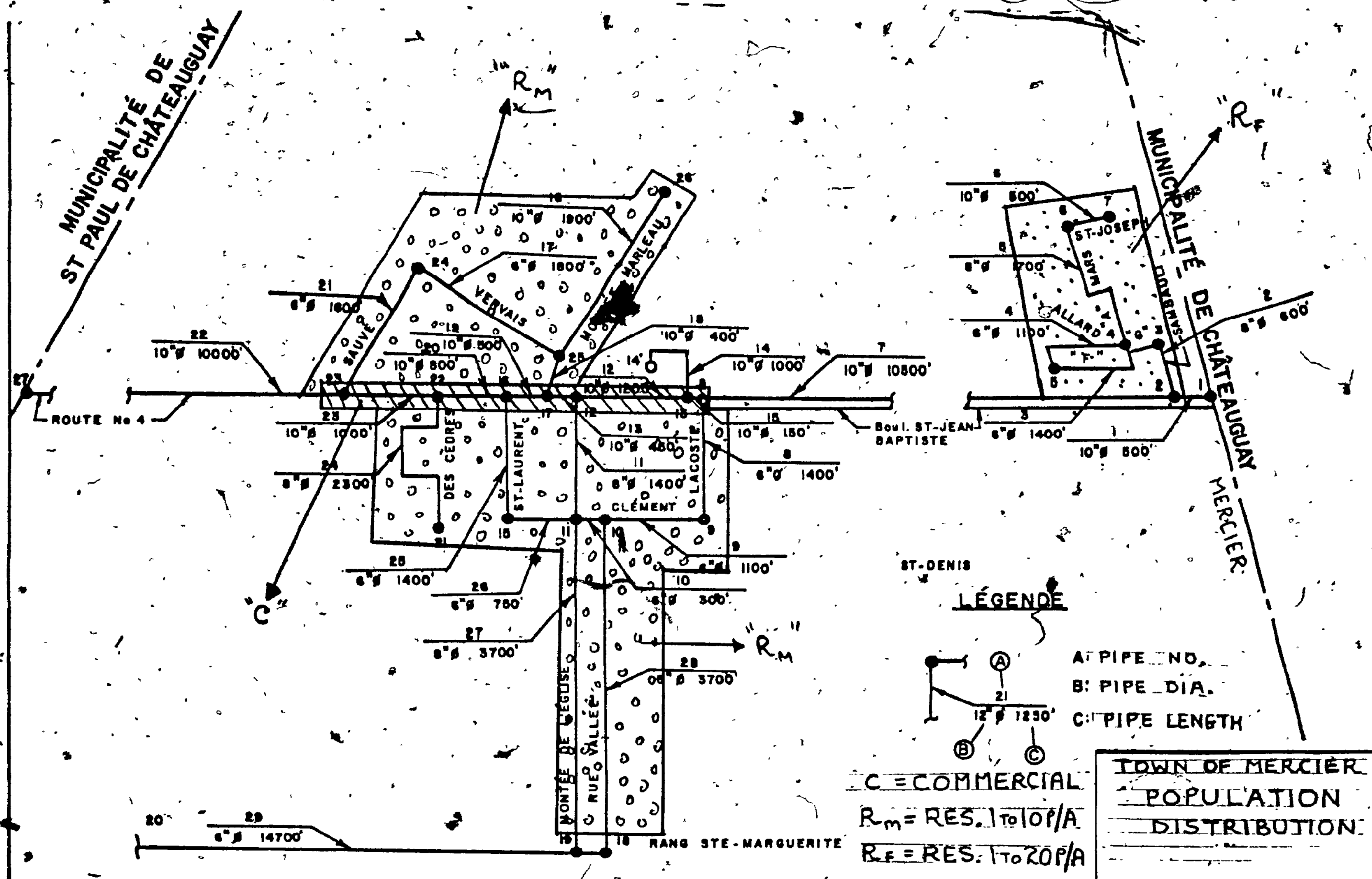

MUNICIPALITÉ DE
ST PAUL DE CHÂTEAUGUAY
MUNICIPALITÉ DE CHÂTEAUGUAY
MERCIER
ROUTE No 4
Boul. ST-JEAN-BAPTISTE
RANG STE-MARGUERITE
ST-DENIS
SAUVÉ
VERVAIS
DES CÈDRES
ST-LAURENT
CLÉMENT
LACOSTE
MONTÉE DE L'ÉGLISE
RUE VALLÉE
ST-JOSEPH
MARS
ALLARD
SAMBAULT
"C"
"R_M"
"R_F"
LÉGENDE
A: PIPE NO.
B: PIPE DIA.
C: PIPE LENGTH
C = COMMERCIAL
R_m = RES. 1 TO 10 P/A
R_F = RES. 1 TO 20 P/A
TOWN OF MERCIER
POPULATION
DISTRIBUTION

FIGURE 1.

TOWN OF MERCIER
POPULATION PROJECTION

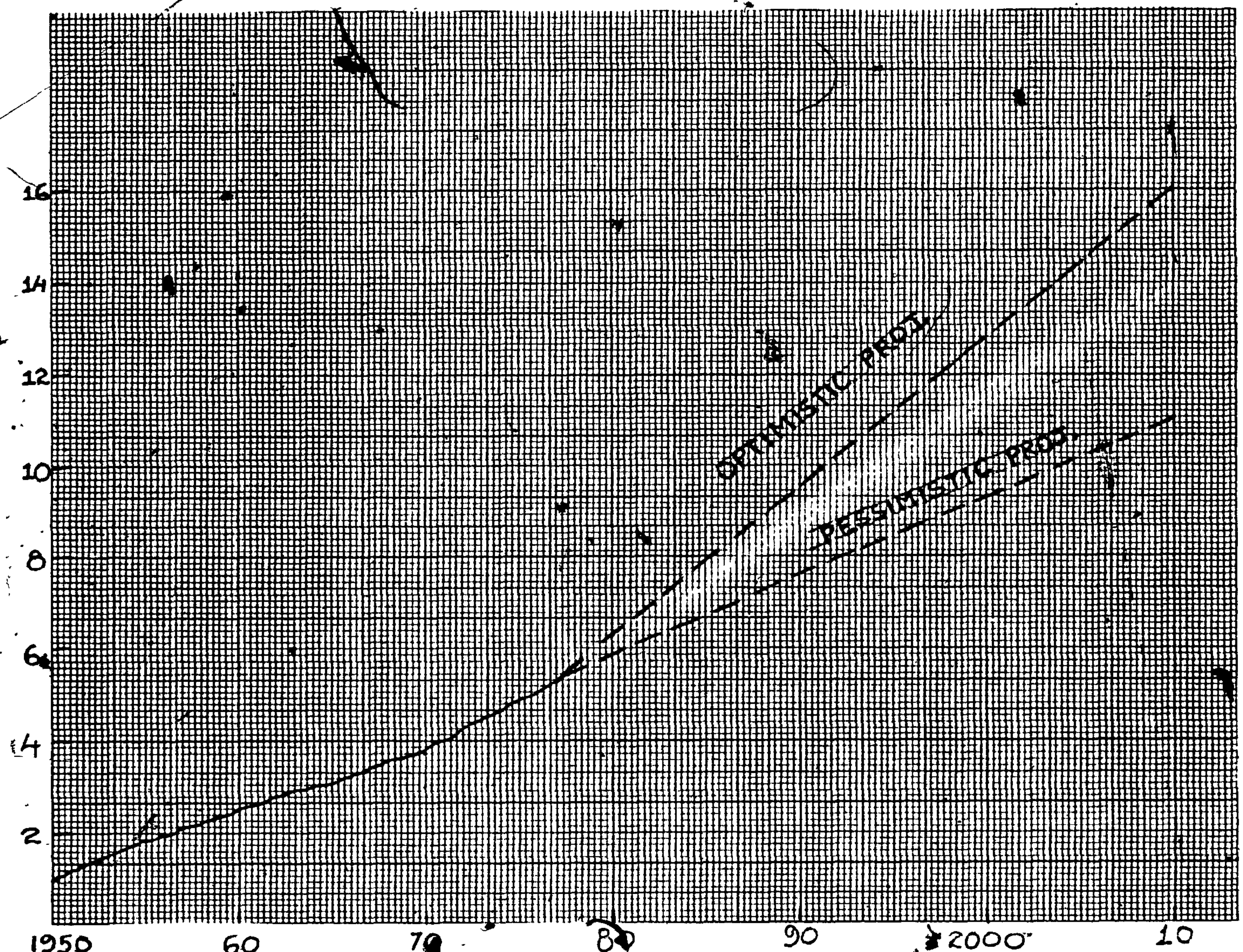
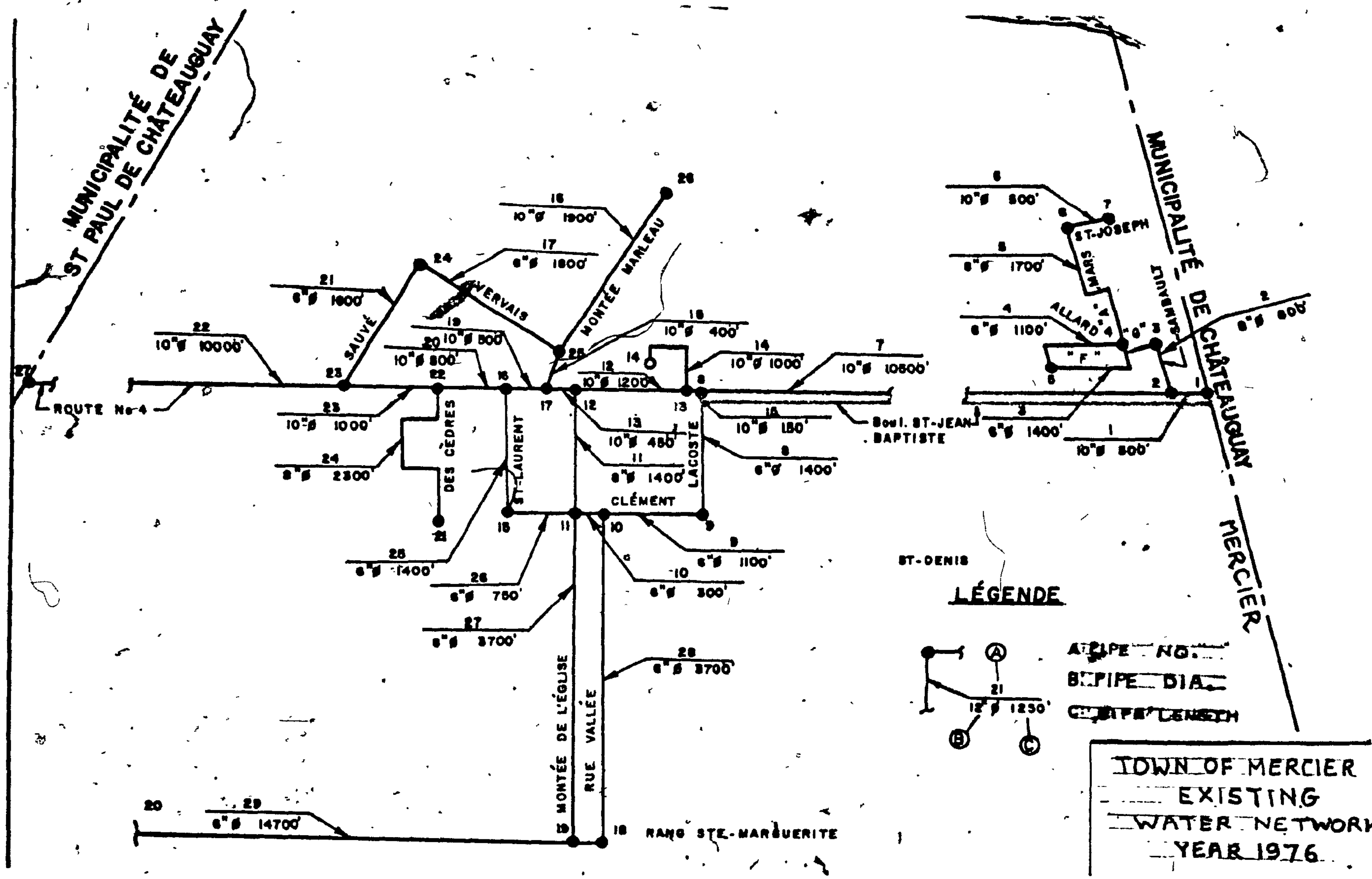


FIGURE 2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MUNICIPALITÉ DE CHÂTEAUGUAY
MERCIER
ROUTE No-4
SAUVÉ
GERVAIS
MONTÉE MARLEAU
DES CÈDRES
ST-LAURENT
CLÉMENT
LACOSTE
Boul. ST-JEAN-BAPTISTE
ST-JOSEPH
MARS
ALLARD
GAMBAULT
ST-DENIS
MONTÉE DE L'ÉGLISE
RUE VALLÉE
RANG STE-MARGUERITE
LÉGENDE
A: PIPE NO.
B: PIPE DIA.
C: PIPE LENGTH
TOWN OF MERCIER
EXISTING
WATER NETWORK
YEAR 1976

FIGURE 3.

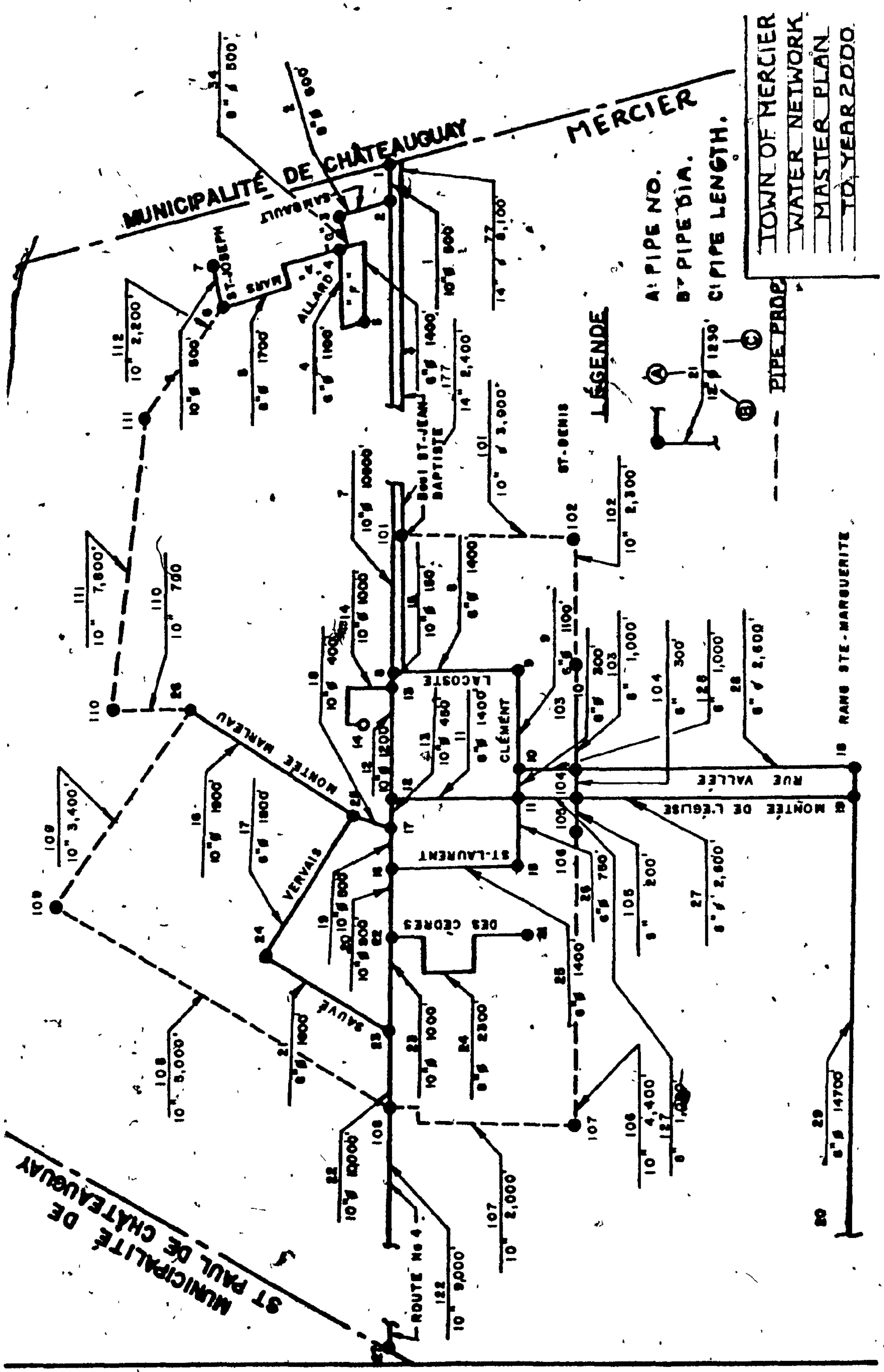
TOWN OF MERCIER
WATER NETWORK
MASTER PLAN
TO YEAR 2000
LÉGENDE
A: PIPE NO.
B: PIPE DIA.
C: PIPE LENGTH.
PIPE PROP.
MUNICIPALITÉ DE CHÂTEAUGUAY
MERCIER
ST PAUL DE CHÂTEAUGUAY
MUNICIPALITÉ DE
ROUTE No 4
MONTÉE MARLEAU
VERVAIS
SAUVÉ
DES CÈDRES
ST-LAURENT
CLÉMENT
LACOSTE
BOUL ST-JEAN BAPTISTE
ST-DENIS
RUE VALLÉE
MONTÉE DE L'ÉGLISE
RANG STE-MARGUERITE
ST-JOSEPH
ALLARD

FIGURE 5.

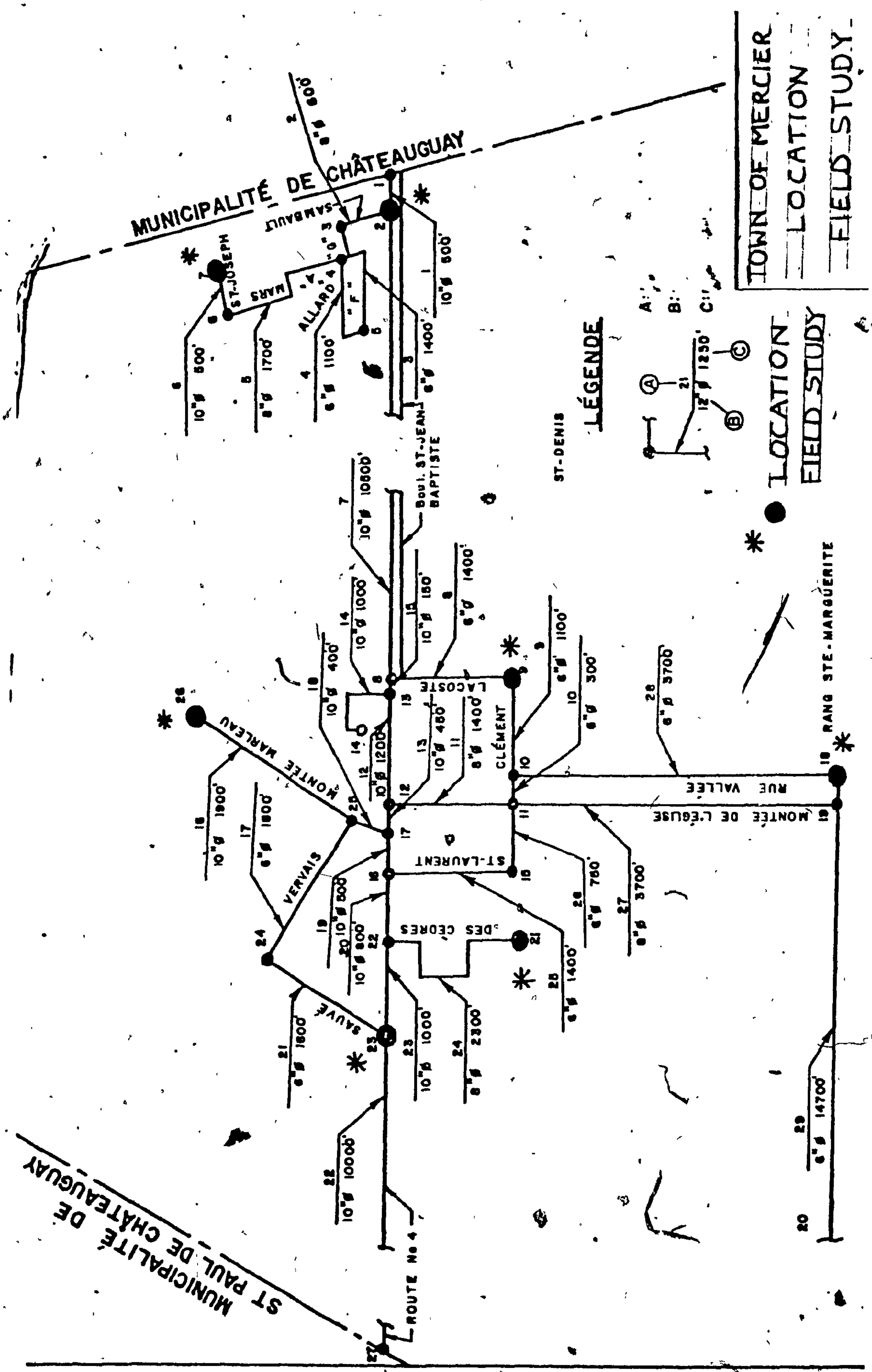
TOWN OF MERCIER
LOCATION
FIELD STUDY
MUNICIPALITÉ DE CHÂTEAUGUAY
MUNICIPALITÉ DE ST PAUL DE CHÂTEAUGUAY
LÉGENDE
A:
B:
C:
LOCATION
FIELD STUDY
ST-DENIS
Boul. ST-JEAN BAPTISTE
ST-JOSEPH
MARS
ALLARD
SAMBAULT
LACOSTE
CLÉMENT
ST-LAURENT
DES CEDRES
MONTÉE MARLEAU
VERVAIS
SAUVÉ
RUE VALLÉE
MONTÉE DE L'ÉGLISE
RANG STE-MARGUERITE
ROUTE No 4

FIGURE 6

APPENDIX 2 : TABLES

Table 1

TOWN OF MERCIER

DISTRIBUTION NETWORK

(1974)

PIPE DIAMETER (inches)	LENGTH (feet)
6	27,600
8	4,650
10	11,800
14	10,000

PUMP	CAPACITY	HEAD
1	350USGPM	115 FT
2	1150USGPM	115 FT

Reservoir - 500,000 I.G. (145 ft) height of water)

Table 2

TOWN OF MERCIER

PRESENT POPULATION DISTRIBUTION

TYPE OF OCCUPANT	ZONES DEVELOPED (ACRES)	ZONES NON-DEVELOPED (ACRES)	TOTAL AREA (ACRES)
Residential 10 pers./acre	130		130
Residential 11 to 20 pers./acre	220		220
Commerce	30		30
Agriculture, green space, Expansion			19620
TOTAL	380	19620	20000

Table 3

TOWN OF MERCIER

POPULATION STATISTICS

YEAR	POPULATION	RATE OF INCREASE %	INCREMENT OF POPULATION
1964	2890	9.0	260
1965	3150	2.5	80
1966	3230	12.4	400
1967	3630	6.3	230
1968	3400	8.8	300
1969	3100	2.7	100
1970	3800	5.3	200
1971	4000	2.5	100
1972	4100	2.4	100
1973	4200	2.4	100
1974	4300	12.8	550
1975	4850	7.2	350
1976	5200		
TOTAL INCREASE (In 12 years)		79.9	2310
AVERAGE ANNUAL INCREASE		6.7	193

Table 4

TOWN OF MERCIER

POPULATION PROJECTION

YEAR	OPTIMISTIC PROJECTION	PESSIMISTIC PROJECTION	PROJECTION RETAINED
1980	6500	5500	5600
1985	8000	6000	6900
1990	9500	6500	8200
1995	11000	7000	9500
2000	13000	8000	10900
2010	16000	9000	13500

Table 5

TOWN OF MERCIER

POPULATION DISTRIBUTION (YEAR 2000)

SUMMARY - TYPE OF OCCUPANT

TYPE OF OCCUPANT	ZONES DEVELOPED (ACRES)	ZONES NON-DEVELOPED (ACRES)	TOTAL AREA (ACRES)
Residential 10 pers./acre	450		450
Residential 11 to 20 pers./acre	320		320
Commerce	60		60
Agriculture, green space expansion			19170
TOTAL	830	19170	20000

Table 6

TOWN OF MERCIER

ANNUAL AVERAGE FLOW

YEAR	ANNUAL CONSUMPTION (MIG)	ANNUAL AVERAGE FLOW IMPERIAL GAL/DAY
1974	133.66	366200
1975	142.01	389100
1976	159.67	437400

Table 7

TOWN OF MERCIER

BREAKDOWN OF UNIT CONSUMPTION FOR 1976 (AVERAGE)

TYPE OF CONSUMER	POSSIBLE VARIATION	CONSUMPTION RETAINED
Domestic	30 to 50 IG/pers/day	40 IG/pers/day
Lawn Watering	200 to 400 IG/acre/day	300 IG/acre/day
Commercial	800 to 5000 IG/acre/day	1700 IG/acre/day
Leaks	100 to 600 IG/acre/day	300 IG/acre/day
Public Use	One Percent (1%) of annual average flow	

Table 8

TOWN OF MERCIER

BREAKDOWN OF TOTAL CONSUMPTION FOR 1976

TYPE OF CONSUMER	AVERAGE CONSUMPTION (IGD)
Domestic	208000
Lawn Watering	60000
Commercial	51000
Leaks	114000
Public Use	4400
Annual Average Flow	437400

Table 9

TOWN OF MERCIER

BREAKDOWN OF TOTAL CONSUMPTION FOR THE YEAR 2000

TYPE OF CONSUMER	AVERAGE CONSUMPTION (IGD)
Domestic	566800
Lawn Watering	137100
Commercial	102000
Leaks	249000
Public Use	10600
Annual Average Flow	1065500

Table 10

TOWN OF MERCIER

FLUCTUATION OF DEMAND

YEAR	ANNUAL CONSUMPTION (MIGD)	ANNUAL AVERAGE CONSUMPTION (MIGD)	AVERAGE FLOW FOR MAXIMUM DAY (MIGD)		PEAK FOR THE MAXIMUM DAY (MIGD)		MIN. FLOW FOR THE MAXIMUM DAY (MIGD)	
1974	133.66	0.366	0.594	1.6	0.941	2.6	0.403	1.1
1975	142.01	0.589	0.622	1.6	1.011	2.6	0.428	1.1
1976	159.67	0.437	0.699	1.6	1.136	2.6	0.481	1.1

Table 11

TOWN OF MERCIER

UNIT CONSUMPTION FOR THE MAXIMUM DAY IN 1976

TYPE OF CONSUMER	FACTOR FOR MAXIMUM DAY		UNIT CONSUMPTION	
	POSSIBLE VARIATION	FACTOR RETAINED	AVERAGE	MAXIMUM DAY
Domestic	1.2 to 1.8	1.5	40 IG/pers/day	60 IG/pers/day
Lawn Watering	2 to 20	3	300 IG/acre/day	900 IG/acre/day
Commercial	1 to 4	1.74	1700 IG/acre/day	2958 IG/acre/day
Leaks	1	1	300 IG/acre/day	300 IG/acre/day
Public Use	1	1	One percent (1%) of annual average flow	

Table 12

TOWN OF MERCIER

BREAKDOWN OF THE FLOW FOR THE MAXIMUM DAY IN 1976

TYPE OF CONSUMER	CONSUMPTION (IGD)
Domestic	312000
Lawn Watering	180000
Commercial	88600
Leaks	114000
Public Use	4400
Total	699000

Table 13

TOWN OF MERCIER

BREAKDOWN OF THE FLOW FOR THE MAXIMUM DAY IN 2000

TYPE OF CONSUMER	CONSUMPTION (IGD)
Domestic	850200
Lawn Watering	111300
Commercial	177500
Leaks	249000
Public Use	10600
Total	1698600

Table 14

TOWN OF MERCIER

UNIT CONSUMPTION AT PEAK HOUR IN 1976

TYPE OF CONSUMER	PEAK VALUE FACTOR: POSSIBLE VARIATION	PEAK VALUE FACTOR: FACTOR RETAINED	UNIT CONSUMPTION: AVERAGE	UNIT CONSUMPTION: PEAK
Domestic	2 to 3	2.3	40 IG/pers/day	
Lawn Watering	5 to 45	7.2	300 IG/acre/day	
Commercial	1 to 4	2.74	1700 IG/acre/day	
Leaks	1	1	300 IG/acre/day	
Public Use	1	1	One percent (1%) of annual average flow	

Table 15

TOWN OF MERCIER

BREAKDOWN OF TOTAL CONSUMPTION AT PEAK HOUR FOR 1976

TYPE OF USE	PEAK HOUR CONSUMPTION (IGD)
Domestic	482600
Lawn Watering	433000
Commercial	102000
Leaks	114000
Public Use	4400
Total	1136000

Table 16

TOWN OF MERCIER

BREAKDOWN OF FLOW AT PEAK HOUR IN 2000

TYPE OF CONSUMER	PEAK CONSUMPTION (IGD)
Domestic	1303600
Lawn Watering	987100
Commercial	204000
Leaks	249000
Public Use	10600
Total	2754300

Table 17

TOWN OF MERCIER

UNIT CONSUMPTION AT MINIMUM HOUR IN 1976

TYPE OF CONSUMER	MINIMUM HOUR FACTOR		UNIT CONSUMPTION	
	POSSIBLE VARIATION	FACTOR RETAINED	AVERAGE	MINIMUM HOUR
Domestic	0.2 to 0.9	0.9	40 IG/pers/day	36 IG/pers/day
Lawn Watering	0.3 to 3.0	2.9	300 IG/acre/day	870 IG/acre/day
Commercial	0	-	1700 IG/acre/day	
Leaks	1	1	300 IG/acre/day	300 IG/acre/day
Public Use	1	1	One percent (1%) of annual average flow	

Table 18

TOWN OF MERCIER

BREAKDOWN OF TOTAL CONSUMPTION AT MINIMUM HOUR

IN 1976

TYPE OF CONSUMER	CONSUMPTION AT MINIMUM HOUR (IGD)
Domestic	187200
Lawn Watering	175400
Commercial	
Leaks	114000
Public Use	4400
Total	481000

Table 19

TOWN OF MERCIER

BREAKDOWN OF FLOW AT MINIMUM HOUR IN 2000

TYPE OF CONSUMER	CONSUMPTION AT MINUMUM HOUR (IGD)
Domestic	510100
Lawn Watering	397600
Commercial	
Leaks	249000
Public Use	10600
Total	1167300

Table 20

TOWN OF MERCIER

RESERVE REQUIRED

(1976-2000)

TYPE OF RESERVE	VOLUME (IG) 1976	2000
Equilibrium reserve	86,000	165,000
Fire reserve	120,000	120,000
Emergency reserve	110,000	250,000
Total	316,000	535,000

Table 21

COEFFICIENT OF FRICTION

Material of Pipe	Age (years)	Diameter (inches)	Coefficient chosen
Iron	0 to 10 11 to 20	6 to 12 6 to 12	100 90
Iron with concrete lining	0 to 10	12	120
HYPRESCON	0 to 20	14 to 30	140
Steel	0 to 10	16	100

Table 22

TOWN OF MERCIER

FIELD STUDY

Node No.	Theoretical Pressure from Test Model (psi)	Field Reading (psi)	Difference
2			
7	77.62	75.0	5
9	43.87	40.0	5
18	33.77	26.0	7.77
21	38.39	40.0	5
23	38.44	41.0	5
26	27.79	22.0	5.79

APPENDIX 3 : REFERENCES