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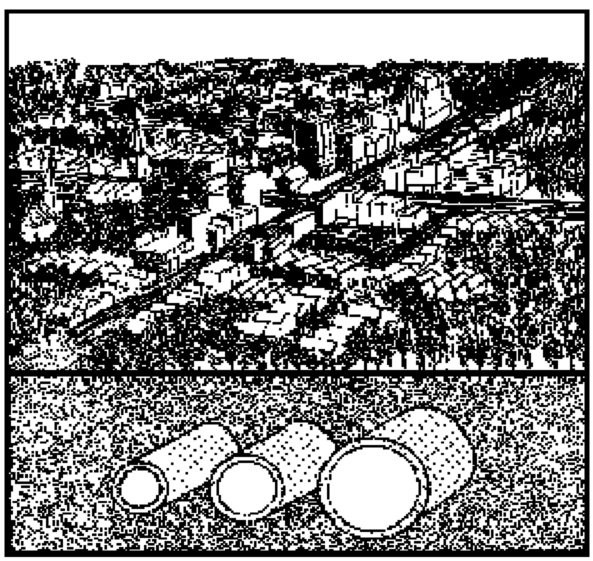
DRAINAGE CRITERIA MANUAL FOR

THE CITY OF WINNIPEG

JAMES F. MacLAREN LIMITED

Willowdale, Ontario



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JAMES F. MacLAREN LIMITED
ENVIRONMENTAL CONSULTANTS
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CHAPTER 1

GENERAL PRINCIPLES OF DRAINAGE DESIGN POLICY

1.1 Introduction

Urban drainage policy and design practices are currently undergoing a transition period with focus on new storm water management techniques and related costs and environmental concerns. Some of the new methods being used or proposed have not yet been entirely proven by practical application since the transition to new techniques is a long term, continual process. As part of this evolving course, the City of Winnipeg has embarked on a program of developing and updating local storm water management design criteria. This report is concerned mainly with the hydrologic aspects of the development of such criteria. Much of the report is based on the extensive literature and experience concerning accepted design practice and recent practical design innovations.

A well defined urban storm drainage policy is an essential part of the master plan required for orderly urban growth. In North America the concept of the dual drainage system is receiving growing attention. Such a system is composed of a major drainage system and a minor or initial system. A master plan can provide a drainage development guide covering the major system which includes major channels (natural or man made) and a definition of any corresponding flood plains. The capacity of the major system must be sufficient to minimize loss of life and major damage. A detailed discussion of the design and functions of the major system is not within the scope of this study. However, consideration is given to the major system insofar as it will influence the design of the initial drainage system.

The initial or minor drainage system can be generally characterized as convenience drainage and includes storm sewers, street gutters, etc. The "protection" provided is generally to reduce localized flooding and complaints from residents. Such flooding does not result in major economic loss on an individual basis or loss of human life and therefore the initial system is designed to handle the more frequent, less intense storms. The required extent of the minor system is obviously a function of the design of the major drainage system and is essentially dependent on the distance to the outfall to the major system and its hydraulic interaction with the initial system. Therefore, the design of the major system can significantly affect the design and extent of the minor system. The main benefit of using the dual drainage concept is the possibility of reducing the size and extent of expensive storm sewer installations. [5,6,7]*

*Numbers in parentheses refer to items in the list of references given at the end of each chapter.

This study is concerned mainly with the underlying design principles of the initial drainage system based on the latest developments in urban hydrology. With regard to hydrologic aspects an attempt is made to indicate the availability of alternative design methods at different levels of sophistication.

1.2 General Principles for Storm Water Management

In the past urban storm water management policy has been directed towards removing storm rainfall from urban areas to the receiving water body as quickly as possible.

This has resulted in rechannelization, straightening, lining, or conversion to storm sewers of natural streams in many urban areas. Since the natural easement for flow can only be denied with great effort, relatively expensive stormwater drainage systems have resulted. Associated detrimental affects such as increased peak flows and corresponding flooding and erosion have also added to the overall cost.

The current trend in design is towards the use of temporary storage facilities which reduce the peak stormwater runoff and hence reduces the size and extent of downstream drainage requirements. Temporary storage can be provided not only by means of artificial facilities but also by maintaining the natural stream conditions wherever possible. Natural streams temporarily store more stormwater than improved channels due to bends and frictional affects. Natural stream storage is especially attractive in relatively flat areas with relatively slow rate of runoff. Where natural streams are to be utilized, the main effort is directed towards controlling erosion and flooding rather than towards achieving a fast rate of runoff. Natural channels also result in a more pleasing urban environment and other benefits related to more available recreational space etc.

Besides encouraging the use of natural channels and artificial storage facilities to reduce the overall cost of the urban drainage system the use of other methods such as the following are being considered more frequently:

i. Discharge of roof drains onto pervious surfaces.
ii. Construction of minor roadways without curbs.
iii. Encouragement of site grading patterns that increase overland flow distances over pervious surfaces.

Implementation of these relatively new methods for controlling storm water runoff requires accurate methods of accounting for the amount of runoff in both time and space for specific storm events. This requires the use of hydrograph methods as opposed to the Rational method which only gives peak flows. For example, sophisticated computer models such as the U.S. Environmental Protection Agency’s Stormwater Management Model (EPA-SWMM) can be used to assess both the quantity and quality of stormwater runoff. [4]

1.3 Planning, Stages in Design

Planning of both the major and minor drainage systems is important in order to create an orderly urban system. A master plan which includes an extensive major system and using natural drainage wherever possible can reduce the size and cost of the minor systems. In turn, proper planning of the minor system to eliminate inconvenience and frequency of minor damage helps create an orderly urban system. Planned use of the drainage system for multiple use facilities should be encouraged wherever possible in order to reduce overall drainage costs.

The first stage of design considers the general feasibility of various alternatives while the second phase is used for detailed design. Phase one considers analysis of several possible solutions in terms of primary estimates of runoff volume and sewer network layout. At this stage it can also be determined whether special structures or policies such as temporary storage facilities will be required. The second phase considers detailed design of the selected alternative. Obviously a more approximate methodology can be used to assess phase one alternatives in a relative sense whereas more accurate methods must be used for final design purposes.

1.4 Data Collection

An important aspect of urban drainage policy is to maintain a detailed program of data collection. In general the physical data required for design facilities includes the following:

i. topography - relief and soils
ii. ground water table
iii. existing structures and rights of way and projections for proposed developments
iv. meteorological records (e.g. rainfall, snowfall, temperature etc.)
v. flow measurements concerning both quantity and quality
vi. infiltration measurements
vii. overflow frequencies of combined sewers
viii. pumping facilities and method of operation
ix. flood levels in the receiving water body.

A rapidly changing urban environment requires an up to date inventory of all available data in order to achieve an optimum design. In addition, establishment of measurement programs to collect quantity and quality data for the purposes of model calibration of the rainfall-runoff process is extremely important. In addition flood damage surveys such as those carried out by Dolhun [2] are useful in assessing the magnitude of the flooding problem associated with large storms.

1.5 Criteria for Assessment of Alternatives

Assessment of various strategies for urban storm water management requires careful analysis of several aspects such as economical, environmental, and operational considerations.

1.5.1 Environmental

The potential for improvement of storm water quality is an important criteria for assessment of alternatives. For example the removal of sediment and debris by storage facilities can improve water quality downstream. Consideration must also be given to means of minimizing overflows from combined sewers and the possibility of future treatment of overflows from combined sewers. A possible method for assessment of alternatives from an environmental viewpoint is by constructing a matrix of alternatives and scoring the related benefits in a relative manner.

Other environmental aspects include the use of natural streams as green belts and preservation of the natural ecology in such areas if possible, and recreational and aesthetic benefits.

1.5.2 Operation and Maintenance

Consideration must be given to normal operation and maintenance aspects such as regular cleaning of inlets and gutters, etc. The use of storage facilities also gives rise to periodic removal of debris and sediment, algae and plant control, mosquito control maintenance of pumps, valves, pipes and screens where applicable. The safety of children in relation to reservoirs and inlet and outlet structures must be considered.

Multipurpose use of drainage facilities as green belt areas requires continuous maintenance of trees and grass cutting.

Consideration should also be given to the fact that drainage systems which improve runoff conditions in streets will reduce street maintenance requirements.

1.5.3 Economics

An economic analysis is one of the most important aspects to be considered when assessing storm drainage alternatives. A generally accepted method of assessment is by means of a cost-benefit analysis which should be carried out for each alternative using various design storm frequencies. The most serious economic losses result from flooding of the major drainage system, however the economic consequences of possible flooding of basements, etc., must also be considered.

In residential and commercial areas the interrelationships between average annual damages, flooding frequency at the ground elevation outside of the building and the stage-frequency relationship should be determined for each alternative. The slope of the stage-frequency curve is a topographic factor that is interrelated with the size of drainage area, characteristics of channel cross-section, and other variables. The assessment of costs related to flood damage of various frequencies can be determined from existing records such as basement flooding reports, etc. [25]

Other costs obviously include:

i. construction and capital costs
ii. direct operation and maintenance expenses
iii. interest, amortization and interim repayment
iv. taxes of various kinds

Annual costs are amortized over the life of the project.

Benefits related to efficient stormwater management programs may be difficult to determine in terms of dollar values but estimates should be made for major designs. Direct benefits of flood control are the most easily defined and include reduction or prevention of physical damage to properties, increased property value, control of water quality downstream of storage facilities (with the potential for decreased treatment costs) and multiple use such as recreation facilities, Indirect benefits of flood control which may be more difficult to assess include prevention of the interruption of services, utilities and transportation and the increased use of water as a resource. Intangible benefits are those to which it may not be possible to assign a monetary value but which can nevertheless be considered in the discussion of alternatives. These include prevention of personal injury, reduction of nuisance, maintenance of public morale and aesthetic factors. [1]

Insofar as possible all costs and benefits should be reduced to annual dollar values with the objective of the analysis being to maximize the benefit/cost ratio. The cost-benefit analysis should be performed for various storm frequencies in order to determine an optimum design, taking into account risk and uncertainty. For each design, it will be evident that protection from less frequent storms will result in high costs with relatively small additional incremental benefit.

1.6 Design Frequency

The planning and design of storm drainage facilities is related to the degree of protection required which is usually expressed in terms of the recurrence interval in years. Normally the emphasis is placed on the 2 to 10 year return frequency for the initial drainage system and on larger less frequent storms for design of the major drainage works. [4,6]

It is important to point out that while most designs are currently based on the concept of identifying a design storm and translating this rainfall into runoff by means of an appropriate model, the return period of the resulting runoff is not necessarily the same as the return period of the design storm. This is because the runoff resulting from a rainfall with a particular return frequency is a function of the antecedent soil conditions (permeability and infiltration) and the areal rainfall pattern in relation to the shape of the drainage area. It is for this reason that continuous monitoring of rainfall and runoff would be required in order to identify the return frequency associated with measured runoff volumes. It is possible that this could result in substantial savings for design for a given return period. [3, 8]

However, continuous measurements of rainfall-runoff are rarely available and therefore practically all designs are currently based on using models to estimate runoff from storm rainfall by assuming a conservative return period. For design of the initial system in Winnipeg use of the 5 and 25 year return periods for design of the pipe network and storage facilities respectively is generally considered to provide acceptable protection. These storm return frequencies are commonly used in Canadian practice [4] and have generally resulted in satisfactory drainage designs. However, as pointed out in Section 1.5.3, the use of these frequencies should be reviewed for each individual design in order to obtain an optimum balance between protection and cost, At the design stage for large drainage networks, the final selection of the design frequency should be based on such factors as economics and safety. For example, if a large storage reservoir is required for storm flow regulation, loss of human life may be possible, thus necessitating the use of a larger design storm corresponding to a longer return period.

CHAPTER 1 REFERENCES

1. Chow, V.T., "Handbook of Applied Hydrology", McGrawHill, 1964

2. Dolhun, S. and E. P. Fedyniak, "Report on Basement Flooding in Winnipeg – 1972", City of Winnipeg, 1973

3. Espey, W. H., "Urban Flood Frequency Characteristics" J. ASCE Hydraulics Division, February, 1974

4. James F. MacLaren Limited, "Review of Canadian Storm Sewer Design Practice and Comparison of Urban Hydrologic Models, 1974".

5. Jones, E.D., "Urban Water Resources Management Affects the Total Urban Picture", in Treatise on Urban Water Systems, Colorado State University, Fort Collins, 1971

6. Rice, G., "Reduction of Urban Runoff Peak Flow by Ponding", ASCE, J. of Irrigation and Drainage Division, September, 1971

7. Wright-McLaughlin Engineers, "Urban Storm Drainage Criteria Manual", Vol. 1, 2, Denver, Colorado, March, 1969

8. Water Resources Engineers and H.E.C. "Management of Urban Storm Runoff", May, 1974.

CHAPTER 2

PRECIPITATION ANALYSIS

2.1 General

The average annual rainfall in Winnipeg (1931-1960) is about 15.22 in. (Labelle [9]*) with a range of from 8.14 in. in 1936 to 24.39 in. in 1878. The highest amount of rainfall which occurred in a 24-hour period was 6.00 inches which fell on June 25, 1901. The year-to-year variation in total rainfall is a function of the showery nature of the precipitation which is characteristic of the region. In addition an average of 5.1 inches water equivalent of snowfall occurs each year.

June, July and August are the wettest months accounting for about 57% of the average rainfall and studies by Lipson [10]* and Simpson [16]* have shown that these are also the months of most frequent thunderstorms. (Table 2.1). From an analysis of eleven years of data from 1953 to 1963 Lipson found an average of 1 or more thunderstorms per week through June, July and August with a peak of 1.8 per week during the 3rd week of July.

Table 2.2 summarizes the number of storms equalling or exceeding the intensity for different durations corresponding to a storm with a return period of 5 years. This clearly demonstrates that June, July and August are the months with the most frequent intense storms.

A mathematical representation of the intensity-duration frequency relationship of rainfall events can be derived by analyzing recording rain gauge records according to some assumed frequency distribution. The computed points can then be fit to an assumed mathematical formula. For example, the design formulae currently used in Winnipeg for storms with return periods of 2 1/2 and 10 years respectively are:

$$i = \frac{100}{t_d + 18} \quad \text{(2 ½ year storm)} \qquad 2.1$$

$$i = \quad \text{(10 year storm)} \qquad 2.2$$

where t_d = storm duration in minutes

* Numbers in parentheses refer to items in the list of references.

TABLE 2.1

COMPARISON OF MONTHLY FREQUENCY OF

THUNDERSTORMS FROM 1953 TO 1963 WITH CORRESPONDING

DATA FROM 1900-1936

(Lipson Ref. 10)

Month	Number of Storms per Month		Monthly Percentage Frequency	
	1900-1936	1953-1963	1900-1936	1953-1963
April	0.6	0.7	3	3
May	1.9	2.4	10	10
June	4.4	5.0	24	21
July	5.3	6.6	29	28
August	3.7	6.2	20	26
September	2.3	2.6	12	11
October	0.3	0.3	2	1

TABLE 2.2

NO. OF THUNDERSTORMS OCCURRING WITH RAINFALL INTENSITY GREATER THAN INTENSITY OF STORM WITH 5-YEAR RETURN PERIOD

	DURATION	INTENSITY FOR 5-YR. RET.PER.	April	May	June	July	Aug.	Sept.	Oct.	Nov.
Analysis of 25 years of Rainfall Records	5	5.7	0	0	1	1	1	0	0	0
	10	4.2	0	0	2	2	0	1	0	0
	15	3.6	0	0	1	2	0	1	0	0
	30	2.4	0	0	1	2	1	1	0	0
	60	1.4	0	0	1	1	2	1	0	0
	120	0.86	0	0	1	1	2	1	0	0
	360	0.35	0	0	0	2	3	1	0	0
	720	0.19	0	0	1	1	2	1	0	0
From "Climate of Winnipeg" [9]	Average days with thunder		7 in 10 years	2	5	7	6	3	1 in 2 yrs.	-
	Days with precipitation (30 year average)		8	9	12	11	10	10	8	10
	Average Precipitation (inches)		1.17	1.97	3.19	2.71	2.76	2.16	1.44	1.14
	Maximum 24-hr rain (inches)		1.74	2.68	6.00	5.26	3.30	3.22	3.04	2.00

Recording rain gauge measurements for Winnipeg are published by the Atmospheric Environment Service for the following three stations:

1. Winnipeg International Airport for the period 1944 to date.

2. North End Sewage Treatment Plant for the period 1961 to date.

3. St. Boniface Waterworks for the period 1961 to date.

A previous analysis has shown that extreme value rainfalls at the Winnipeg International Airport (WIA) are typical of the city since urban influences are not of practical significance in the distribution of excessive quantities. [6] Therefore, measurements taken at the WIA are used in this study since these represent the longest period of record. When data from the WIA records were missing, records from the other two stations were used in assessing extreme rainfall events for the various durations considered.

2.2 Method of Analysis

The extreme rainfall events at WIA were analyzed for durations of 5, 10, 15, 30, 60, 120, 360 and 720 minutes using both Gumbel and Log-Pearson Type III extreme value distributions for comparison purposes.

The Gumbel analysis computes the rainfall (P) with a specified duration and return period according to the following equation:

$$P = \overline{P} + KS \qquad (2.3)$$

Where P and S are the mean and standard deviation respectively of the extreme value series for a specified duration.

K = Gumbels frequency factor which is a function of the number of data in the series and the required return period. (The K value can be computed or obtained from tables, e.g. Gray [5]).

The Gumbel method is generally accepted for extreme value analysis of rainfall events and is used by the Atmospheric Environment Service.

The following similar equation is used for the Log Pearson Type III analysis:

$$LOG\ P = \overline{P}' + K'S' \qquad (2.4)$$

where $\overline{P}'$, S′ are the mean and standard deviation respectively of the logarithms of the extreme value series for a specified duration.

K′ = Pearson Type III frequency factor which is a function of the return period and the coefficient of skew of the data set. (The K′ value is usually obtained from tables, e.g. Ref. 19).

The Log-Pearson Type III analysis probably fits the data better since it takes into account the skew of the data.

The data were corrected for analysis to account for the fact that the use of the partial duration series is more correct than using the annual series, for which data were more readily available.

Extreme rainfall records at the WIA were analyzed for individual months and for combinations of months. The analysis confirmed that the major storms take place during the period June, July, August. The results of comparing the Gumbel and Log-Pearson Type III distributions are given in Figure 2.1. From a practical design point of view the difference between the two methods is small and therefore the results of the Gumbel method were selected for further analysis since it is more widely used.

The results of the intensity-duration-frequency analysis summarized on Figure 2.1 are similar to those given for Winnipeg by Heino and Labelle [6,9]. However, the present analysis covers a longer period of record and it is anticipated that the rainfall analysis will be updated at regular intervals in the future as more data continually becomes available.

The 24-hour duration rainfall was also considered using 101 years of daily rainfall extremes combined from records at St. Johns College (1872-1938) and the WIA (1938-1973). The daily data were corrected by a factor of 1.13 suggested by McKay [12] to convert the observation-day statistics to clock-hour days. The results of the Gumbel analysis for 24hour duration are given in Figure 2.2.

2.3 Snowmelt

2.3.1. Snowfall and Accumulation

Table 2.3 summarizes snowfall precipitation at Winnipeg as given by Thomas [18] and extended by analysis of additional records for the period 1931-1973.

TABLE 2.3

	MEAN MONTHLY SNOWFALL (inches)										
	Period	S	O	N	D	J	F	M	A	M	Annual
Winnipeg	1931-1973	-	2.9	8.5	8.6	9.4	7.6	8.3	4.4	0.9	50.6

The greatest annual recorded winter snowfall during the period 1874-1973 was 99.5 inches which occurred in 1955-1956, and the greatest daily snowfall was 15.0 inches on March 4, 1935. January is the month with the greatest average snowfall (9.4 inches). However, since melt can occur during the winter months, the most important statistic is the amount of snow on the ground at the start of the melt season. Potter [14] summarizes the depth of snow on the ground at Winnipeg Airport for 20 winters, beginning in 1941-42 according to Table 2.4:

TABLE 2.4

	OCCURRENCE OF SNOW COVER OF 1" OR MORE			DEPTH OF SNOW COVER (inches)							
	Date of 1st Snow Cover	Days With Snow Cover	Date of Last Cover	Winter Max.	Oct 31	Nov 30	Dec 31	Jan 31	Feb 28	Mar 31	Apr 30
Earliest or Least	Oct 3	77	Mar 29	8	0	0	0	0	1	0	0
Latest or Greatest	Dec 1	166	May 15	36	5	20	26	35	32	26	2
Mean	Nov 3	126	Apr 11	18	-	-	-	-	11	-	-
Median	Nov 4	122	Apr 9	16	0	1	5	11	9	2	0

For the 20 years summarized the maximum amount of snow on the ground at the end of March was 26 inches. Also, according to Potter, at the end of March, the snow cover is greater than 5" only about 20 percent of the time. Additional snow depth data for the period 1962-1973 were examined in order to update the extremes in the above table, with the changes being maximum depths of snow on the ground at the end of October and April of 7 and 4 inches occurring in 1971 and 1966 respectively.

2.3.2 Snowmelt Analysis

Thunderstorms in Winnipeg are rare in March, but a few can occur in April. [9,10] Since the snow cover at the end of March has been recorded up to 26" deep, it is assumed that runoff from a rain on snow event is most critical at the end of March and throughout April.

The total snowmelt produced at a point is given by the following equation:

$$M = M_{rs} + M_{rl} + M_{ce} + M_p + M_g \qquad (2.5)$$

For snowmelt during rainfall conditions Gray [5] gives the following expressions for the above quantities.

$$M_{rs} = 0.00508\ R_{si}\ (1\text{-}a)\ \text{inches/day} \qquad (2.6)$$

= melt due to shortwave radiation

where a = albedo

R_{Si} = effective solar radiation in Langleys/day

$$M_{r1} = 0.029\ (Ta\text{-}32)\ \text{inches/day} \qquad (2.7)$$

= melt due to longwave radiation

where Ta = air temperature at 10 ft. height

$$M_{ce} = 0.0084\ (Ta\text{-}32)V\ \text{inches/day} \qquad (2.8)$$

= melt due to condensation and convection

where V = wind speed in mph at the 50 ft. height

$$M_p = 0.007\ P(Ta\text{-}32)\ \text{inches/day} \qquad (2.9)$$

= melt due to rainfall

where P = amount of rainfall (inches/day)

$$M_g = 0.02 \qquad (2.10)$$

= melt due to heat transfer from the ground

It must be noted that the above equations were developed by the U.S. Army Corps of Engineers to describe basin snowmelt essentially for rural conditions. However, by using the following snowmelt parameters, it is assumed that a conservative estimate of snowmelt associated with rain on snow can be made.

$$a = 0.50 \text{ (for melting, old snow)}$$

$$R_{si} = 40 \text{ Langley/day (during rain)}$$

$$Ta = 48^{o}F$$

$$V = 30 \text{ mph}$$

Using the above assumptions the total melt is

$$M = .187 + (P \times .00455) \text{ inches/hour} \quad (2.11)$$

(where P is the rainfall in inches/day)

The extreme value rainfall analysis for April is shown on Figure 2.3 and the curves obtained from equation 2.11 for combined rainfall and snowmelt are also indicated. By comparing the latter curves with the rainfall-intensity duration curves for the summer months, it is evident that the summer curves can account for April rainfall plus snowmelt for basins where the time of concentration is less than about 6 hours, considering a return period of 5 years.

The unlikely assumption that the maximum recorded snowpack (26") at the end of March could melt in one day was also considered. This led to the result that the summer curves can account for rainfall plus snowmelt for the 5-year storm for durations shorter than about 2.5 hours.

From this analysis it can be concluded that the summer rainfall-intensity-duration curves give higher intensities for small catchments having a time of concentration less than 6 hours than an April rain plus snowmelt. Therefore, Figure 2.3 should not be used for design purposes. However, estimates of total runoff from major drainage systems should include an assessment of rain on snow in the spring.

2.4 Areal Distribution

For design purposes in some drainage basins, point rainfall sometimes requires conversion to areal rainfall estimates. The corresponding reduction in rainfall intensity can allow for a more economical design of the storm sewer drainage system for relatively large areas. The relationship between point and area rainfall is usually obtained by analysis of specific storm events. The analysis of 18 storms which occurred in Manitoba (published in the "Analysis of Storm Rainfall in Canada" [2] for the period 1911-1968) has indicated that there is practically no reduction in point intensity for areas less than 10 square miles. This conclusion is also maintained by McKay [12] who assumes that "…point rainfall is representative of the rainfall over a 10 square mile area…" McKay has analyzed the storms for the Prairie Provinces and recommends Figure 2.4 for conversion of rainfall extremes for areas larger than 10 square miles.

Several other literature sources also indicate that for areas up to approximately 10 square miles, no reduction in point intensity is necessary, for example, Urban Storm Drainage Criteria Manual [19], G. E. Stout [17], D. M. Hershfield [7]. In addition, Stout concludes that "...in a 10 square mile area, a point rainfall record is a satisfactory index of the frequency distribution of areas mean rainfall...and that urban influences, if present, are not of practical significance in the distribution of excessive rainfall quantities."

2.5 Design Storms

Design storms for various frequencies can be derived from the frequency intensity-duration curves using the methods outlined by Keiffer [8], Bandyopadhyay [1] and others. The points computed by the Gumbel analysis can be fit to an equation of the following form for each return frequency.

$$i_{av} = \frac{a}{(t_d + b)^c} \tag{2.12}$$

where t_d is the duration and a, b and c are constants. Therefore, for any duration the total volume of water is

$$P = i_{av} \frac{t_d}{60}$$

$$= \frac{a}{(t_d + b)^c} \frac{t_d}{60} \tag{2.13}$$

For a so called "completely advanced" rainstorm where the peak occurs immediately

$$P = \frac{1}{60} \int_o^{t_d} i \, dt_d \tag{2.14}$$

Differentiating equation 2.14 the rate of change of the total volume of rainfall can be expressed as

$$\frac{dP}{dt_d} = \frac{i}{60}$$

(2.15)

Differentiating equation 2.13

$$\frac{dP}{dt_d} = \frac{a}{60}\left[\frac{(1-c) \quad t_d \quad + \quad b}{(t_d \quad + \quad b)^{c \ + \ 1}}\right]$$

(2.16)

Now combining equations 2.15 and 2.16

$$i = \frac{a\left[(1-c) \quad t_d \quad + \quad b\right]}{(t_d \quad + \quad b)^{c \ + \ 1}}$$

(2.17)

Equation 2.17 is the equation for a completely advanced storm pattern but can be modified for intermediate storms by assuming that within the maximum period of any rainfall the duration t_d can be segregated into the period t_b occurring before the most intense moment and the one t_a after the most intense moment by defining $r = t_b/t_d$ which results in the following expressions

$$t_b = r \ t_d$$

(2.18)

$$t_a = (1-r) \quad t_d$$

(2.19)

Where t_b is measured from the peak to the left and t_a is measured from the peak to the right (Figure 2.7).

The following equations which represent the intensity before and after the peak are derived from the latter 3 relationships.

$$i_b = \frac{a\left[(1-c)\dfrac{t_b}{r} + b\right]}{\left(\dfrac{t_b}{r} + b\right)^{1+c}}$$

(2.20)

$$i_a = \frac{a\left[(1-c)\dfrac{t_a}{1-r} + b\right]}{\left(\dfrac{t_a}{1-r} + b\right)^{1+c}}$$

(2.21)

The synthetic hyetograph determined from equations 2.20 and 2.21 will have the same average intensity for all times of concentration as the intensity-duration curve from which the constants a, b, and c are derived. These constants are estimated for each return period by fitting the Gumbel intensity-duration data to the general form of equation 2.12 and using a regression equation of the following form:

$$\text{Log } i = \text{Log } a - c \text{ Log } (t_d + b)$$

(2.22)

Table 2.5 gives the values of a, b, and c and the resulting equations which were calculated to fit the Gumbel intensity-duration data for the period June, July, August. The curves are also shown in Figure 2.5.

TABLE 2.5

RAINFALL INTENSITY EQUATIONS DERIVED FROM GUMBEL ANALYSIS

Return Period (Years)	a	b	c	Intensity Equation (Inches/Hour)	Coefficient of Correlation
2	32.4	7	0.813	$i = \frac{32.4}{(t_d + 7)^{.813}}$	-0.9999
5	47.2	8	0.828	$i = \frac{47.2}{(t_d + 8)^{.828}}$	-0.9997
10	60.2	9	0.842	$i = \frac{60.2}{(t_d + 9)^{.842}}$	-0.9996
25	72.5	9	0.842	$i = \frac{72.5}{(t_d + 9)^{.842}}$	-0.9994

The 2, 5 and 10 year intensity-duration design curves defined in Table 2.5 are compared with the old design curves currently used by the City of Winnipeg (equations 2.1 and 2.2) in Figure 2.6. The new equations generally fit the data better and it is apparent that the return frequencies associated with the old curves should be modified slightly.

The value of r in equations 2.20 and 2.21 was evaluated by an analysis of past excessive rainfall events in Winnipeg. The method followed was used by Keiffer and Bandyopadhyay [8, 1] and involves calculating the average antecedent mass and time to peak within various rainfall durations for a series of excessive rainfall events. The required data was extracted from the recording rain gauge charts for the selected events. The weighted average value of r thus determined from a series of over 60 excessive rainfall events is $r = 0.31$.

The design storms resulting from the solution of equations 2.20 and 2.21 for the 2, 5, 10 and 25 year frequencies are shown in Figures 2.7 to 2.10 respectively. The use of these hyetographs for design purposes and the corresponding method of discretization for practical applications is discussed in Section 3.0.

For completeness, the Gumbel analysis was carried out for the months of April and May and the results are given on Figures 2.11 and 2.13 respectively. The corresponding design hyetographs were also developed and are presented in Figures 2.12 and 2.14.

CHAPTER 2 REFERENCES

1. Bandyopadhyay, M. "Synthetic Storm Pattern and Runoff for Gauhati, India", Journal of the Hydraulics Division, #3887, May, 1972, HY5

2. Canada, Dept. of Transport, Meteorological Branch, Toronto, Ont. Continuing publication: "Storm Rainfall in Canada"

3. Department of Transport, Meteorological Branch, Daily Climatological Data, Winnipeg, Manitoba, March, 1872 February, 1938 at St. John’s College March, 1938 December, 1939 at Winnipeg. International Airport

4. Ferguson, H.L. and D. Storr. "Some Current Studies of Local Precipitation Variability over Western Canada". Proc. of American Water Resources Assoc. Symposium, Banff, June Series No. 7

5. Gray, D.M. (ed.) "Principles of Hydrology", NRC, Ottawa, 1970

6. Heino, R.L. "Design Curves for the High-Intensity, Short Duration Rainfall at Winnipeg, Manitoba". Undergraduate thesis, Department of Civil Engineering, University of Manitoba

7. Hershfield, D.M. "Extreme Rainfall Relationships", Journal of the Hydraulics Division, ASCE, November, 1962.

8. Keifer, C.J. and H.H. Chu. "Synthetic Storm Pattern for Drainage Design", Journal of the Hydraulics Division, Paper #1332, August, 1957, HY4

9. Labelle, J.J., R.J.Brown and M.D. Hasinoff, "The Climate of Winnipeg", Department of Transport, Meteorological Branch, Cir 4437, CL 135, 1966

10. Lipson, E.D. "A Study of Winnipeg Thunderstorms", Dept. Of Transport, Meteorological Branch, Cir 4198, TEC562, March, 1965

11. Langley, R.W. "The Climate of the Prairie Provinces", Environment Canada, Atmospheric Service Climatological Studies, No, 13, Toronto, 1972

12. McKay, G.A. "Statistical Estimates of Precipitation Extremes for the Prairie Provinces", Canada Department of Agriculture, Prairie Farm

Rehabilitation Administration, Engineering Branch - Hydrology Division, 1965

13. Newton-Smith, W.H. and G.D.V. Williams, "The Relationship Between Rain Gauge Network Density and Areal Rainfall Estimates on the Prairies", Dept. of Transport, Met. Brach, Cir 4137, TEC-542, October

14. Potter, J.G. "Snow Cover", Climatological Studies No. 3, Toronto, 1965

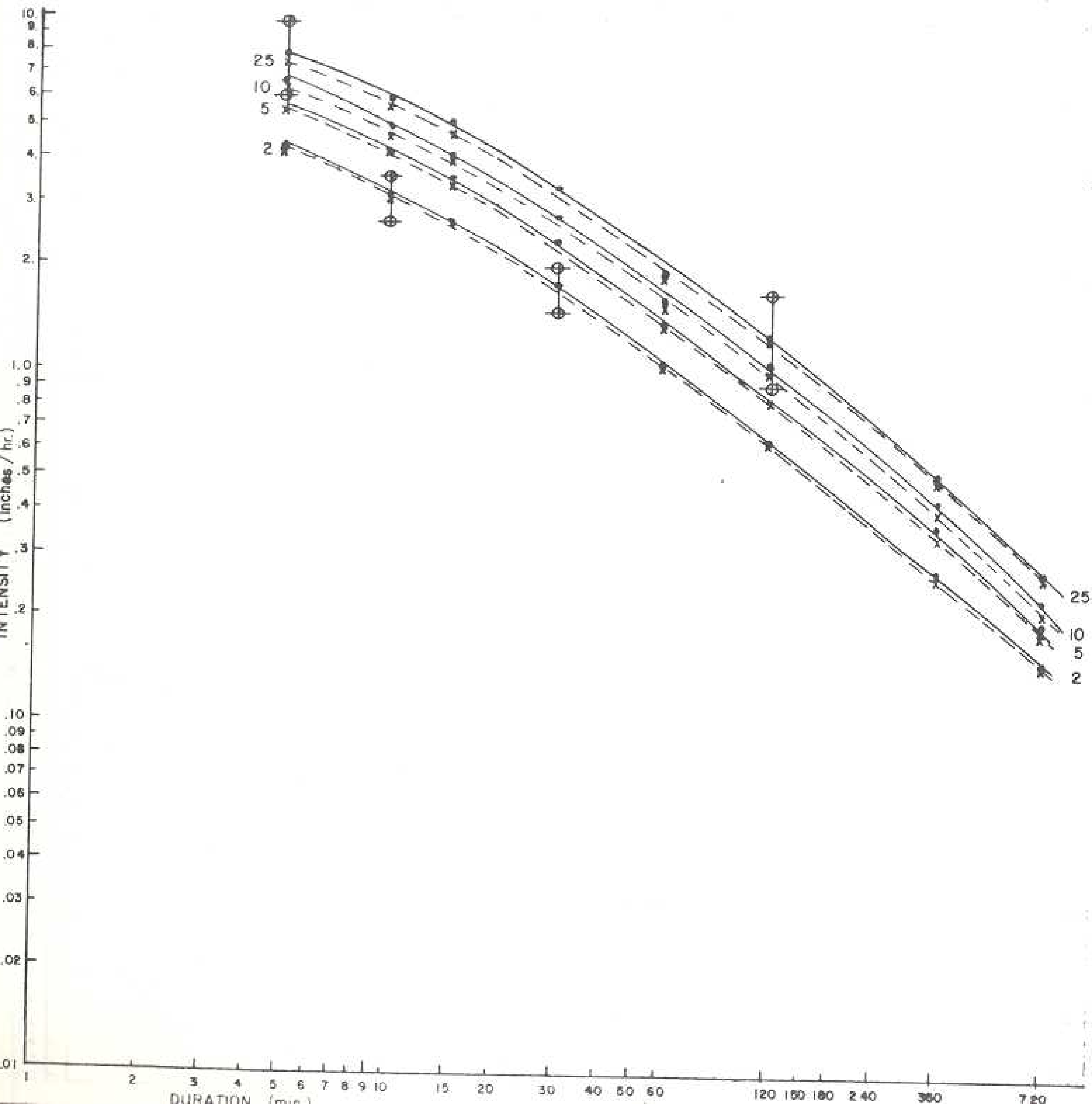
15. Preul, H.C. and C.N. Papadakis, "Development of Design Storm Hyetographs for Cincinnati, Ohio", Water Resources Bulletin, Vol. 9, No. 2, April, 1973

16. Simpson, W. and C.D. Henry, "Dry and Wet Spells at Winnipeg", Dept. of Transport, Met. Branch Cir. 4507, TEC-630, November, 1966

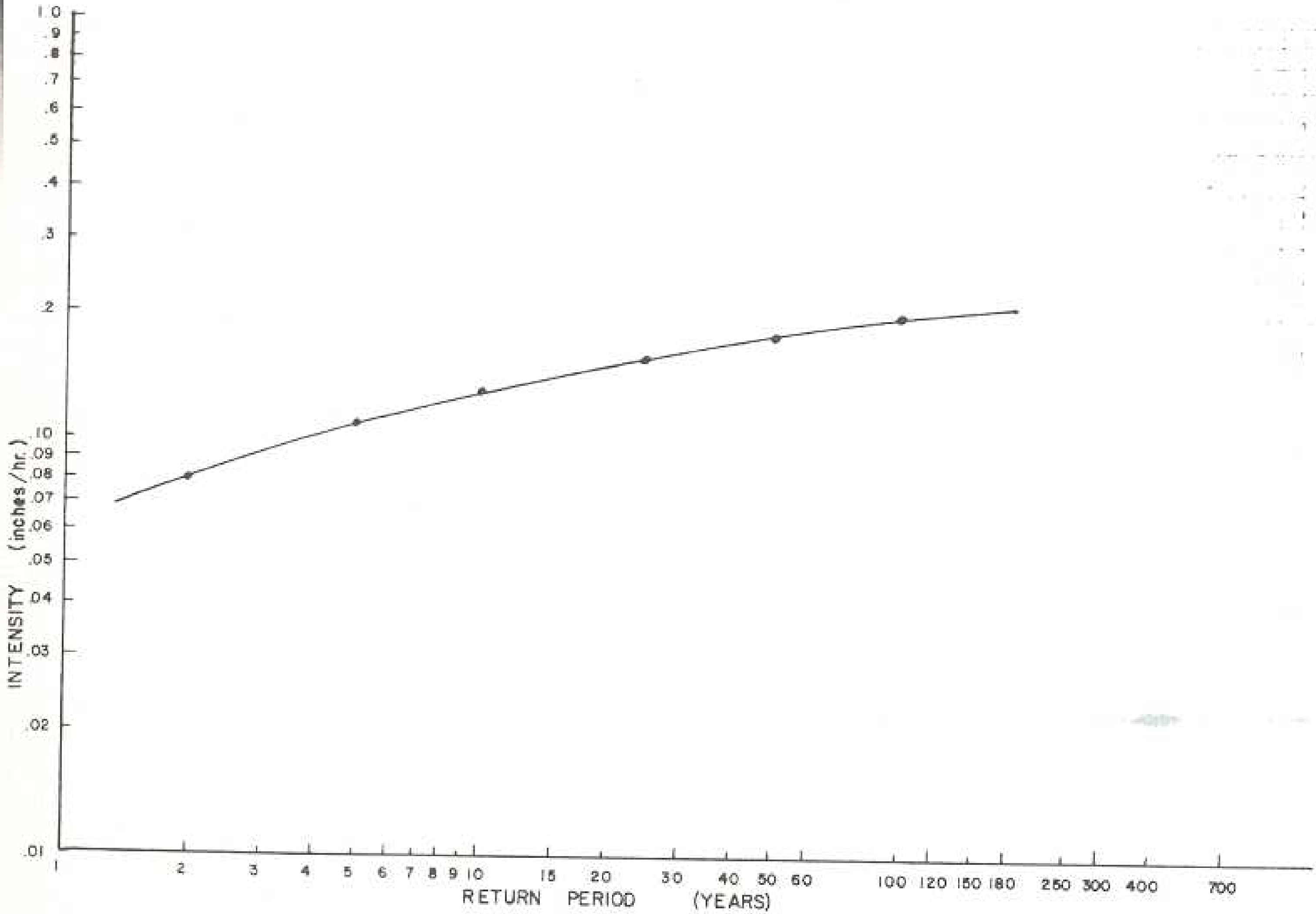
17. Stout, G.E., E.A. Huff, "Studies of Severe Rainstorms in Illinois", Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 4, July, 1962, pp. 129-146

18. Thomas, M.K., "Snowfall in Canada", Dept. of Transport, Meteorological Branch, Circ. 3977, TEC-503, January 24, 1964

19. Wright-McLaughlin Engineers, "Urban Storm Drainage Criteria Manual", Denver, Colorado, March, 1969



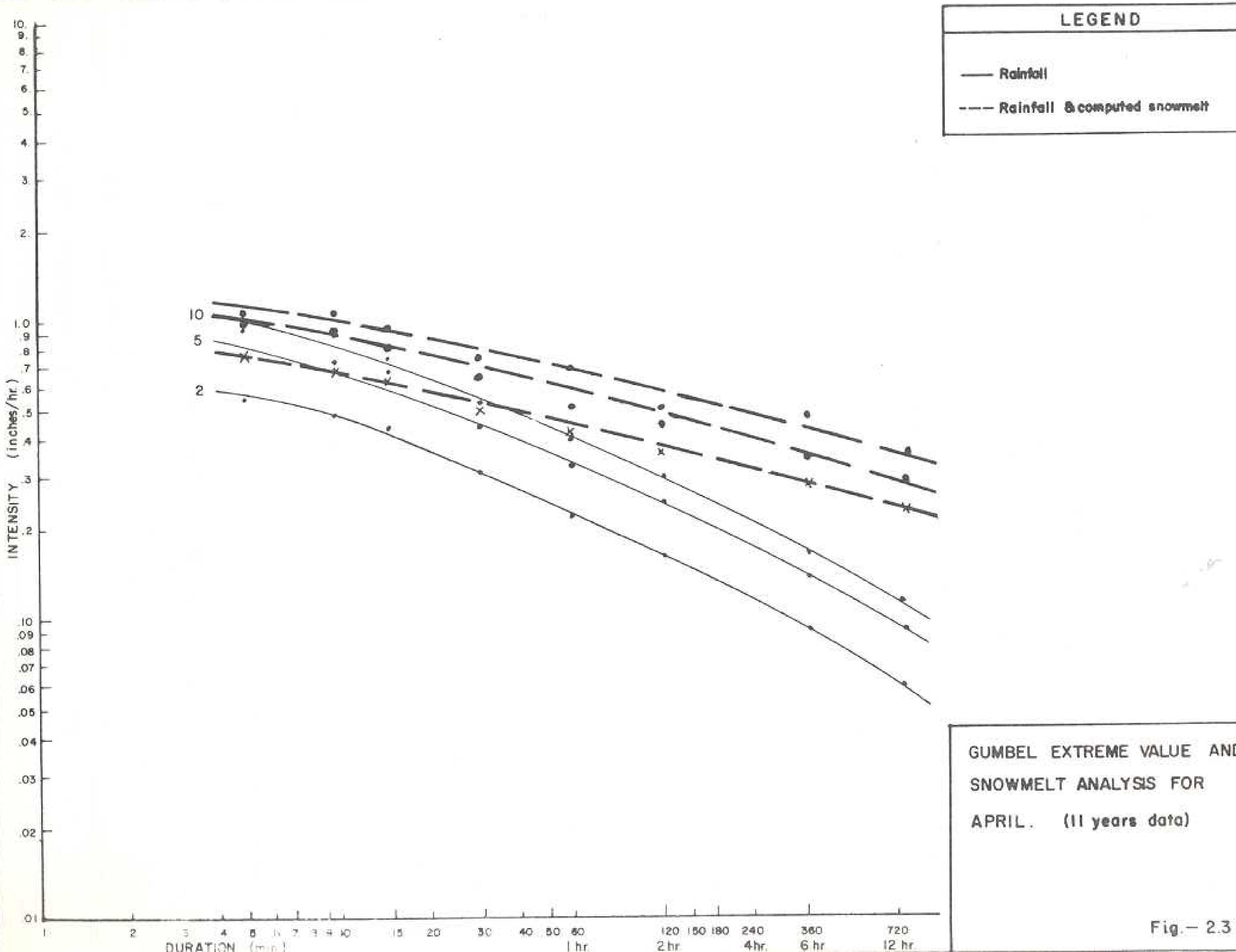
COMPARISON OF GUMBEL AND LOG-PEARSON TYPE III EXTREME VALUE RAINFALL ANALYSIS FOR JUNE, JULY AND AUGUST. (25 years data)



GUMBEL RAINFALL ANALYSIS FOR 24 HOUR DURATION.

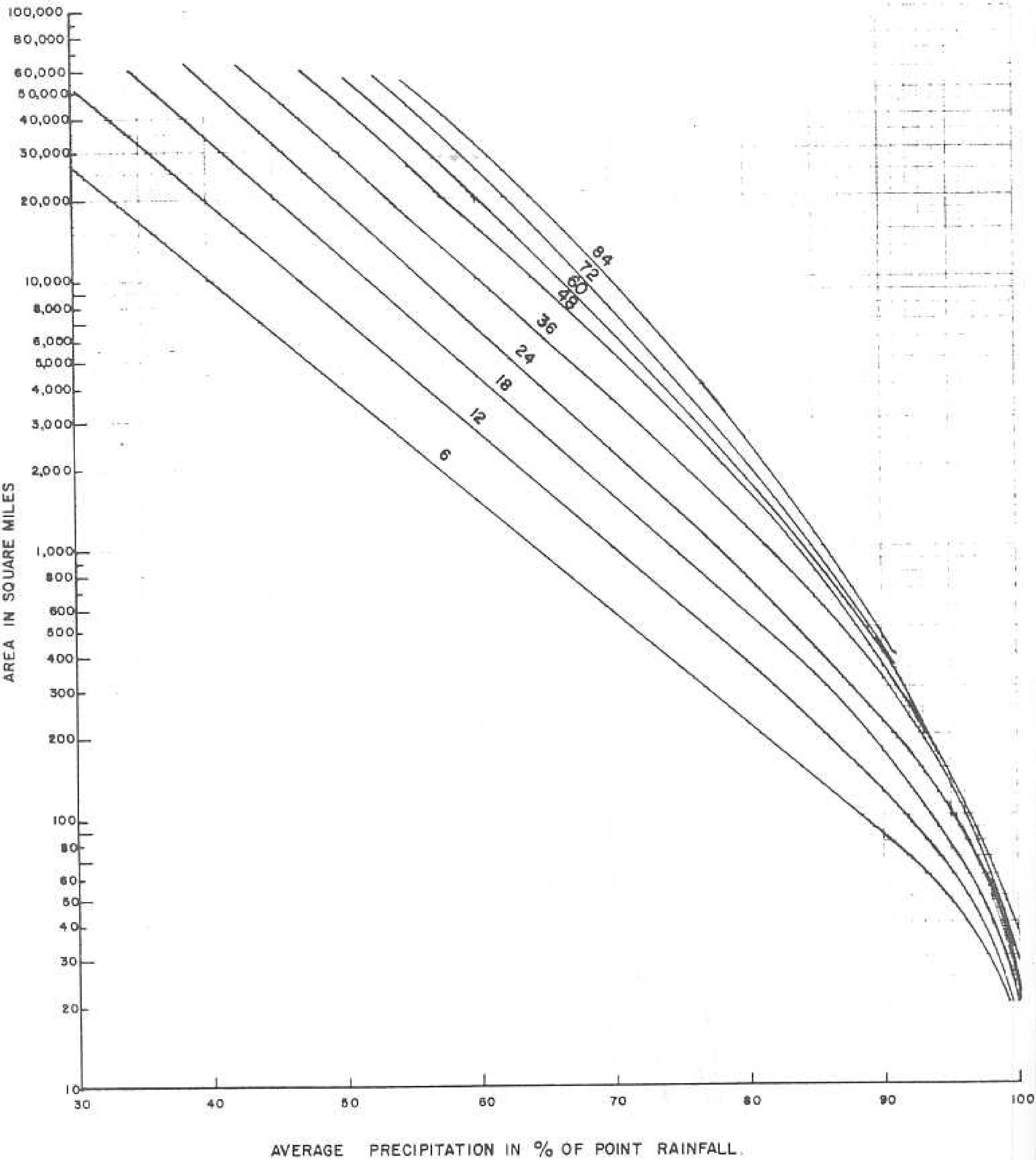
(101 years data)

Fig. - 2.2



GUMBEL EXTREME VALUE AND SNOWMELT ANALYSIS FOR APRIL. (11 years data)

Fig.— 2.3



LEGEND

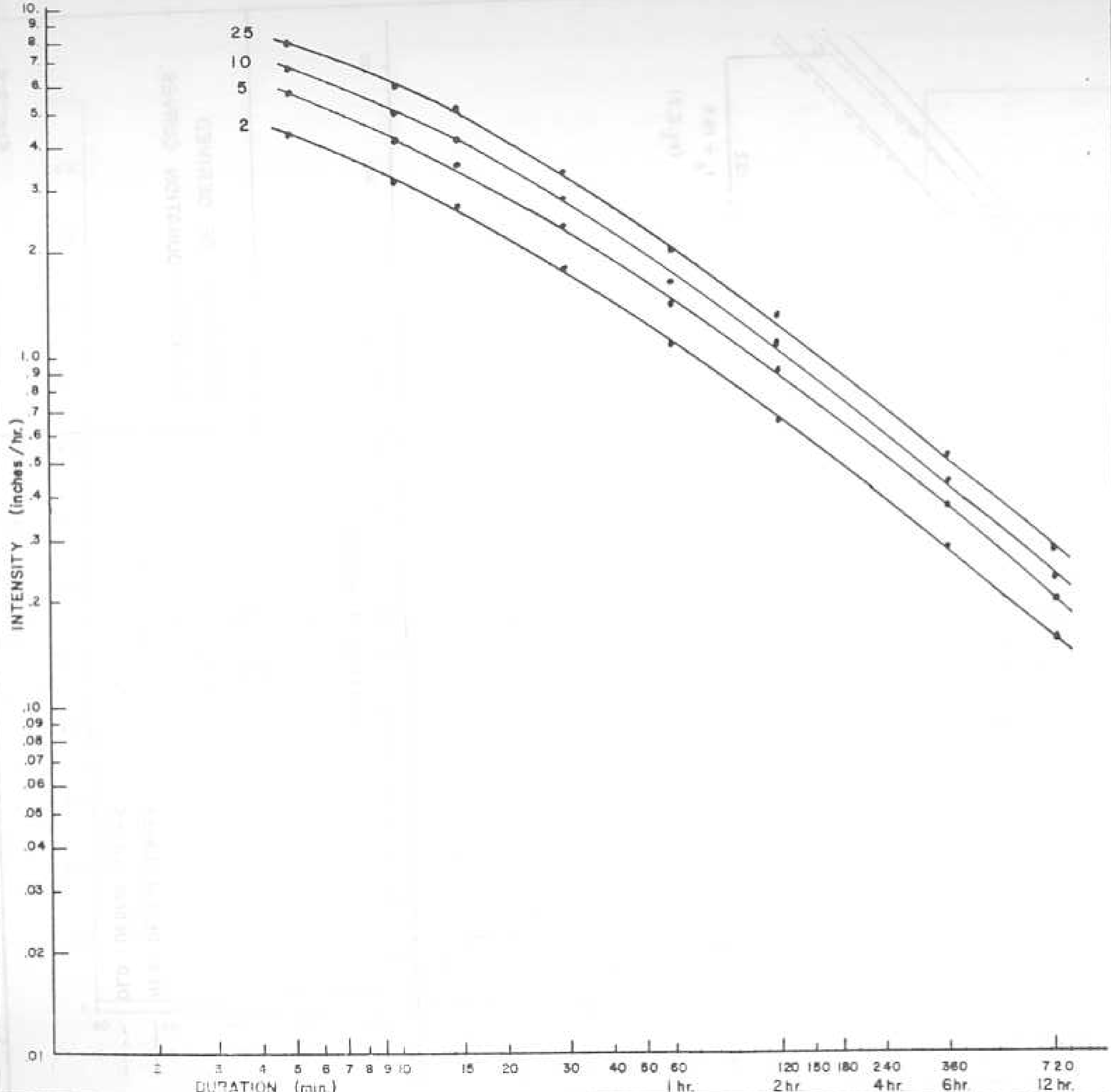
72 Duration in hours.

CONVERSION OF RAINFALL EXTREMES

ACCORDING TO AREA.

BASED ON STORMS OF 24 HOURS OR LONGER DURATION (McKay Ref. 12)

Fig. 2.4



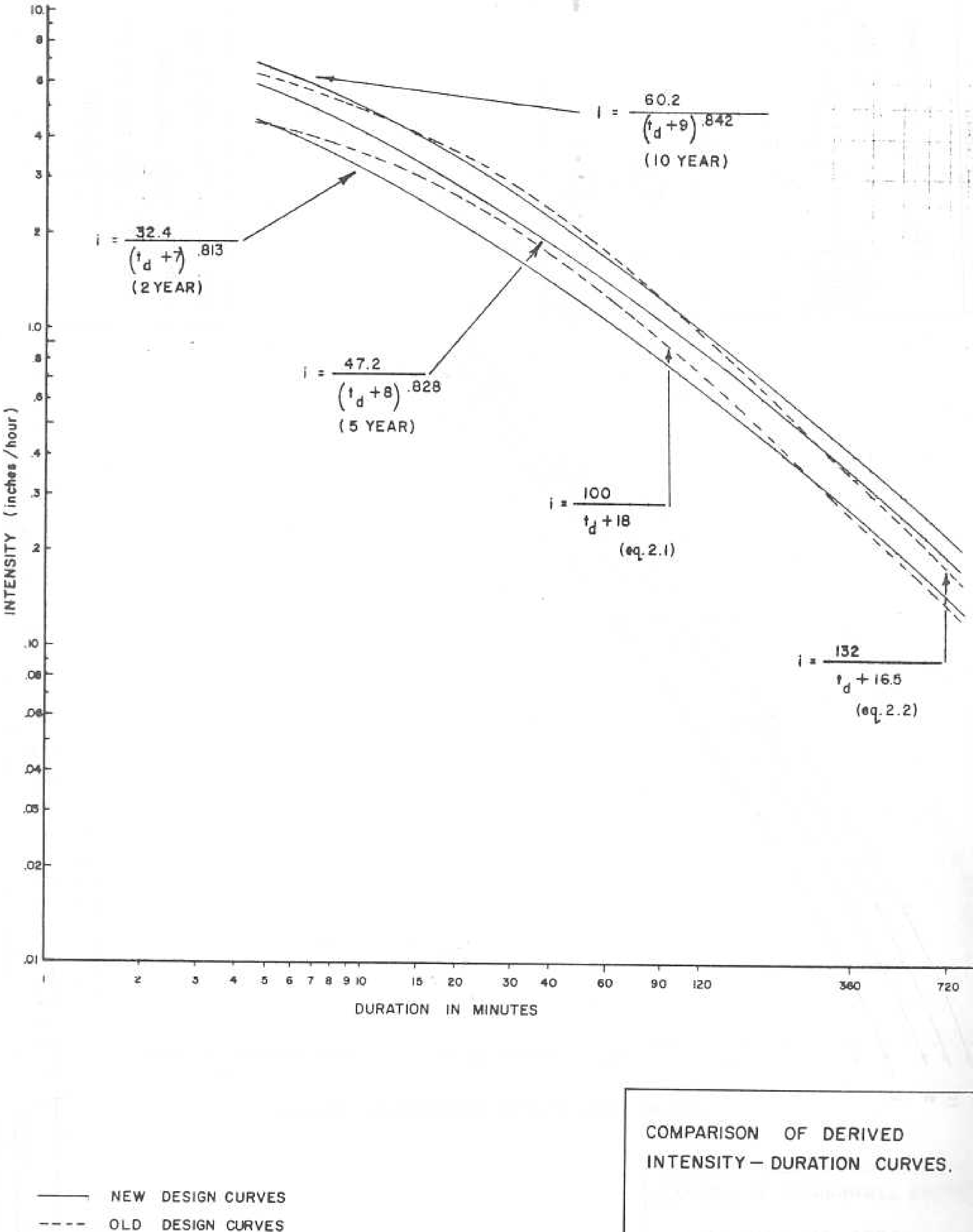
LEGEND

- • Points computed from historical data.
- —— Fitted intensity-duration curve.

RETURN PERIOD	EQUATION OF CURVES FITTED TO DATA
2	$i = \frac{32.4}{(t_d + 7)^{.813}}$
5	$i = \frac{47.2}{(t_d + 8)^{.828}}$
10	$i = \frac{60.2}{(t_d + 9)^{.842}}$
25	$i = \frac{72.8}{(t_d + 9)^{.842}}$

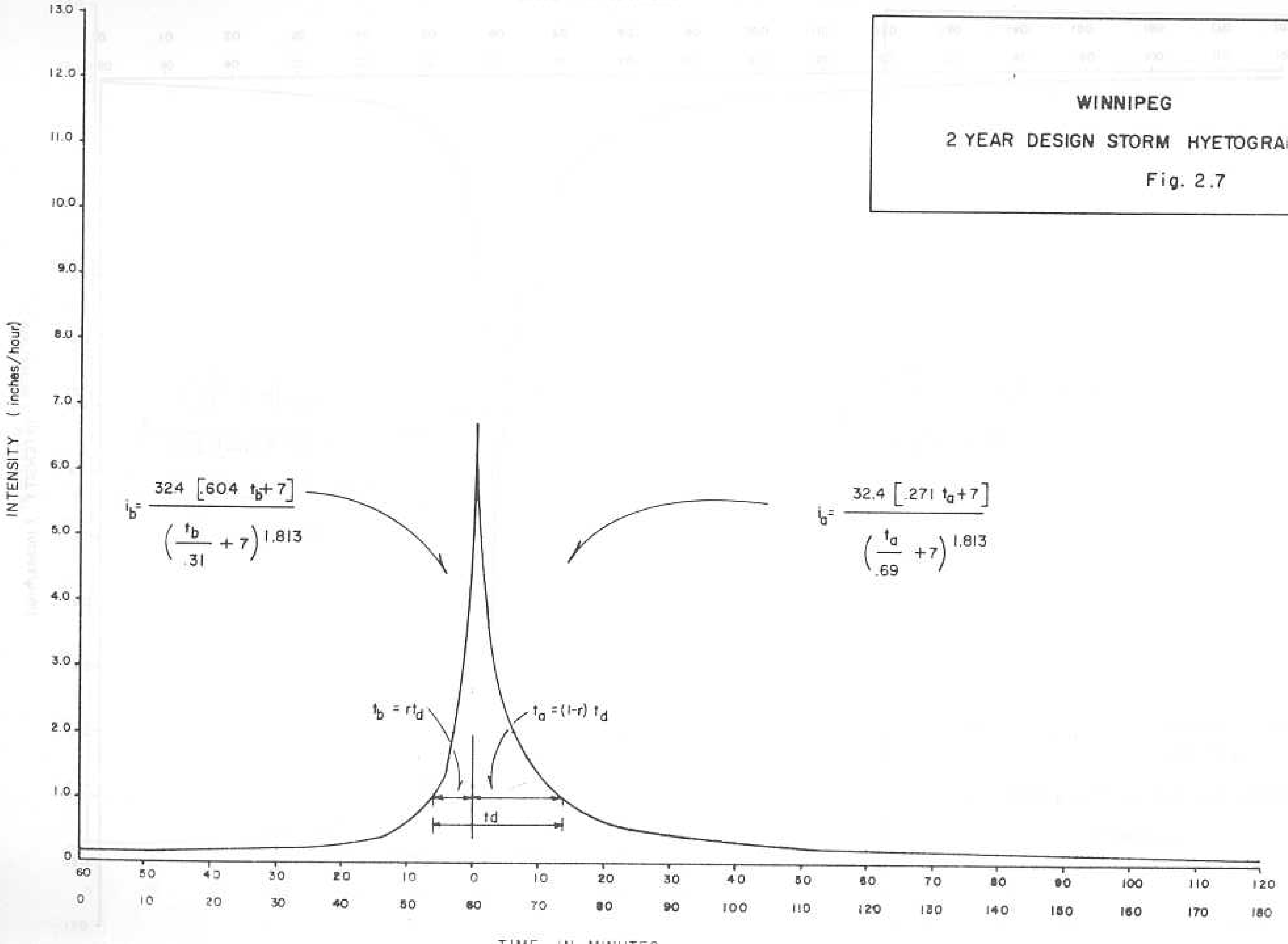
FITTED INTENSITY-DURATION CURVES. GUMBEL ANALYSIS FOR JUNE, JULY AND AUGUST.
(25 years data)

Fig.- 2.5



COMPARISON OF DERIVED INTENSITY – DURATION CURVES.

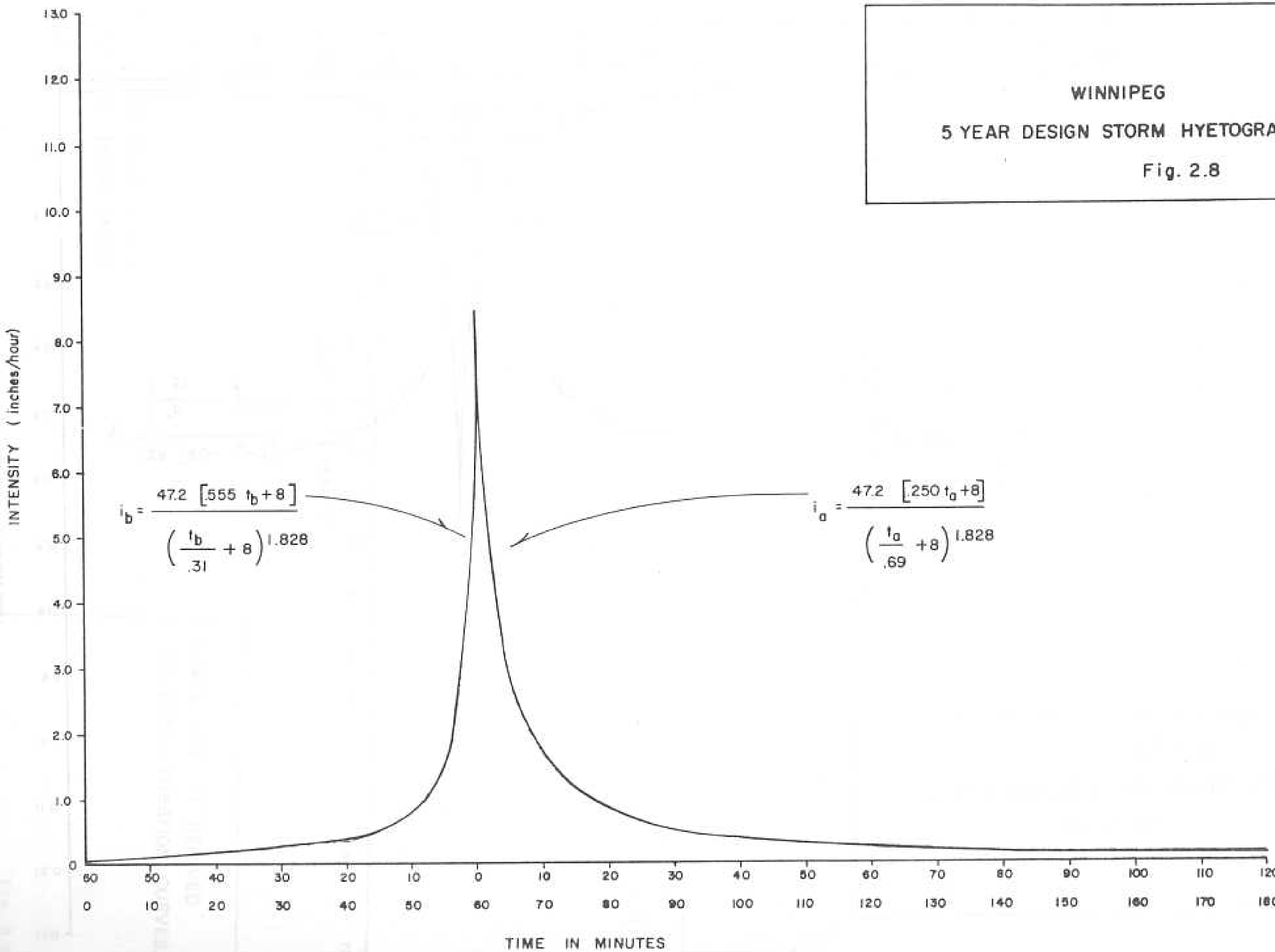
Fig. 2.6



WINNIPEG

2 YEAR DESIGN STORM HYETOGRAPH

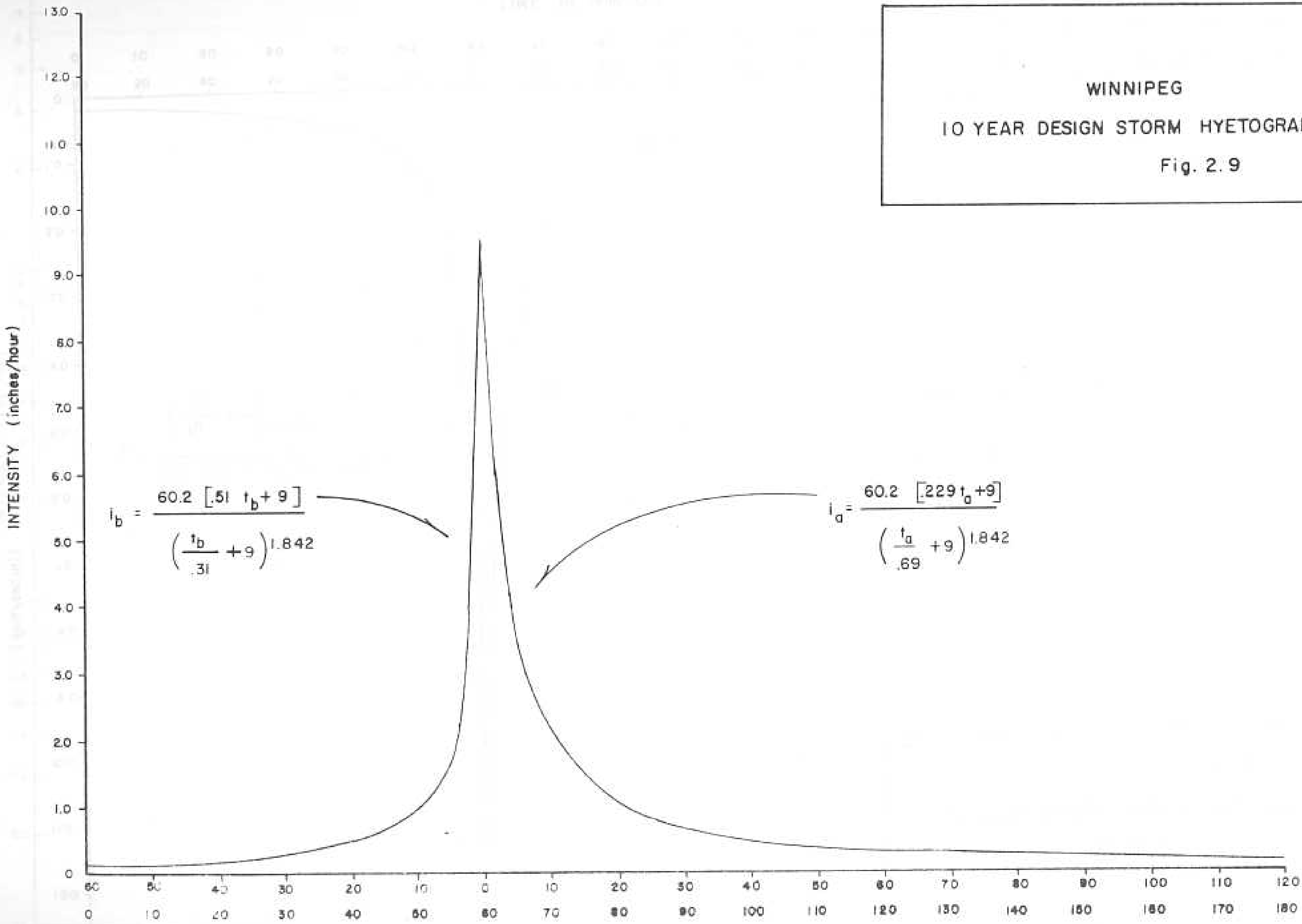
Fig. 2.7



WINNIPEG

5 YEAR DESIGN STORM HYETOGRAPH

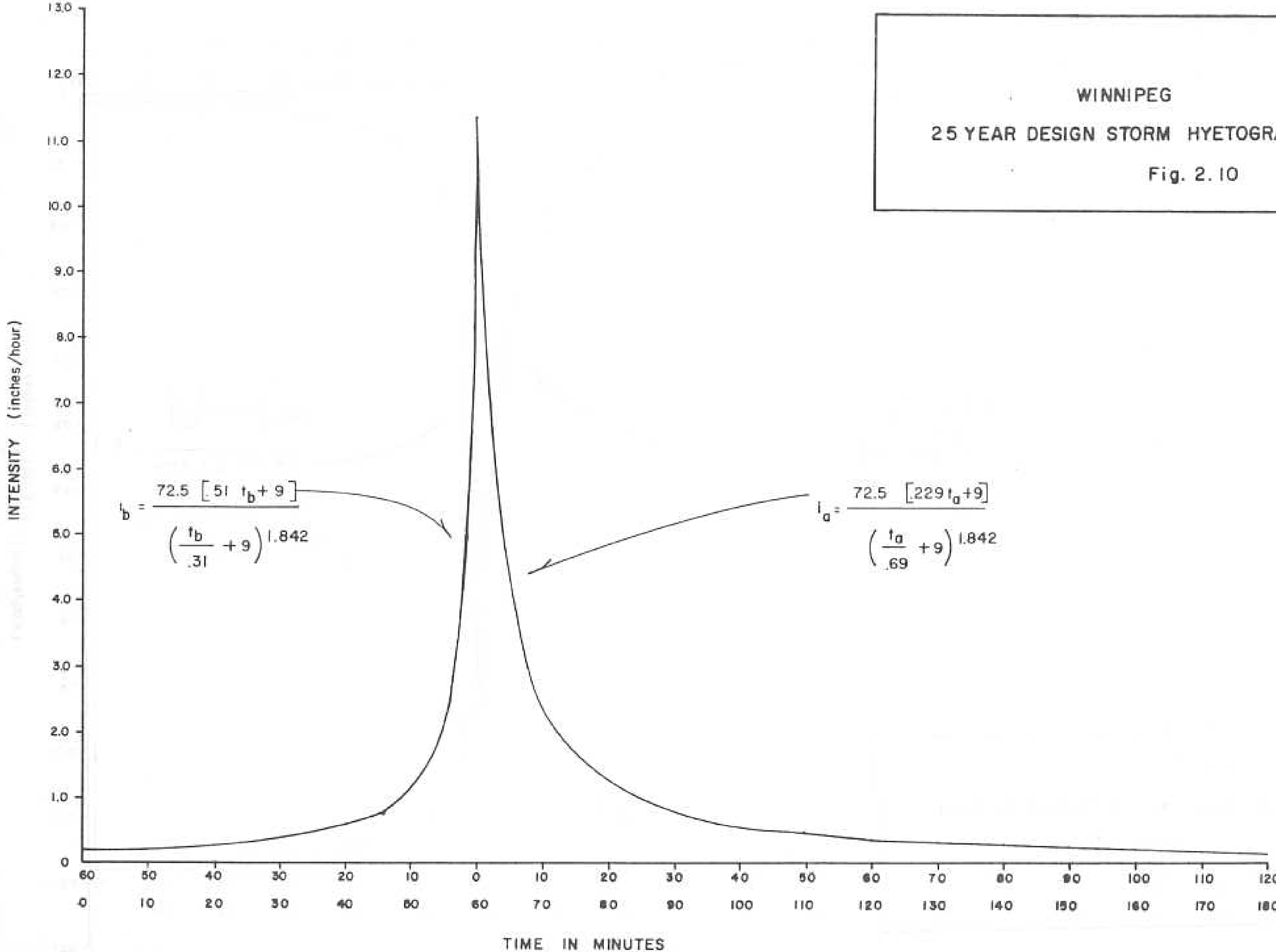
Fig. 2.8



WINNIPEG

10 YEAR DESIGN STORM HYETOGRAPH

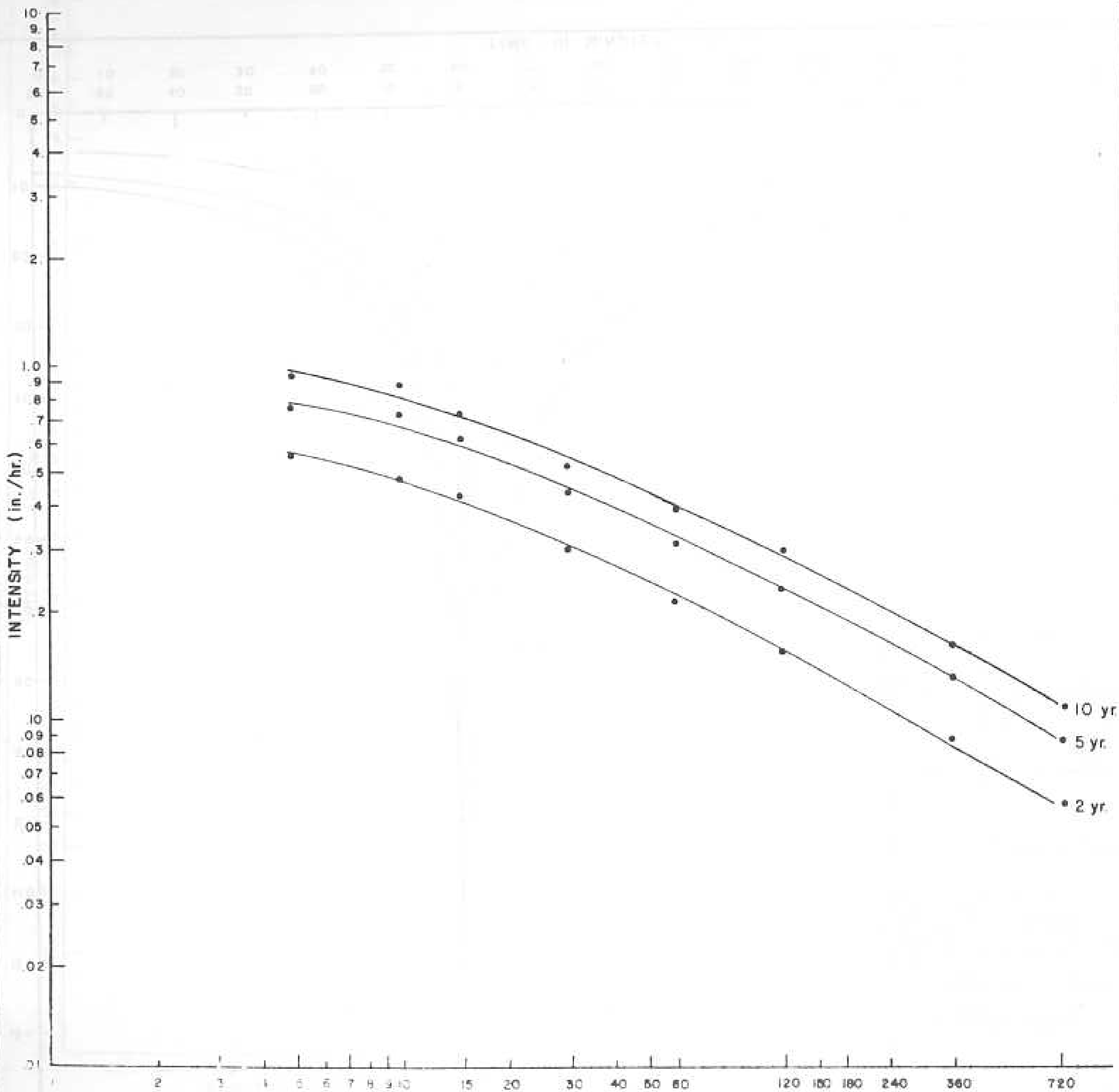
Fig. 2.9



WINNIPEG

25 YEAR DESIGN STORM HYETOGRAPH

Fig. 2.10



LEGEND

- • Points computed from historical data.
- —— Fitted intensity–duration curve.

RETURN PERIOD	EQUATION OF CURVES FITTED TO DATA
2	$i = \frac{2.44}{(t_d + 8)^{.5687}}$
5	$i = \frac{3.46}{(t_d + 9)^{.5570}}$
10	$i = \frac{4.08}{(t_d + 9)^{.5500}}$

FITTED INTENSITY-DURATION CURVES. GUMBEL ANALYSIS FOR APRIL.

(11 years data)

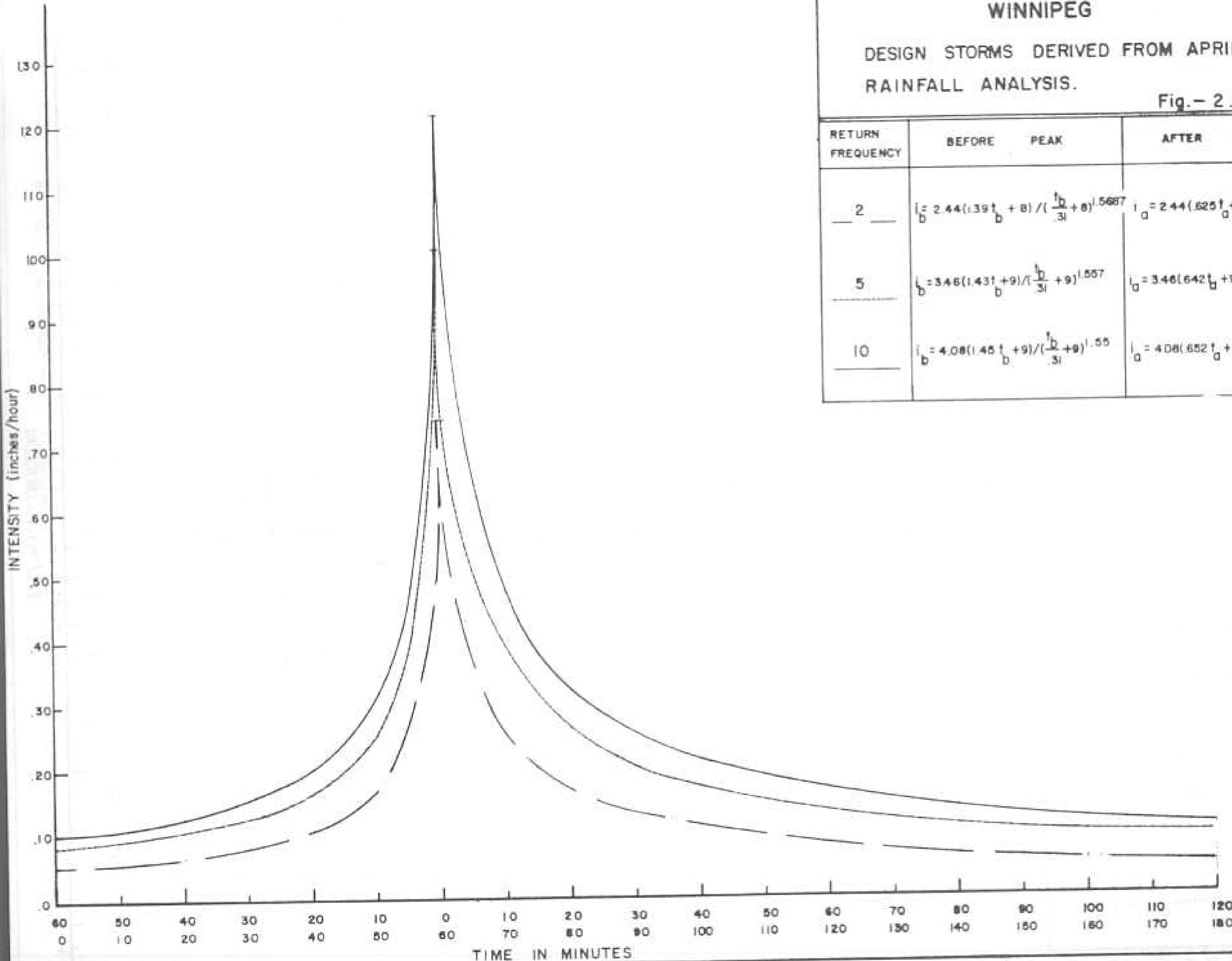
Fig. - 2 11

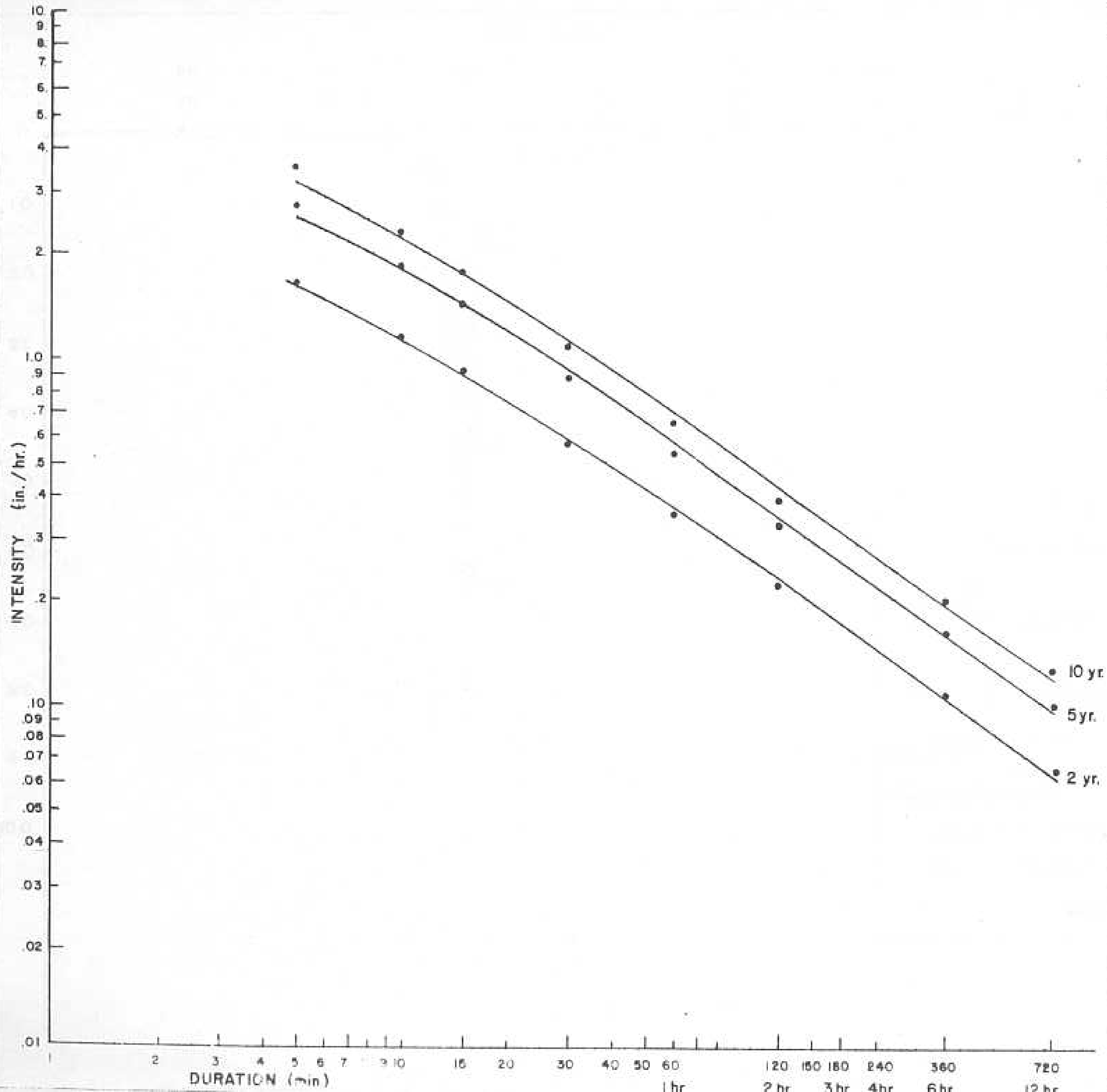
WINNIPEG

DESIGN STORMS DERIVED FROM APRIL RAINFALL ANALYSIS.

Fig.– 2.12

RETURN FREQUENCY	BEFORE PEAK	AFTER PEAK
2	$i_b = 2.44(1.39 t_b + 8)/(\frac{t_b}{.31} + 8)^{1.5687}$	$i_a = 2.44(.625 t_a + 8)/(\frac{t_a}{.69} + 8)^{1.5687}$
5	$i_b = 3.46(1.43 t_b + 9)/(\frac{t_b}{.31} + 9)^{1.557}$	$i_a = 3.46(.642 t_a + 9)/(\frac{t_a}{.69} + 9)^{1.557}$
10	$i_b = 4.08(1.45 t_b + 9)/(\frac{t_b}{.31} + 9)^{1.55}$	$i_a = 4.08(.652 t_a + 9)/(\frac{t_a}{.69} + 9)^{1.55}$





LEGEND

- • Points computed from historical data.
- —— Fitted intensity – duration curve.

RETURN PERIOD	EQUATION OF CURVES FITTED TO DATA
2	$i = \frac{7.26}{(t_d+3)^{.7243}}$
5	$i = \frac{11.9}{(t_d+3)^{.7399}}$
10	$i = \frac{15.03}{(t_d+3)^{.7438}}$

FITTED INTENSITY-DURATION CURVES. GUMBEL ANALYSIS FOR MAY.

(26 years data)

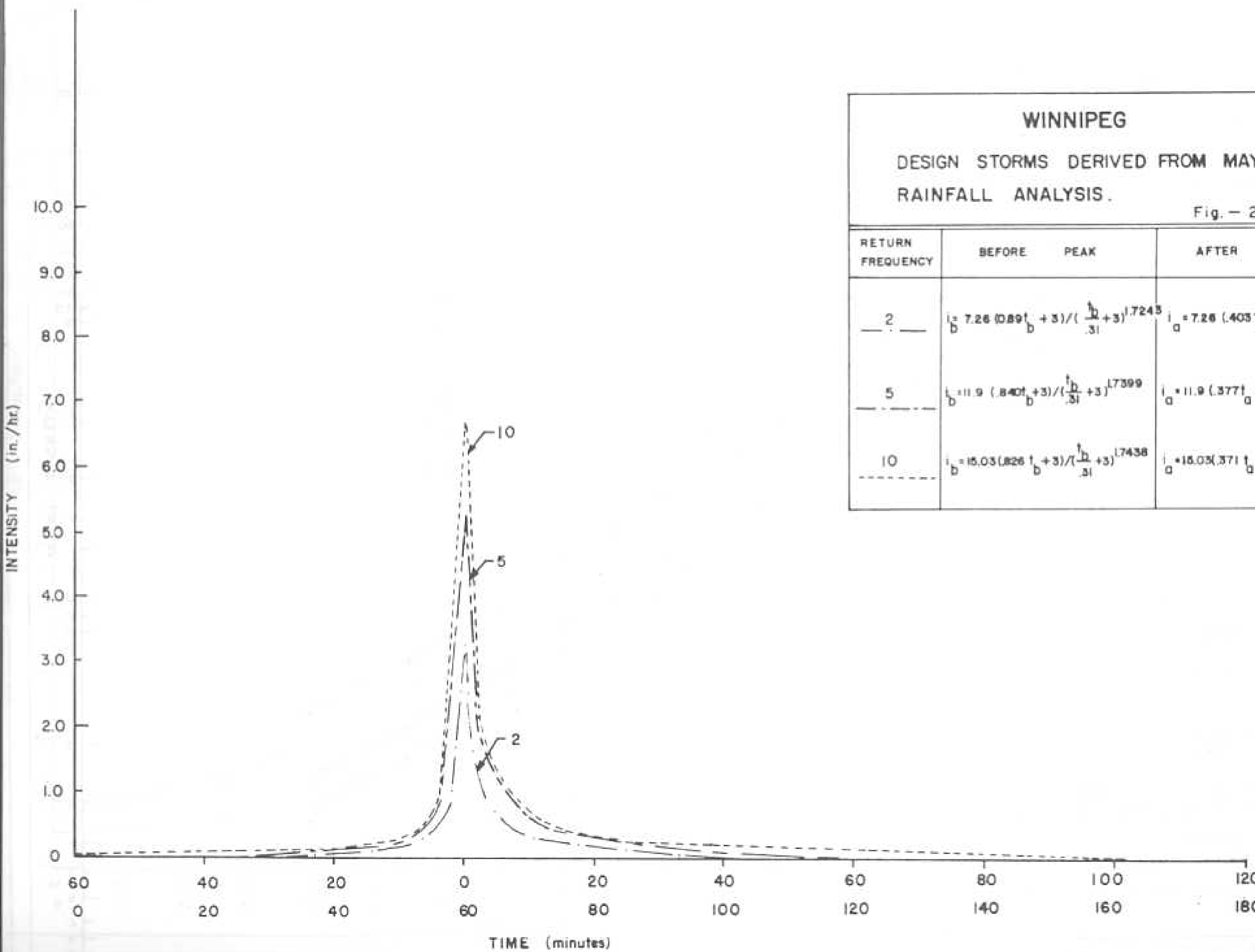
Fig. – 2.13

WINNIPEG

DESIGN STORMS DERIVED FROM MAY RAINFALL ANALYSIS.

Fig. — 2.14

RETURN FREQUENCY	BEFORE PEAK	AFTER PEAK
2	$i_b = 7.26\,(0.89 t_b + 3)/(\frac{t_b}{.31} + 3)^{1.7243}$	$i_a = 7.26\,(.403 t_a + 3)/(\frac{t_a}{.69} + 3)^{1.7243}$
5	$i_b = 11.9\,(.840 t_b + 3)/(\frac{t_b}{.31} + 3)^{1.7399}$	$i_a = 11.9\,(.377 t_a + 3)/(\frac{t_a}{.69} + 3)^{1.7399}$
10	$i_b = 15.03\,(.826 t_b + 3)/(\frac{t_b}{.31} + 3)^{1.7438}$	$i_a = 15.03\,(.371 t_a + 3)/(\frac{t_a}{.69} + 3)^{1.7438}$



CHAPTER 3

RUNOFF

3.1 General

This chapter deals with the examples of methods for estimating stormwater runoff volume and peak flow. In view of the general drainage principles outlined in Chapter 1.0, different methods at various levels of sophistication can be used in order to complement the concepts of preliminary and final design. Reviewing briefly, sophisticated computer hydrograph models can be used when sufficient input data are available (for final design stages) and when their use is economically justifiable (for large drainage areas, for example). In general, the simple rational formula can be used for sewer design within certain limitations (discussed in section 3.3.3) and for rough preliminary designs. So-called intermediate hydrograph methods (such as the Isochrone method given in section 3.4.4) lend themselves either to computer or fairly easy hand computations. Such methods can also be used at initial stages of design but require some knowledge of the underlying runoff characteristics of a drainage basin and therefore tend to "bridge the gap" between the oversimplified approach of the rational method and detailed computer models.

From a theoretical point of view, hydrograph methods are generally superior to the rational method for several reasons. For example, hydrograph methods can model the actual runoff distribution in time and space since they account not only for the actual storm pattern (i.e. the storm concept given in Chapter 2.0) but also can account for spatial variations in other physical parameters such as infiltration, overland flow (slope) and surface detention and depression storage. Hydrograph methods can also take into consideration pipe storage affects which tend to reduce the peak flow. These concepts are explained in more detail in the following sections.

On the other hand, among other deficiencies the rational method lumps all physical runoff parameters into one coefficient. However, with respect to preliminary design of the storm sewer system, the rational method is adequate in order to obtain an initial layout of the drainage system. For final design of pipes, the rational method should only be used for relatively small areas.

For final design of pipe networks and storage facilities a detailed hydrograph model such as the EPA-SWMM is required.

Some of the theory and limitations of the models which can be used for various design levels are discussed from the point of view of runoff in the following sections. The design of storage facilities is discussed in detail in Chapter 4.

3.2 General Description of the Rainfall-Runoff Process

The rainfall/runoff process on an urban watershed can be divided into hydrologic and hydraulic components. Precipitation, losses to soil infiltration and depression storage and the resulting routed overland flow comprise the hydrologic regime from which an inlet hydrograph is produced. The summation of inflow hydrographs and the routing of the resultant flows through the sewer network considering the influence of storage, friction, manhole and junction losses and flow divisions under free surface or pressurized flow conditions comprise the hydraulic regime.

The hydraulic problems involved in routing flows through conduits have been well understood for some time so that when the urban runoff hydrograph models were developed it only remained to determine what degree of accuracy is necessary for the purpose of analysing or designing sewer networks. The hydrologic phenomena, on the other hand were not well understood and involve a much more complex interrelationship between many different physical processes.

An urban watershed contains two different types of elements: collecting channels (gutters, lateral, main and trunk sewers) and surfaces, which may be impervious with direct connections to the sewer system or pervious. Pervious areas may also contain scattered disconnected impervious areas. The degree of imperviousness, which is the ratio of the directly connected impervious area to the total watershed area, is a parameter defining the overall influence of urbanization.

During the precipitation on a pervious area, water is continuously being abstracted by infiltration. Other abstractions considered in rural hydrology such as interception and evaporation are usually negligible for urban storm events.

Various formulations of the "infiltration process" have been developed but most reflect the Characteristic exponential decrease in infiltration Capacity from a high initial (dry) value to a lower (saturated) value. For example, Horton’s equation for infiltration is

$$F = f_o + (f_1 + f_o) e^{-\alpha t} \qquad \textbf{3.1}$$

where F = the infiltration Capacity

f_l, f_O = the initial and ultimate infiltration rates respectively

4 = a Constant depending upon soils and vegetation and

t = the time from the start of the precipitation in minutes.

The actual infiltration which occurs at any time, depends upon the volume of rainfall which has preceded that time. If the total precipitation is less than the total infiltration capacity up to a certain time then some capacity for infiltration remains in excess of that which would be available considering only the duration of the storm to that point. There will be no water available for runoff, of course, if the accumulated precipitation never exceeds the accumulated infiltration capacity. Precipitation in excess of the infiltration capacity becomes available to depression storage.

Depressions on a natural surface vary greatly in size. Therefore, there is some instantaneous runoff from areas without depression, and as the accumulated volume of precipitation increases, more and more depressions are filled until the entire area is contributing to runoff. At the same time, infiltration is still taking place at a constant rate, so that if the rain falls below this constant rate, the depressions will begin to drain.

On impervious areas, infiltration is assumed not to occur. Depression storage, although much less than on pervious areas cannot be disregarded, however, and behaves similarly to the depression storage phenomena of pervious areas (without infiltration).

As the depression storage capacity decreases, more water becomes available to overland flow. The rainfall causes the depth of the surface flow to increase which in turn results in an increase in the runoff rate. There is a time lag between the inflow (rainfall) and outflow (runoff) however, as the inflow rate is attenuated by the volume water stored in the surface flow. This water temporarily stored in the overland flow depth is the detention storage. This detention storage plus the depression storage comprises the total surface storage.

The overland flow from both pervious and impervious areas is collected in gutters and ditches and thereby conveyed to a sewer inlet. The estimation of the overland supply by routing methods is theoretically possible for a plane surface. Prediction for real situations is complicated however, because of the non-uniformity of the areas involved and the difficulty in estimating the parameters for the various losses. The routing of the supply hydrographs through the collecting system is a hydraulic problem which can be handled by proven methods. Several routing methods are available with different degrees of sophistication. It is not necessary to always apply the same routing method for all the sewer elements of the network. Storage effects in the small lateral sewers are less significant than in the main sewer system so that simplified routing procedures are usually adequate in the laterals. For trunk sewers or interceptors, however, the hydraulic and storage effects can be very important and accurate transport routines should be employed.

3.3 Rational Formula

3.3.1 General

The rational method was first introduced in 1889 and is currently widely used for design of storm sewer systems in both the United States and Canada. For example, a recent survey [6] of major Canadian urban centres has indicated that most use the rational formula for storm sewer design. The rational method has been found by experience to be an adequate means of estimating the peak rate of runoff for relatively small areas.

This section discusses the assumptions and limitations of the rational formula and gives methods of estimating the runoff coefficient and inlet time required for meaningful application.

3.3.2 Description and Assumptions

The Rational formula takes the following simple form:

$$Q = CiA \tag{3.2}$$

where Q is the peak runoff rate in cfs,

i is the average rainfall intensity in in/hr,

A is the drainage area in acres,

and

C is a runoff coefficient depending on the characteristics of the drainage area.

(Actually Q has the units of inches per hour per acre but since this is equal to 1.008 cfs the units are assumed to be cfs for all practical purposes).

The use of the rational formula is restricted by the following basic underlying assumptions:

1. The computed maximum rate of runoff at the design point is a direct function of the average rainfall rate during the time of concentration to that point.

2. The frequency of the peak discharge is the same as the frequency of the average rainfall intensity used.

3. The maximum runoff rate occurs when the entire area is contributing flow. This is determined by the time of concentration which is the time required for the runoff to become established and flow from the most remote point of the drainage basin to the point under design.

Of the parameters in the rational formula only the area, A, can be precisely defined by measurement for a given subcatchment. The rainfall intensity, i, determined for the subcatchment depends on analysis of point rainfall data, as described in Chapter 2.0, the return frequency selected, and the time of concentration estimated for the subcatchment. The intensity is determined from the rainfall-intensity-duration curves for the selected frequency by setting the time of concentration equal to the duration of the rainfall. The most difficult parameter to estimate is the runoff coefficient, C, which varies with the land use and can also vary with time.

3.3.3 Limitations of the Rational Method

The underlying assumptions of the rational method given in section 3.3.2 lead to several limitations of the method which must be recognized in order to achieve optimum design.

According to Watkins, McPherson [9, 15] and others, one of the most serious limitations is the fact that the rational method does not take into account the real storm pattern. Both the time variation of the rate of rainfall and the variation of area and velocity contributing to the flow are therefore not accounted for.

Another limitation is that the underlying physical factors affecting runoff are lumped together into the runoff coefficient, "C", and therefore cannot be analysed or individually modified. For example, surface storage factors such as depression and detention losses play a subjective role in selecting the correct runoff coefficient. Furthermore, according to Mitci [11] among others, the runoff coefficient is not constant with time as is usually assumed for application of the rational formula. This is due to variations in antecedent moisture and rainfall which cannot be accounted for without some knowledge of the storm pattern. A similar problem is also encountered where "C" tends to increase with the design frequency selected, since larger storms generally have more antecedent rainfall. According to McPherson, there is no universally accepted fundamental principle involved in current design practise in selecting the correct runoff coefficient. [9]

With respect to determination of rainfall intensity, the estimates of the inlet time and time of concentration are difficult, especially for flat areas. None of the factors influencing the inlet time can be accurately computed and assumptions of full flowing pipes tends to overestimate the travel time in the sewers, leading to an incorrect time of concentration which results in erroneous design. Furthermore, since the time of concentration varies in each portion of the drainage basin, each part of the pipe network is actually designed by pieces of different storms, from which the rainfall-intensity-frequency curve was derived. This means that the larger the subcatchment area, the more unlikely the concurrrent occurrence of design rainfalls for all catchment components; that is, only a portion of the network would be at design capacity for a given storm.

Since the physical phenomena is inadequately described, it is "almost impossible to verify the rational method" according to McPherson [9]. For example, the C value varies from storm to storm and real storms of design intensity are infrequently monitored in test basins.

Mitci [11] states that "one of the major shortcomings of the rational formula is that although it allows an estimate of peak runoff, it does not give the actual storm runoff hydrograph needed for design of detention storage, pumping, and interceptor facilities". The actual hydrograph is also sometimes required for the planning of multipurpose development of stormwater runoff, to quantify the possible pollution from storm and combined sewer systems, and to route the runoff through the drainage facilities. Furthermore, according to Gray [4] "whereas the rational theory can account for lag effects due to travel time, it does not allow for retardation by storage and momentum of flow in channels. These discussions are particularly significant when applying the method to areas such as the Prairies, which are characterized by flat topography with poorly defined drainage ways. On such areas the rational method will only provide, at best, approximate estimates of peak flows."

All the above mentioned limiting factors result in the fact that the rational formula can only be accurately used for design of pipe networks in relatively small drainage basins.

For example, it is obvious that the formula can be used successfully on small areas where the minimum pipe size available will be used no matter which design method is used. Also it may be assumed that the rational formula gives relatively small errors in peak flow estimates for small areas. Therefore, pipe sizes up to about 18" can be estimated since the error introduced by the rational formula is generally within the limits of incremental capacity differences of available pipes below 18" in size. Depending on the inlet time and the coefficient of runoff, a pipe size of 18" roughly corresponds to areas up to about 2-5 acres. This is only an approximate area limitation for accurate pipe sizing.

3.3.4 Runoff Coefficient

The runoff coefficient C, is the most uncertain parameter in the rational formula since several physical aspects of the runoff phenomenon must be lumped together. For example, the runoff coefficient characterizes the following variables, among others; antecedent precipitation, soil moisture, infiltration, ground slope, ground cover, surface and depression storage, the shape of the drainage area and overland flow velocity. Several literature sources give recommended values of the runoff coefficient for various land use types. [1,9]

An estimate of the runoff coefficient for a particular application requires a high degree of engineering judgment and experience. Several procedures can be used to assess the accuracy of the estimated value.

A recent survey of drainage practices indicated that for design purposes in the City of Winnipeg the following C values are currently in use:

TABLE 3.1

Typical C Values

Land Use	C
Single Family Housing	0.35
Industrial	0.90
Commercial & Downtown	0.90

However, for most applications of the rational formula the land use is mixed and a weighted average C value is generally calculated. One of several methods which can be used for estimating the weighted average C value is by measuring the pervious and impervious areas and then using the following formula:

$$Cavg = \frac{C\ perv\ X\ A\ perv + C\ imp\ X\ A\ imp}{A\ perv\ X\ A\ imp} \qquad \textbf{3.3}$$

where C perv. = is the C value for the pervious area (0.1 for example)

C imp. = is the C value for the impervious area (0.9 for example)

Other empirical formulas for estimating Cavg have been derived (e,g. Schaake,[ref 131 etc), but equation 3.3 is relatively simple and gives similar results. However, equation 3.3 or the C values summarized in Table 3.1 are not recommended for general use since the depression storage and infiltration start at the beginning of precipitation, and therefore the C value should be increased with time to reflect these physical changes during the time of concentration. Empirical curves which can be used for this purpose are given in Figure 3.1. Mitci has developed these curves by interpolating curves derived by Horner [3]. The runoff coefficient is computed as a function of percent imperviousness and time from the beginning of the storm. However, the curves given in Figure 3.1 cannot be used directly since the beginning of the rainfall and the start of the time of concentration do not coincide. This follows from the definition of the rational formula since the average intensity used is not fixed in a time sequence. Therefore, some knowledge of the precipitation antecedent to the time of concentration is required.

As a method of estimating the antecedent rainfall, the design rainfall hyetographs given in Chapter 2.0 can be used by assuming that the maximum rainfall obtained from the rainfall-intensity-duration curves according to the time of concentration for the basin is distributed according to the relations rt_c before the peak and $(l-r)t_c$ after the peak. Now the antecedent time can be calculated by subtracting rt_c from the time to peak for the design storm. The resulting value is assumed to be the time, t from the beginning of rainfall to the start of the design intensity which lasts t_c minutes. Figure 3.1 can then be used to estimate C at time t and time $t + t_c$ by using the curve corresponding to the percent imperviousness of the basin. The value of C used to compute the peak flow can then be assumed to be the average of these 2 values.

Using this method will generally give more realistic values for the average C value since the antecedent moisture conditions are accounted for. An example is given in Section 3.6.2.

For major storms with recurrence intervals greater than 10 years a frequency factor is sometimes used to adjust the C factor. However, according to Schaake [13] from a practical point of view C is constant for return frequencies of 1-10 years and furthermore, using the method described above accounts for the antecedent rainfall. The use of frequency factors therefore does not seem to be necessary.

The method discussed above can be used to assess runoff coefficients estimated by engineering judgment through experienced use of the rational method.

3.3.5 Inlet Time

The inlet time is the overland flow time for runoff to reach the drainage channel. The time of concentration is the inlet time plus the travel time in the sewer. The travel time in the sewer can be calculated from hydraulic considerations but the inlet time is usually more difficult to estimate as it varies with length of flow path, surface slope, surface roughness, antecedent rainfall intensity, infiltration capacity and depression storage.

According to the WPCF Manual of Practice No. 9 "in well developed districts in the relatively flat slopes, an inlet time of 10 to 15 minutes is common". This is consistent with the inlet time reported by the City of Winnipeg in a recent survey of urban drainage practices [6]. The same source states that an inlet time of 5 minutes is often used where impervious surfaces drain directly to storm sewers through closely spaced inlets.

The inlet time varies with intensity, and basin characteristics such as length and slope of gutter and percent imperviousness. Schaake has developed the following formula for estimating the lag time between the centre of mass of rainfall and runoff:

$$t_e = 0.68L^{0.27}\ S^{-0.l3}\ Imp^{-.38} \qquad \textbf{3.4}$$

where: Imp. is the ratio of the imperviousness area to the pervious area S is the slope of the paved gutter (.06 >S> .005)

L is the length in feet of the paved gutter to the inlet (6000 >L> 150 ft.)

Schaake assumes that t_e is a good estimate of the averaging time used in the rational equation.

Also, according to Chow [1], this assumption is valid for small drainage basins where the time of concentration is very close to the lag time of peak flow.

Assuming that L = 300 - 400 ft. S = 0.007 and Imp = 37% for typical Winnipeg conditions, t_e is close to 5 minutes. It therefore appears that the inlet time may be significantly less than 15 minutes for design in Winnipeg, depending on the configuration of the drainage basin. In any case an inlet time of 5 minutes will result in a more conservative design for any given frequency.

3.3.6 Application Procedure

It must be recognized that the previous discussions concerning the runoff coefficient and the inlet times can only be regarded as possible means of assessing the accuracy or validity of parameters used in rational method. That is, using the method correctly is essentially a matter of engineering judgment and experience for each specific application.

The method of application of the rational formula for estimation of peak runoff and design of pipe networks is well documented in the literature. For example, the procedural outline and tables given in the WPCF Manual of Practice No. 9 can be used bearing in mind the limitations and recommendations presented in the preceeding sections.

3.4 Hydrograph Methods

3.4.1 General

While the rational method can be classed as a lumped model which is strictly valid only to estimate peak flow, several distributed computer models are available to simulate runoff hydrographs from urban areas, In this respect, most hydrograph models are superior to the rational method. The models attempt to simulate each physical process sequentially in time and can therefore closely follow the natural phenomena. Generally, distributed models characterize the drainage basin by dividing it into several sub-catchments and specifying the runoff parameters including, for example, infiltration, slopes, roughness, antecedent precipitation and depression storage depth for both pervious and impervious areas. Such deterministic models allow a great deal of flexibility since the parameters can reflect the underlying runoff processes of each subcatchment. Also, since all parameters are physically based, they can be measured in the field and are therefore assumed to be transferable from one region to another for similar basins. On the other hand, some engineering judgment must always be employed when using such models since some of the parameters (such as depression storage, for example) are difficult to measure exactly.

With reference to the various levels of application discussed briefly in Section 3.1, and in Chapter 1.0, the hydrograph models considered herein for study of runoff in Winnipeg are the U.S. Environmental Protection Agencies Storm Water Management Model (EPA-SWMM) and a simple Isochrone technique. The latter method of determining the runoff hydrograph can be used for studies of small basins or for preliminary studies by those without access to the computer.

Generally speaking, hydrograph methods are continuing to gain wider acceptance, particularly in large cities such as Denver, Chicago, Montreal, Toronto, etc. where significant economic savings are realized by using temporary retention facilities. Copies of nonproprietary computer programs are readily available from such agencies as the U.S. E.P.A. and the C.C.I.W. In particular, the EPA has embarked on a program of dissemination of information by means of seminars and courses, aimed at increasing the use of hydrograph models. The City of Winnipeg could also make copies of the computer programs and instructions for use available on request for the use of local consultants and developers.

Several proprietary hydrograph models at various levels of sophistication are also currently available from various consultants. In most cases the general underlying principles of these models have been published in the literature and comparison of simulation results with other hydrograph models has generally been favourable. Therefore in general proprietary hydrograph models can also be recommended as a viable alternative to the rational method.

3.4.2 Design Storm Discretization

Hydrograph methods generally require a discrete form of the design hyetograph; that is, the design rainfall curve is reduced to a set of discrete data. The obvious criteria for discretization of the design storm representing a particular return frequency are that the time step chosen should be realistic and economically feasible from the point of view of computing time and the total rainfall mass must be the same for the same time step. Since the design hyetographs given in Chapter 2.0 were originally derived from discrete data with a minimum time interval of rainfall measurement of 5 minutes, it can be assumed that 5 minutes represents a reasonable time step for discretization purposes. Furthermore, experience with hydrograph models (e.g. EPA SWMM) has indicated that as long as the antecedent rainfall mass is the same at any time, variations in the discretization interval produce essentially the same hydrograph for small time steps. Experience has also indicated that a purely graphical method of discretization should be avoided since discrete values near the peak can easily be incorrectly selected due to the steep slopes of the design hyetograph. The following producedure is recommended in order to discretize the design hyetograph for a selected return frequency.

(i) A time step of Ît = 5 minutes should be used

(ii) The discrete point representing the peak rainfall

is computed using equation 2.12 and the selected

time step for $t_{d.}$

$$i_p = \frac{a}{(\Delta t + b)^c} \qquad \textbf{3.5}$$

The time interval selected is distributed around the peak as rÎ t of the calculated discrete peak intensity occurring before the peak of the design curve and (l-r)Î t after this peak, as indicated on Figure 3.2. Therefore, by definition of the design hyetograph, for the duration At around the peak, the volume of rainfall from -rÎ t to (l-r)Î t (taking t = 0 at the peak) is equal to the volume $i_{pÎ}$ t.

(iii) Additional points before and after the peak are computed by integrating the design curve and calculating the ordinate (y) for the discrete intensity by equating the volumes for each time increment at. (Figure 3.2) The general integral forms of the design hyetograph before and after the peak are given by the following equations respectively.

$$\int_{t_{bl}}^{t_{b2}} i_b \, dt_b = \left[\frac{a t_b}{(b + \frac{t_b}{r})^c} \right]_{t_{bl}}^{t_{b2}} \qquad \textbf{3.6}$$

$$\int_{t_{al}}^{t_{a2}} i_a \, dt_a = \left[\frac{a t_a}{(b + \frac{t_a}{1-r})^c} \right]_{t_{al}}^{t_{a2}} \qquad \textbf{3.7}$$

where t_b is the time measured from the peak to the left and t_a is the time measured from the peak to the right (see figure 3.2) Therefore the next discrete intensity value immediately to the right of the peak intensity calculated in Step (ii) is given by the following equation

$$y\,\Delta t = \left[\frac{a t_{a2}}{\left(b + \frac{t_{a2}}{(1-r)}\right)^{c}}\right] - \left[\frac{a t_{al}}{\left(b + \frac{t_{al}}{(1-r)}\right)^{c}}\right] \quad \textbf{3.8}$$

where in this case t_{al} = 3.45

t_{a2} = 8.45

r = .31

a,b,c, vary according to the storm frequency

(iv) Step (iii) is applied before and after the peak until the calculated intensity ordinates are insignificant. Since the smallest rainfall values which can be measured are .01 inches in a 5-minute period, it is assumed that intensities below about .12 inches per hour (60/5 x .01) can be neglected for all practical purposes. This absolute limit is reached at various times before and after the peak depending on the design frequency chosen. For the 2, 5, 10, and 25 design storms given in Chapter 2.0, this limit is reached at 46, 60, 68 and 82 minutes respectively before the peak.

The discretized 5 and 25 year design storms are shown on Figures 2.2a and 3.2b respectively. The corresponding rainfall intensity values in inches per hour for 5 minute intervals are given in Table 3.2.

An alternate graphical method of discretization using the mass curve is shown on Figure 3.3.

3.4.3 Routing

Hydrograph models generally consider routing of both overland flow and pipe routing. Overland flow routing is generally accounted for by the nature of the runoff computation. Conduit or channel routing requirements are not the same for all parts of an urban drainage system. Local collectors and tributaries are usually relatively small in size and have no important backwater or storage effects and in some cases it is possible to ignore pipe routing for small systems. On the other hand, trunk lines and interceptors may have significant storage and backwater effects and may contain a variety of storage facilities, diversion structures, or pumping stations.

Different methods are employed by various models to account for the routing characteristics of smaller conduits. For example the RRL (Road Research Laboratory) method (which is essentially a sophisticated isochrone method) uses a simple hydrologic routing technique which considers each conduit to be a reservoir in which inflow equals outflow plus storage. The volume of storage is determined using an assumption of a constant depth of flow throughout the pipe for each time step.

The EPA SWMM has the capability of using 2 different routing techniques; one for the conduits of the subcatchments (RUNOFF BLOCK), which is similar to that described above, and one for the trunk sewer lines which uses a more sophisticated method in its TRANSPORT block [10]. Recent studies have found no significant difference when comparing the different levels of sophistication for routing in local collector systems. The use of the EPA SWMM RUNOFF BLOCK has the additional feature that surcharging pipes in the local network are identified by the program and can be resized for subsequent runs. However, when considering major sewer networks, a more detailed method is required and use of the EPA SWMM's TRANSPORT block is recommended.

3.4.4 Isochrone Method

An approximate, simple method which can be used for preliminary estimates of the peak flow and the runoff hydrograph is the isochrone method. The general principles of the method are outlined on Figure 3.4. The method is described in detail by Watkins [15].

The first step is to define the runoff time-area curve. This can be done by estimating the velocity of flow and the corresponding time of flow for overland runoff or alternatively assuming full flowing pipes in each area. A unit time step is then selected and the contributing areas determined from the time area curve as indicated by Figure 3.4.2.

The pervious and impervious areas for each sub-area are estimated and the net rainfall intensity and corresponding runoff hydrograph calculated according to the method given in Figure 3.4. The 2 resulting hydrographs are combined to produce the total hydrograph. The advantage is that the net rainfall intensity can be defined separately for the pervious and impervious areas. The net rainfall intensity for the impervious area is calculated by subtracting detention and depression storage.

For the pervious area, the additional infiltration abstraction must be accounted for. This can be done by using field measurements of average infiltration rates or by estimating the rates from published values and using a mathematical expression to model the change of infiltration with time during a rainstorm. For example, Horton's equation given in Section 3.2 can be used to estimate the infiltration capacity during a storm.

A procedure sometimes used in simulating runoff for specific storms is to shift the infiltration mass curve in time until it is tangent with the mass of rainfall, as illustrated on Figure 3.5. While this shift can account for some of the unused capacity at the beginning of the storm, it is not as conservative as starting the infiltration at the beginning of the storm since for design events it is safer to assume that the soil may already be partially saturated. The EPA - SWMM does not shift the infiltration curve.

TABLE 3.2

DISCRETIZED DESIGN STORMS FOR WINNIPEG

Time (min)	5-Year (in/hr)	25-Year (in/hr)
0	0.0	0.0
5	1.12	0.12
10	0.13	0.13
15	0.15	0.15
20	0.17	0.17
25	0.19	0.19
30	0.25	0.21
35	0.30	0.23
40	0.35	0.26
45	0.50	0.29
50	0.92	0.33
55	2.09	0.42
60	5.65	0.53
65	2.90	0.76
70	1.58	1.24
75	1.08	2.96
80	0.80	7.86
85	0.60	3.93
90	0.50	2.29
95	0.45	1.54
100	0.40	1.17
105	0.35	0.92
110	0.31	0.75
115	0.29	0.65
120	0.27	0.58
125	0.25	0.51
130	0.23	0.45
135	0.22	0.40

140	0.21	0.35
145	0.20	0.30
150	0.19	0.26
155	0.18	0.22
160	0.17	0.19
165	0.16	0.17
170	0.15	0.16
175	0.14	0.15
180	0.00	0.14
185	-	0.13
190	-	0.12
195	-	0.12
200	-	0.00

Advantages of the Isochrone method are:

i. Its relative simplicity allows hand computation for those without access to a computer.
ii. A flow hydrograph is produced using a design storm pattern.
iii. It is an intermediate method between the rational formula and more complex hydrograph methods. That is, it can directly consider some of the physical parameters of runoff phenomena such as depression and detention storage and infiltration.
iv. Runoff contributions from both the pervious and impervious areas can be estimated.
v. Data requirements are about the same as using the rational formula.

Disadvantages of the Isochrone method are:

i. Due to the simple nature of the flow routing the method is applicable only to relatively small areas for which the time area curve can be easily estimated.
ii. There may be some difficulty in estimating the time area curve of runoff, especially for basins without existing pipe networks or basins with a complicated geometrical configuration.
iii. Since the method is only an approximate one, use of a more detailed hydrograph method is required for final design purposes.
iv. Pipe routing is not directly accounted for. However, a more sophisticated version of the isochrone method known as the Road Research Laboratory Method (RRL) takes account of pipe routing and is available as a computer program [15].

3.4.5 EPA - SWMM

The Environmental Protection Agencies Storm Water Management Model is one of the most comprehensive urban runoff models currently available. The model combines sophisticated computer sub-routines to describe runoff quantity and quality and its affect on the receiving water body by means of producing hydrographs and pollutographs. The model is described in detail in the users manuals and several publications [6, 10]. A revised version of the model which includes a pipe sizing routine is now available from the University of Florida and the corresponding users manual will soon be published.

The runoff portion of the model has been recently studied [6] and it was concluded that runoff simulation by the EPA model could be generally recommended for use. The EPA SWMM is readily available for general use, is quite flexible and has the option of considering pollutant loadings.

The EPA SWMM accounts for the basic underlying runoff phenomena in a drainage basin and requires the following input data:

- one or more rainfall hyetographs
- infiltration parameters for Horton Equation
- depression storage values for pervious and impervious areas
- slope, width, area and percent imperviousness of each subcatchment
- length, slope, diameter and roughness of each conduit
- percent of impervious area with zero depression storage.

A copy of the input format description is given in Appendix A.

The model is obviously very flexible in describing the distributed runoff characteristics of an urban drainage basin. Furthermore, the flexibility can allow preliminary use of the model using a very coarse subdivision of the basin thereby reducing data requirements for preliminary design. Therefore, the model can be used at all stages of design but becomes particularly useful for the final design stage where accurate estimates of runoff and storage can be made by using a more detailed basin discretization.

However, as with all urban runoff models, some engineering judgment and experience is required in order that the selection of parameters reflects the physical conditions of individual drainage basins.

3.5 Design Parameters for Winnipeg

3.5.1 General

The location of Winnipeg on the plains of the Red River valley results in a situation where temporary detention of stormwater is a very attractive storm water management alternative. The flat topography results in a low hydraulic head thereby simplifying design of detention facilities.

The terrain is the result of a former glacial lake, the glacial drift being overlain with lacustrine deposits of highly plastic clays which have the capacity to hold large quantities of moisture. [8]. The clay layer is about 30-40 feet thick and is generally overlain by 1-3 feet of silt and 2-3 feet of fill.

The quantity of runoff and corresponding storage requirements are determined by the nature and storm pattern of precipitation as defined in Chapter 2.0, and the method of determining the runoff hydrograph.

Also important for runoff design considerations is the downstream location of Winnipeg in the Red River Drainage basin. Autumn rains with early frosts and a cold winter combine to seal moisture in the ground preventing evaporation. When followed by heavy winter snowfalls and rapid spring melt with rainstorms, the large drainage basin area results in high spring floods at Winnipeg. However, operation of floodway diversions considerably reduces the possibility of severe spring flooding in the city.

The following section deals with a definition of some of the design parameters necessary for the models previously discussed.

3.5.2 Spring and Summer Conditions

As pointed out in Chapter 2.0, the summer precipitation conditions are usually the most critical from the point of view of the magnitude of the design storm. However, due to the flat topography, spring floods in conjunction with April or May rainstorms on frozen ground may be more critical for some designs, depending mostly on location.

During spring, the most conservative assumption is frozen ground with no infiltration. Since the Red River Floodway has been put into operation, the probability of severe spring floods has been considerably reduced (Figure 3.6). Analysis of spring rainfall events and Red River water levels has indicated very little day to day correlation of local storm events and water levels in the Red River. For all practical design purposes, the probability of the storm (P(S)) and the probability of the water level (P(L)) can be considered to be independent, and the joint probability of occurrence can be estimated to be the product, which is represented by the following equation:

$$P\ (\text{DESIGN}) = P(S) \times P(L) \qquad \textbf{3.9}$$

Analysis of various combinations of storm and water levels should be made in order to find the critical condition, as described in Chapter 1 . For example the joint occurrence of high water level and moderate spring rain plus snowmelt may result in critical design conditions depending on the land slope and nearness of the drainage basin to the river. (That is, drainage design for relatively high ground may depend mainly on the design storm frequency).

With the flood control works in operation, a design flood level of elevation 749 feet or less (1/100 years, Figure 3.6) should be chosen, depending on the return frequency of the spring storm event and the economic consequences of flooding. According to a recent land drainage study done by the Waterworks and Waste Disposal Division of the City of Winnipeg, a design storm with a 5-year return period gives satisfactory protection in view of the increased cost associated with the use of a less frequent design storm. The use of the 5-year frequency is consistent with the practice of most Canadian and American cities. [3,6] However, it must be stressed that, in general, each design should be assessed on its own merit by a cost/benefit analysis. This is discussed in more detail in Chapter 1.

Analysis of stage records for 1966 - 1974 has indicated that the flood stage of elevation, 746 feet at James Avenue, was exceeded in 3 different years, compared to a return frequency of about 1/60 years given by the curve of Figure 3.6. This apparent discrepancy is currently being investigated by City of Winnipeg staff and the flood frequency curve with the flood control works in operation is in the process of being updated.

During the summer, the water level in the Red River generally remains constant at about elevation 734 feet, due to downstream regulation. Severe summer fluctuations are very rare and probably need not be considered in conjunction with summer storms.

For application of the rational method, to estimate runoff, the C value can be assessed by the method given in Section 3.3.4. Ideally, for application of hydrograph methods to specific drainage areas, the infiltration capacity should be measured in the field. Alternatively, real storms can be used to calibrate infiltration and surface storage parameters by fitting the measured and computed runoff hydrography to determine optimum parameter values for each specific application. However, for initial estimates of the runoff, the use of values recommended in the literature may be justified. For example, Denver [16] recommends typical values of 0.1 and 0.3 inches for the combined value of the depression and detention storage on impervious and pervious areas respectively. The EPA-SWMM default values are .06 and .18 inches respectively for depression storage. The EPA model considers surface detention storage separately by means of the runoff computation. Chow recommends about .25 inches for the depression storage on pervious areas.

Infiltration is a difficult parameter to assess without field measurements or calibration, but the EPA SWMM uses default values of 3.0 and .52 in/hr. for f_l and f_o respectively and % = 0.069 for use in Horton’s equation (t in min.). The infiltration rate is obviously affected by the soil group and vegetative cover and varies from area to area. Denver [16] recommends making infiltrometer tests at a density of about 1 per 160 acres of drainage basin, or for first estimates to use generally a constant value of about .50 inches per hour. This is consistent with the asymtotic value of .52 used by the FPA.

3.6 Example Computations and Comparison of Recommended Methods

3.6.1 General

A typical Winnipeg residential subdivision of about 61 acres in size was selected to demonstrate the application of the various models discussed in previous sections (see Figure 3.7). The subdivision drains to a temporary storage pond with a normal surface area of about 3.5 acres and a normal depth of 5 feet. Generally the area is very flat, the surface slope being less than 3% on the average. The overall percent imperviousness of the area was estimated to be 32% when considering the house roofs draining onto the grass (i.e. considering the roofs do not contribute directly) and about 42% when assuming that the roofs are directly connected to the storm sewer system.

The 5 year design rainfall starting at 60 minutes before the peak and discretized into 5 minute intervals was used as input to the hydrograph methods. (Figure 3.2a)

3.6.2 Rational Method

Using equation 3.3 the values computed for the average runoff coefficient, C for the basin are 0.36 and .44 assuming the roofs to be draining onto the grass and directly connected to the storm sewer respectively. Assuming a time of concentration of 12 minutes (estimated during pipe sizing, see ref. 3 for example), the corresponding intensity obtained from the 5 year rainfall-intensity duration curve (Figure 2.5) is about 3.9 inches/hour. Therefore applying the rational formula and the above C values, the peak flows are 86 and 106 cfs respectively for the 61 acre test basin.

Since the runoff coefficient actually varies with time due to antecedent rainfall the C value can be better estimated by using the method described in section 3.3.4. This results in values for the runoff coefficient of 0.65 and 0.69 for the cases of unconnected and connected roofs. The corresponding peak flows are 156 and 166 cfs respectively.

The rational method was used to calculate initial pipe sizes for an assumed pipe network in the test basin. This preliminary design of the pipe network was further refined by using the runoff block of the EPA-SWMM.

3.6.3 Isochrone Method

The simple isochrone method of overland flow routing described in section 3.4.4 was applied to the typical Winnipeg test basin for the configuration given on Figure 3.8. A 5-minute time interval and 3 contributing areas were used in conjunction with the discrete values of the 5 year design storm given in Table 3.2.

In the absence of infiltration data Horton’s infiltration curve using the parameter values recommended by the EPA were used.

$$F = .52 + (3.0 - .52)\, e^{-0.069t} \qquad \textbf{3.10}$$

where t is in minutes from the start of application of the equation. In this example infiltration was started at the beginning of the storm. Parameters of equation 3.10 can be modified to reflect real conditions where infiltration data is available. Abstractions of 0.1 and 0.25 inches were assumed for the impervious and pervious areas respectively. An example of the computation for 42% imperviousness is given in Table 3.3.

TABLE 3.3

ISOCHRONE METHOD CALCULATIONS OF THE TYPICAL SUBCATCHMENT

	Runoff from Impervious Area					Runoff from Pervious Area						
Time (min)	I (in/hr)	A_1=8.0 (cfs)	A_2=11.3 (cfs)	A_3=5.9 (cfs)	Pervious (flow) (cfs)	i (in/hr)	A_1=15.0 (cfs)	A_2=11.7 (cfs)	A_3=9.1 (cfs)	Impervious (flow) (cfs	Total Flow (cfs)	Mass Volume (cu ft)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
5	0.0	0.0	-	-	0.0						0.0	
10	0.0	0.0	0.0	-	0.0						0.0	
15	0.0	0.0	0.0	0.0	0.0						0.0	
20	0.0	0.0	0.0	0.0	0.0						0.0	
25	0.0	0.0	0.0	0.0	0.0						0.0	0
30	0.0	0.0	0.0	0.0	0.0						0.0	1476
35	0.11	0.88	0.0	0.0	0.88						0.88	8583
40	0.35	2.80	1.24	0.0	4.04						4.04	58014
45	0.50	4.00	3.96	0.65	8.61			-		0.0	8.61	112638
50	0.92	7.36	5.65	2.07	15.08	0.0	0.0	0.0	-	0.0	15.08	160308
55	2.09	16.72	10.40	2.95	30.07	0.0	0.0	0.0	-	60.45	30.07	200454

60	5.65	45.2	23.62	5.43	74.25	4.03	60.45	47.15	0.0	82.70	134.70	215061
65	2.90	23.2	63.85	12.33	99.38	2.37	35.55	27.73	0.0	80.15	182.08	222489
70	1.58	12.64	32.77	33.34	78.75	1.05	15.75	12.29	36.67	42.11	158.90	228123
75	1.08	8.64	17.85	17.10	43.59	0.55	8.25	6.44	21.57	20.2	85.70	232671
80	0.80	6.40	12.20	9.32	27.92	0.28	4.20	3.28	9.56	9.49	48.12	236577
85	0.60	4.80	9.04	6.37	20.21	0.08	1.20	0.94	5.01	3.49	29.70	239988
90	0.50	4.00	6.78	4.72	15.50	0.0	0.0	0.0	2.55	0.73	18.99	243075
95	0.45	3.60	5.65	3.54	12.79	0.0	0.0	0.0	0.73	0.0	13.52	245859
100	0.40	3.20	5.09	2.95	11.24				0.0	0.0	11.24	248340
105	0.35	2.80	4.52	2.66	9.98				0.0		9.98	250521
110	0.31	2.48	3.96	2.36	8.80						8.80	251442
115	0.29	2.32	3.50	2.07	7.89						7.89	
120	0.27	2.16	3.28	1.83	7.27						7.27	
125	0.25	2.00	3.05	1.71	6.76						6.76	
130	0.23	1.84	2.83	1.59	6.26						6.26	
135	0.22	1.76	2.60	1.48	5.84						5.84	
140	0.21	1.68	2.49	1.36	5.53						5.53	
145	0.20	1.60	2.37	1.30	5.27						5.27	
150	0.19	1.52	2.26	1.24	5.02						5.02	
155	0.18	1.44	2.15	1.18	4.77						4.77	
160	0.17	1.36	2.03	1.12	4.51						4.51	
165	0.16	1.28	1.97	1.06	4.26						4.26	
170	0.15	1.20	1.81	1.00	4.01						4.01	
175	0.14	1.12	1.70	0.94	3.76						3.76	
180	0.13	1.04	1.58	0.89	3.51						3.51	
185			1.47	0.83	2.30						2.30	
190				0.77	0.77						0.77	

The resulting outflow hydrographs are shown in Figures 3.10 and 3.11 for the cases of roofs draining onto the grass and directly connected to the storm sewers respectively. The resulting peak flows are 174 Ifs for the case of roofs draining onto the grass and 182 cfs for roofs directly connected to the sewer system.

3.6.4 EPA Model

The runoff block of the EPA Storm Water Management Model was applied to the test basin by assuming a pipe network and sub-dividing the drainage area into 37 subcatchments, each representing drainage to a catchbasin or manhole (see Figure 3.9). Initial pipe sizes were calculated using the rational formula and the method given in the WPCF Manual of Practice No. 9. A complete description of input requirements for the EPA model are detailed in the users manual and input descriptions for the RUNOFF and TRANSPORT blocks are attached as Appendix A. The EPA SWMM was then run using the discretized 5 year design storm. Experience with the model has indicated that for design purposes the optimum rainfall time increment and computational time step are 5 and 2 minutes respectively. Output from the runoff block indicates which pipes, if any, are undersized and by increasing the pipe size and modifying the input data to the program subsequent computer runs finalized the pipe design and resulting runoff hydrographs.

The hydrographs for the cases of roofs draining onto the grass and directly connected to the sewer system are shown in figures 3.10 and 3.11 and the peak flows are 143 and 172 cfs respectively.

3.6.5 Discussion and Comparison of Results

According to the results of a recent study of Urban Runoff by James F. MacLaren Limited the EPA-SWMM gives good overall results compared with the other hydrograph methods considered for simulation purposes. [6].

The EPA-SWMM is capable of giving a good representation of the spatial variation in land use (depending on the degree of sub-basin discretization) since the percent imperviousness, surface roughness, storage and slope, infiltration and pipe characteristics can vary with each sub-area. Also, since the flow is modelled in a fairly sophisticated deterministic manner it is felt that the results of the 2 cases for roofs draining onto the grass and directly connected to the sewer system can represent the "actual" or base condition for comparison to the other methods of estimating runoff.

A comparision of the peak flows and time to peak is given in table 3.4.

TABLE 3.4

COMPARISON OF PEAK FLOWS FOR DIFFERENT METHODS USING A 5-YEAR DESIGN STORM

RUNOFF MODEL	ROOFS DRAINING ONTO GRASS % IMPERVIOUSNESS = 32		ROOFS DIRECTLY CONNECTED TO SEWER %IMPERVIOUSNESS = 42	
	PEAK FLOW (cfs)	TIME TO PEAK (min)	PEAK FLOW (cfs)	TIME TO PEAK (min)
EPA SWWM	143	62	172	62
ISOCHRONE METHOD	174	65	182	65
RATIONAL METHOD (see Sect. 3.3.4)	156 (C = .65)	-	166 (C = .69)	-
RATIONAL METHOD (Equa. 3.3)	86 (C = .36)	-	106 (C = .44)	-

The EPA-SWMM was also run using the 5 year April design hyetograph, (Chapter 2, Figure 2.12) assuming the ground is frozen and no infiltration can occur. The resulting peak flow was only 14 cfs due to the low intensity and since the EPA model accounts for surface storage and detention. Assuming the time of concentration to be about 12 minutes the C coefficient corresponding to this peak is only about 0.30, which is an unrealistically low value for spring runoff conditions. The basic reason for this is that the runoff coefficient, C does not accurately take into account surface storage, detention and infiltration losses as a function of the design storm intensity and the percent imperviousness.

However, for fairly large storms, the peak values computed using the rational formula are comparable (Table 3.4) to the peaks given by the hydrograph methods provided that the antecedent rainfall is used to modify the runoff coefficient. Therefore Figure 3.1 and the method described in section 3.3.3 is recommended as a means of assessing the runoff coefficient selected for a particular basin. Furthermore, it should be noted that the use of C = 0.35 for Single Family Housing as given in Table 3.1 will also give a low peak. The use of such generalized, tabulated values for estimating C should be avoided wherever possible.

As noted previously, a realistic hydrograph cannot be easily constructed using the rational method. As indicated by Figures 3.10 and 3.11, the isochrone method can give a hydrograph which is quite comparable to the EPA SWMM. It should be noted that the surface storage and infiltration parameters should be modified to be representative of the physical conditions of each specific application.

Alternatively any other hydrograph model which takes into account the physical runoff parameters could be used to estimate the runoff hydrograph. For example, the Road Research Laboratory computer model (RRL Model) could be used to account for pipe routing provided that the model is modified to account for contributions from the pervious area.

Comparison of the results for the two cases of the roofs draining onto the grass and directly connected to the storm sewers indicates that this can be an important drainage factor since the peak flow increases with the imperviousness. The increase will vary for different basins according to the land use characteristics.

It must be noted that the comparison of results for one test case should not be used to generalize Winnipeg design conditions, rather, the idea is to stress the fact that the use of hydrograph methods must be introduced into Winnipeg design practise. Future testing and calibration of various hydrograph methods on real storm events will lead to an extremely useful design tool.

A real storm which occurred on May 20, 1974 was digitized and used as input to the EPA model for comparison purposes. The results are attached as Appendix B. and also represent a good example of the type of output from the EPA-SWMM. The return frequency of this storm is estimated to be about 1 in 2 years (using the June, July, August curves since the storm occurs late in the month; using May curves the return frequency is about 1 in 10 years). The peak flow was about 118 cfs and the shape of the hydrograph corresponds to that produced by theoretical design storms.

CHAPTER 3 REFERENCES

1. Chow, V. T. "Handbook of Applied Hydrology" McGrawHill, 1964.

2. Chen,C.W. and R.P. Shubinski, "Computer Simulation - of Urban Storm Water Runoff" Journal of the Hydraulics Division, ASCE, February 1971.

3. Committee of the Water Pollution Control Federation "Design and Construction of Sanitary and Storm Sewers" WPCF Manual of Practice No. 9, 1969.

4. Gray, D. M. (ed.) "Handbook on the Principles of Hydrology" Secretariat, Canadian National Committee for the International Hydrological Decade, Ottawa, 1970.

5. Heeps, D. P. and R.G. Mein, "An Independent Evaluation of Three Urban Stormwater Models" Civil Engineering Research Reports, Monash University, Report No. 4, 1973.

6. James F. MacLaren Limited, "Review of Canadian Storm Sewer Design Practice and Comparison of Urban Hydrologic Models", Environment Canada, 1974

7. Lyons, D. J. RRL Users Manual (RRL Road Note No. 35) Road Research Laboratory, Crowthorne Berkshire 1970.

8. Labelle, J. J. "The Climate of Winnipeg", Department of Transport, Meteorgological Branch, Cir. 4437, CLI 35 June 1966.

9. McPherson, M.B. "Some Notes on the Rational Method of Storm Drain Design" ASCE Urban Water Resources Research Program Technical Memorandum No. 6 Jan. 22 1969.

10. Metcalf and Eddy Inc., University of Florida, and Water Resources Engineers Inc., "Storm Water Management Model, 4 volumes, Federal Environmental Protection Agency, Water Pollution Control Reserach Series, 11024 DUC, 1971.

11. Mitci, C. "Determine Urban Runoff the Simple Way", Water and Waste Engineering Journal, January, 1974.

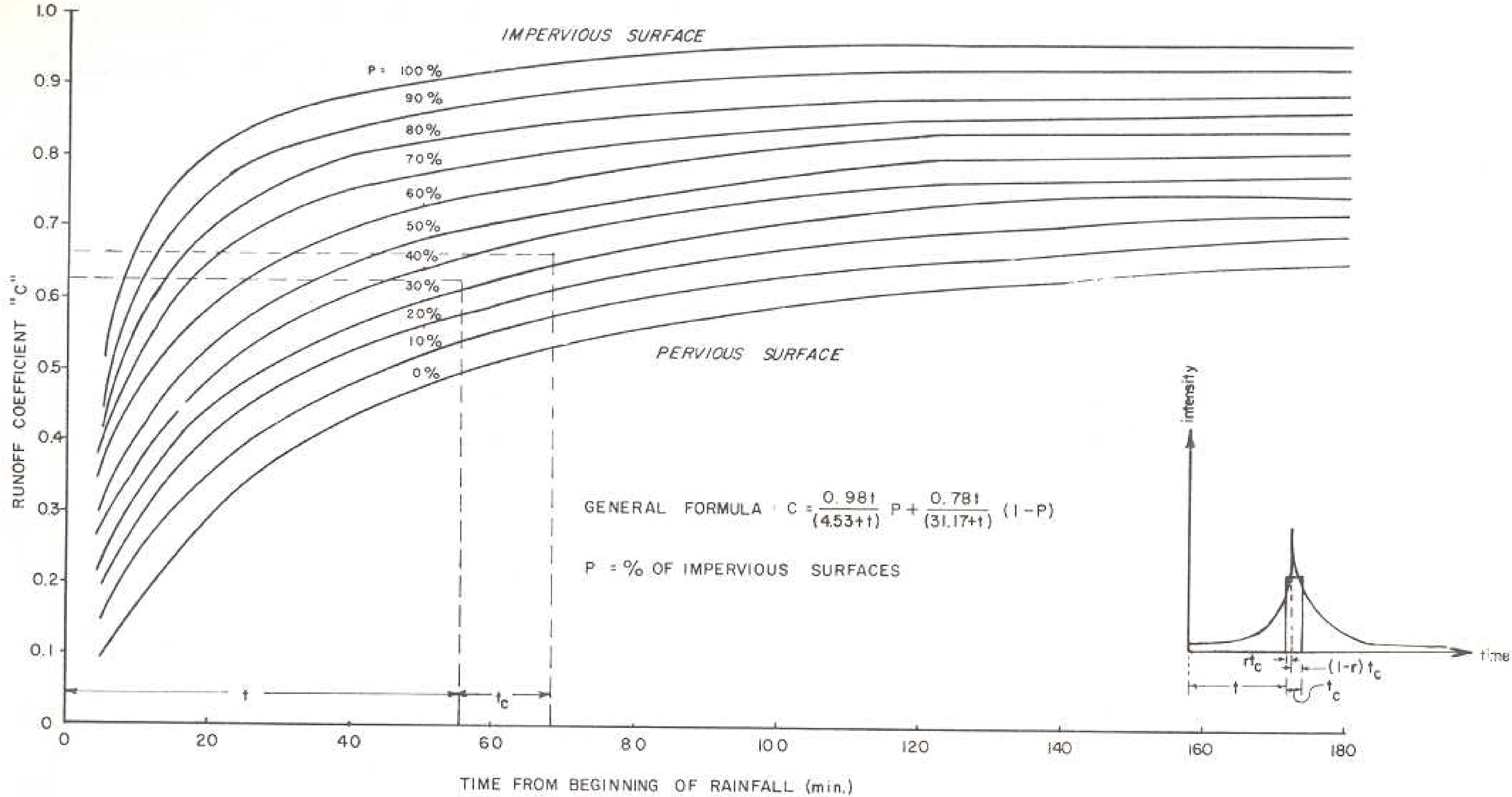
12. Render, F.W. "Geohydrology of the Metropolitan Winnipeg Area as Related to Groundwater Supply and Construction:. Water Control and Conservation Branch, Department of Mines and Natural Resources, Winnipeg, Manitoba Canadian Geotechnical Journal, 7, 243 (1970).

13. Schaake, J. C. et al "Runoff Coefficients in the Rational Method" Technical Report No. 1, July 1964 John Hopkins University, Baltimore Maryland.

14. Terstriep, M.L. and J.B. Stall, "Urban Runoff by Road Research Laboratory Method" Journal of the Hydraulics Divisions ASCE, Nov. 1969.

15. Watkins, L.H. "The Design of Urban Sewer Systems" Road Research Technical Paper No. 55 Department of Scientific and Industrial Research London, England, 1962.

16. Wright-McLaughlin Engineers, "Urban Storm Drainage Criteria Manual" Vol. 1, 2, Denver, Colorado, March, 1969.



RUNOFF COEFFICIENT AS A FUNCTION OF PERCENT IMPERVIOUSNESS AND ANTECEDENT RAINFALL.

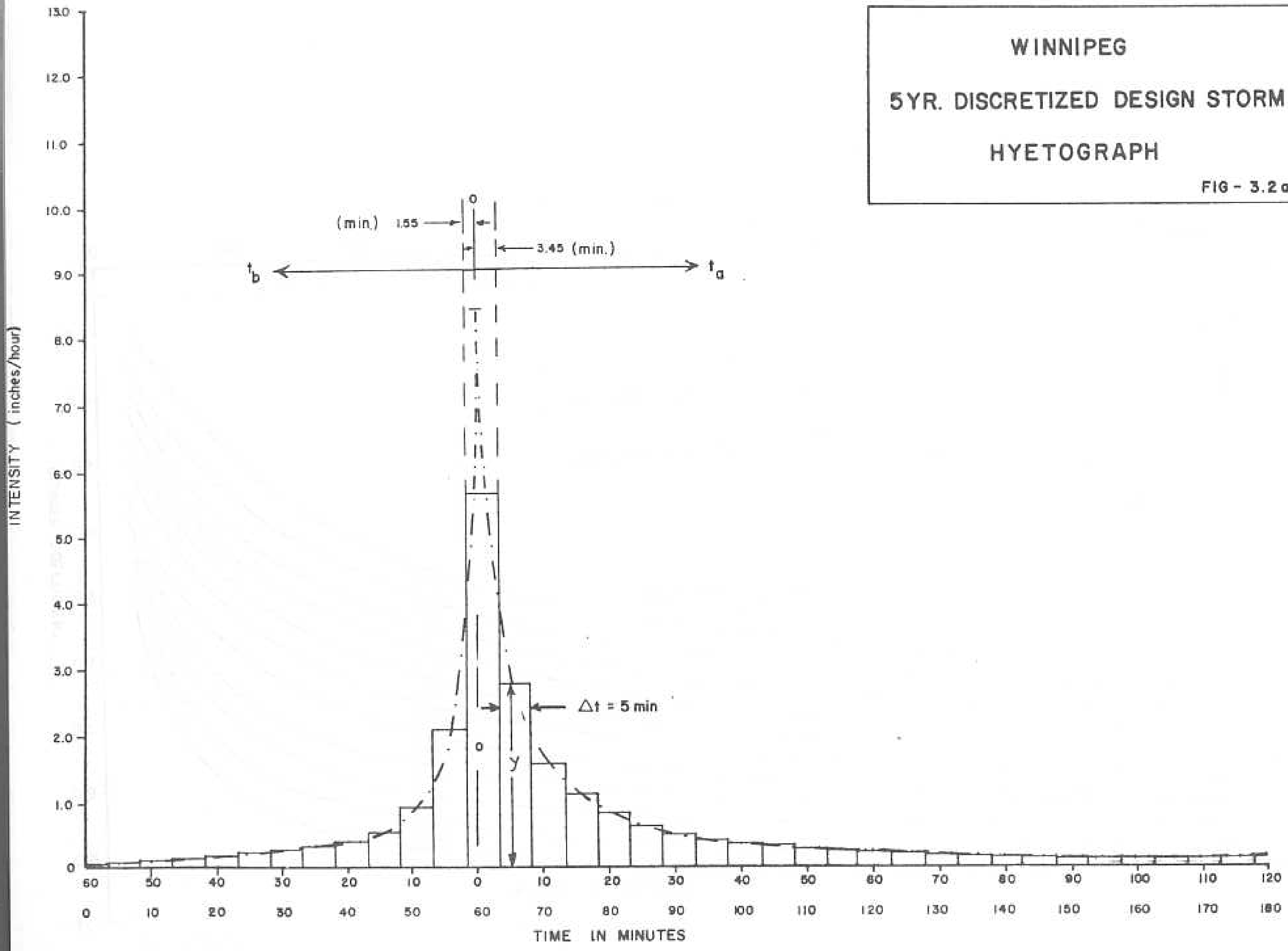
Fig.— 3·1

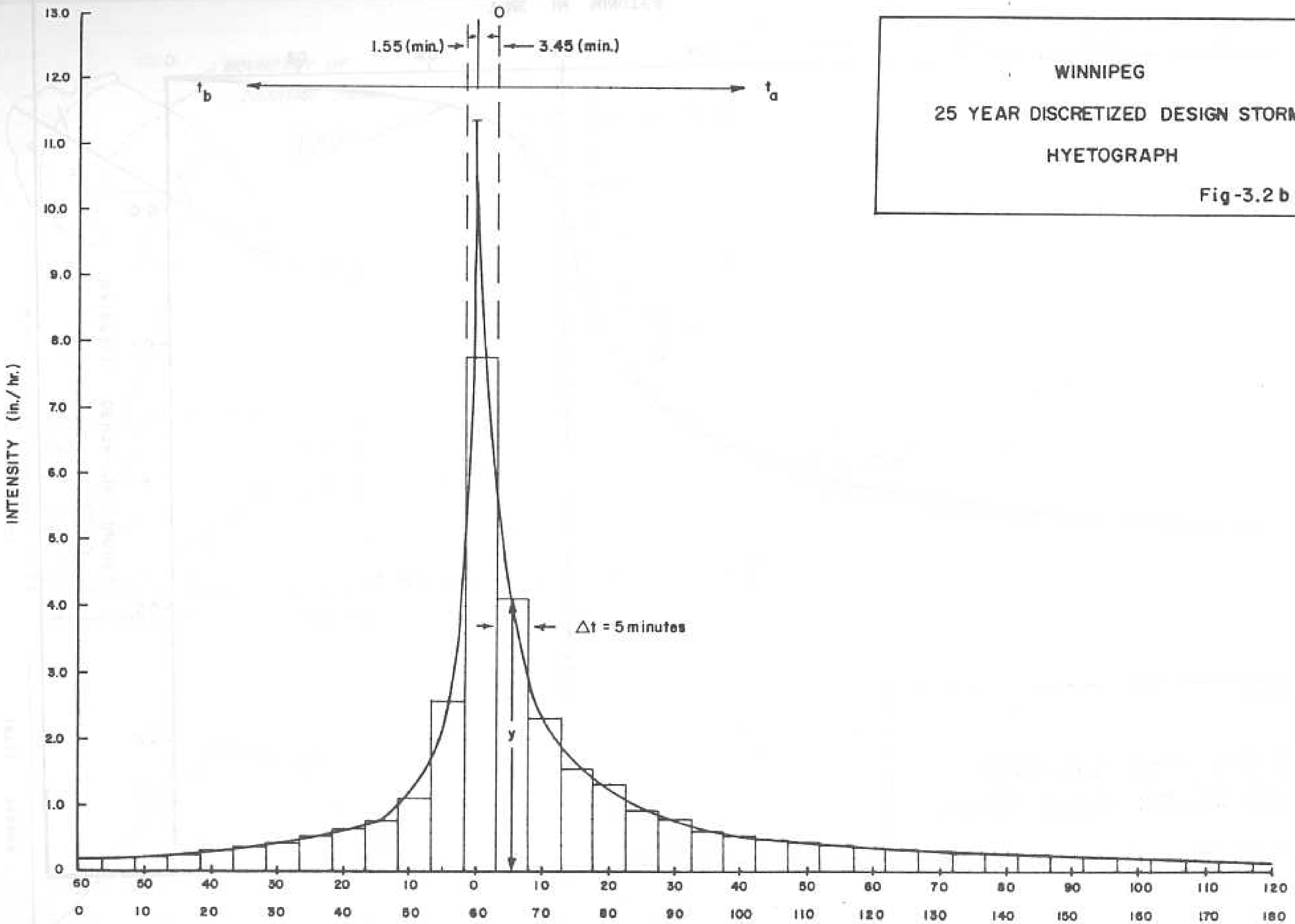
WINNIPEG

5YR. DISCRETIZED DESIGN STORM

HYETOGRAPH

FIG - 3.2a





WINNIPEG

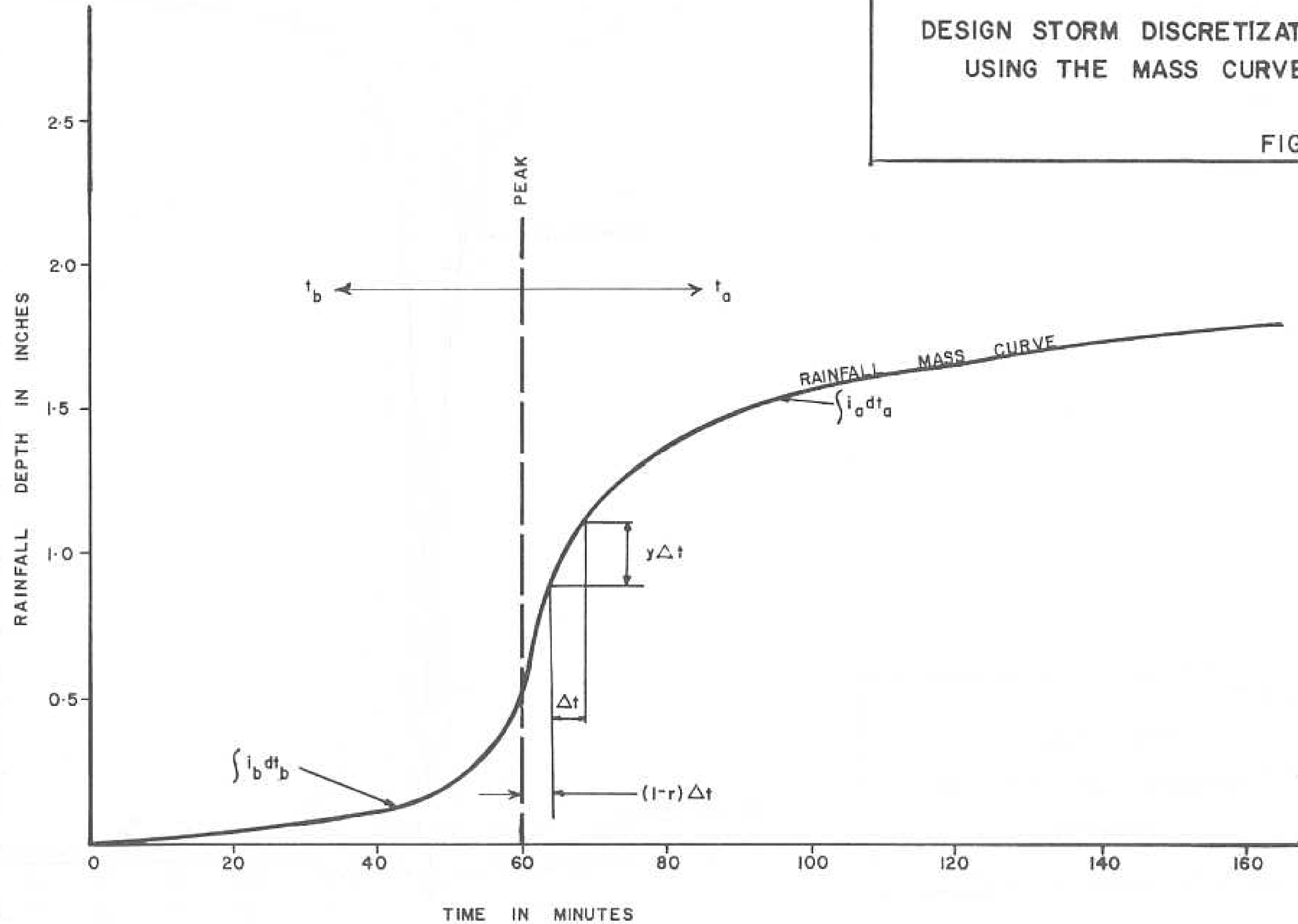
25 YEAR DISCRETIZED DESIGN STORM

HYETOGRAPH

Fig-3.2 b

DESIGN STORM DISCRETIZATION USING THE MASS CURVE

FIG. 3.3



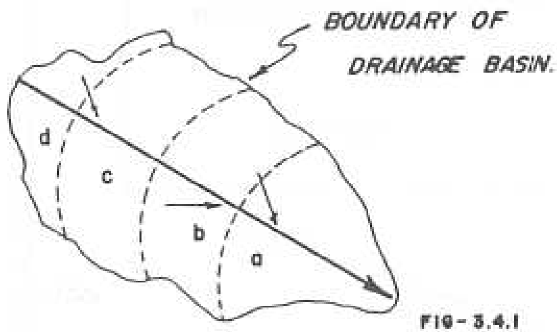


FIG-3.4.1

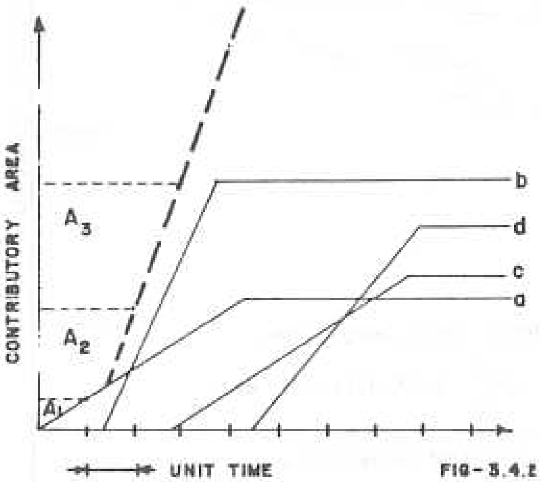


FIG-3.4.2

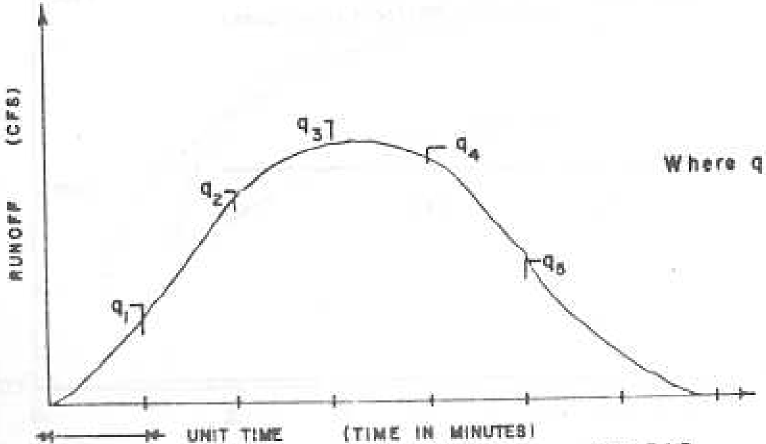


FIG-3.4.3

Where q =

$q_1 = i_1 A_1$

$q_2 = i_2 A_1 + i_1 A_2$

$q_3 = i_3 A_1 + i_2 A_2 + i_1 A_3$

$q_4 = i_4 A_1 + i_3 A_2 + i_2 A_3$

$q_5 = i_5 A_1 + i_4 A_2 + i_3 A_3$

etc.

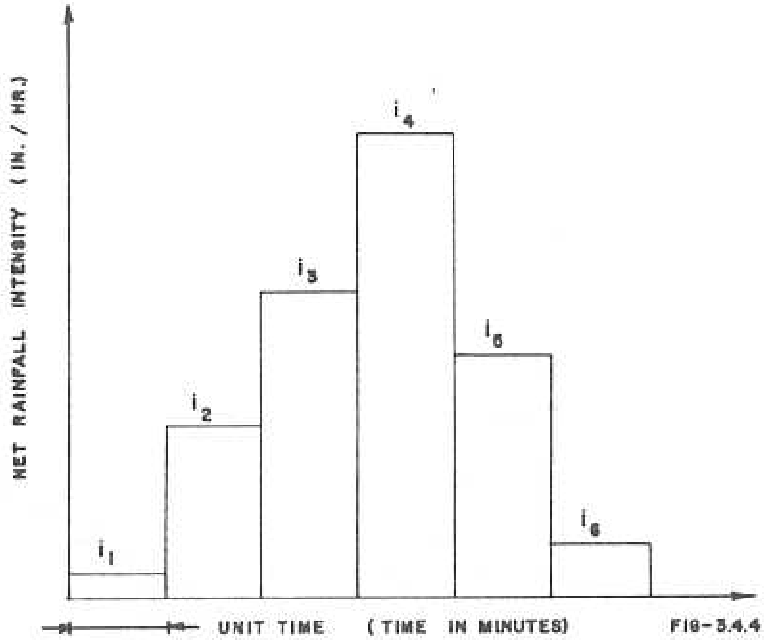
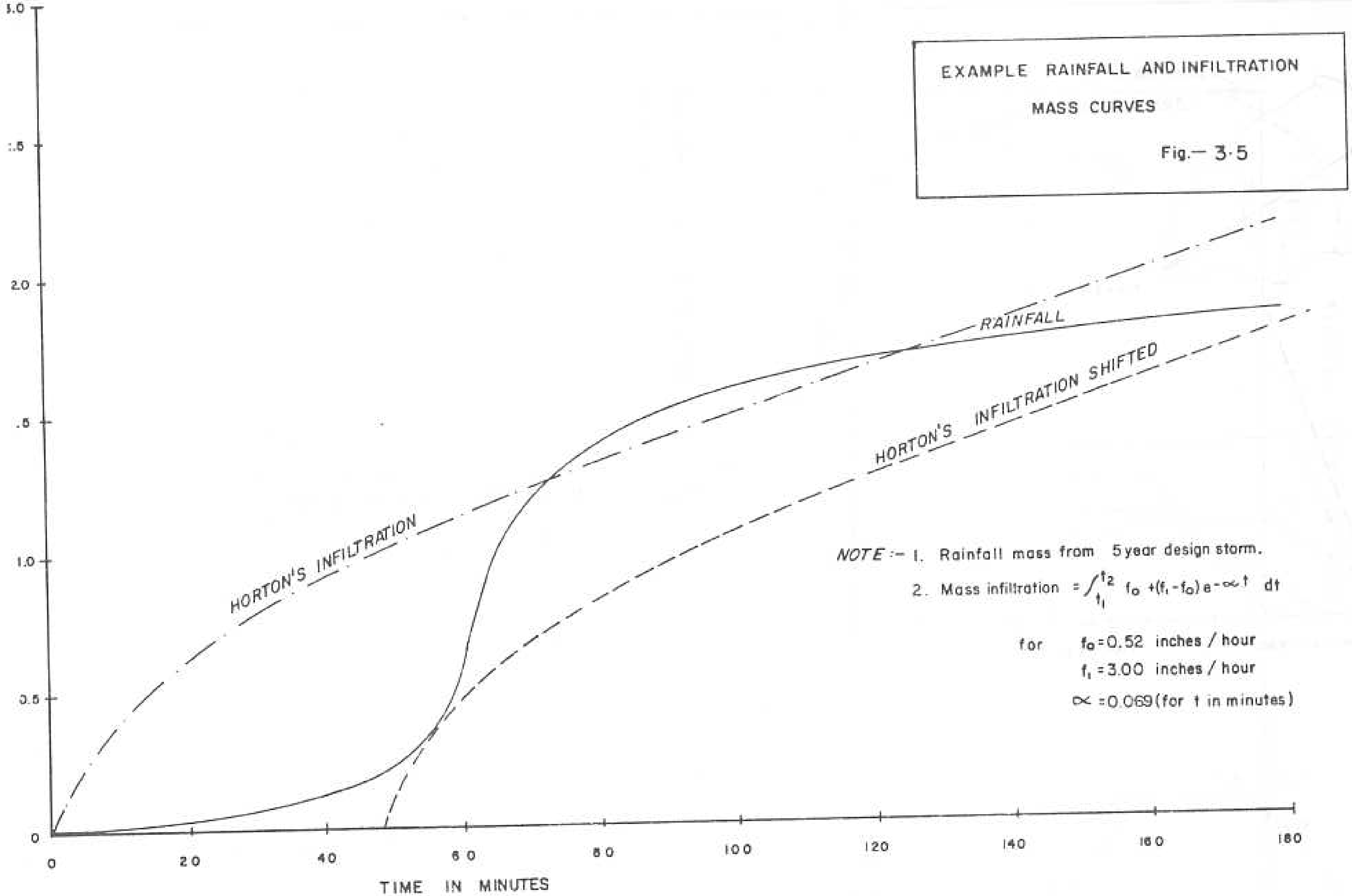
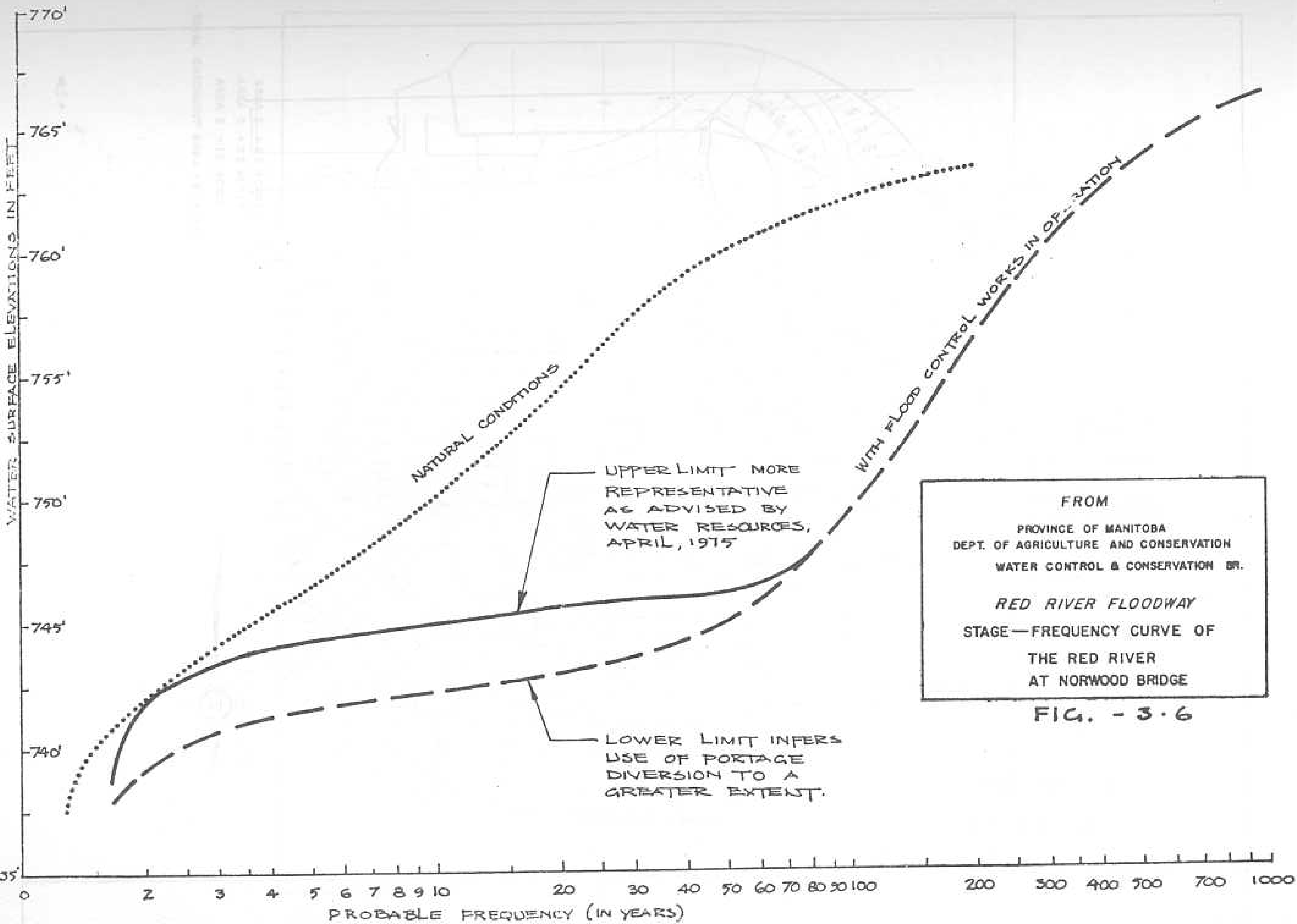


FIG-3.4.4

ISOCHRONE METHOD

FIG. — 3.4


EXAMPLE RAINFALL AND INFILTRATION
MASS CURVES
Fig.— 3·5
RAINFALL
HORTON'S INFILTRATION
HORTON'S INFILTRATION SHIFTED
NOTE :- 1. Rainfall mass from 5 year design storm.
2. Mass infiltration $= \int_{t_1}^{t_2} f_o + (f_1 - f_o) e^{-\alpha t} \, dt$
for $f_o = 0.52$ inches / hour
$f_1 = 3.00$ inches / hour
$\alpha = 0.069$ (for t in minutes)
0
0.5
1.0
.5
2.0
.5
.0
0
20
40
60
80
100
120
140
160
180
TIME IN MINUTES



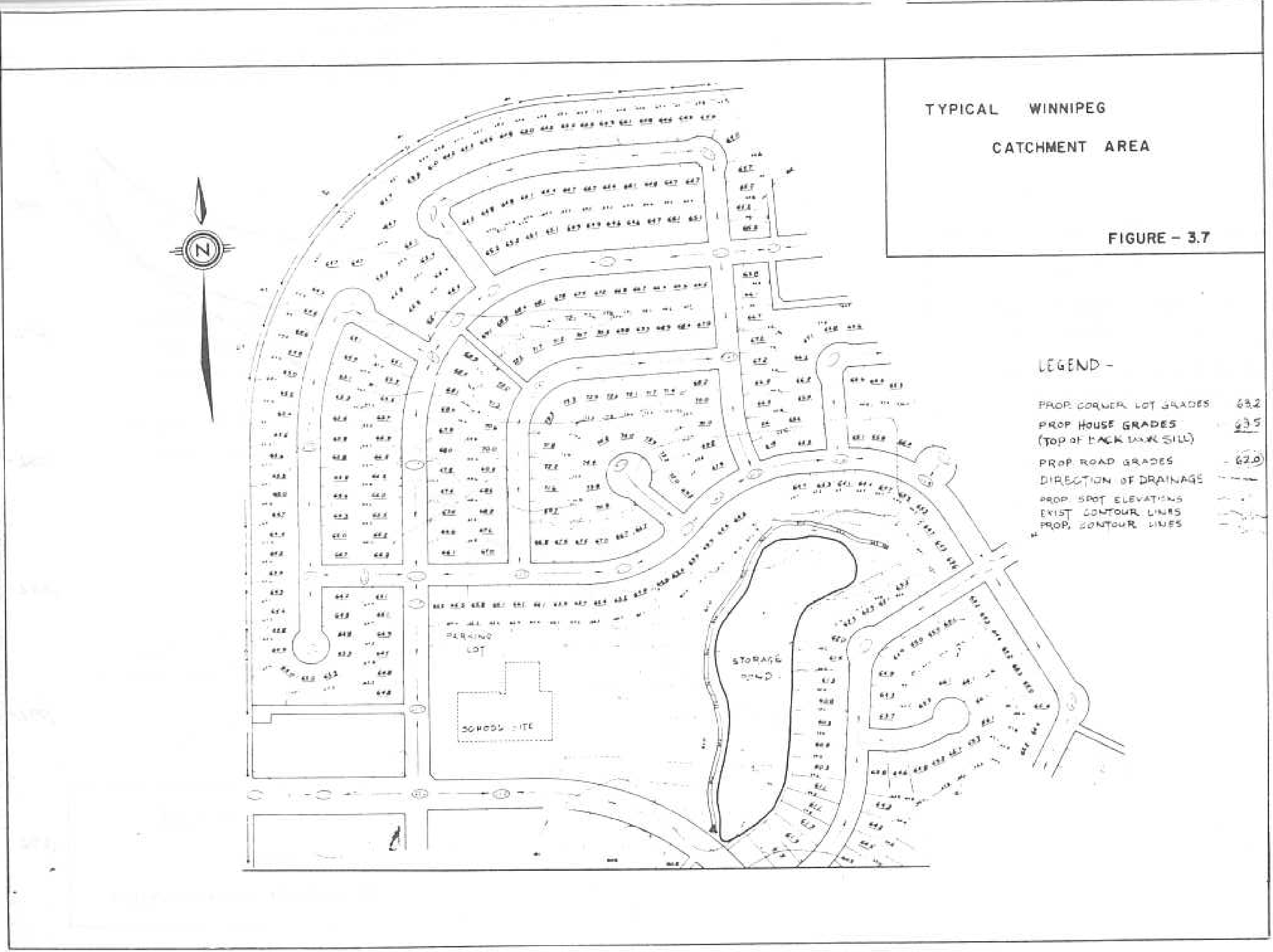
FROM

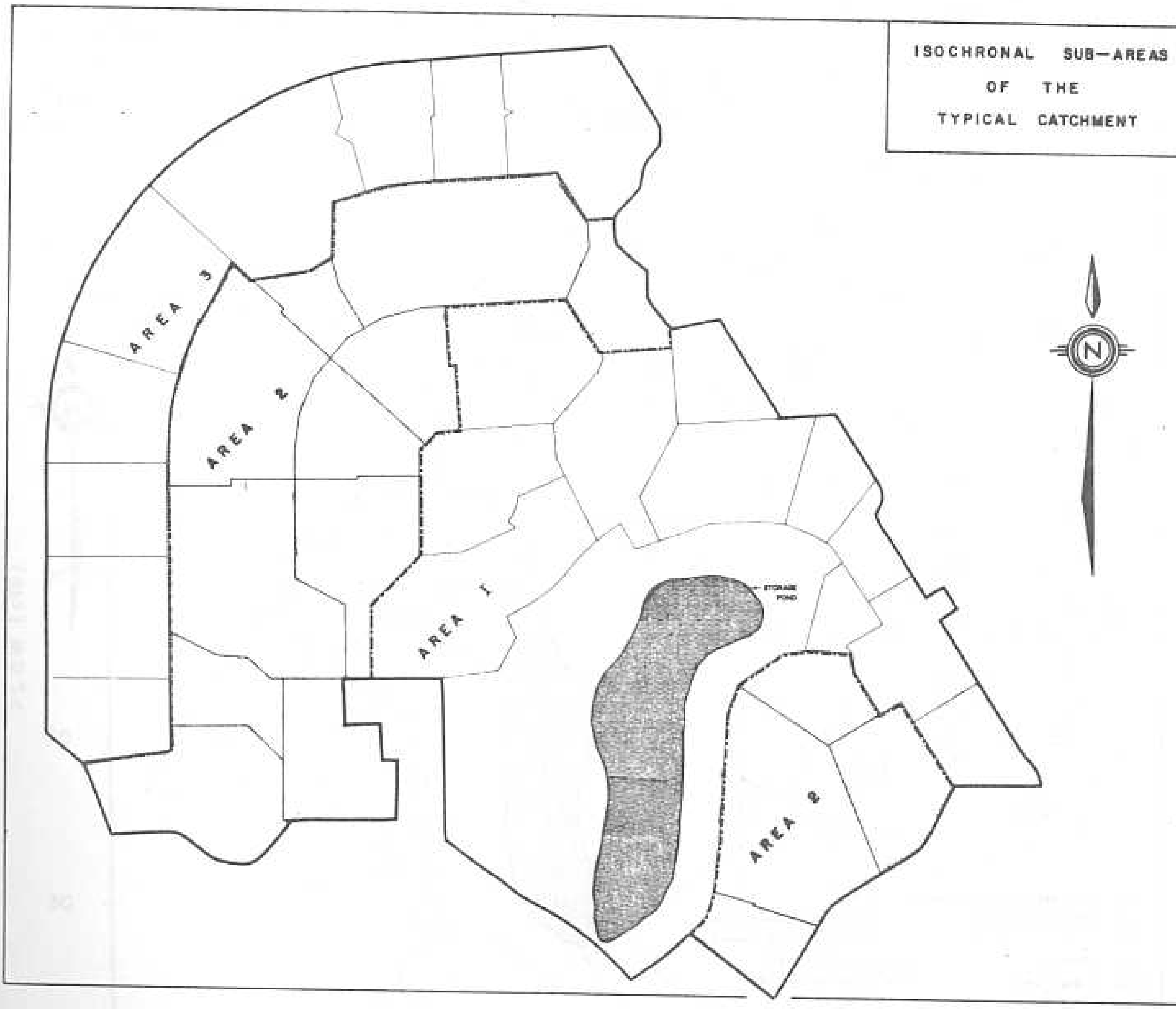
PROVINCE OF MANITOBA
DEPT. OF AGRICULTURE AND CONSERVATION
WATER CONTROL & CONSERVATION BR.

RED RIVER FLOODWAY

STAGE — FREQUENCY CURVE OF
THE RED RIVER
AT NORWOOD BRIDGE

FIG. - 3·6


TYPICAL WINNIPEG
CATCHMENT AREA
FIGURE – 3.7
LEGEND -
PROP. CORNER LOT GRADES 63.2
PROP. HOUSE GRADES (TOP OF BACK DOOR SILL) 63.5
PROP. ROAD GRADES (62.0)
DIRECTION OF DRAINAGE
PROP. SPOT ELEVATIONS
EXIST. CONTOUR LINES
PROP. CONTOUR LINES
PARKING LOT
SCHOOL SITE
STORAGE POND
N



AREA I = 23 ACRES
AREA 2 = 23 ACRES
AREA 3 = 15 ACRES
TOTAL CATCHMENT AREA = 61 ACRES

FIGURE - 3.8

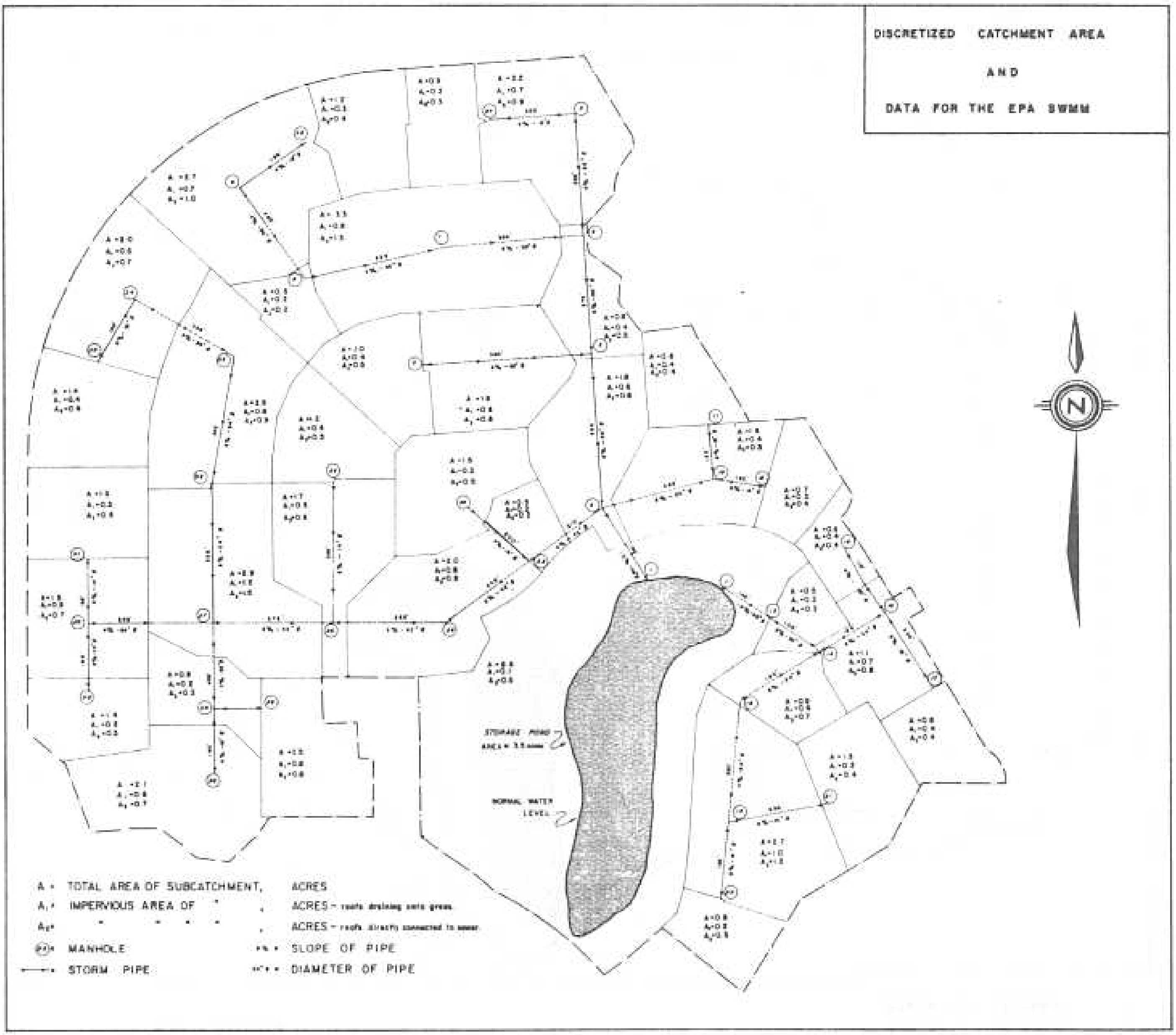
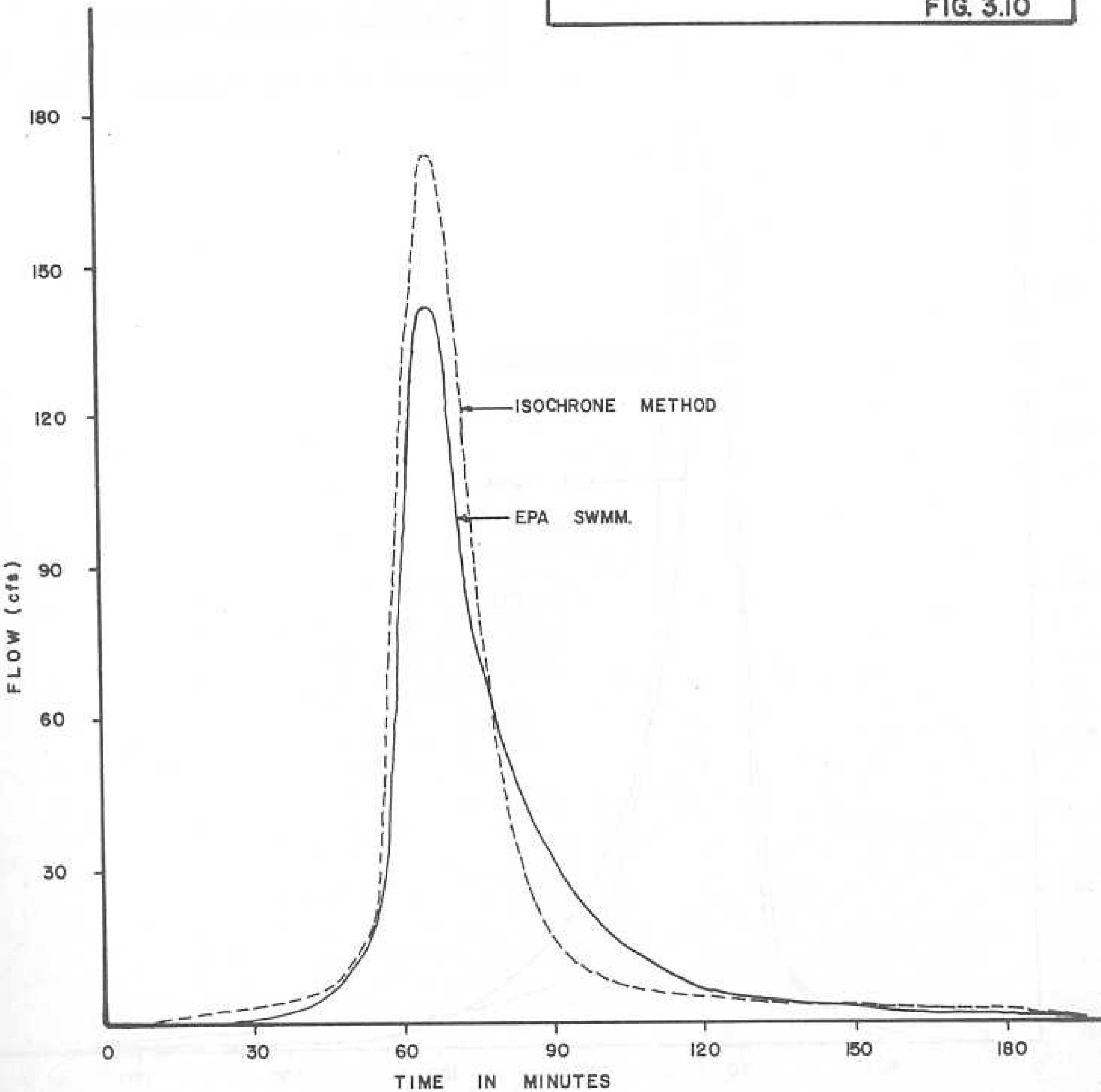
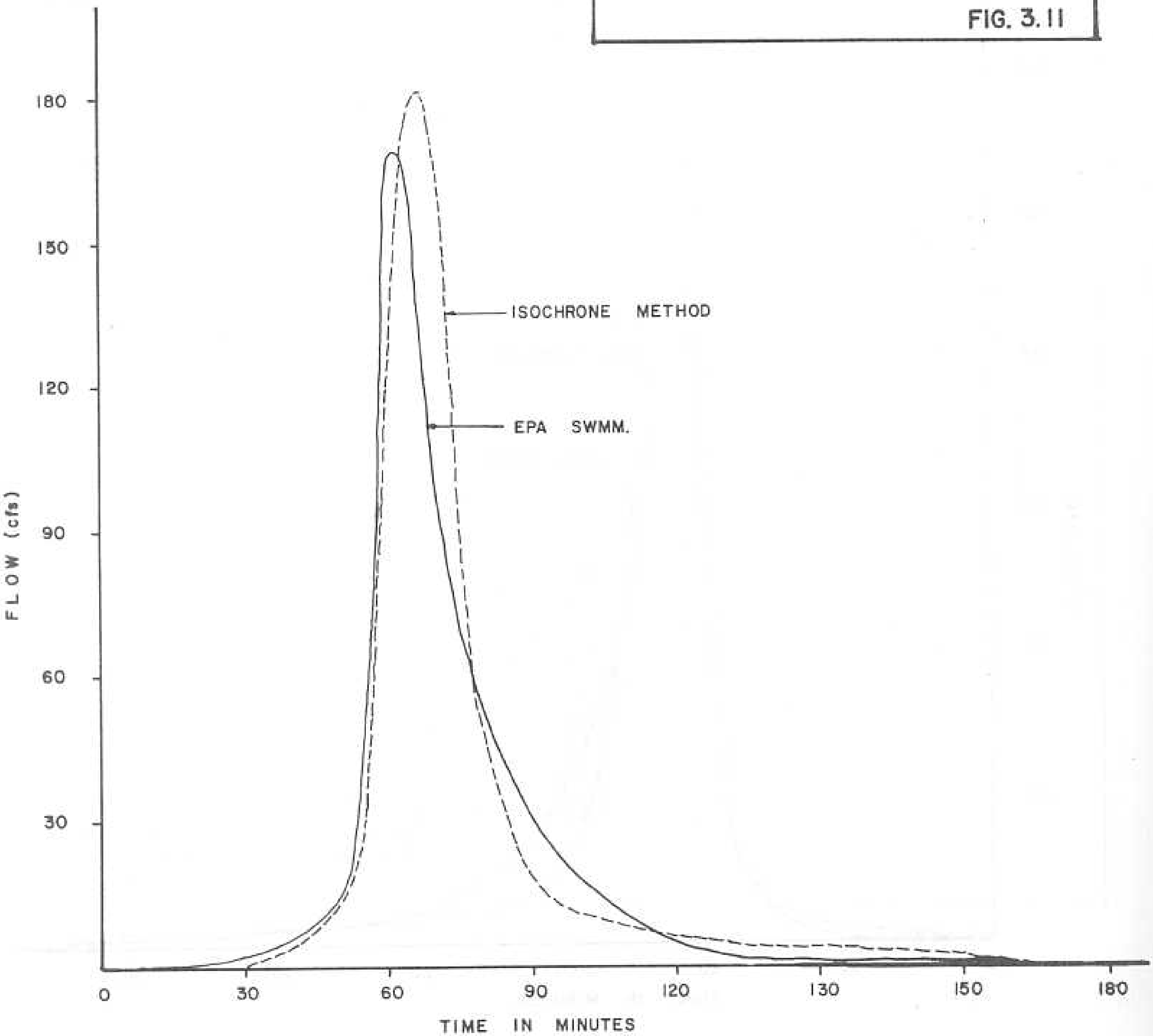

DISCRETIZED CATCHMENT AREA
AND
DATA FOR THE EPA SWMM
NORMAL WATER LEVEL
A = TOTAL AREA OF SUBCATCHMENT, ACRES
A_1 = IMPERVIOUS AREA OF " , ACRES – roofs draining onto grass
A_2 = " " " " , ACRES – roofs directly connected to sewer
MANHOLE
STORM PIPE
SLOPE OF PIPE
DIAMETER OF PIPE
N

FIGURE – 3.9

HYDROGRAPHS FOR 5 YR. STORM
FOR 61 ACRE TEST BASIN
ROOFS DRAINING ONTO GRASS
PERCENT IMPERVIOUSNESS - 32%
FIG. 3.10



HYDROGRAPHS FOR 5YR. STORM
FOR 61 ACRE TEST BASIN
ROOFS DIRECTLY CONNECTED
TO SEWER
PERCENT IMPERVIOUSNESS - 42%
FIG. 3.11



CHAPTER 4

STORAGE

4.1 General

As areas become urbanized there is a corresponding increase in volume and peak rate of runoff which requires construction of expensive drainage facilities. The use of storage as a means of reducing peak flows has long been an established practice in flood control on rural catchments but is only now becoming more widely used in the field of urban drainage. Reduction of peak flows results in a more economical pipe network design and also is an effective means of reducing the possibility of downstream flooding. Other advantages of on-site detention storage include use of the stored runoff for multipurpose uses or for supplementary water supplies, for erosion control, for aesthetic purposes and for recreational activities. Another important result of onsite detention is the prevention of overflows of combined sewers, thereby reducing stormwater pollution. Also, a potential application results from the fact that flow rates in sewers may be reduced to levels at which treatment of urban runoff may become feasible.

The City of Winnipeg has reviewed the present status of storage impoundments in Winnipeg and it has been stated that "Stormwater management schemes are being implemented by various developers in the Winnipeg area largely on the basis of economics, with varied emphasis on the recreational, aesthetic, and the standard of protection provided." [8] The review included the types of impoundments in use and presented general guidelines for physical design, maintenance and operation and assessment of benefits. However, design guidelines from a hydrologic point of view were not considered.

Correct design of storage facilities requires a method for computing the time distribution of runoff for a particular real storm event or design storm. The required storage volume is a function of the capacity of the outlet structure and also depends on economic and safety considerations, which are generally accounted for by a judicious selection of the design storm return frequency. For large ponds, a method of routing the inflow through the reservoir is required in order to accurately estimate the storage volume. The above problems are discussed in frame of the general storage concepts given in the following sections of this chapter. For the purposes of this discussion, "upstream" and "downstream" storage facilities are described; retention or detention facilities can be designed in both cases.

4.2 Upstream Storage Facilities

4.2.1 General

Upstream storage facilities capture and detain water near or at the point of rainfall occurrence before the runoff enters the drainage system. Thus, the greatest beneficial potential exists for reducing the overall costs of the urban drainage network. The possibility of using such facilities should, therefore, be considered an integral part of the design process for all new drainage works in Winnipeg. Possible upstream storage facilities include the following: [14]

i. Roof tops
ii. Parking lots
iii. Recreation areas
iv. Property line swales
v. Parks
vi. On site ponds

Such storage facilities should be planned for in the early stages of a development and utilized as a means of reducing both upstream and downstream drainage costs. Small ponds also have an aesthetic benefit in residential areas, and should therefore continue to be encouraged for use in residential areas in Winnipeg.

According to a recent report by Dolhun [3], basement flooding is a widespread problem in Winnipeg for large storms of design magnitude. This flooding problem could be alleviated by constructing relief sewers or by using upstream storage techniques to regulate the inflow into the sewer system. For example, modifications could be made to the inlets of existing roofs and parking lots, etc.

The amount of roof top ponding depends primarily on the structural design of the building. The National Building Code (1970) allows a maximum roof loading of snow of 45 lbs. per square foot which is equivalent to 7" of water. [7]. Denver allows a roof load of 30 lb. per square foot and uses a safety factor of 2 thereby limiting the maximum rainfall storage on roofs to about 3 inches. Denver also recommends a release rate of about ½" per hour. Overflows must be provided in order to ensure that structural loading requirements are not exceeded for large storms.

Large paved parking lot areas easily lend themselves for use as temporary upstream storage facilities. The allowable depth is a function of safety and convenience to users. Again Denver uses an approximate criteria of 3" of storage with a controlled release of ½" per hour.

In some cases, temporary upstream storage can also be obtained by using property-line swales, park storage and small on-site ponds. Swales along back property lines can be used for temporary storage and released slowly or allowed to infiltrate into the ground. Prior to implementation of such designs, soil conditions should be investigated in order to avoid adverse effects to foundations, etc. Parks and recreational fields contribute little runoff during moderately intense storms and such areas can also be designed to detain water temporarily. Such temporary ponding can generally be designed to minimize potential conflict in functions by allowing for relatively rapid drainage after the end of the storm. Also, by combining parks, ball fields and green belt use with the storage of storm runoff water, the cost of drainage can be reduced and the cost of green belts, parks and playing fields can be reduced. This can create measurable and significant benefits for the public, the degree of which must be assessed for individual projects.

It is obvious that the design and use of any of the above mentioned temporary upstream storage facilities depends mainly on the type and extent of development being planned. Therefore, designs for individual facilities should be considered according to their merit with respect to simplicity of design and construction, safety and economic considerations.

Generally speaking, for all types of upstream surface storage, hydrograph methods such as the isochrone method or the EPA-SWMM are required to design the facility in order to assess its usefulness with respect to the benefits derived from modification of the time distribution of flow to the downstream drainage system. The return frequency of the design storm depends on the purpose and type of the storage design. For example, if the purpose of upstream storage is only to reduce the cost of the local pipe system, then the same design frequency could be used for both. For additional benefits from localized flooding, a larger storm could be used to design the storage.

4.2.2 Example - Temporary Roof and Parking Lot Storage

In order to demonstrate the effect of upstream storage on design of the drainage network, a simple example was executed using the EPA-SWMM and a subcatchment of the typical Winnipeg area described in Chapter 3.0. The subcatchment was chosen to contain the school site and associated parking lot shown on Figure 3.7. Details of the subcatchment are indicated on Figure 4.1. The grassed area is assumed to drain to the parking lot. For simulation purposes, the roof and parking lot are connected to the system by two separate pipes, the diameters of which control the rate of flow from the ponded areas. When the capacity of the pipes is reached, the EPA-SWMM stores the surcharge at the upstream end, until the storm intensity decreases, allowing the stored water to flow into the system at the pipe capacity. For the purposes of this demonstration, the 5 year rainfall was used and the pipe size was chosen such that the maximum surcharge was equivalent to a surface ponding of about 1" on both the roof and parking lot. The 5-year storm hydrographs with and without ponding are shown on Figure 4.1. It is evident that application of upstream storage facilities such as roofs and parking lots to large drainage areas can result in significant savings with regard to the design of the downstream pipe network and storage facilities. Therefore, due consideration should be given to the use of such facilities.

4.3 Downstream Storage

4.3.1 General

Downstream retention or detention storage facilities are located downstream from the drainage area. Runoff may be derived from one or several upstream tributary catchments, and for this reason the operation and maintenance of such facilities may become the responsibility of local government. Such facilities are usually designed to reduce peak runoff and prevent flooding in the main drainage system downstream, and are therefore supplementary to upstream storage facilities. Such facilities are particularity applicable to flat areas of minimum hydraulic head. The use of hydrograph methods for design is essential in order to account for the distribution of runoff and the resulting reservoir fluctuations.

Types of facilities include channel storage and onstream and offstream ponds. Open channel storage is particularly effective in flat areas where the flood wave is relatively slow moving. As the flow increases, the depth and storage increases,resulting in reduced peaks downstream. Channel storage can be designed by routing the incoming hydrograph using relatively simple techniques such as the well-known Muskingum flow routing method. Several texts describe this routing procedure. (e.g. Chow [1], Gray [4])

Offstream storage facilities generally operate only during peak flows by utilizing side-channel spillways. Such facilities are therefore infrequently inundated and are ideal for multipurpose uses such as parks and recreational fields.

Onstream storage facilities such as ponds and reservoirs are generally more common, and provide various benefits. In addition to reducing the peak flow downstream, the ponds act as settling basins for sediment and debris, thereby improving downstream water quality. Also, by storing the water or re-using it in the watershed, the quantity of nutrients released to the receiving water may be reduced

In some cases, reduction in the peak flow will also result in a corresponding reduction in erosion in open channels downstream. Care must be taken to design adequate emergency spillways to avoid structural failure which might result in serious economic losses downstream.

In the design stage, it is important to evaluate the disadvantages associated with each alternative, such as sediment removal, safety, general maintenance, etc.

Hydrograph methods of design must be used for all of the above mentioned downstream storage facilities. For illustration purposes, the design of an onstream reservoir storage facility is discussed in more detail in the following section.

4.3.2 Storage Design

Current status of the use of impoundments in Winnipeg and considerations such as maintenance, benefits, economics and physical design parameters are discussed by Penman [8]. From the hydrologic point of view Penman states that "the normal storm water collection system is designed to carry a once-in-5-year storm with excess waters temporarily stored on the street surfaces. To assure protection of the local residential properties, however, larger storm design frequencies are required for impoundments." A review of current design practises in Winnipeg has indicated that many impoundments are currently designed for storms with a return frequency of 25 years or more depending on general considerations of the economic consequences. However, some designs are based on incorrect use of the Rational formula as a means of estimating the mass curve of inflow. With reference to Figure 4.2, the following general procedure, using the Rational method has been widely used:

(i) Rainfall intensity-duration-frequency curves are derived as described in Chapter 2.0.

(ii) An average C value for the basin is estimated, generally by assuming it to be a function of the percent imperviousness of the basin.

(iii) A storm return frequency is chosen and intensities i_l to i_n are determined for selected storm durations t_d ranging from t_l to t_n.

(iv) The total runoff volume V_d, corresponding to each duration is computed according to the relationship

$$V_d = i_d \, t_d \, CA \qquad \textbf{4.1}$$

where V_d = runoff volume in ft

i_d = rainfall intensity corresponding to duration t_d in inches/hour

td = rainfall duration in seconds

A = drainage area in acres

C = runoff coefficient

In addition to all the disadvantages of the Rational method discussed in Chapter 3.0, this procedure has the following weaknesses:

(i) No consideration is given to the actual storm pattern and the effects of rainfall antecedent to the peak intensities.

(ii) The contributing area is assumed to be evenly distributed in time. This assumption of a linear time area curve can result in considerable error.

(iii) Since no runoff hydrograph is developed, no reservoir routing technique can be used. That is outflow must be considered independent of inflow and storage, which is generally not the case. Also, the possibility of examing various reservoir operation strategies cannot be realized.

(iv) Variations in various physical parameters affecting the runoff such as changes in infiltration and surface storage cannot be accounted for.

In view of these limitations, hydrograph methods such as the Isochrone method and the EPA-SWMM described in Chapter 3.0 and used in conjunction with the design storms developed in Chapter 2.0 are recommended to derive the runoff inflow hydrographs and mass curves for the purpose of determining the required storage.

As mentioned previously, the outflow from a storage pond is a function of inflow, storage and type of outflow structure. Simple reservoir routing techniques such as that represented by the following equation are widely used:

$$I = D + \frac{dS}{dt} \qquad \textbf{4.2}$$

This equation can be approximated by a time interval t, and the terms rearranged to yield:

$$\frac{1}{2}(I_1 + I_2)t + (S_1 - \frac{1}{2} D_1 t) = (S_2 + \frac{1}{2} D_2 t) \qquad \textbf{4.3}$$

where 1 and 2 are subscripts at the beginning and end of the time, t, respectively.

I = inflow

D = discharge

T = routing period

S = storage

The terms on the left hand side of equation 4.3 are known. A simple graphical procedure described in detail by Wilson [11] and others is recommended for solving equation 4.3 for the storage and outflow at the end of each time interval. An outflow hydrograph and corresponding mass curve can then be produced, and the required storage is the largest difference between the inflow and outflow mass curves. In many cases, the outflow mass curve will become linear when the capacity of the outflow structure is reached. An example is given in Section 4.3.3.

Other factors which must be considered in the design of storage ponds are the detention depth and the relationship between the elevation of the water in the storage pond and the elevation in the receiving water body. For example, during the spring in some cases, depending on location, the Red River may be high enough to affect the outflow and storage in the proposed pond. As indicated in Section 3.5.2, the frequency of the local design storm hydrograph and water levels in the Red River can be treated as independent events for all practical purposes. Each storage pond design should be checked for any possible influence from the Red River and appropriate design changes made where necessary. For example, pumping may be required during spring flow conditions for some designs.

4.3.3 Example - Storage Design

An example computation of storage requirements has been made using the typical Winnipeg drainage area described in Chapter 3.0. The 25-year design storm given in Chapter 2.0 was used together with the methods of computing runoff described in Chapter 3.0. Computations involving the isochrone method used the 3 areas indicated on Figure 3.8. In order to obtain a conservative estimate of storage when using the 25 year storm rainfall as input to the EPA-SWMM, it is necessary to assume that most of the runoff occurs as overland flow. That is, the pipe network designed using the 5 year storm will not accommodate all of the runoff resulting from the 25 year storm. The EPA-SWMM stores excess runoff at each surcharged manhole and releases it later in the storm according to the capacity of the pipe. However, in the real case, some of the excess runoff would be stored at the manhole and some (an unknown amount) would run overland to the storage facility, following the natural drainage. Since the relative amounts stored and running overland are very difficult to estimate, it is more conservative to ignore the pipe network when using the EPA-SWMM for design of storage facilities. In this example, the typical catchment shown on Figure 3.7 was divided into the 3 main natural drainage areas indicated on Figure 4.3. Thus a coarse basin discretization can be used with the EPA-SWMM for obtaining a conservative design of storage facilities. The hydrographs for the 25year storm using the isochrone method and EPA-SWMM are shown on Figure 4.4. By comparison an EPA-SWMM simulation which used 37 subcatchments and the pipe network designed for a 5 year rainfall resulted in a peak flow about 10% lower than the peak calculated ignoring the pipes and using a coarse discretization. The resulting mass curves derived from the hydrographs and the mass curves derived by the Rational method outlined in Section 4.3.2 for the C values of 0.65 and 0.36 determined in Chapter 3.0 are shown on Figure 4.5.

In order to determine the size of the required storage facility, a reservoir shape (i.e. depth-storage relationship) was assumed and the inflow hydrograph produced by the EPA-SWMM was routed through the reservoir using the methodology outlined in Section 4.3.2. The routed hydrograph and resulting outflow mass curve are shown on Figures 4.4 and 4.5 respectively. Since a limit of 20 cfs was assumed for the outflow the isochrone mass curve of outflow would be very similar to the mass curve of outflow derived by routing the EPA-SWMM inflow hydrograph.

As previously discussed, the reservoir routing procedure cannot be used with the Rational method, and therefore, in this case, the mass curve of outflow is assumed to be constant at 20 cfs starting at the beginning of the storm. Reference to Figure 4.5 indicates that for this example using the test basin and the 25 year design storm, the hydrograph methods give the most conservative storage estimates, with a difference of only about 4% between the storage estimated using the EPA-SWMM and the Isochrone method.

On the other hand for this example, the Rational method for C = 0.65 gives a storage about 20% smaller than that estimated by the EPA-SWMM. For C = 0.36 the storage value estimated by the Rational method is, by comparison, unrealistic when compared to the other values.

It is therefore essential that in general some kind of hydrograph method should be used for design. Furthermore, a reservoir routing technique should also be used since the routed outflow also affects the storage design. Details of the methods of computation finally selected by the designers should be provided for review with the presentation of the final storage design. In general it is felt that methods such as the isochrone method can be used for preliminary storage design and more sophisticated methods such as the EPA-SWMM should be used for final design purposes.

CHAPTER 4 REFERENCES

1. Chow, V.T. "Handbook of Applied Hydrology", McGraw-Hill, 1964

2. Committee of the Water Pollution Control Federation "Design and Construction of Sanitary and Storm Sewers", WPCF Manual of Practice No. 9, 1969

3. Dolhun, S. and E. P. Fedyniak "Report on Basement Flooding in Winnipeg – 1972", City of Winnipeg, 1973

4. Gray, D.M. (ed.) "Handbook on the Principles of Hydrology"

Secretariat, Canadian National Committee for the International Hydrological Decade, Ottawa, 1970

5. McPherson, M.B. "Better Design of Stormwater Drainage Systems", APWA Reporter, May, 1974

6. Metcalf and Eddy, Inc., University of Florida and Water Resources Engineers, Inc., "Storm Water Management Model", 4 volumes, Federal Environmental Protection Agency, Water Pollution Control Research Series 11024, DUC, 1971

7. National Building Code, Canada, 1970

8. Penman, A. "Stormwater Management by Use of Impoundments" Waterworks, Waste and Disposal, City of Winnipeg

9. Rice, L. M, "Reduction of Urban Runoff Peak Flows by Ponding", Journal of the Irrigation and Drainage Division, ASCE, 1971

10. Underwood, McLellan & Associates Limited "Analysis of Storm Runoff and Lake Storage", City of St. James, Assiniboia, 1970

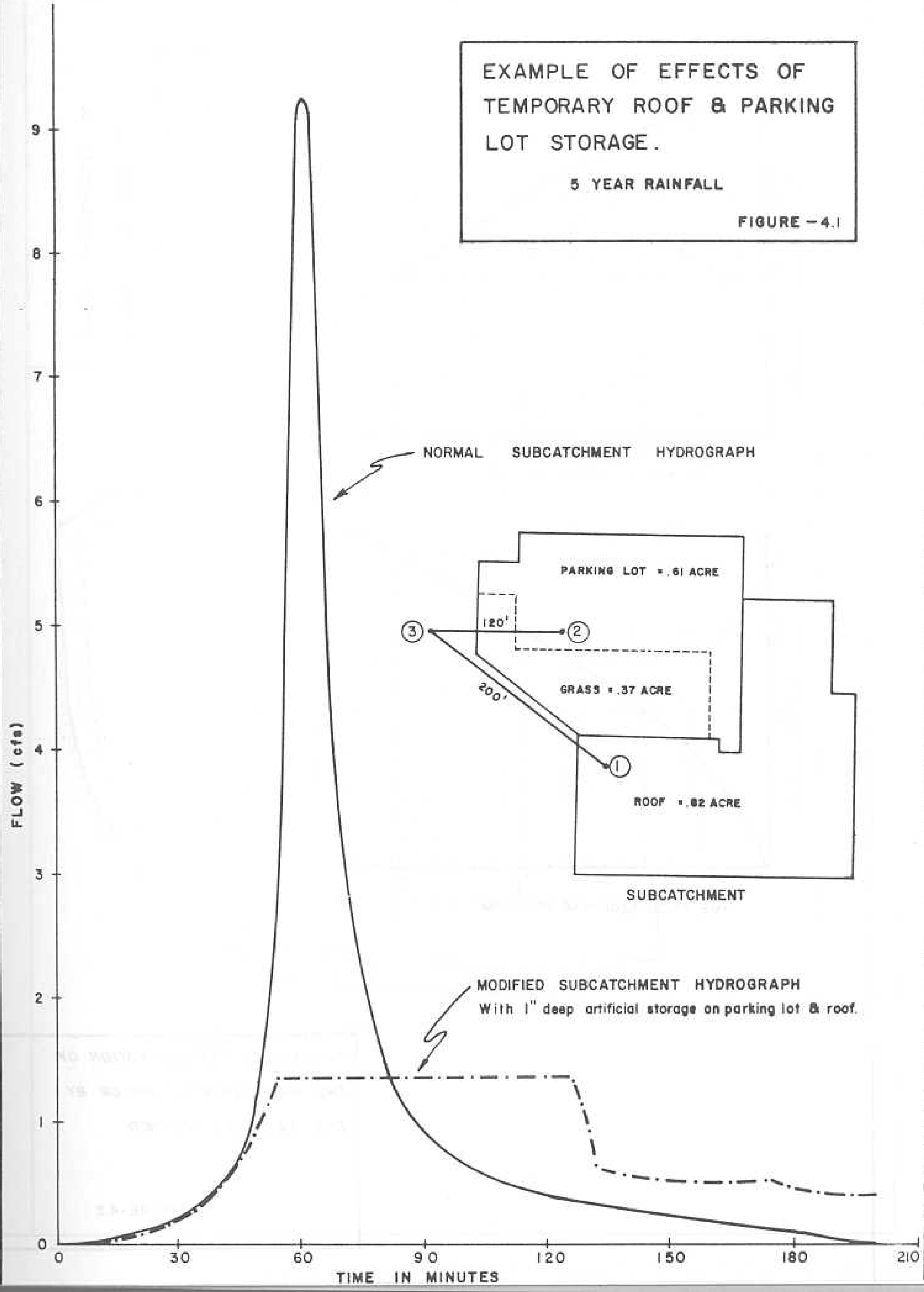
11. Underwood, McLellan & Associates Limited "Rural Municipality of Fort Garry Storm Drainage Study", 1971

12. Winnipeg "Land Drainage Study", Waterworks and Waste Disposal Division, Winnipeg, 1972

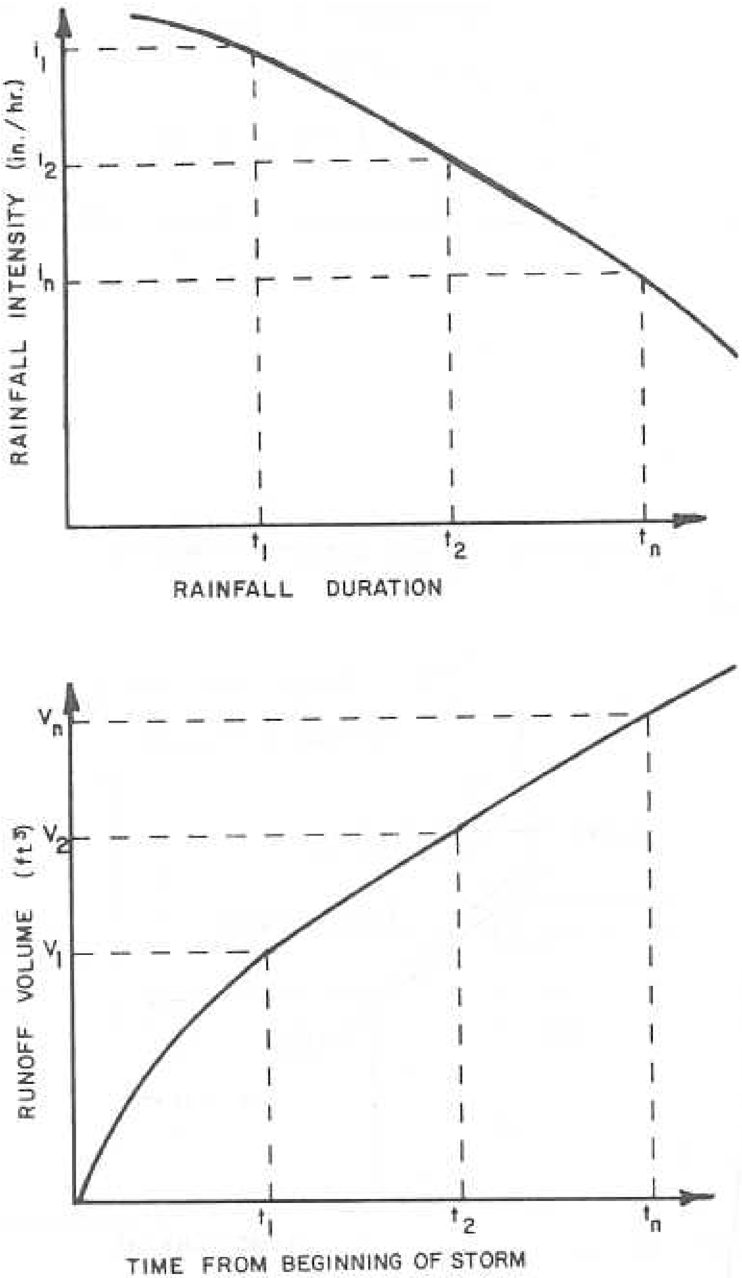
13. Wilson, E.M. "Engineering Hydrology", MacMillan, 1969.

14. Wright-McLaughlin Engineers "Urban Storm Drainage - Criteria

Manual", Vol. 1, 2, Denver, Colorado, March, 1969

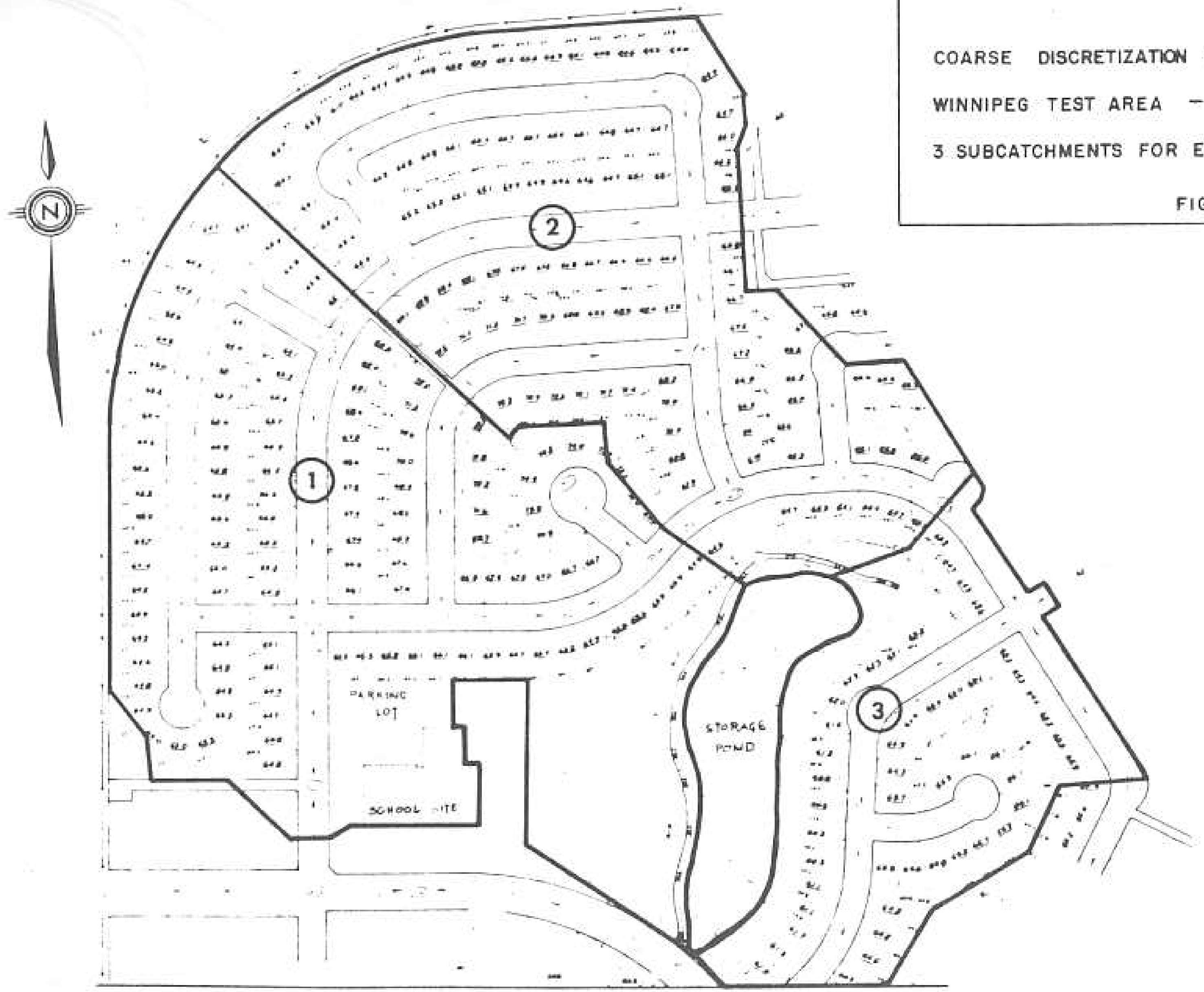


EXAMPLE OF EFFECTS OF TEMPORARY ROOF & PARKING LOT STORAGE.
5 YEAR RAINFALL
FIGURE — 4.1
NORMAL SUBCATCHMENT HYDROGRAPH
PARKING LOT = .61 ACRE
120'
200'
GRASS = .37 ACRE
ROOF = .82 ACRE
3
2
1
SUBCATCHMENT
MODIFIED SUBCATCHMENT HYDROGRAPH
With 1" deep artificial storage on parking lot & roof.
FLOW (cfs)
0
1
2
3
4
5
6
7
8
9
30
60
90
120
150
180
210
TIME IN MINUTES



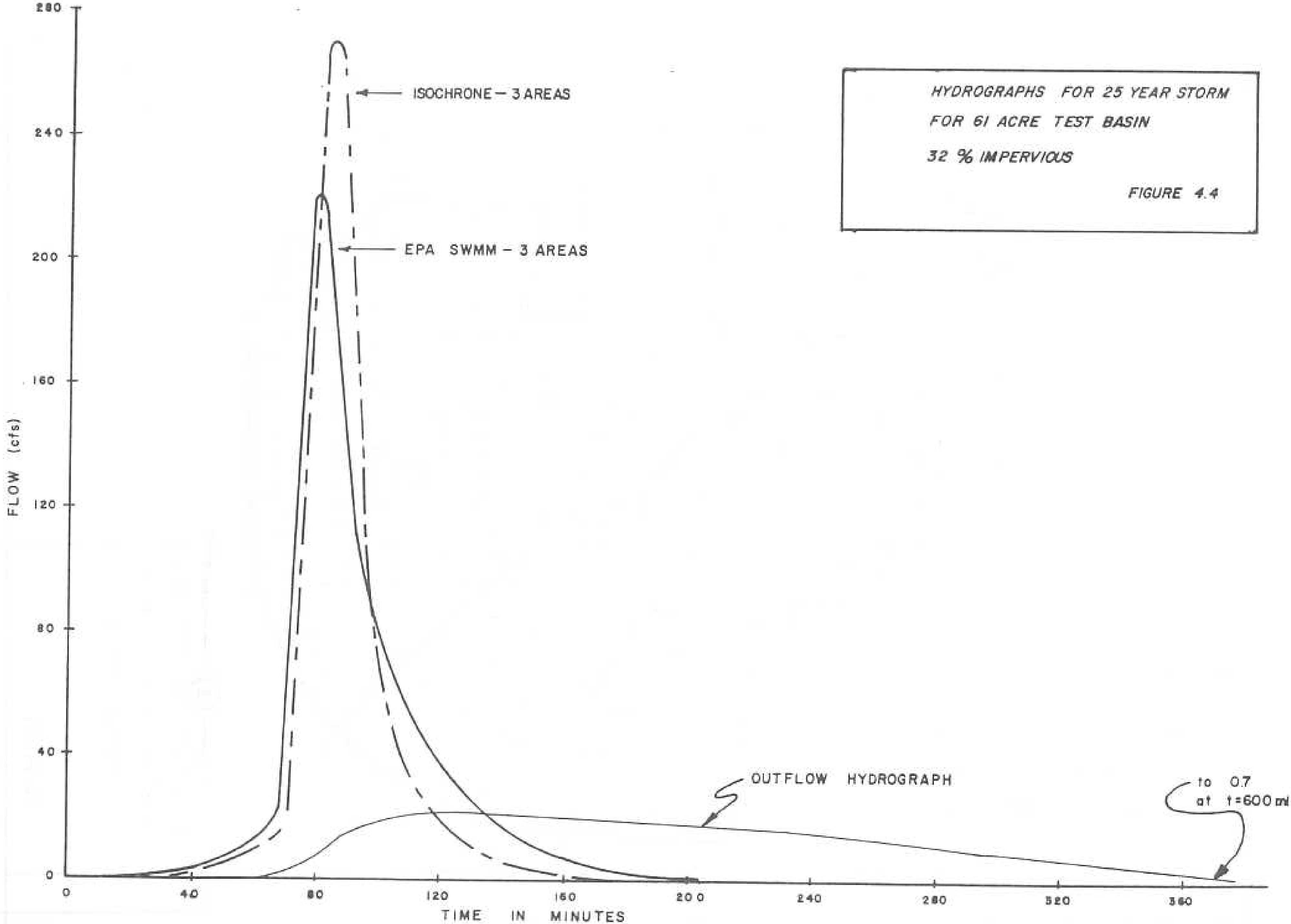
PROCEDURE FOR ESTIMATION OF THE MASS CURVE OF INFLOW BY THE RATIONAL METHOD.

FIGURE-4.2



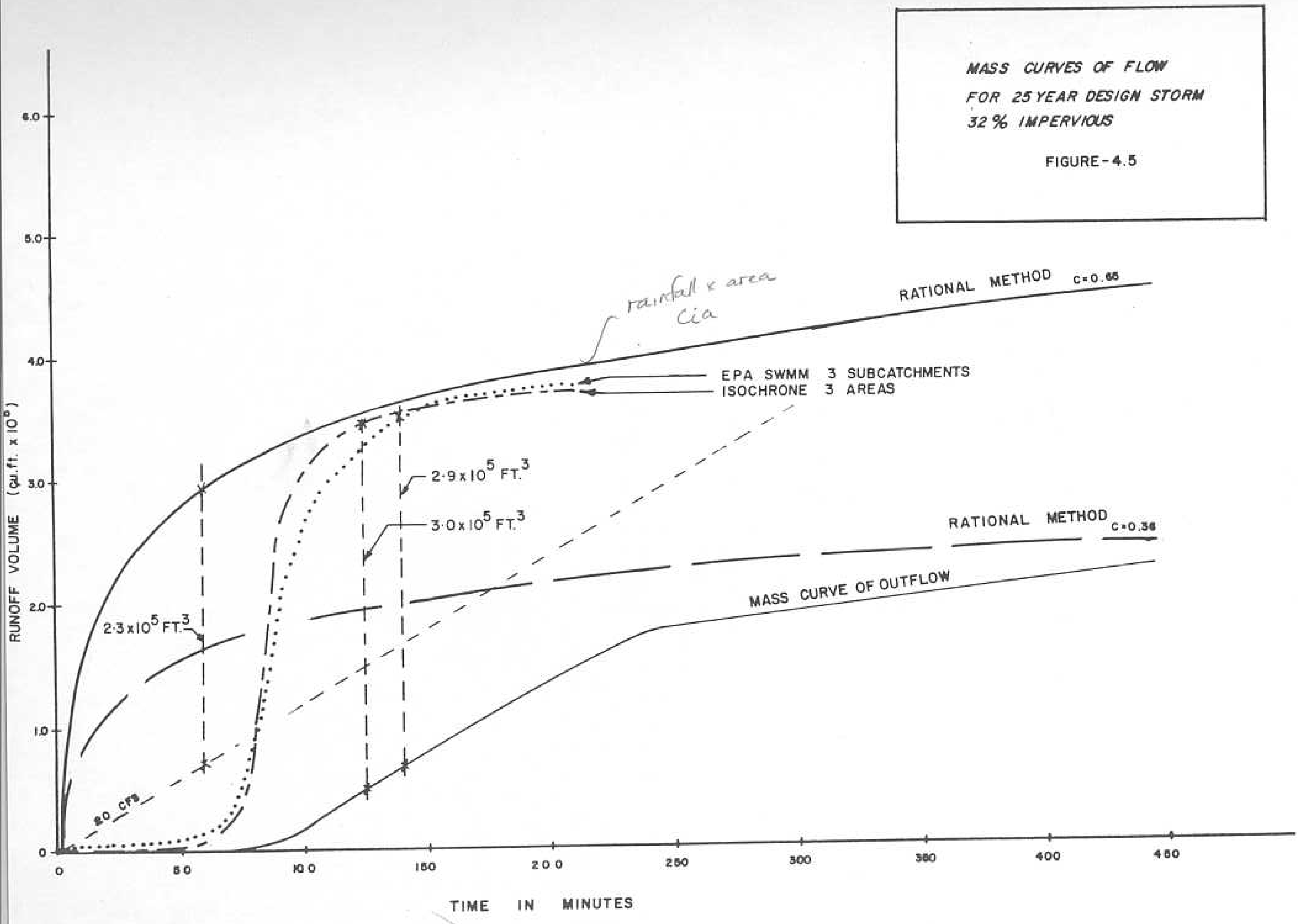
COARSE DISCRETIZATION OF WINNIPEG TEST AREA — 3 SUBCATCHMENTS FOR EPA-SWMM

FIGURE – 4.3



HYDROGRAPHS FOR 25 YEAR STORM
FOR 61 ACRE TEST BASIN
32 % IMPERVIOUS

FIGURE 4.4


MASS CURVES OF FLOW
FOR 25 YEAR DESIGN STORM
32 % IMPERVIOUS
FIGURE-4.5
RATIONAL METHOD C=0.65
rainfall x area
Cia
EPA SWMM 3 SUBCATCHMENTS
ISOCHRONE 3 AREAS
2·9x10^5 FT.^3
3·0x10^5 FT.^3
RATIONAL METHOD C=0.36
MASS CURVE OF OUTFLOW
2·3x10^5 FT.^3
20 CFS
RUNOFF VOLUME (cu.ft. x 10^5)
0
1.0
2.0
3.0
4.0
5.0
6.0
0
50
100
150
200
250
300
350
400
450
TIME IN MINUTES

CHAPTER 5.0

RECOMMENDATIONS

5.1 General Recommendations

In summarizing, the following general comments can be made concerning the usefulness and design of drainage and storage facilities:

1) Until extensive continuous measurements are available, the summer design storms developed in Chapter 2 should be used to obtain estimates of the volume and time distribution of runoff. Rainfall-intensity duration design curves should be updated about every 5 years as more data becomes available. However, continuous measurement programs of rainfall and stormwater flows should be maintained in order to eventually determine the most severe real storms, their rainfall pattern and associated frequencies. Such measurements are extremely valuable in calibrating mathematical models representing the rainfall runoff process. For example, existing quantity and quality measurements on the Bannatyne area in Winnipeg have been used to calibrate the EPA-SWMM. Such measurements are also useful in defining the frequency of overflows in combined sewer areas.

2) Drainage designs which minimize the amount of surface runoff over impervious surfaces should be encouraged. This reduces the peak flows and hence the downstream drainage and storage costs.

3) Upstream and downstream storage facilities should be used wherever possible in order to reduce the cost of the drainage system. Consideration should be given to other possible benefits such as aesthetic value, recreational uses of ponds, erosion and siltation control and reduction of downstream pollution and flooding. The selection of the type of storage facility will depend on the benefits for each particular case.

4) Negative aspects of storage facilities should also be examined for each particular proposed application. These include general inconvenience (e.g. interference of parking lot storage to pedestrian movement), safety to children, land cost, legal aspects, pumping requirements, maintenance and the possibility of developing adverse environmental conditions such as algae growths and mosquito breeding areas.

5.2 Design Recommendations

1) For areas larger than about 5 acres, hydrograph methods are preferred for the design of drainage facilities. Hydrograph methods should be used for the design of all storage facilities. Such methods must account for the physical parameters describing runoff and storage such as the following factors - storm pattern and frequency, surface depression and detention storage, infiltration, the pipe network and flow routing.

2) Design storm frequencies of 5 years and 25 years should be used as the guideline for the network and storage design respectively. Selection of other frequencies should be justified for each application on the basis of an economic cost benefit analysis.

3) The designer should use the synthetic summer rainfall patterns developed in Chapter 2 as input to the hydrograph models. A time step of 5 minutes (or less) is recommended but this could be modified provided the designer gives a complete justification.

4) For catchments having a time of concentration greater than about 6 hours, the designer should consider spring rainfall plus snowmelt as input to a hydrograph model.

5) For new drainage networks, the designers should give due consideration to use of the following storage facilities as a means of reducing system costs:

- roof top storage
- parking lot and street storage
- multiple use storage (i.e. recreation areas, parks, etc.)
- property line swales
- on site ponds and reservoirs
- open channel storage

6) In general, the designer should follow the criteria suggested for physical design of storage facilities outlined in Chapter 4.0. For large storage ponds, provision must be made for emergency spillage. Modification to these principles should be completely justified.

7) For large storage reservoirs, a routing method such as that outlined in Section 4.3.2 should be used by the designer. Routing the hydrograph is necessary to obtain a correct design for the reservoir and outlet structure.

8) For each desiqn the relationship between water levels in the receiving water body and the drainage system or storage reservoir should be considered.

9) Drainage designs should consider the possible effect of the joint occurrence of high water levels in the receiving streams and a spring rain plus snowmelt.

10) Where the Rational Method is used for small drainage areas, the procedure outlined in Section 3.3.4 should be adopted to assess the C value. Use of constant, "tabulated" C values should be avoided.

11) The designer shall provide details of the methods of computation for runoff and storage and sample calculations should be provided upon submitting the final drainage design for approval. This would include stating any calibration results and coefficients that have been used.

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