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TITLE OF THESIS/*TITRE DE LA THÈSE* The Impact of Urbanization on Peak Flows in the Lower Mainland of British Columbia

UNIVERSITY/*UNIVERSITÉ* Simon Fraser University

DEGREE FOR WHICH THESIS WAS PRESENTED/*GRADE POUR LEQUEL CETTE THÈSE FUT PRÉSENTÉE* Master of Science

YEAR THIS DEGREE CONFERRED/*ANNÉE D'OBTENTION DE CE GRADE* 1981

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DATED/*DATÉ* 7th October 1980 SIGNED/*SIGNÉ*

PERMANENT ADDRESS/*RÉSIDENCE FIXÉ* [redacted]

THE IMPACT OF URBANIZATION ON PEAK FLOWS IN THE LOWER MAINLAND OF BRITISH COLUMBIA

by

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B.A. (Hons.), University of Malaya, 1978

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A THESIS SUBMITTED IN PARTIAL FULFILLMENT

OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

in the Department

of

Geography

SIMON FRASER UNIVERSITY

September 1980

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The Impact of Urbanization on Peak Flows in the Lower Mainland of British Columbia

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ABSTRACT

This study examines the impact of urbanization on peak flows in the Lower Mainland of British Columbia. Due to the short term of records available for the selected watersheds, a comparative approach is used throughout the study.

Four watersheds were selected from the Lower mainland of British Columbia: two from the City of Vancouver and another pair from the Municipality of Surrey. For the purpose of this study, the chosen watersheds differ in terms of the degree of urbanization. Rainfall, sewerflow and streamflow records, maps and aerial photographs were collected from various government agencies.

The analytic tools used for determining the urbanization impact on peak flows were the unit hydrograph, distribution graph and runoff coefficient. The results obtained are then compared with the urbanization index (impervious cover) between watersheds.

Although the principal conclusion is that urbanization does have a considerable impact on peak flows, it should be noted that, this is more meaningful when other things are assumed constant. This is because of the uniqueness of each individual watershed in terms of geomorphologic and morphologic

characteristics. Finally, it is suggested that comparisons of seasonal variations be made on the storm runoff characteristics because the comparison may improve the understanding of the effectiveness of urbanization on peak flows.

To my parents,

sister and brothers.

ACKNOWLEDGEMENTS

This study would not have been possible if not for the support, encouragement and assistance provided by a good number of people, both far and near:

- Low Kwai Sim, Peh Cheng Hock (both from University of Malaya) and Colin Leigh: I am especially grateful to Kwai Sim who gave me the initial interest in hydrological research and in many ways has made it possible for me to pursue this program.

- Mike Roberts as my Senior Supervisor, Ted Hickin and Colin Crampton on my Supervisory Committee: their comments and criticisms during the course of research are deeply appreciated.

- Paul Ham for being the external examiner.

- Hew McConnell from the Sewer Design Division of City Hall, for providing the necessary data.

- Glen McDonnell from Sigma Engineering for giving me access to the data and providing time for periodic discussions.

- Lambert Chu from the Engineering Division of Surrey Municipal Hall for providing the necessary maps and information.

- Jack Nagtegaal from the GVRD for providing the rainfall records.

- Ian Hutchinson and Tuck Wong for their valuable critisms and discussions during the final stages of research.

- Ray Squirrel who advised on some cartographic techniques and Margaret Wheat for photographing the maps.

- fellow graduate students: Wayne Luscombe, who introduced me to the WYLBUR text-editing system and provided editorial assistance; Ken Rood, who has given very useful views and criticisms during the course of writing; Tom Naughten, Larry Smith, Anne Shuttleworth, George Khroda and; many others in the Geography Dept.

- Moyna Gick who did some additional typing.

- and finally, Helen, who has been patient and has given me abundance encouragement during my course of study.

TO ALL OF YOU--THANK YOU.

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CHAPTER ONE

INTRODUCTION

1.1 General

In recent years, population in urban areas has been rising and this has been attributed mainly to increased industrialization. The consequent movement of people from rural to urban areas has cause a phenomenal increase in urban spread (Sarma, et al. 1969). It was estimated, for instance, that by the year 2000, 80% of the population in the United States of America would reside in urban areas (Brater and Sangal 1969). In Canada, the urban population has increased from 50% to 75% of the total population since 1950 (Waller 1977).

The concentration of urban population brings about not only social and economic problems but also physical ones: for example, the task of meeting a heavy demand for water for industrial and domestic purposes, bringing about a concomittant increase in the planning and construction of water supply and drainage facilities (Savini and Kammerer 1961). Additionally, the natural water balance is altered and disposal of wastes may contaminate streams and groundwater.

From the hydrological viewpoint, man's impact is nowhere more intensive than in urban areas (McPherson 1977). Urban development produces inevitable changes in land use and configuration of the land surface with urban features such as buildings, houses, parking lots, sidewalks, sewers and improved natural channels (Roberts 1972). These urban structures increase the impervious cover of an area and, with the installation of sewers or improved natural channels, produce higher flood frequencies and flood peaks.

1.2 Literature Review

Urbanization may be defined as the human habitation and development of previously uninhabited land (Sarma, et al. 1969); its significance on the hydrology of the affected areas has been intensively studied in recent decades. Most investigators have used the time lag, magnitude, and frequency of peak discharges to detect urbanization impacts, though the use of time lag is most common. Savini and Kammerer (1961) were among the first to discuss the hydrological response to urbanization in a general way. This review of the literature will center on peak discharges, runoff volume and time lag.

There are three major approaches undertaken to evaluate the hydrological impact of urbanization: a) "before-after" method; b) synthetic method and; c) "paired watershed" method.

a) "Before-after" method--this method involves the analysis of long-term streamflow records for this reflects the changing trend in discharge and time lag characteristics of a drainage basin undergoing urban development (Fig. 1.1a). The major advantage of this method is that the basin characteristics such as area, mean basin slope and basin shape will remain relatively constant which facilitates detection of any change in the streamflow characteristics as a result of urbanization.

Changes in peak discharges have been frequently defined in terms of the unit hydrograph of the mean annual flood (Espey, et al. 1969). Seaburn (1969) in a study of East Meadow Creek, Nassau County, compared unit hydrographs at different periods (1937 to 1962) during the transition of the drainage basin from the rural to urban conditions. The average peak of the derived 1-hour unit hydrograph increased from 313 cfs from 1937-1943 to 776 cfs in 1960-1962, an increase of about 2.5 times. An analysis of the rainfall-runoff relations for the pre-urban and urban conditions shows direct runoff to have increased from 1.1 to 4.6 times depending on the individual storm.

The first study actually documenting the impact of urbanization using the unit hydrograph was by Van Sickle in 1962. Unit hydrographs were derived by Van Sickle using results from Brays Bayou (Harris County, Texas) for the period July 1939 to July

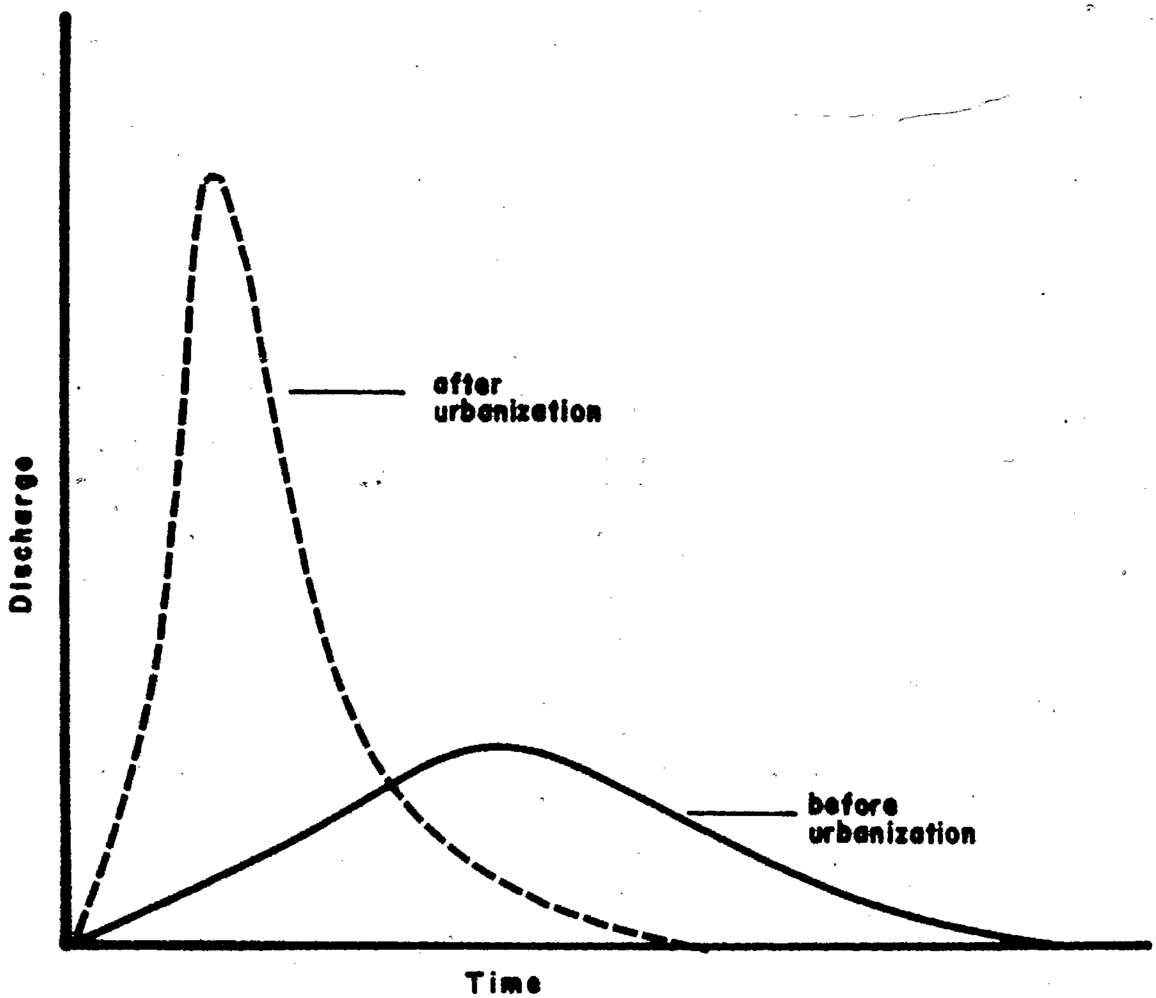


Fig. 1.1a Diagram illustrating the "before-after" urbanization on the hydrograph.

1960 during which time the rural watershed was slowly being transformed into an urban one. The peak of the unit hydrograph increased from about 1500 cfs to 4800 cfs during these 21 years with the time lag being reduced from 12 to 3 hours.

A drainage basin of 11.4 sq. km on the North Shore of Auckland was examined by Williams (1976) to evaluate the urbanization effects on its hydrology. The mean height of the flood hydrographs was found to increase directly with the percentage of urban land from 1963 to 1971. A concomittant decrease in time lag resulted during the same period - from 70.6 to 32.7 hours. Although only a couple of studies have been discussed in detail, a large number of works have used the before and after approach (Anderson 1963; Crippen 1965; Walling and Gregory 1970; Hollis 1974, 1975; Ichikawa 1975). Similar changes ocurred for all before and after urbanization hydrographs (though they may vary in magnitude) where runoff rates were accelerated and peak flow increased. Shorter time lags and larger volumes of direct runoff were simultaneously detected.

(b) Synthetic method--this involves the use of synthetic hydrographs to predict the hydrologic conditions of the watershed prior to urban development. This approach is undertaken for the major reason that data are not available prior to urbanization. Often, this involves the derivation of empirical formulae to provide the values of time and magnitudes of the peak, time base of the unit hydrograph, and basin lag from measures of basin characteristics.

Carter (1961) developed an empirical equation to determine the effect of urbanization on the mean annual flood for suburban catchments near Washington D.C. It was based on time lag, drainage area and percentage of impervious cover. The equation is:

$$\bar{Q} = 223\ KA^{0.85}\ T_l^{-0.45} \quad \text{.............(1.1)}$$

where, $\bar{Q}$ = mean annual flood in cfs A = area in sq. mi. T_l = time lag in hours (centroid of rainfall excess to centroid of runoff) K = an adjustment factor.

Factor K is:

$$K = 0.30 + 0.0045I/0.03 \quad \text{.............(1.2)}$$

where, I = impervious cover in per cent.

Dempster (1974) in a similar approach, derived a regional flood frequency equation for Dallas relating the flood peak for an assumed return period with storm and physical watershed characteristics using a stepwise multiple regression technique. The most important watershed variable was area followed by impervious cover, and finally, the length-slope parameter. The equation is:

$$Q_T = aA^b K^c (L/\sqrt{S})^d \quad \text{..................} (1.3)$$

where, Q = discharge in cfs; A = area in sq. mi.; L = length of longest collector in mi.; S = slope in ft./mi.; K = impervious factor.

Factor K is:

$$K = 1 + 0.015I \quad \text{......................} (1.4)$$

where, I = impervious cover in per cent.

A somewhat similar approach was undertaken by Espey, et al., (1966) to derive a relationship using data from 22 urban and 11 rural watersheds. The equations were derived to model the peak discharge characteristics under future urban conditions. The equations are:

a) Under urban conditions:

$$Tr = 20.8\phi\ L^{0.29}\ S^{-0.11}\ I^{-0.61} \quad \text{.............(1.5)}$$

$$Q = 1.93 \times 10^{4}\ A^{0.91}\ Tr^{-0.94} \quad \text{..............(1.6)}$$

b) Under rural conditions:

$$Tr = 2.65L^{0.12}\ S^{-0.52} \quad \text{......................(1.7)}$$

$$Q = 1.70 \times 10^{3}\ A^{0.88}\ Tr^{0.30} \quad \text{..............(1.8)}$$

where, Tr = time to peak from beginning of runoff in mins.; Q = peak discharge in cfs; A = area in sq. km; L = length of main channel in ft.; S = slope of main channel in ft./ft.; I = impervious cover in per cent; ϕ = channel roughness factor.

In a comparison of Carter's equation to the 38th and 23rd Street watersheds of Waller Creek (Houston, Texas) used by Espey, et al. (1966), good agreement was obtained between the measured and predicted mean annual flood for 'present' conditions. The application of Espey's rural equations to Common's (1942) and Mockus's (1955) hydrographs from Waller Creek watershed were similarly in good agreement. Again, comparisons of the urban equation to Beargrass Creek (Louisville, Kentucky) by Espey, et al. (1966) showed similar results. However, contrasting results were obtained with

Dempster's equation because of differences in pedology and channel roughness as noted by Schulz and Lopez (1974). Hence, regional variability has to be considered rather carefully when applying empirically derived equations as appreciable errors may be involved in using them.

(c) Comparative watershed method--this method (also known as "paired watershed" method) involves the comparison of different watersheds with similar basin characteristics (Fig. 1.1b). Many studies have been done to evaluate the hydrological differences by comparing urban with rural watersheds. Stall and Smith (1961) compared 2 unit hydrographs derived from watersheds in Champaign, Illinois: one completely rural while the other, urbanized with 38.1% impervious cover. Though similar in area, mean basin slope and stream channel shapes, the unit hydrograph peak discharge from the urban watershed was approximately 4 times that for the rural watershed. In Tokyo, Ikuse, et al. (1975) reported a difference in peak discharge of 5 times for a developed Nagayama watershed compared to its counterpart, Bessho watershed which is a natural one.

Waananen (1969) compared three small basins over time to determine the hydrologic changes occurring from rural to suburban land use development. These basins are located near the foothills west of San Francisco Bay near Palo Alto

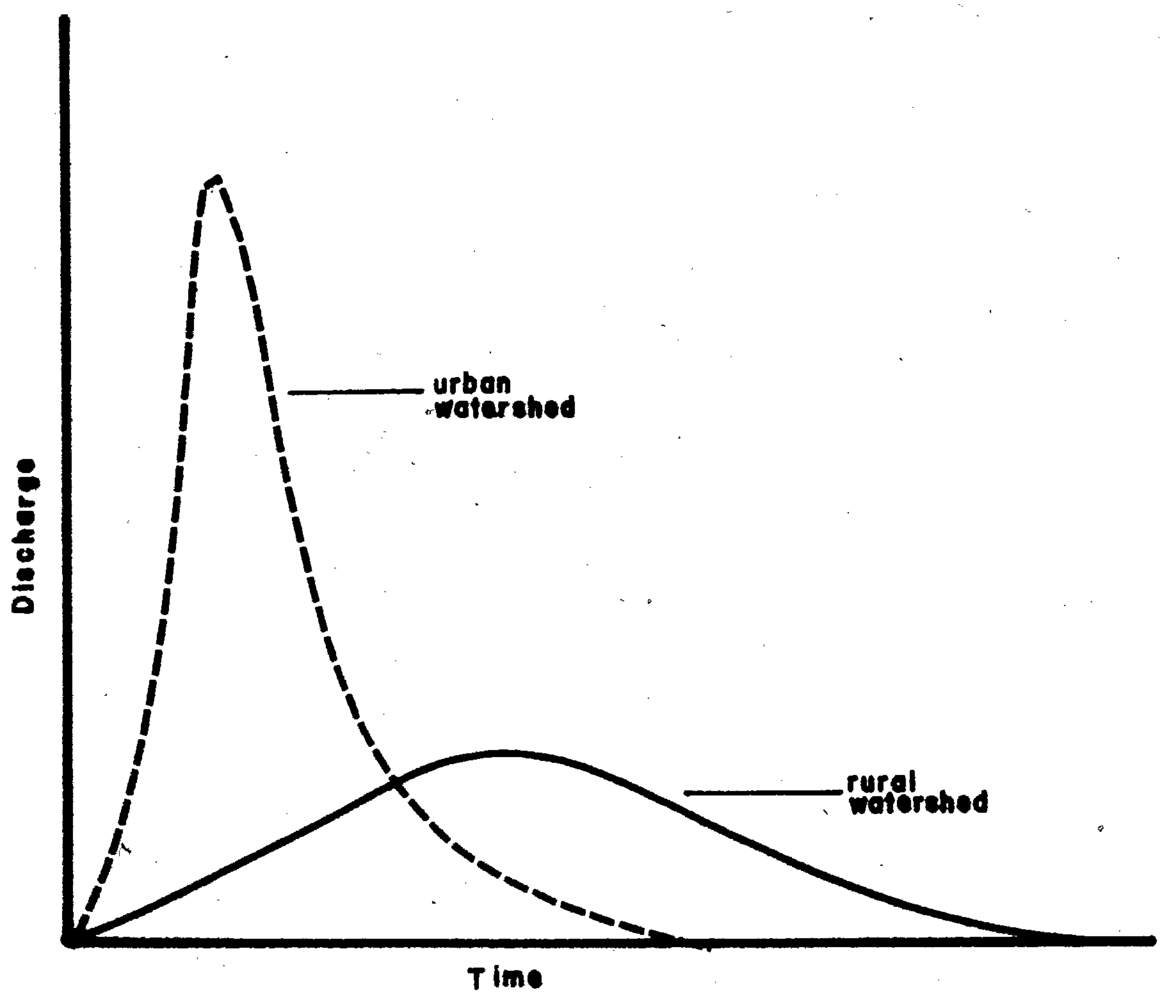


Fig. 1.1b Diagram illustrating the comparison method.

California. Known as the San Francisquito project, Sharon Creek and a San Francisquito Creek tributary were the suburban ones while Los Trancos acted as a rural control. The results showed that considerable hydrologic changes could be attributed to urban development. Waananen concluded that there was an increase in annual discharge; a modification of low flow of streams resulting from the importation of wastewater; and a decrease in groundwater recharge. The increase in direct runoff due to urban development is shown in Fig. 1.2 which illustrates the precipitation-discharge relations from 1959-1965.

From the three approaches discussed above, it appears that the effect of urbanization on the hydrologic conditions has been to a very large extent carried out on a macro level. This is to say that contrasting land uses were examined and evaluated. Most of these studies had used the degree of imperviousness caused by roofs, sidewalks, parking lots and other features as the urbanization index. Indeed, such features significantly increase the imperviousness as a region urbanizes. Some investigators however, have suggested other criteria such as population density (Brater 1968; Hammer 1973; Dunne and Leopold 1978), the degree of industrialisation or drainage density and the number of dwellings per unit area (Brater and Sangal 1969).

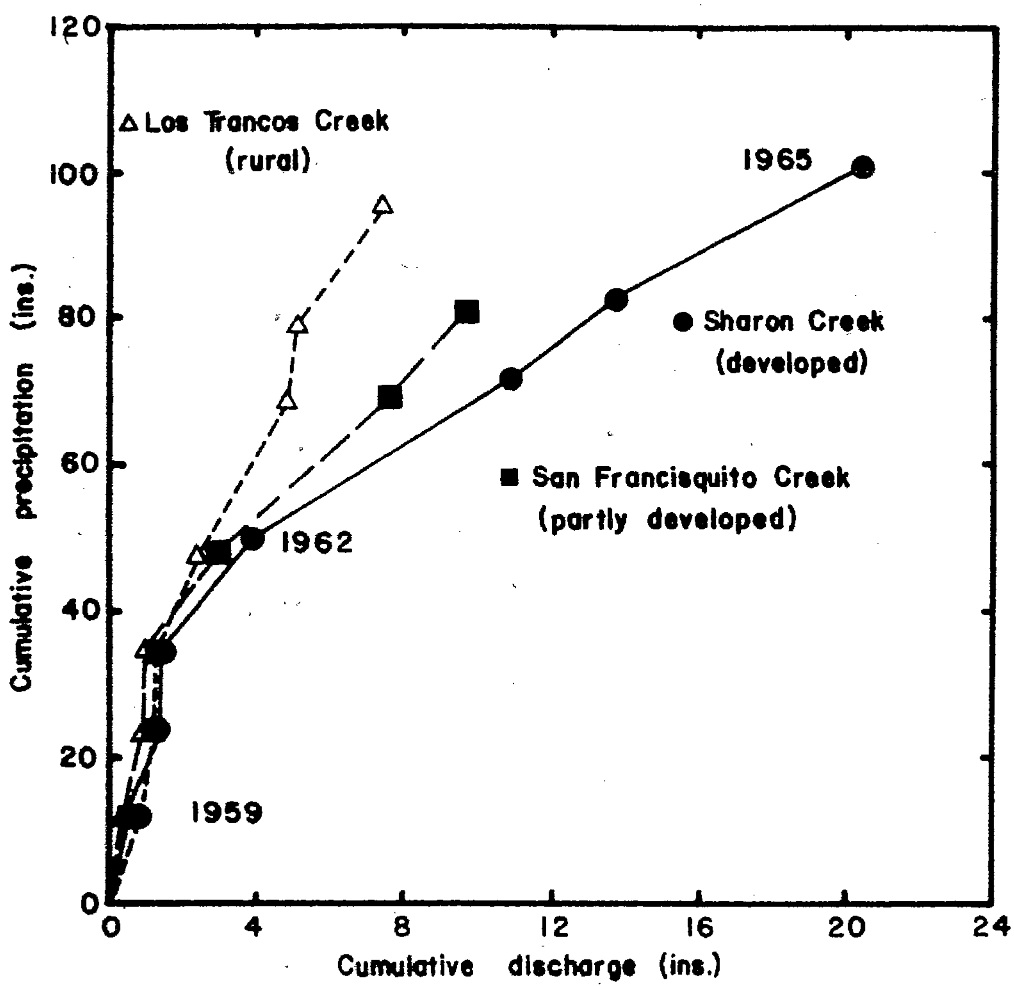


Fig. 1.2 Precipitation - discharge relations in San Francisquito Project, 1959-1965 (after Waananen 1969).

The urbanization index selected in this thesis is the percentage of impervious cover; there are several advantages in using this criteria. Besides being the principal feature which is increased with urban development, it provides a common basis for measurement and comparison. It has advantages over the other criteria mentioned in the above paragraph. For example, because of the different types of premises (such as residential and commercial) found within and between watersheds, the choice of the number of dwellings would not provide a reasonably consistent urbanization index. The choice of drainage density as an urbanization factor would also be unsatisfactory. This is so since different combinations of drainage facilities are used: the City of Vancouver watersheds are served by a combined severage system while a combination of storm sewers, open ditches and watercourses is used in the Surrey watersheds.

On a micro level, however, (meaning the use of urban watersheds only) most of the studies are directed towards storm sewer design. Such studies (Keifer and Chu 1957; Eagleson 1962; Terstriep and Stall 1969) center around the computation of runoff estimates and design hydrographs as this parameter forms the basis of storm drainage design. However, others (Viessman 1966, 1968; Brater 1968; Miller and Viessman 1972) did analyse the hydrologic conditions empirically. Though there is substantial evidence that the hydrologic conditions are altered in various stages of urbanization, there are still very few studies done to evaluate the role of urbanization among urban watersheds. A recent modelling study by Beard and Chang (1979) in the Tulsa District, Texas, indicated that increases in imperviousness, particularly buildings and pavements, did not seem to have affected the rapidity of runoff. Instead, they concluded that the channel and storm sewer improvements were the main cause.

If this literature review is to be complete then the the increasing importance of deterministic models for the estimation of runoff from urban basins must also be mentioned. Perhaps the most popular models are the Stormwater Management Model (SWMM) (U.S.A.), and the Road Research Laboratory Hydrograph Model (RRL) (U.K.). These deterministic models attempt to mathematically conceptualize the hydrologic processes that occur in a watershed and utilize the rainfall data as a way of generating streamflows.

Two types of models can be recognized in terms of their use; i) generalized models and; ii) specific models. The SWMM and RRL models are generalized ones in the sense that they can be applied by changing only the parameters. On the other hand, the specific models (e.g. Chicago Hydrograph and Haan-Johnson

Model) are intended for application to either a single basin or a specific type of land or land use. To quote Larson (1972), ..."If watershed models can be made to represent the actual hydrologic processes, they can be applied under a variety of conditions. Thus, hydrologic modelling is likely to become the principal means of utilizing or extending our limited data to predict the effects of land use changes on water resources." p. 113. The description and use of these models have been described by Larson (1972), Roesner, et al. (1972), and Aitken (1973).

1.3 Local studies

In the Lower Mainland of British Columbia, sewerflow/streamflow and rainfall records are quite readily available in some of the urban drainage basins. There is a fair amount of literature but the reports done for the past number of years were mainly concerned with urban drainage design problems and developments. Some reports on the City of Vancouver had been comprehensively done by the Greater Vancouver Sewerage and Drainage District (Mechler 1977).

In the Municipality of Surrey, the Engineering Division works closely with Sigma Consultants Ltd. in a Master Drainage Program. Most of the reports were for storm sewer design purposes. There are, however, attempts in minimising the impact of urbanization on peak flows. The most common methods adopted are the increased use of natural watercourses and detention storage basins.

Other governmental agencies, such as the Water Survey of Canada; and the B.C. Ministry of Environment do gauge several watersheds in Surrey. However, most of the data are either kept as records or used for water quality analyses. Perhaps the only local studies attempting to evaluate the role of urbanization on the hydrological regime known by the writer is that by Dr. M.C. Roberts of the Department of Geography, Simon Fraser University.

1.4 Objectives of Study

The general objective of this study is to analyze the impact of urbanization on peak flows using a comparative watershed approach (Fig. 1.1b). To the best knowledge of the author, this kind of study has not been done for any basins in the Lower Mainland of British Columbia.

The comparative approach is used because the length of records for local streams does not permit the classical analysis of using the "before-after" urbanization method (Fig. 1.1a). This type of analysis is based on a long record of runoff that permits the investigator to compare hydrographs when the watershed was rural with hydrographs when the watershed was urbanized. However, this approach demands a lengthy temporal sequence of runoff data which is not available in the local area. Therefore, the study will use short records of runoff but from watersheds of contrasting degrees of urbanization. In a sense, this is the substitution of space for time.

The specific objectives are:

a) To examine the unit hydrograph characteristics of watersheds and, therefore, to evaluate the role of urbanization on peak flows.

b) To examine the relationship of runoff to rainfall in the context of the urbanized watersheds.

The characteristics of the study areas are described in Chapter 2 while Chapter 3 discusses the methodology of this study. Chapter 4 presents the results of hydrograph and analyses while a discussion of the results is dealt with in Chapter 5. In Chapter 6, the findings are summarised and suggestions for further research are proposed.

CHAPTER TWO

CHARACTERISTICS OF THE WATERSHEDS

2.1 Location

The watersheds selected for this study lie within the Greater Vancouver Regional District (GVRD) and are situated in the City of Vancouver and the fast-growing Municipality of Surrey (Fig. 2.1):

a) City of Vancouver

12th & MacKenzie

10th & Alberta

b) Municipality of Surrey

Robson Creek

Hyland Creek

A detailed description of each watershed will be discussed later in this chapter, but first there is a summary of the climate, geology and soils for the general region in which the watersheds are located.

2.2 Climate

The Lower Mainland of British Columbia enjoys a mild maritime climate. It is dominated by the onshore flow of Pacific air

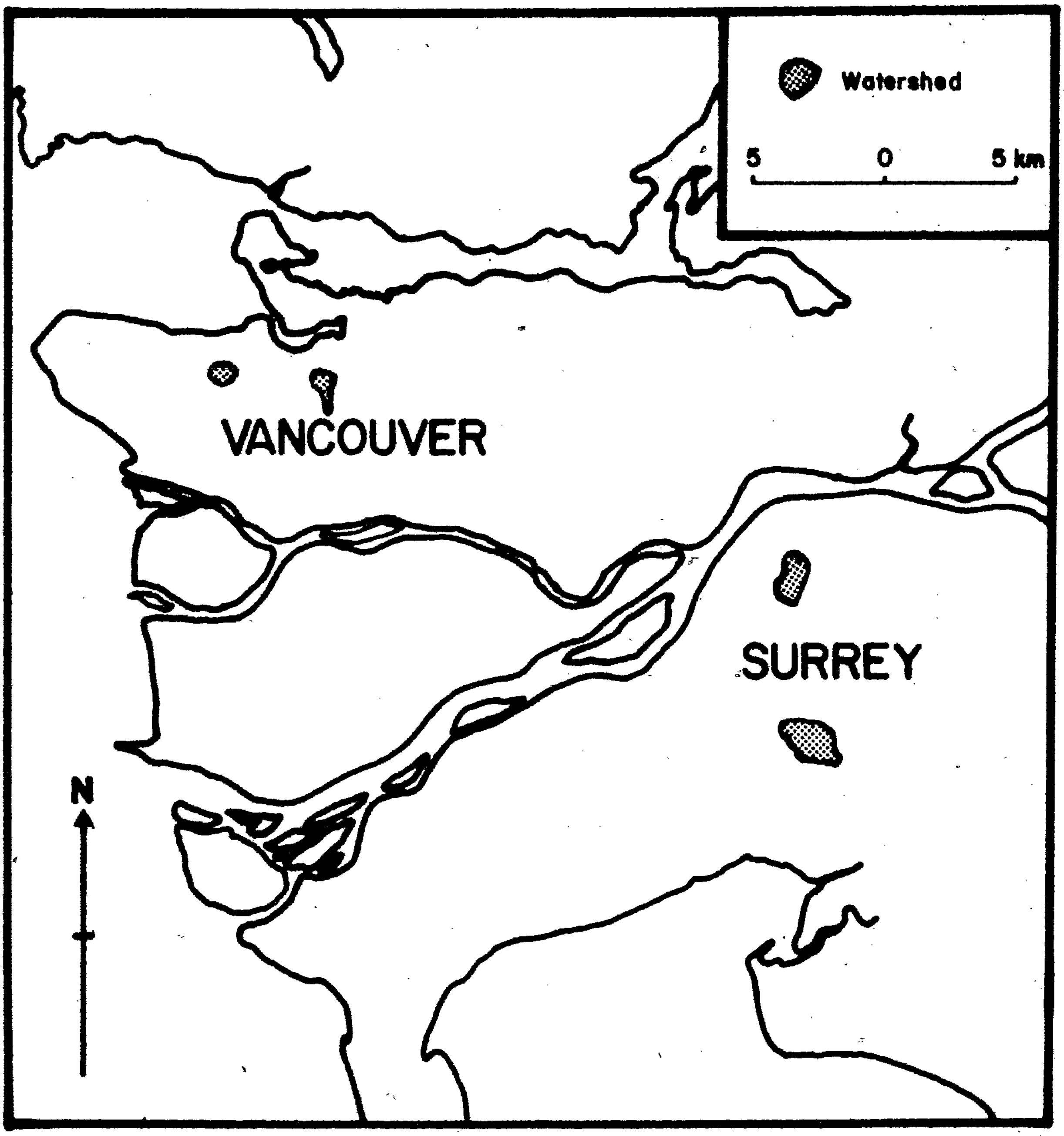


Fig. 2.1 Location of watersheds in the study.

which produces warm temperate rainy winters and cool summers (Hare and Thomas, 1974). The presence of the Coast Mountains has an influential effect: the windward slopes experience very high precipitation levels because of the orographic effect.

Weather stations close to the watersheds were selected to show the representative mean temperature (Table 2.1) and mean precipitation (Table 2.2) for the period 1941-1970. The mean annual temperature for the Vancouver Dunbar South weather station is 10.5°C (51°F) while that for Surrey Newton it is 9.4 °C (49°F). A very slight difference is also found during the seasons: 2.8°C (37°F) in January and 18.3°C (65°F) in July for Vancouver Dunbar South while for Surrey Newton, the temperature is 2.2°C (36°F) in January and 16.7°C (62°F) in July. Infiltration capacity therefore is low during the winters because subsurface flow is constantly being replenished by precipitation.

Precipitation varies seasonally in the Lower Mainland with a maximum during winter and a minimum in summer, while the fall has more than spring. The mean precipitation amount for Vancouver City Hall is 160.0 mm (6.3 ins.) in January and 32.3 mm (1.27 ins.) in July (Table 2.2). Due to the relatively larger amount of precipitation during the winter months, the infiltration capacity is very low and may practically reach

Station	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
Surrey Newton	36	40	42	47	53	58	62	62	58	50	42	38	49
	2	4	6	8	12	14	17	17	14	10	6	3	9*
Vancouver Dunbar South	37	42	44	50	56	61	65	64	60	52	44	40	51
	3	6	7	10	13	16	18	18	16	11	7	4	11*

Table 2.1 Mean Temperature (in ^{o}F) (based on 1941-1970 Average)
* - in ^{o}C

Source: Br. Columbia Dept. of Agriculture, Report for 1976 p. 15

Station	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
Surrey Newton	7.70	5.86	4.61	3.44	2.45	2.11	1.27	2.07	3.19	6.74	7.45	7.84	54.73
	195.6	148.8	117.1	87.4	62.2	53.6	32.3	52.6	81.0	171.2	189.2	199.1	1390.4*
Vancouver City Hall	6.30	4.91	4.29	2.97	1.99	2.03	1.24	1.47	2.83	5.47	5.93	7.07	46.50
	160.0	124.7	108.9	75.4	50.5	51.6	31.5	37.3	71.9	138.9	150.6	179.6	1180.9 *

Table 2.2 Mean Precipitation (in ins.) (based on 1941-1970 Average)

* - in mm

Source: Br. Columbia Dept. of Agriculture, Report for 1976, p. 66

zero after a lengthy period of rain. Also, with lower temperature levels, the rate of evapotranspiration (EVTR) is reduced. These variations result in a higher runoff volume for a given amount of rainfall in comparison to the summer months.

2.3 Geology and soils

The B.C. Lower Mainland is an area primarily of Quaternary deposition with relatively low relief stretching 150 km east to Hope. Much work of the Pleistocene geology of the area has been done by John E. Armstrong (1965, 1968, 1977) of the Geological Survey of Canada. This region has been subjected to repeated glaciations during the Wisconsin Ice Age and possibly earlier. At least 3 main stages during the Wisconsin are suggested (Armstrong 1977); an advance stage which is characterised by coalescing piedmont glaciers; a maximum stage when the ice reached a thickness of 1800 m or more which overrode much of the adjacent mountainous areas; and finally a retreat or deglaciation stage.

During each major glaciation, there were isostatic, eustatic and tectonic processes involving changes in sea-levels of up to 200 m or more. This accounts for the thick Quaternary sequence in the Lower Fraser valley (300 m or more) that is a mixture of glacial, non-marine and marine deposits. The surficial geology of the Lower Mainland is subdivided into 19 groups but only 4 major groups are found in the watersheds (Table 2.3).

Generally, four main groups (Fig. 2.2) are found in the study areas: the Newton Stony Clay, Surrey Till, Haney Outwash and Sunnyside Sand. The Newton Stony Clay is glacio-marine by origin. It is composed of stoney and clayey silt, poorly sorted till-like mixtures, and minor clayey silt, silty clay and sand. The thickness of this layer may be up to 200 ft. thick. The Surrey Till (glacial deposits) is composed of sandy to silty till. The Haney Outwash is glacio-fluvial by origin while Sunnyside Sand is composed of raised littoral or beach deposits of medium to coarse sands. Thickness may be up to 25 ft. (Table 2.3).

Although the detailed surficial geology and soil textures vary from one watershed to another, their distribution within each watershed could allow a general assumption that they are reasonably similar.

2.4 The watersheds

The watersheds were chosen mainly because of the availability of data. Geographically, the four watersheds selected are relatively close to one another, being within an approximate

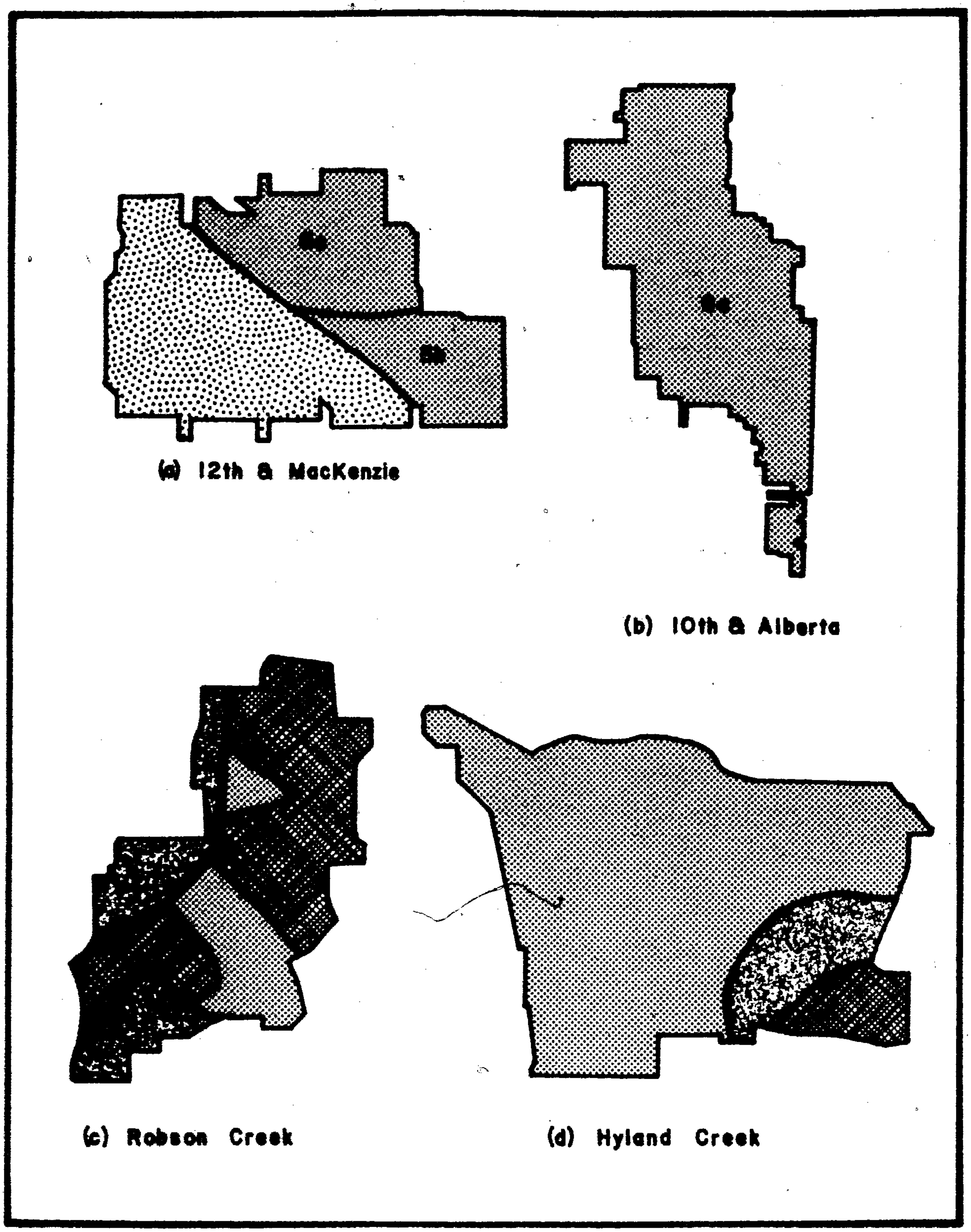


Fig. 2.2 Surficial geology.

Legend

4a	-SURREY TILL	(glacial deposits)
5 (5a, 5b)	-NEWTON STONY CLAY	(glacial - marine deposits)
6	-HANEY OUTWASH	(glacio - fluvial deposits)
10b	-SUNNYSIDE SAND	(raised littoral and beach deposits)

Surficial Geology Group	Description
4a - Surrey Till (glacial deposits	Sandy to silty till and minor sub-stratified drift up to 75 ft. thick but generally less than 25 ft. thick. Throughout much of the area Surrey Till Deposits are mantled by Bose Gravel.
5 - Newton Stony Clay (glacial-marine deposits)	5a - sandy to silty and minor sub-stratified drift up to 60 ft. thick but generally less than 20 ft., overlain in most places by glacio-marine stony clayey silt, and minor interbedded marine clayey silt, silty clay, and sand, up to 25 ft. thick but generally less than 10 ft.; 5b - sandy to silty and minor sub-stratified drift generally less than 25 ft. thick.
6 - Haney Outwash (glacio-fluvio deposits)	Clayey silt, silty clay, and stoney, clayey silt, up to 25 ft. or more.
10b - Sunnyside Sand (raised littoral and beach deposits)	Medium to coarse sand up to 25 ft. thick; probably includes floodplain and channel sand, and littoral and beach sand.

Table 2.3 Surficial Geology Groups

Source: **Armstrong** and Brown (1957), Surficial Geology, **New Westminster** Area radius of 14.3 km of one another. It is permissible for us to assume that they are climatically similar (Section 2.2). the underlying materials of the basins are Quaternary deposits composed of alluvial, marine and glacial materials. It was hoped that the geology of the basins would be relatively similar as this will remove some of the variance in explaining hydrologic response. Finally, because of the comparative objective of this study, the urban land use intensity of the watersheds varies from a high 50.4% impervious cover to a low 11.6% (Table 2.4). The watersheds in Vancouver are entirely sewered while those in Surrey are drained by a combination of storm sewers, open ditches and watercourses or creeks.

2.4.1 12th & MacKenzie

It is located at about 10 blocks west of Granville Street in the City of Vancouver. It covers a total area of 0.26 sq. km (Fig. 2.3).

Approximately rectangular in shape (circulatory ratio, Rc - 0.78) (Table 2.5), the mean basin slope is approximately C.02. It should be mentioned however that the distribution of slopes within the watershed is not even. A field survey of the areas reveals that some parts of the basin are gently rolling while the north western part has quite moderate slopes. It is

Watershed	Landuse				
	Open Spaces (%)	Build- ings (%)	Pavements Roads (%)	Total Area (%)	Impervious Cover (%)
12th & MacKenzie	54.6	20.2	25.2	100	45.4
10th & Alberta	49.5	21.5	29.0	100	50.5
Robson Creek	71.5	12.1	16.4	100	28.5
Hyland Creek	88.4	4.2	7.4	100	11.6

Table 2.4 Land use of watersheds in study

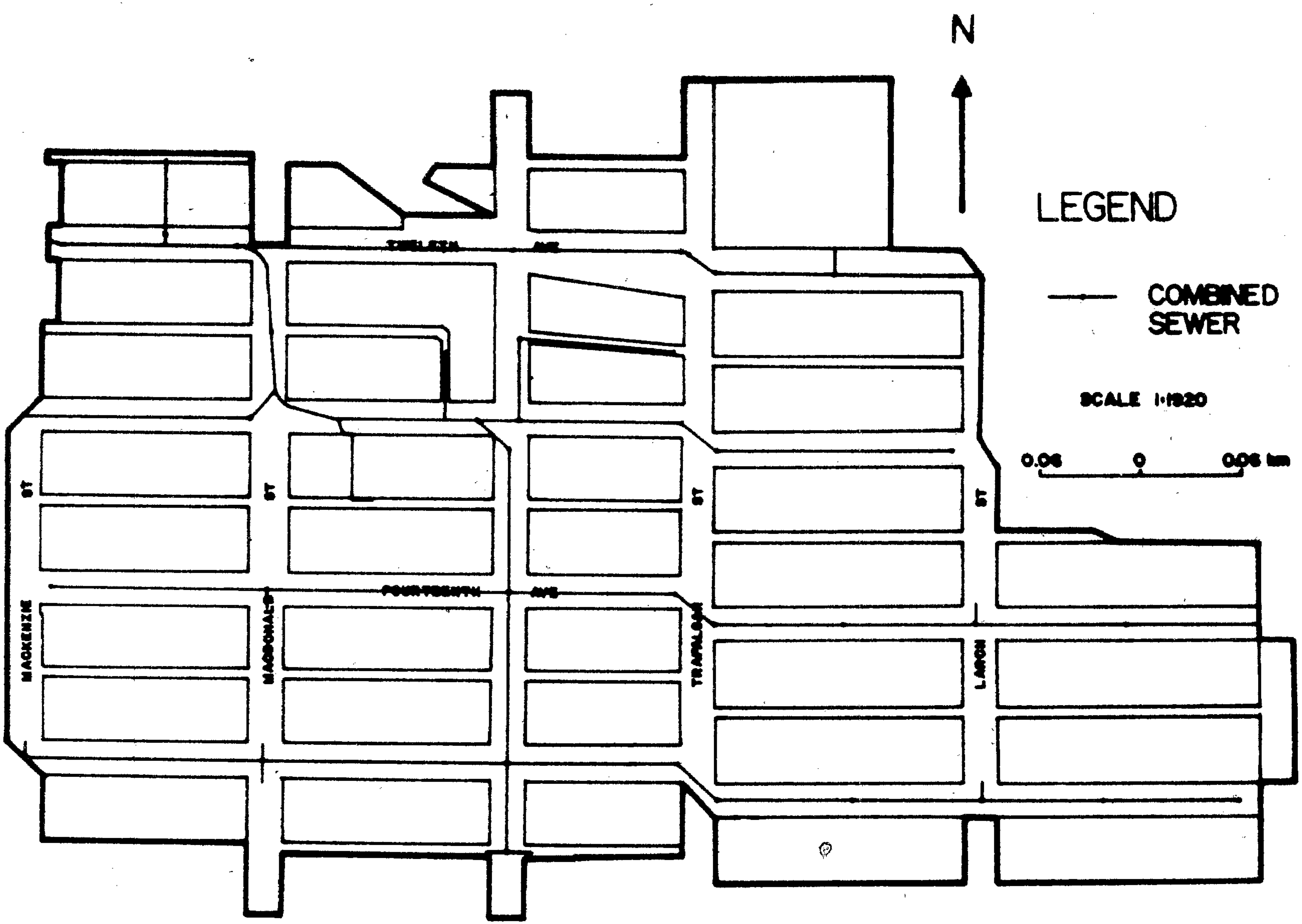


Fig. 2.3 12th & MacKenzie watershed.

Watersheds	Area (sq. km)	Mean Basin Slope	Maximum Relief (m)	Drainage Perimeter (km)	Circulatory Ratio
12th & MacKenzie	.26	0.02	11	2.06	0.78
10th & Alberta	.74	0.03	37	5.24	0.34
Robson Creek	2.03	0.02	23	7.92	0.41
Hyland Creek	4.71	0.02	61	10.7	0.52

Table 2.5 Topographic characteristics of watersheds in study

completely drained by a combined severage system which runs along practically every block (Fig. 2.3). The total length of the sewers measured on the map is approximately 3.21 km with a drainage density of 12.35 km/sq. km. Measurements of other characteristics such as the length of the longest channel, main channel and slope of the main channel are listed in Table 2.6.

The surficial geology is made up of a mixture of marine, glacio-marine and glacial deposits. Cloverdale sediments, Newton Clay and Surrey Till are the major Pleistocene units found (Fig. 2.2). Texturally, they vary from clayey silt to sandy till (Table 2.3).

The entire area of the watershed is urban (Figs. 2.4a and 2.4b). It consists of houses, lawns, pavements, roads and a few vacant areas; the area is entirely residential. Most of the houses are single-storey built mainly on 33' lots; each with a lawn. The computation of the impervious cover (Appendix A) indicates that it makes up 45.5% of the total area (Table 2.4).

2.4.2 10th & Alberta

The watershed is located 1.5 km east of 12th & MacKenzie and 22 blocks east of Granville Street. The total area is 0.74 sq.km (Fig. 2.5).

Watershed	Drainage Density (km/km^2)				Length of Main Channel (km)	Length of Longest Channel (km)	Length of Centre of Watershed (km)	Mean Slope of Main Channel
	Total Length of Sewers (km)	Total Length of Open Ditches (km)	Total Length of Water-cover Creeks (km)	Total Density (km/km^2)				
12th & MacKenzie	3.21	-	-	12.4	0.85	0.96	0.48	0.01
10th & Alberta	11.63	-	-	15.7	2.38	2.38	1.19	0.02
Robson Creek	8.95	8.16	0.30	8.6	0.64	1.08	0.32	0.01
Hyland Creek	10.0	11.4	2.8	5.1	1.31	1.91	0.66	0.01

Table 2.6 Drainage characteristics of watersheds in study

Fig. 2.4a 12th & MacKenzie: closely built residential units.

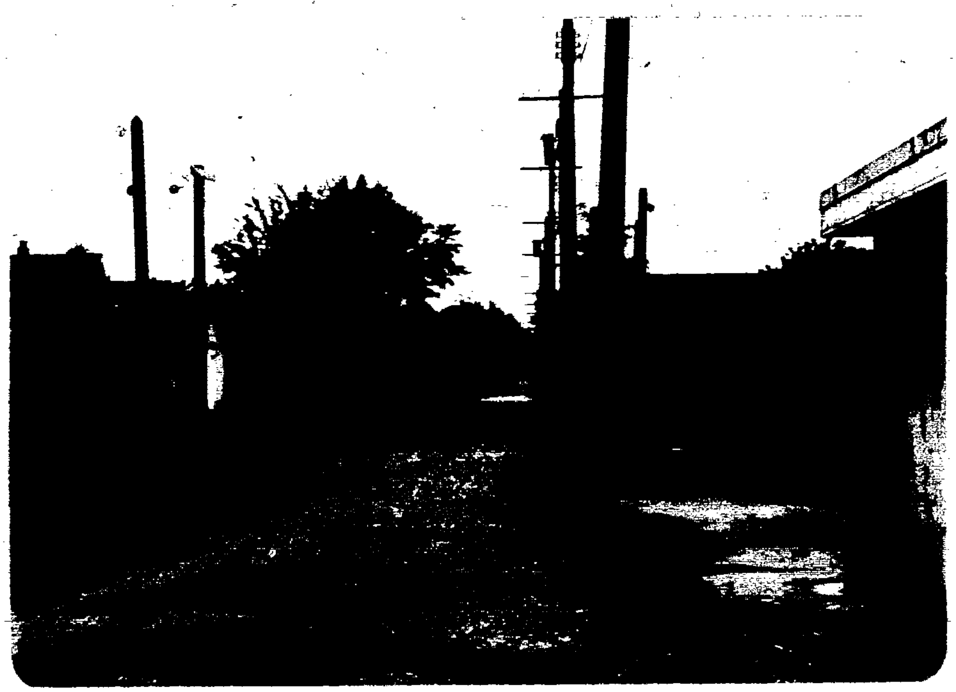


Fig. 2.4b 12th & MacKenzie: graveled back roads.

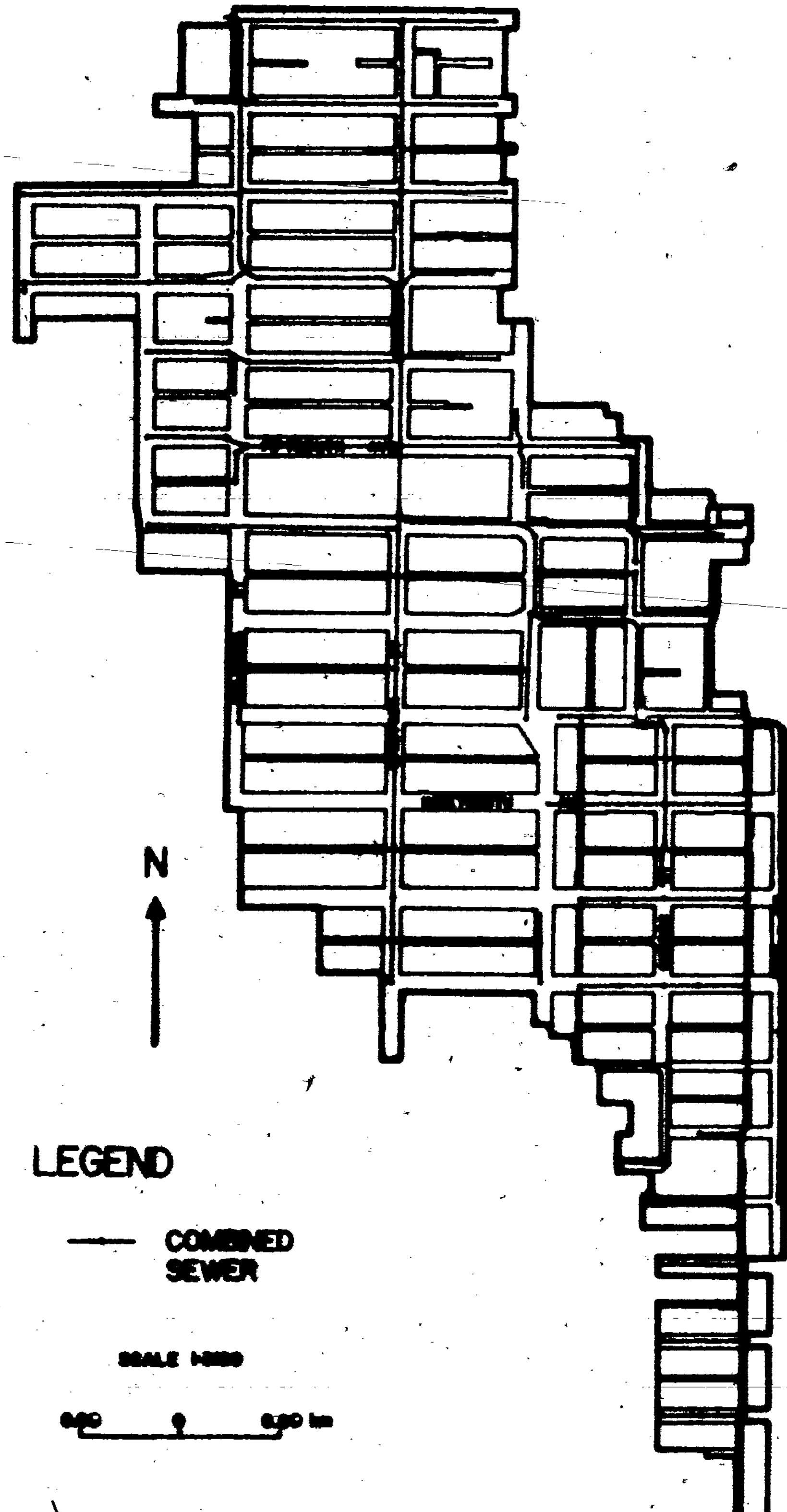


Fig. 2.5 10th & Alberta watershed.

Unlike the other basins, 10th & Alberta is rather unusually shaped; it is an elongated basin which tapers to the south. The circulatory ratio (Rc) is measured at 0.34. The mean basin slope is 0.03, though it does not slope evenly throughout the watershed. The area is entirely drained by a combined sewerage system which also runs along practically every block (Fig. 2.5). Due to its shape, the number of sewers constituting the drainage network are far greater than 12th & MacKenzie, hence explaining a higher drainage density of 15.72 km/sq. km. The total length of sewers draining this area is 11.63 km. Table 2.6 shows the measurements for other drainage characterisics.

The surficial geology of this basin is made up of glacio-marine and glacial deposits (Fig. 2.2). Consisting mainly of the 5b Newton Stony Clay group, the sandy to silty till materials are similar though the deposits are generally less than 25 ft. thick (Table 2.3).

The watershed is entirely urban. Unlike 12th & MacKenzie, this basin has commercial land uses as well as residential, especially in those blocks flanking Main Street along the eastern boundary (Fig. 2.6a). Residences occupy a very large portion of the watershed area. In addition, there is a school (Simon Fraser School) and a community park (Mt. Pleasant Community Park) along 16th Ave. and a number of parking lots

Fig. 2.6a 10th & Alberta: commercial premises along Main Street.

Fig. 2.6b 10th & Alberta: parking lots at 26th Ave.

along Main Street (Fig. 2.6b). There is a mixture of single-storey and double-storey houses which are more closely built, especially along Quebec St. The commercial block renders up to about 100% imperviousness. This is evident as the impervious cover is 50.5%, much of which results from the commercial buildings and associated parking lots.

2.4.3 Robson Creek

Robson Creek is located in the Municipality of Surrey near (0.5 km east) the King George Highway on 104 Ave. (Fig. 2.1). It is a subwatershed of a bigger Whalley drainage basin (Fig. 2.7). Robson Creek has an area of 2.03 sq. km.

The watershed approximates an elongated fan in shape. The mean basin slope is 0.02 with certain parts of the basin sloping more steeply than the rest, for example, along 129 St. The maximum relief is approximately 37 m: see Table 2.5 for quantitative geomorphologic details. The entire area is drained by a combination of storm sewers, open ditches and watercourses. It is quite likely that some of the rainfall excess would not reach the main channel because of pondage (as a field inspection of the open ditches revealed). Even if there is any flow, the abundance of vegetation in most ditches would impede a smooth passage for the flow, thus reducing the

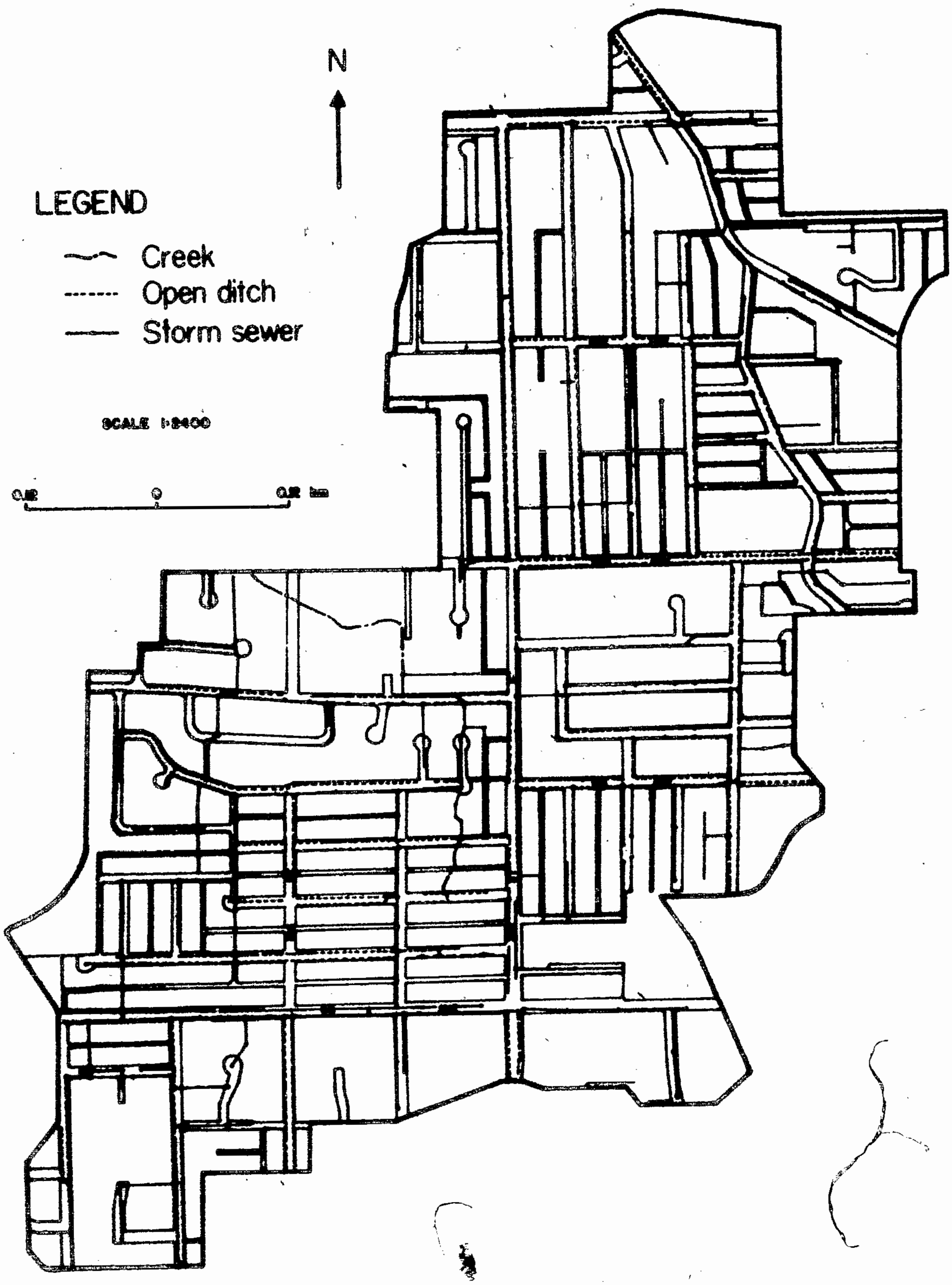


Fig. 2.7 Robson Creek watershed.

rate of flow. Consequently, the peak volume of runoff reaching the main outlet will be greatly reduced; this will be discussed in Chapter Five.

The surficial geology for the watershed ranges from marine, glacio-marine, glacio-fluvial to alluvial deposits (Fig. 2.2). The marine and alluvial deposits are sandy, belonging to the Salish and Capilano groups. The reason for the presence of alluvial materials is probably due to its nearness to the Fraser River where the materials may have been deposited during the post-glacial period. Texturally, the glacio-fluvial materials range from less important medium to coarse gravels of the Maryhill Outwash deposits, to more important silty materials.

The urban landscape consists mainly of residences and commercial activities located in clusters, e.g. along 96 Ave. and 128 St. Unlike the City of Vancouver watersheds, the residences have wide lawns and are on larger lots - a clear suburban characteristic. There are vacant or open spaces situated sporadically over the area. The impervious cover for Robson Creek is 28.5% (Table 2.4). A larger portion of the area, therefore, is divided among lawns and open spaces (Figs. 2.8a and 2.8b) than in the city.

Fig. 2.8a Robson Creek: runoff storage in open ditches.

Fig. 2.8b Robson Creek: moderately low housing density.

2.4.4 Hyland Creek

Hyland Creek is located at approximately 2 km south of Robson Creek (Fig. 2.1). The King George Highway runs through the watershed from 73 Ave. and 66A Ave. It is a sub-watershed of the Newton drainage basin and is the largest watershed in the study covering an area of 4.71 sq. km (Fig. 2.9).

Approximately rectangular in shape, with an irregular drainage divide, the mean slope is about 0.02. The slope is generally gentler in the northwest section, steepening as one approaches the outlet in the southeast section of the watershed. The maximum relief is approximately 61 m (200 ft.).

Like Robson Creek, the drainage system is made up of storm sewers, open ditches and watercourses or creeks. The main channel, Upper Hyland Creek, is about 1.67 km long and is fed by a few smaller creeks as it drains from northwest to southeast (Fig. 2.9).

A very large portion of the watershed is made up of Newton Stony Clay and Surrey Till (glacio-marine and glacial deposits). Maryhill Outwash deposits (glacio-fluvial) occur locally in the southwest, over even less extensive deposits of the Salish and Capilano groups (marine and alluvial deposits) (Fig. 2.2 and Table 2.3).

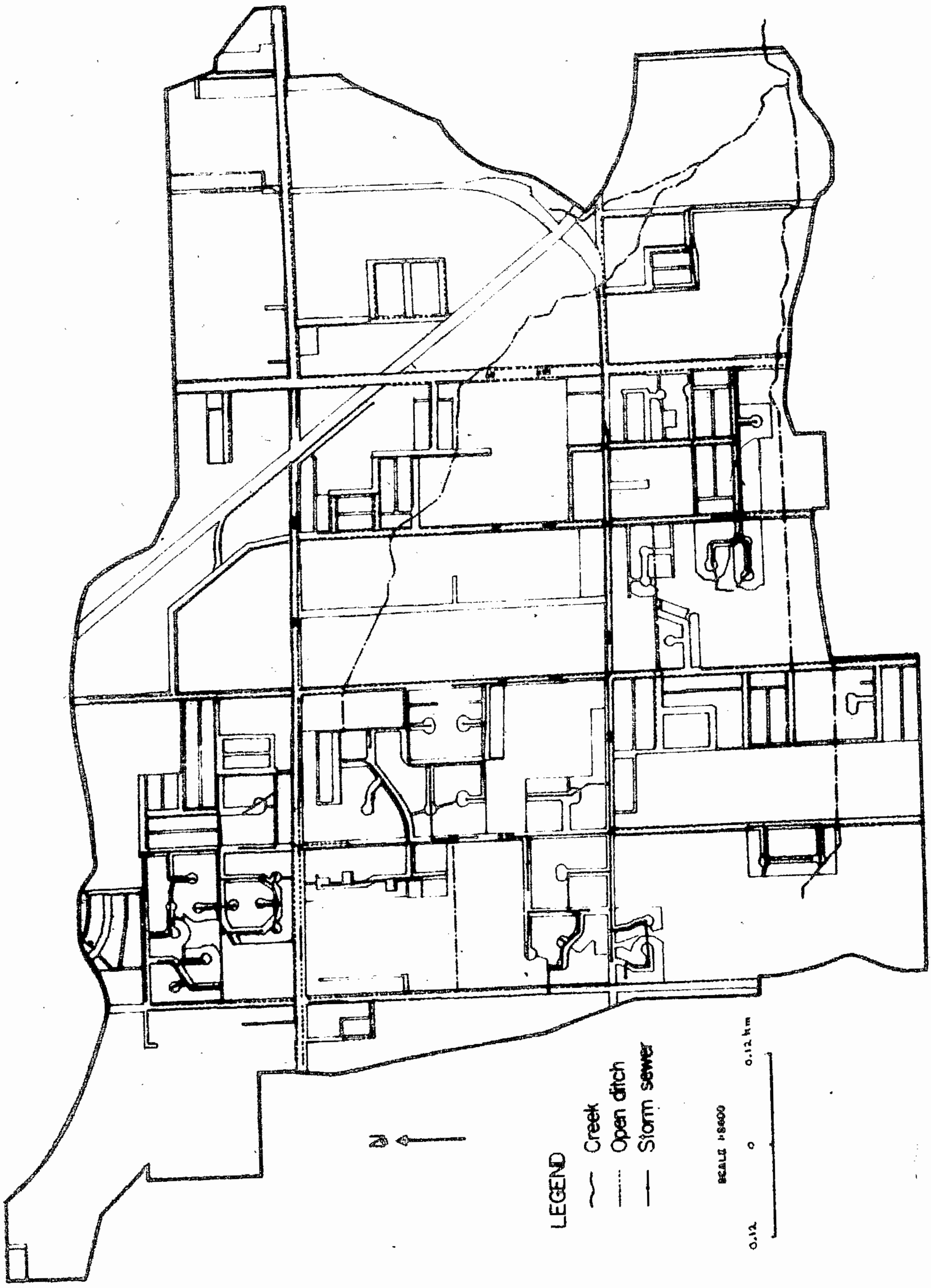


Fig. 2.9 Hyland Creek watershed.

Hyland Creek is the least developed or urbanized of the four watersheds in the study. Commercial buildings are found along both sides of the King George Highway (Fig. 2.10a). There are also light industrial premises concentrated along both sides of 72 Ave. up to 130 St. Residences occupy a larger portion of the area, with a mixture of small and wide open lawns. There are more open and vacant spaces in the western part of the watershed and the impervious cover is only 11.6% (Fig. 2.10b). This area however, is undergoing rapid development which will ensue drastic changes to the landscape in a few years time.

2.5 Summary

The four watersheds in the study are urbanized but with differing degrees of land use intensity. This is reflected by the chosen urbanization measurement in the study, i.e. the percentage of impervious cover. The basin physiography is dissimilar with areas ranging from 0.26 to 4.71 sq. km. Shape differs from an almost rectangular-type, 'elongated' fan-type to an elongated one. Also, the nature of the drainage systems found in the 2 municipalities is not the same. Consequently, all these factors will have to be taken into account when comparing the results from the analyses in Chapter Four and Chapter Five.

Fig. 2.10a Hyland Creek: commercial center along 72 Ave. and King George Highway.

Fig. 2.10b Hyland Creek: wide vacant lots.

CHAPTER THREE

METHODS OF ANALYSIS

3.1 INTRODUCTION

The purpose of this chapter is to present the methods used in analysing the data obtained. The first part of this chapter discusses the methods of analysis; a) unit hydrographs and; b) distribution graphs. The second part of the chapter deals with the data and its collection.

3.2 Methods used

Since this study is concerned with the comparison of several basins, the unit hydrograph principle is the main analytical tool used. This is because a common basis of comparison such as the unit hydrograph is fundamental such that all runoff responses will be the result of 1 cm of rainfall excess. Also, it is a straight forward method to derive the rainfall-runoff relations from observed hydrographs. Though originally designed for use on large watersheds where effective rainfall could be determined from daily rainfall records (Chow 1952), a number of investigators (Brater 1940 ;Chow 1952; Minshall 1960; Eagleson 1962; Espey et al. 1966; Viessman 1966, 1968) have used it successfully on small watersheds such as those used in this study. Additionally, the distribution graph, which is an advancement of the unit hydrograph, and observed volumetric runoff coefficients will be computed and incorporated for further analysis.

3.2.1 The unit hydrograph

The basic theory of the unit hydrograph was proposed by L.K. Sherman in 1932. It was defined as a hydrograph of direct runoff resulting from rainfall excess occurring in a specified period of time. The unit hydrograph of runoff represents the integrated effects of all the basin constants including area, shape, stream pattern, channel properties, land slopes and other physiographic characteristics. However, in the course of development and refinement, the unit hydrograph has come to be defined as the hydrograph of 1 cm of rainfall excess generated uniformly in intensity within a specified period of time.

The basic assumptions of the unit hydrograph theory may be summarised as follows:

a) For a given watershed, storms of equal duration will produce surface runoff hydrographs with approximately equivalent time bases, regardless of the intensity of rain; b) for a given watershed, the ordinates of the surface runoff hydrographs from similar storms of equal duration are proportional to the volume of surface runoff; c) for a given watershed, the time distribution of surface runoff from a particular storm period is independent of precipitation from antecedent or subsequent storm periods.

The varying effects of channel storage, in a way, do not exactly reflect truthfully the first assumption. However, as Linsley, et al. (1949) noted, because the recession curve approaches zero asymptotically, a practical compromise is possible without excessive error. The second assumption is applicable provided that the selected time unit is less than the minimum concentration time (Sherman 1940). Finally, the antecedent conditions are important as the volume of rainfall excess resulting from a given amount of rainfall will have to depend on the soil infiltration rate, depression and detention storage (Gray 1970).

Though accepted by most hydrologists as a valuable hydrologic tool it has limitations which must be taken into account. The ideal conditions for the derivation of unit hydrographs from storms can be summarised as:

a) the rain must have fallen within the recorded time interval or time limit, such as a day or hour; b) Tte storm must have been well distributed over the watershed, with all stations showing an appreciable depth of rainfall; c) the runoff following the storm must not have been affected by low temperature or by melting snow and ice; d) the storm period must be comparatively isolated in record. It should follow a period of low stream flow, and there should be no further rainfall until the peak is well passed (Morgan and Johnson 1962).

Identical runoff producing storms occurring over the same watershed are rare, particularly over large watersheds or odd-shaped basins, especially those which are long and narrow. It is also not suitable for areas subjected to orographic rainfall where areal distribution of such rainfall is usually quite uneven. Hence, unit hydrographs are best suited to areas which are less than 2000 sq. miles (Linsley, et al. 1949).

The unit hydrograph is deemed to be a very suitable and valuable hydrologic tool for the comparative objectives of this thesis. With basin areas ranging from 0.26 to 4.71 sq. km, it is reasonable to assume that the distribution of rainfall is not too uneven. Moreover, the rainfall data selected to derive the unit duration show relatively even intensities. Though 10th & Alberta is an 'odd-shape' basin, we can safely accept that the unusual shape is compensated by the smallness of its area of 0.74 sq. km.

3.2.1.1 Derivation of unit hydrographs

Several steps are taken when deriving a unit hydrograph from observed hydrographs. The procedures are explained below:

a) Plotting the observed hydrograph - The observed hydrographs for 12th & McKenzie and 10th & Alberta were recorded with the percentage of maximum flow against time. These were converted to discharges in cumecs mathematically (for a further discussion, see Section 3.3.3, p. 63). The watersheds in Surrey have the stage of water level plotted against time in the observed hydrographs. The stages were converted to discharges in cumecs by means of their respective rating curves (Appendix B).

b) Baseflow separation - There are several methods for separating baseflow. The accurate determination is a matter of uncertainty since the exact laws which govern baseflow are unknown (Chow 1952). It should be noted that the exact method used in the analysis is not critical to unit hydrograph development provided the same method is always applied. Three important points of the observed hydrographs are needed to derive a unit hydrograph (Fig. 3.1b), namely, the beginning of direct runoff (point A), the peak discharge (point B) and the point of the recession curve which represents the end of direct

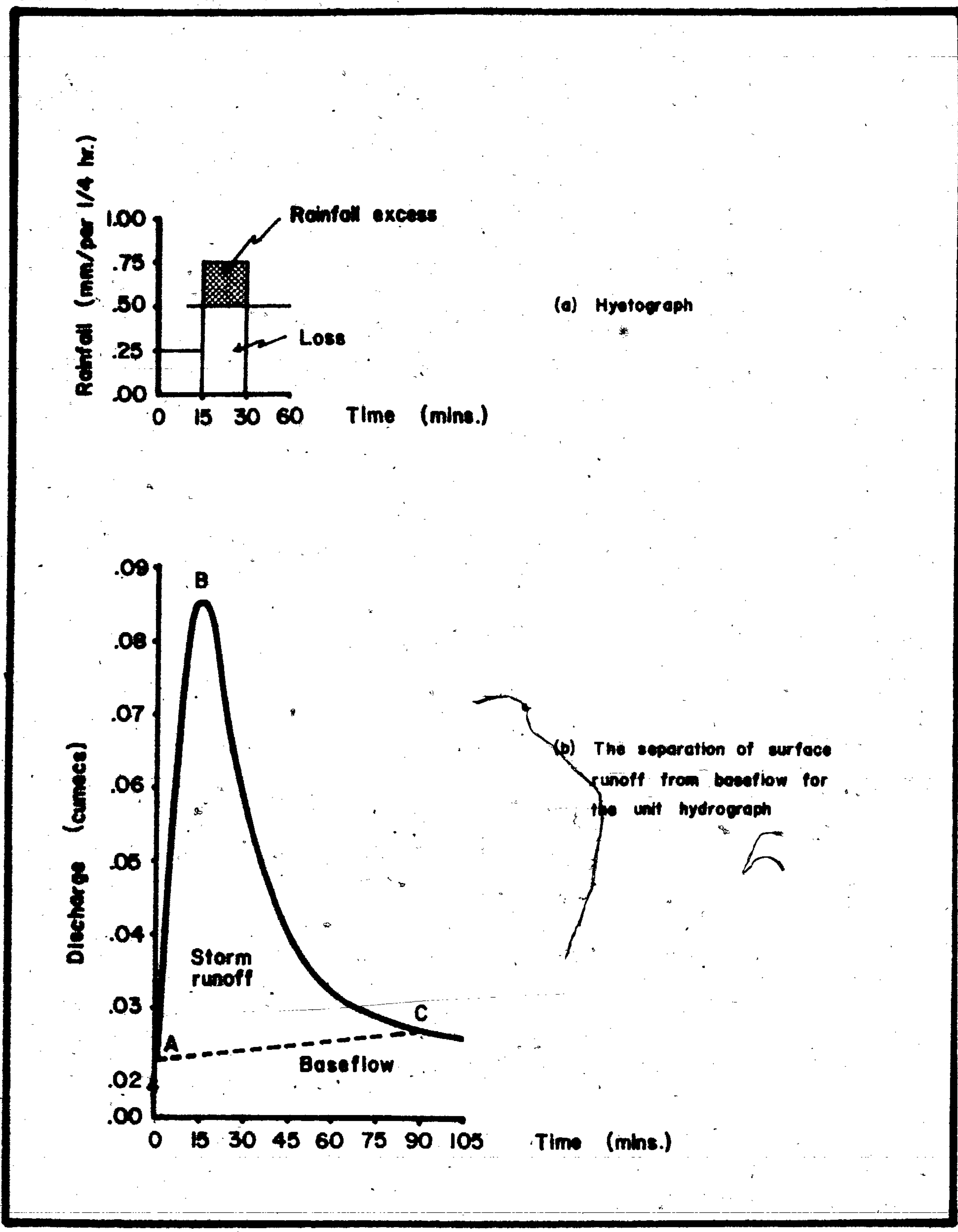


Fig. 3.1 The event of March 25th, 1977 (12th & MacKenzie basin) used in the derivation of the unit hydrograph.

runoff (point C). It is with the determination of the third point on the recession curve that difficulties arise. However, a consistent method of locating this point is to plot the observed hydrograph on a semilogarithmic paper (after Barnes 1939). A straight line will be produced by the reccessional limb. Gray (1970) comments that this will usually occur when interflow and groundwater is negligible in comparison to the contribution by surface runoff. The point where the recessional limb meets the baseflow line is taken to represent the third point, which is the end of direct runoff. A line is then drawn from point A to C to represent the baseflow (Fig. 3.1b). The entire area, ABC, is therefore taken as the total direct runoff for a particular storm.

c) Deriving the hydrograph - The ordinates of the direct runoff hydrograph are reduced by taking the difference between the observed hydrograph and baseflow. The modified hydrograph represents the hydrograph of runoff.

d) Calculation of direct runoff depth - The area of the hydrograph derived above represents the total volume of direct runoff. The hydrograph is plotted accordingly onto a graph paper and the squares below the hydrograph counted. The volume of direct runoff is calculated:

$$y = a.t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.1)$$

where, y = discharge volume in cubic meters; a = area as in number of squares; t = value of each square in cubic meters.

The depth of direct runoff is calculated:

$$D = y/A \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.2)$$

where, D = depth of direct runoff in millimeters; y = dicharge in cubic meters; A = area of watershed in sq. km.

e) Computing the unit hydrograph - The ordinate of the direct runoff hydrograph divided by the total depth of direct runoff will give the ordinated of the unit hydrograph for the unit duration for a given storm (Appendix C).

3.2.1.2 Determination of unit duration

There are several techniques used in deriving the unit durations or the duration of rainfall excess. Several opinions have also been expressed over their use.

For instance, for small watersheds (less than 10 sq. mi.), unit hydrographs will result from short, isolated storms with durations less than the time of rise. On the other hand, the unit duration for larger watersheds may be less than the time of rise but no more than half as long. Linsley, et al. (1949) found that the unit duration should approximate 1/4 of the time lag (time from centroid of rainfall excess to peak discharge), with a tolerance level of + 25%.

In the determination of unit duration, it has been accepted by some that before overland flow can resume, a portion of the initial rainfall is stored and permanently abstracted from surface runoff by interception (which later evaporates) and depression storage (which later evaporates or infiltrates below the soil surface). Through experiences in research, various investigators have arrived at a range of retention values. Tholin and Keifer (1960) suggested values of retention of 1/16 inch for pavements and 1/4 inch for grassland. Viessman (1966) reported retention ranges of 0.04 to 0.10 ins. for small paved areas. Brater (1968) concluded an average initial retention of 0.02 inch for 3 urban watersheds studied in Detroit.

To estimate infiltration, depression storage and interception loss, a wide range of techniques have been used. Some (Keifer and Chu 1957; Schulz and Lopez (1974) assumed that a portion of the initial rainfall is lost and does not contribute to the channel. Others (Brater 1968; Eagleson 1962) used a decay-type curve to estimate the volume lost. Minshall (1960) made use of the Antecedent Precipitation Index (API) for this purpose. Yet others like Chow 1952, 1964) and Willeke (1966) drew a horizontal line across the hyetograph of a particular storm which, in actuality, gives a mean ordinate of the decay curve and the attainment of a phi-index.

In this study, the method used is similar to that of Chow (1952, 1964) and Willeke (1966). A horizontal line is drawn across the hyetograph such that the volume of rainfall above the line equals the direct runoff for that particular storm (Fig. 3.1a). The area under the hyetograph however, represents the losses by infiltration, interception, retention and depression storages. Though this technique gives only a first approximation to rainfall loss, it proves to be a satisfactory method for deriving unit duration in this study. The unit duration is defined by the time between the beginning and the end of rainfall excess.

3.2.1.3 Derivation of unit hydrograph for other than the unit duration of the original storm

Unit hydrographs for unit durations longer than the original one is computed through the addition of the original hydrographs from the selected events. Since representative unit hydrographs are to be derived, it is considered appropriate to develop the few 15-minute unit hydrographs into 30-minute durations (30 minute unit hydrographs were mostly obtained compared to 15-minute cases). This is done by using the summation technique. For example, the summation of two 15-minute unit hydrograph will yield a 30-minute graph with 2 cm of direct runoff. The ordinates of this derived graph are then divided by 2 to obtain a 30-minute unit hydrograph (Appendix D).

3.2.1.4 Derivation of a mean unit hydrograph

Still a further step is needed for the common basis of comparison of the unit hydrographs. This is where the development of a mean unit hydrograph is necessary. In this study therefore, mean 30-minute unit hydrographs are derived. This is done by averaging several 30-minute unit hydrographs to derive a mean 30-minute unit hydrograph. Accordingly then, each mean 30-minute unit hydrograph would then be assumed to be representative for the watersheds. It is graphed by calculating and locating the mean peak discharge, time of rise and time base of the 30-minute unit hydrographs. The area below the graph equals 1 cm of runoff while resembling the individual graphs as much as possible (Linsley, et al 1949).

3.2.2 The distribution graph

The distribution graph is a development of the unit graph and was developed by Merill M. Bernard in 1934. It is a modified unit hydrograph to show the proportional relation of its ordinates whereby the ordinates are expressions of the total surface runoff volume in percentages. Gray (1970) commented that because the total area under the distribution graph equals 100% of surface runoff, differences in the runoff characteristics between watersheds are reflected in the respective shapes of their distribution graphs - a valuable tool for runoff analyses. This method is used in conjunction with the unit hydrograph since the watersheds in this study are of different sizes. Therefore, it is useful in the analysis to attempt an explanation on the characteristics of surface runoff.

3.2.2.1 Construction of the distribution graph

Though the derivation of the mean distribution graph from several similar graphs have been cited in most literature (Brater 1940; Meyer 1940), it is considered reasonable to construct mean distribution graphs from the already derived mean unit hydrographs for the watersheds. This is because similar conditions of rainfall distribution, duration and other factors required for the unit hydrograph are also assumed in the distribution graph. The reasons underlying this consideration are representativeness and consistency; representativeness because the mean 30-minute unit hydrographs are assumed to be representative graphs, and consistency in order to avoid any 'errors' that might incur during the averaging process of the distribution graphs.

The mean unit hydrographs are interpolated into time intervals of 15 minutes. The ordinate values are summed and the percentage of the total value occurring at every time interval calculated. These percentage values are the numerical representation of the distribution graphs. It should be noted that the time intervals selected are similar because the percentage value changes accordingly if different time intervals are selected for each of the mean unit hydrographs. For example, if 15-minute and 30-minute intervals are selected for each case, then the percentages would be different byone half.

3.3 Collection of data

The data used in this study are the rainfall, sewerflow and streamflow records for the basins. Other information included are aerial photos and maps needed to determine the urbanization index as well as for determining the quantitative geomorphology of the watersheds. Because of the geographical distribution of the watersheds, the data, aerial photos and maps were collected from a variety of governmental sources; the City of Vancouver Engineering Department, Greater Vancouver Sewerage and Drainage District, Sigma Engineering Consultants Ltd. and the Engineering Division of the Surrey Municipality. Table 3.1 shows the respective sources for the rainfall, sewerflow and streamflow data.

3.3.1 Rainfall stations

Each watershed in the study has a corresponding recording rain gauge located within or very close to the watershed (Fig. 3.2). Though some investigators have recommended at least 2 rain gauges are needed for the evaluation of a representative rainfall depth for an area, one recording rain gauge is deemed sufficient because of the smallness of the drainage basins.

The locations of the respective rainfall gauges are shown in Fig. 3.2. For 12th & MacKenzie, the recording rain gauge is located on the rooftop of the GVRD Head Office (10th & Vine St.) which is just outside the watershed boundary. It is a long term (records up to periods of 90 days) tipping bucket recording rain gauge. The time resolution is 10 minutes with

Drainage Areas	Type of Data	*Source	Period of Analysis
12th & MacKenzie	Rainfall	GVS & DD	Feb. '76-Mar. '77
	Sewerflow	VCED	Feb. '76-Mar. '77
10th & Alberta	Rainfall	VCED	Oct. '76-Mar. '77
	Sewerflow	VCED	Oct. '76-Mar. '77
Robson Creek	Rainfall	SEC	Jan. '77-Oct. '77
	Streamflow	SEC	Jan. '77-Oct. '77
Hyland Creek	Rainfall	SEC	Dec. '76-Oct. '78
	Streamflow	SEC	Dec. '76-Oct. '78

* GVS & DD - Greater Vancouver Sewerage and Drainage District

VCED - Vancouver City Engineering Department

SEC - Sigma Engineering Consultants Ltd.

Table 3.1 Data source

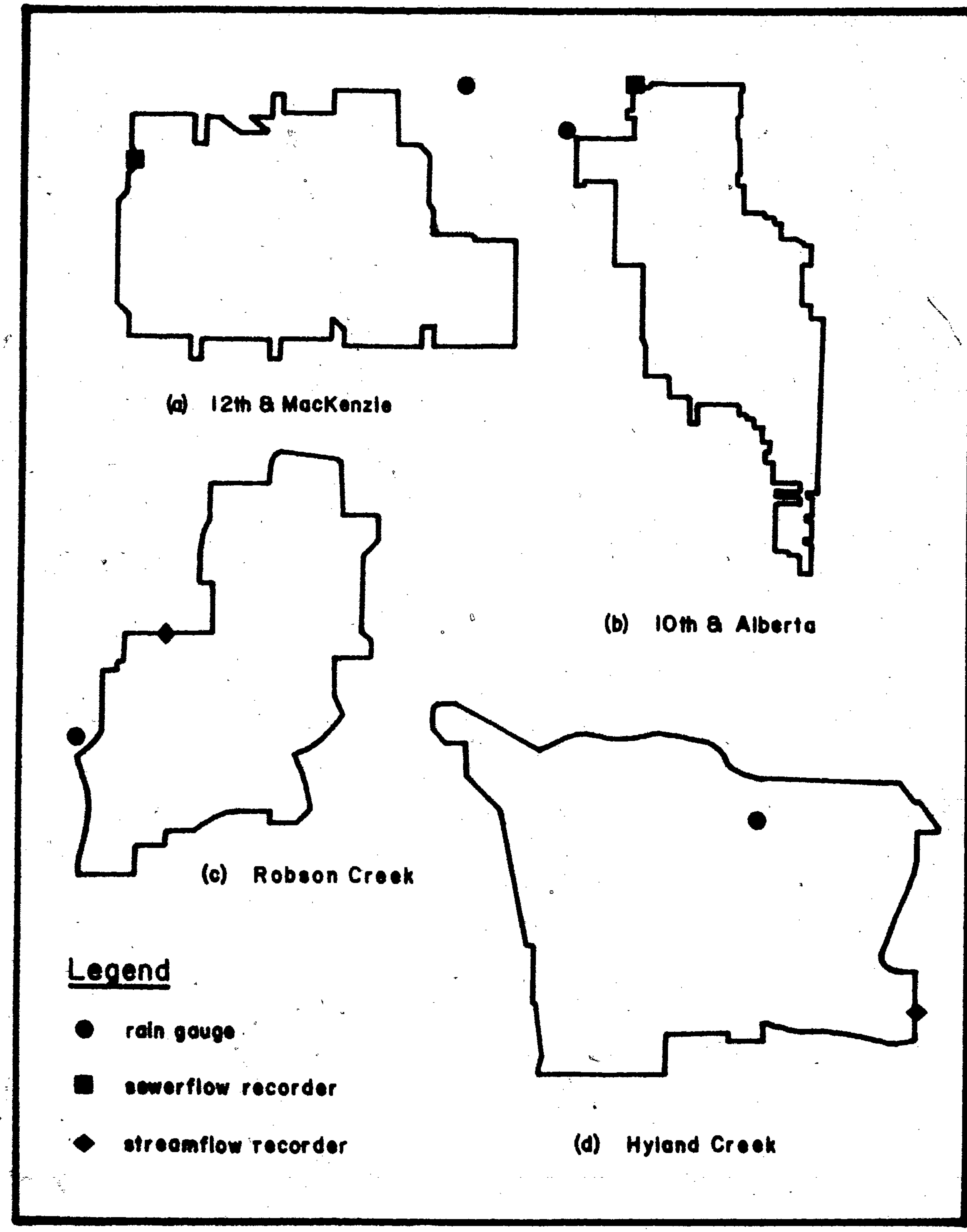


Fig. 3.2 Instrumentation of the watersheds.

each 0.01 inch (approx. 0.25 mm) of rain causing the bucket to tip. In 10th & Alberta, the recording rain gauge is situated on the rooftop of Vancouver City Hall. It is a tipping bucket type with a time resolution of 15 minutes while each step traced over indicates a 0.02 inch of rainfall. As for Robson Creek and Hyland Creek, the rain gauges maintained are of the same design, which is a tipping bucket long term recording gauge with a time resolution of half hour and a tracer step of 0.01 inch. The gauge in Robson Creek is situated just outside the watershed boundary. It is located at the south side of the roof of Cedar Hills School at 124 St. and 98 Ave. In Hyland Creek, the gauge is located on the rooftop of the storage building at the Fire Hall at No. 10 on 72 Ave. near King George Highway.

3.3.2 Sewerflow and streamflow gauging stations

The sewerflow measurements for the Vancouver watersheds are recorded by stage-discharge recorders (Manning's Portable Dipper Flow Meter) installed within the sewers: these are clockwork driven. These instruments are located at the junction of 12th Ave. and MacKenzie St. and at the junction of 10th Ave. and Alberta St.

The streamflow measurements for the Surrey watersheds are recorded by Type R20 Ott Water Level Recorders. These clockwork driven, float-actuated instruments are installed together with stilling wells. Broad crested weirs of the rectangular type are used and installed across the outlet. The location of the streamflow gauging station for Robson Creek is at the culvert headwall at 100 Ave. near 125 St. As for Hyland Creek watershed, the streamflow gauging station is situated in the mainstream of Upper Hyland Creek at the 140 St. bridge crossing.

3.3.3 Stage-discharge relations

The stage-discharge relation, or more simply the rating curve is established by current metering the creeks near the gauges to obtain the discharge for a given stage of the water level. A representative line is drawn through the plotted stage-discharge points (Appendix B). Any discharge volume to be known would simply be extrapolated from the curve with a known stage value and, in our case, from the hydrographs.

As a result of the type of instruments used for sewerflow gauging in 12th & MacKenzie and 10th & Alberta, a somewhat different stage- discharge relation is noted. Instead of determining the discharge volumes from rating curves like Robson Creek and Hyland Creek, the discharge ordinate structure is expressed in percentages of the maximum discharge for the sewer. The maximum discharge is represented as 100%. If the discharge in cfs of, say, the peak of the hydrograph is to be known, the peak discharge is then computed by a simple mathematical calculation. The maximum flows are 0.91 cubic meters per second (cumecs) (8.81 cfs) and 2.32 cumecs (31.98 cfs) for 12th & MacKenzie and 10th & Alberta respectively.

3.4 Evaluation of data

A preliminary examination of the available rainfall, sewerflow and streamflow data was done before deriving unit hydrographs. The primary events chosen were isolated, clear-cut and single peaked events except for the storm dated 11th December 1977 for Hyland Creek which was a complex one. Even then this complex storm hydrograph is the resultant event from the first of a series of storm bursts so that it can be separated from the suceeding hydrographs by extending the recessional limb that overlaps the subsequent hydrograph. The analysis was also designed to choose those events occurring during the winter months. This is important as the seasonality will have a pronounce effect on runoff response (Taylor 1977). However, wetter and drier antecedent conditions are possible because the rainfall instruments do not gauge any light showers less than 0.01 inch. Errors may even be greater for 10th & Alberta since there will only be recordings when rainfall of 0.02 inch or larger occurs. Limitations were also found in the sewerflow records. The time resolution has an interval of 3 hours which is too long for a short interval (say, 5 mins.) discharge ordinate to be calculated. Conversely, because events were carefully chosen to reduce 'theoretical errors' to a minimum, only a very small number of cases were used given the available short period of records (Table 3.1).

3.5 Measurement of impervious cover

The impervious cover includes rooftops, car parks, pavements, sidewalks, roads, and other surfaces restricting infiltration. The extent of these features was measured from aerial photos and maps. Information used for 12th & MacKenzie and 10th & Alberta are aerial photos of scale 1 inch to 200 ft. while, maps drafted from aerial photos of scale 1 inch to 400 ft. were used for Robson Creek and Hyland Creek.

There are several ways in which the impervious cover can be measured. Among the most popular ones are the planimeter and sampling method. However, many attributes to inaccuracy when using a planimeter may be involved, such as, whether the operator has a steady hand, whether the map has a smooth or rough surface and whether the area being measured is conveniently located far enough from the edge of the map to allow tracing without the wheel of the instrument running off the edge (Wood 1954). On the other hand, the sampling method will give a coarse estimate. In this study, the means of measuring the impervious cover is the random dot planimeter. It is a reliable means of measuring the impervious cover of the watersheds with an accuracy of 97% (Dept of Agric. 1970). Additionally, this technique is fast and also simple to use.

The random dot planimeter works on the principle that the number of randomly spaced dots falling within a boundary is proportional to the area enclosed. In other words, if a value in square units is assigned to each dot, the area enclosed by the boundary will be the number of dots times the value of each dot. A positive of the random dots (in rectangular grids) is placed over the area enclosed by the watershed. The total number of dots falling within the boundary is counted to represent the total area. The next step is to count the dots falling within the impervious features as described earlier. These are then summed up and divided by the total number of dots computed initially. This ratio is then the proportion of impervious area within the watershed expressed as a percentage (Appendix A).

CHAPTER FOUR

THE RESULTS

4.1 Introduction

In the preceding chapters, the analytical procedures for the rainfall, sewerflow, streamflow data, and the measurements of basin characteristics of the watersheds in the study have been presented. In this chapter, the results of the use of the derived unit hydrograph, distribution graph and the runoff coefficients in the analysis of the impact of urbanization on peak flows will be reported.

4.2 Urbanization and peak flows

4.2.1 Unit hydrographs

Unit hydrographs for the watersheds were derived from the procedures outlined in Chapter Three. Caution was taken in choosing the appropriate events (see Section 2.2) and only a small number of events were selected (Table 4.1) for the derivation of unit hydrographs (Tables 4.2 and 4.3).

Watershed	Drainage Area (sq. km)	Storm No.	Storm Date	Duration of Rainfall (mins.)	Duration of Rainfall Excess (mins.)	Rainfall (mm)	Rainfall Excess (mm)
12th & MacKenzie	0.26	1	27th Feb.'76	30	15	1.27	0.14
		2	17th Mar.'76	45	30	1.27	0.11
		3	16th Feb.'77	30	15	1.27	0.27
		4	25th Mar.'77	30	15	1.02	0.37
		5	27th Mar.'77	30	15	2.29	0.42
10th & Alberta	0.76	1	7th Nov.'76	105	30	4.57	0.36
		2	18th Nov.'76	60	15	3.05	0.54
		3	20th Dec.'77	105	60	2.03	0.36
		4	4th Feb.'77	30	30	1.02	0.20
		5	26th Mar.'77	30	30	2.03	0.40
Robson Creek	2.03	1	23rd Mar.'77	75	15	2.29	0.47
		2	26th Mar.'77	90	30	10.41	0.64
		3	25th Oct.'77	105	45	5.08	0.58
Hyland Creek	4.71	1	25th Dec.'76	450	30	9.40	0.32
		2	11th Feb.'76	540	150	19.30	2.87
		3	28th Feb.'77	285	30	9.14	0.35
		4	4th Apr.'78	360	135	11.18	2.07
		5	20th Oct.'78	270	75	9.65	0.45

Table 4.1 Selected storm characteristics

Watershed	Storm No.	Unit Duration (mins.)	Peakflow (cumecs.)	Peakflow Per Unit Area
12th & MacKenzie	*1	15	1.93	7.42
	2	30	1.99	7.65
	*3	15	1.37	5.27
	*4	15	1.70	6.54
	5	15	1.41	5.42
10th & Alberta	*1	30	3.50	4.72
	2	15	2.55	3.44
	3	60	2.72	3.68
	*4	30	3.45	7.19
	*5	30	5.33	4.66
Robson Creek	*1	30	7.24	3.57
	*2	30	6.08	1.29
	3	45	4.85	2.39
Hyland Creek	*1	30	7.83	1.66
	2	150	3.10	0.66
	*3	30	6.58	1.40
	4	135	8.21	1.74
	5	75	8.21	1.74

* Selected events for deriving mean 30-min. unit hydrographs

Table 4.2 Derived unit hydrograph characteristics

Watershed	Storm No.	Storm Date	Duration of Rainfall Excess (mins.)	Qpk (cumecs.)	Qpk (cumecs. per sq. km)	Time of Rise (mins.)	Base Width (mins.)
12th & MacKenzie	1	27/02/76	30	1.23	4.73	30	60
	3	16/02/77	30	1.24	4.77	30	70
	4	25/03/77	30	1.11	4.27	30	90
10th & Alberta	1	7/11/76	30	3.34	4.51	45	105
	4	4/02/77	30	2.84	3.84	45	105
	5	26/03/77	30	3.30	4.46	45	90
Robson Creek	1	23/03/77	30	6.12	3.01	45	210
	2	26/05/77	30	5.75	2.83	60	225
Hyland Creek	1	25/12/76	30	6.95	1.48	60	210
	3	28/02/77	30	6.46	1.37	60	240

Table 4.3 Derived 30-minute unit hydrograph characteristics

4.2.1.1 Peak flows

Fig. 4.1 shows the average 30-minute unit hydrographs that have been derived for the watersheds under study. Despite the varying degrees of urbanization, the total peak discharge is still, to a very considerable extent, a function of the drainage area explaining the direct relationship (12th & MacKenzie - 1.19 cumecs; 10th & Alberta - 3.16 cumecs; robson Creek - 5.94 cumecs and; Hyland Creek - 6.71 cumecs). In order to compare the storm runoff yields of these basins, the discharge in transformed into cumecs per square kilometer (Table 4.4 and Fig. 4.2).

From Fig. 4.2, the hydrographs show that the unit peak flow is higher in the Vancouver watersheds than the ones in Surrey. Furthermore, the Vancouver unit hydrographs appear steeper and narrower than the broad based ones of Surrey watersheds (Hyland Creek in particular). Although the direct relationship between impervious cover and the unit peak flow is apparent (Fig. 4.3), unit yields from 10th & Alberta raise some questions. Despite having the highest computed impervious cover of 50.5%, the unit peak flow is only 4.27 cumecs/sq. km compared to 4.58 cumecs/sq. km for 12th & MacKenzie which is 45.4% impervious. Results from Robson Creek appear to be reasonably consistent. Overall, the interpretation of the data appears to be fairly

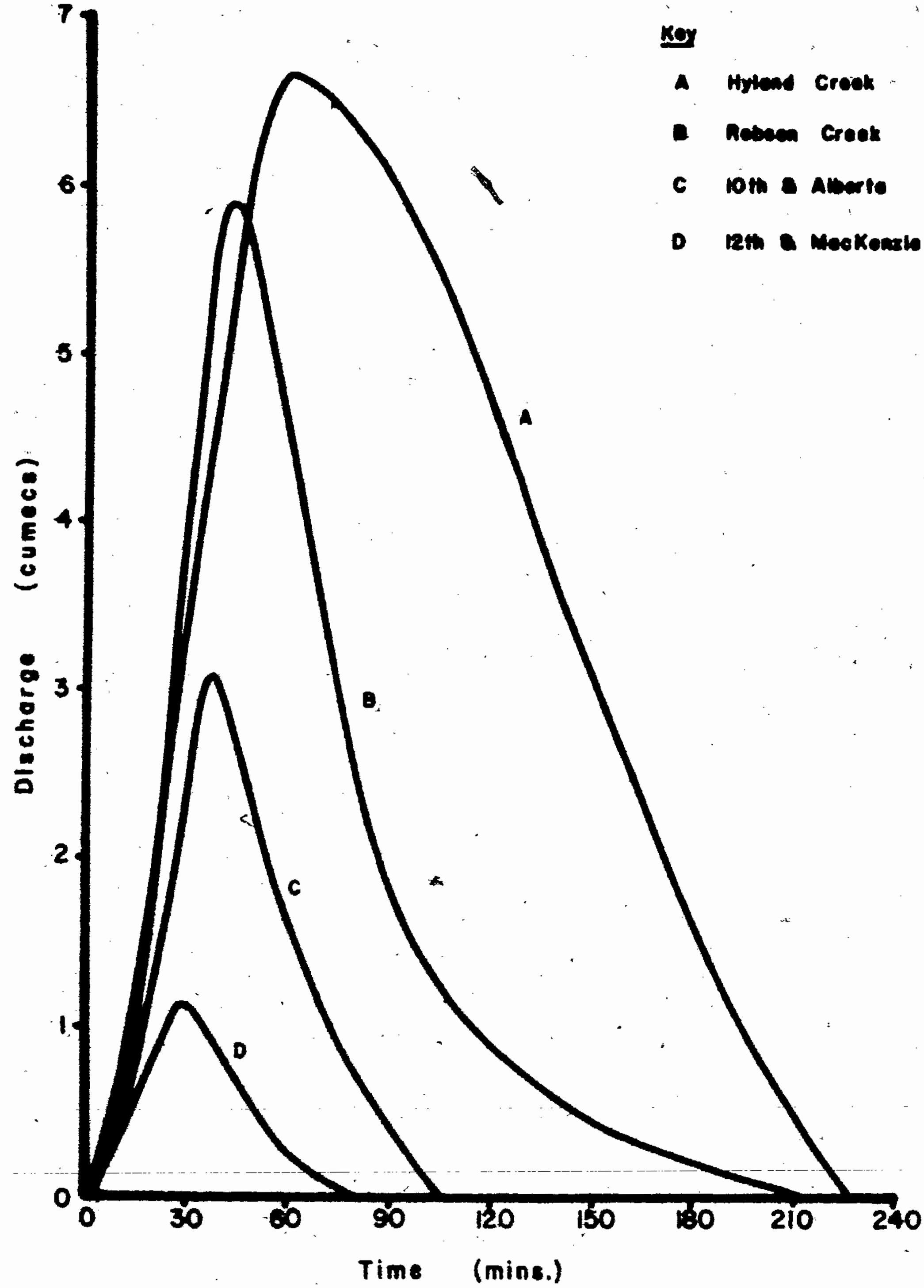


Fig. 4.1 Mean 30-minute unit hydrographs for the four watersheds.

Watershed	Duration of Rainfall Excess (mins.)	Average Qpk (cumecs.)	Average Qpk (cumecs. per sq. km)	Average Time of Rise (mins.)	Average Width (mins.)
12th & MacKenzie	30	1.19	4.58	30	73
10th & Alberta	30	3.16	4.27	45	100
Robson Creek	30	5.94	2.93	53	218
Hyland Creek	30	6.71	1.42	60	225

Table 4.4 Average 30-minute unit hydrograph characteristics

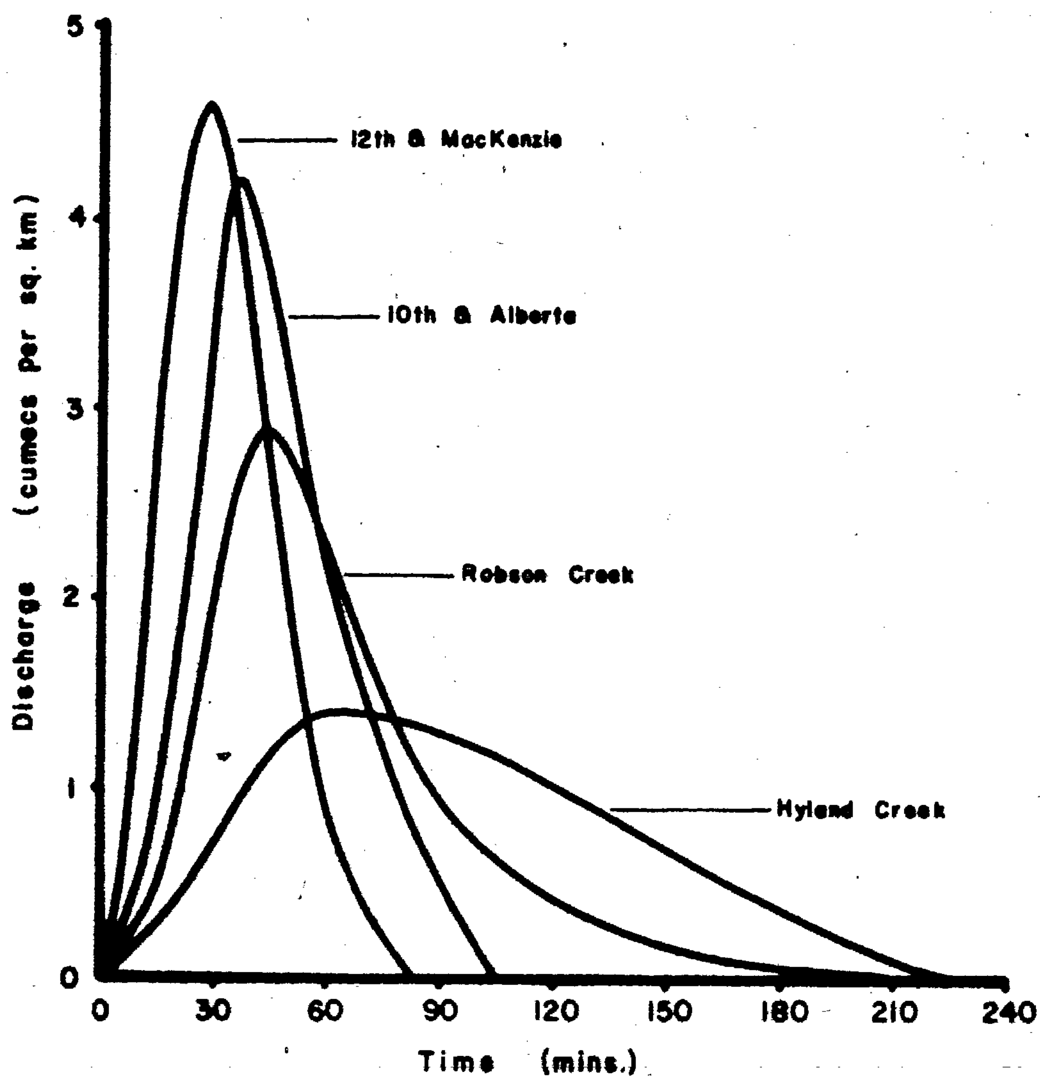


Fig. 4.2 Mean 30-minute unit hydrographs (per unit area) for the watersheds.

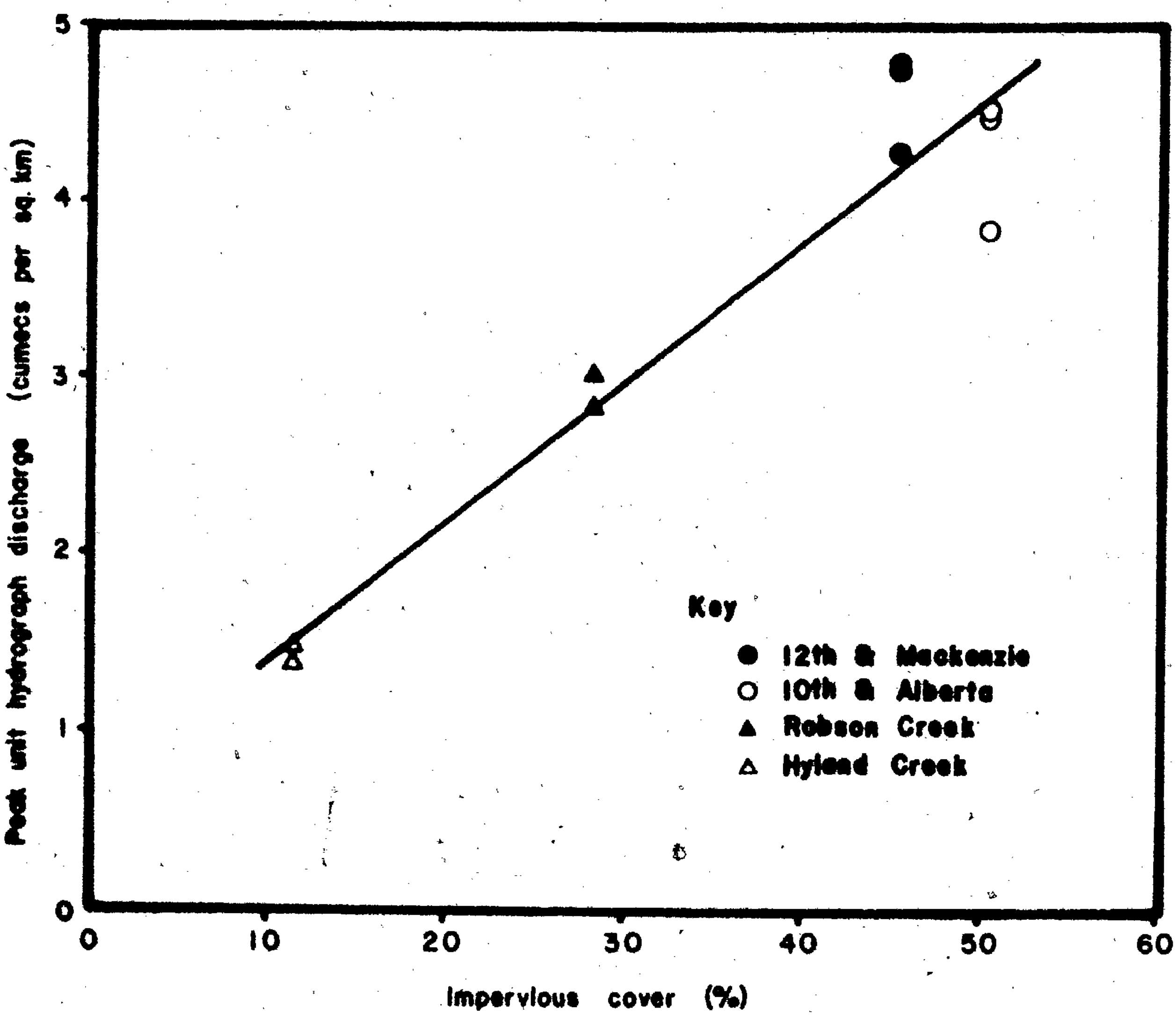


Fig. 4.3 Relationship between peak discharges (cumecs/sq. km) and the degree of urbanization.

consistent with the general assumption of the thesis with the minor exception of 10th & Alberta.

4.2.2 Distribution graphs

Like the unit hydrograph, the distribution graph, however, gives a more explicit quantitative indication on the timing of the rate of surface runoff. Though it would define the hydrograph less explicitly than instantaneous flow (Linsley, et al. 1958), it is deemed an appropriate tool for comparison as it shows the percentage contribution characteristics of the net storm. It is also preferable, as mentioned earlier, since the study includes different sized watersheds.

Fig. 4.4 illustrates the distribution graphs derived at 15-minute intervals. Note that the percentage of flow increases rapidly for 12th & MacKenzie, with a peak contribution of 42.3% of the total storm runoff (Fig. 4.5 and Table 4.5). Compared to the rest which are more gradual, the ratios of percentage peak flows computed are about 40:25:23:12 for 12th & MacKenzie, 10th & Alberta, Robson Creek and Hyland Creek accordingly. A greater portion (66.9%) of the storm runoff is delivered out of the watershed at the time of peak discharge for 12th & Mackenzie (Table 4.5). Slightly over 1/2 (53.3%) is accounted for 10th & Alberta, while 45.7% and 32.4%

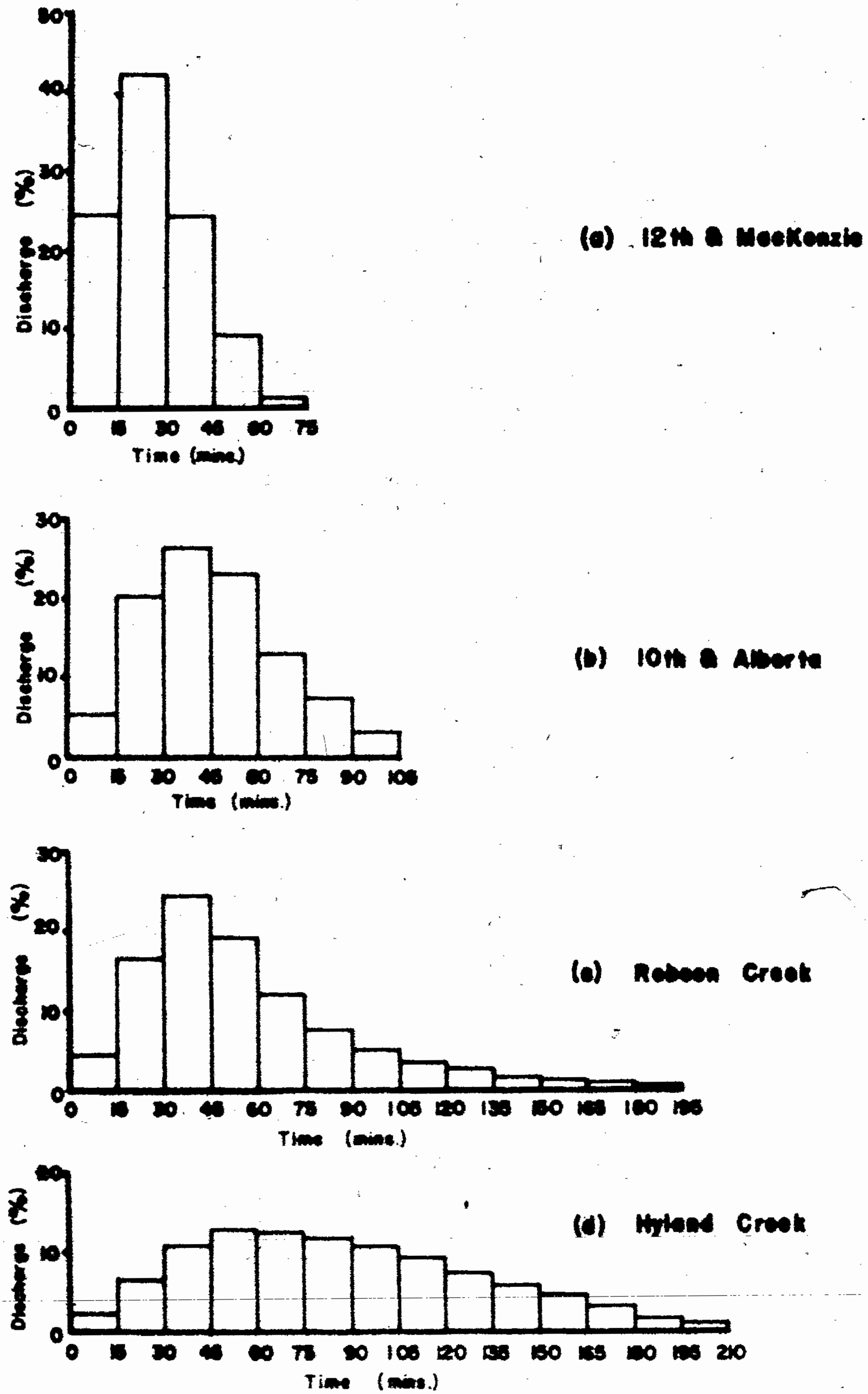


Fig. 4.4 Distribution graphs from mean 30-minute unit hydrographs.

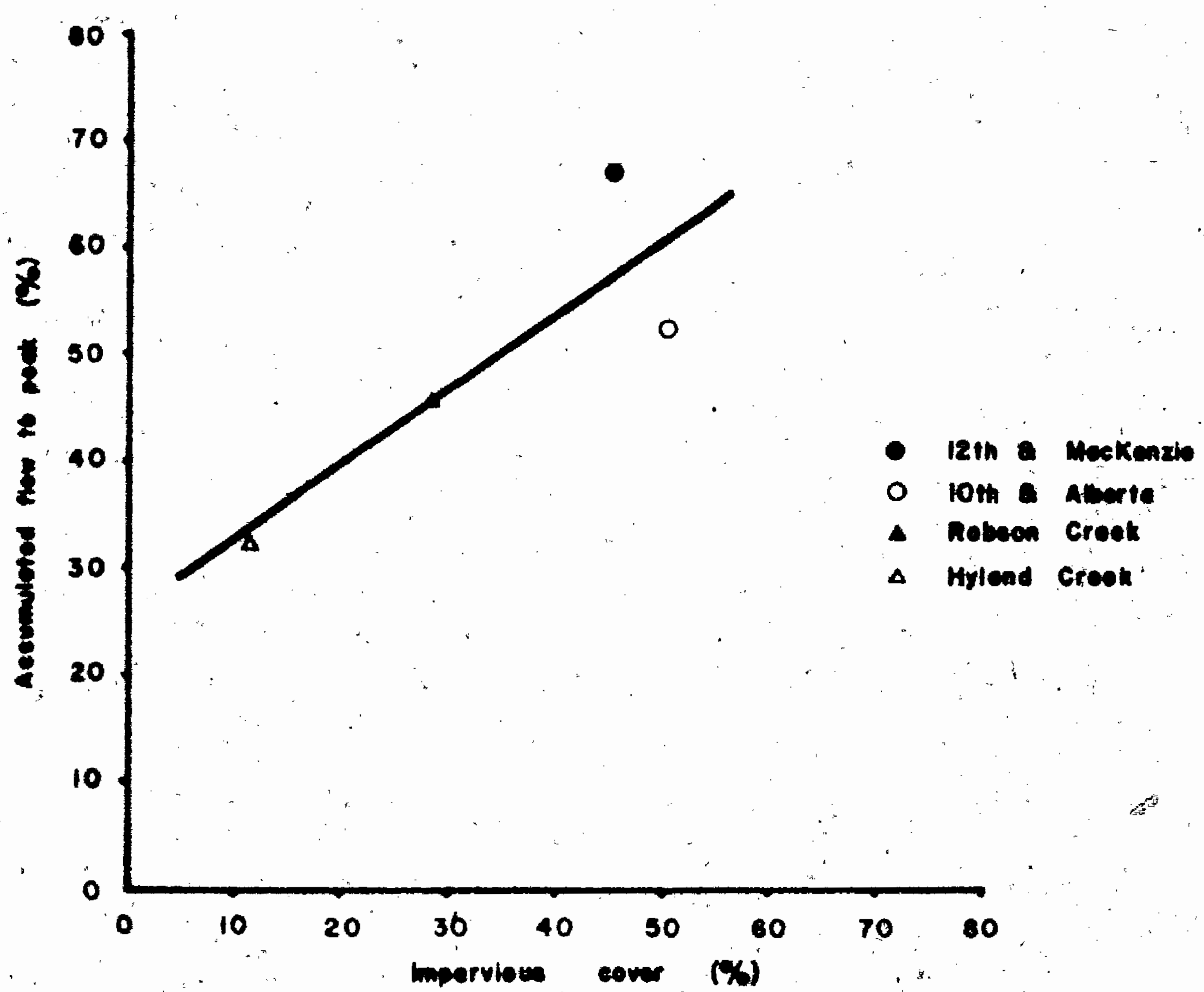


Fig. 4.5 Relationship between distribution flow and impervious cover.

Time Ordinates (mins.)	12th & MacKenzie	10th & Alberta	Robson Creek	Hyland Creek
0	0.00	0.00	0.00	0.00
15	26.90	5.85	4.17	2.12
30	46.20	20.07	16.69	6.55
45	26.90	26.42	24.79	10.78
60	8.10	23.41	19.20	12.92
75	1.20	13.38	12.10	12.52
90	0.00	7.53	7.51	11.94
105		3.34	5.09	10.59
120		0.00	3.76	9.05
135			2.51	7.32
150			1.67	5.97
165			1.25	4.47
180			0.84	3.08
195			0.42	1.73
210			0.00	.96
225				0.00
240				

Table 4.5 Ordinates of distribution graphs in percentages

are similarly computed for Robson Creek and Hyland Creek respectively. These figures give some indication of the rate of flow, which in turn implies the rate of delivery by the drainage system.

4.3 Urbanization and hydrograph shape

4.3.1 Time of rise

The time of rise (sometimes known as the period of rise) may be defined as the time from the beginning of storm runoff to peak discharge (Fig. 4.6). Generally, the time of rise for a small watershed is a function of three factors:

a) surface properties, which involves such factors as: impervious cover, channel characteristics, land use, soil moisture and geology.

b) geometry of watershed, as defined by area, length, slope and shape.

c) storm characteristics, as expressed by size, duration and intensity.

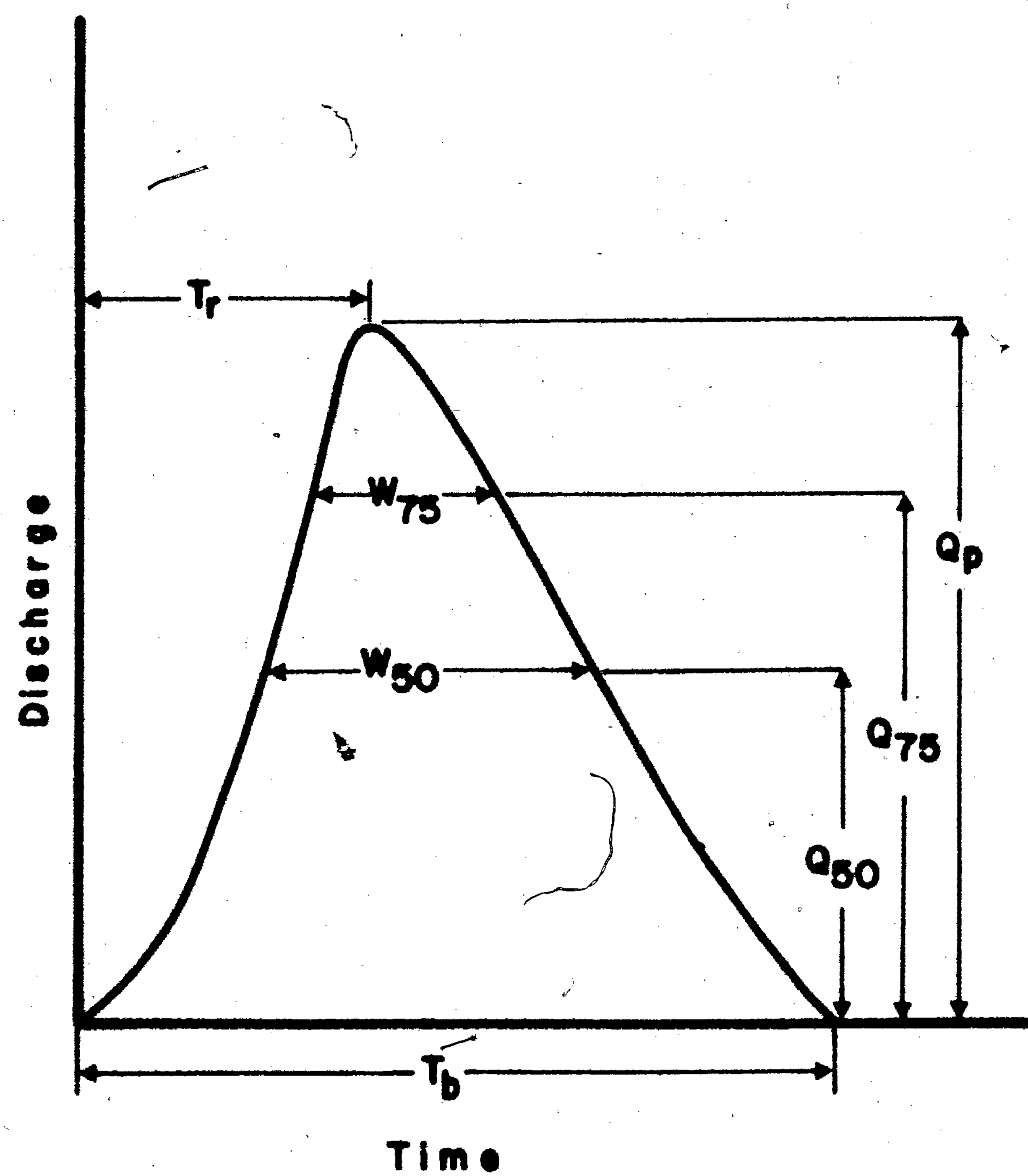


Fig. 4.6 Unit hydrograph properties (see text for definitions).

By the selection of storms having essentially similar charcteristics, the time of rise can be considered a function of the surface properties and watershed geometry. Since the surface properties are reasonably stable over a period of time, it is permissible for us to assume that this factor and the watershed geometry is constant.

Table 4.3 shows the time of rise computed for the watersheds as a result of 30-minute rainfall excess. It is quite clear that shorter times of rise are experienced by the Vancouver watersheds. Fig. 4.2 may illustrate this more clearly when the average 30-minute unit hydrographs are derived (see also Table 4.4). 12th & MacKenzie has a time of rise of 30 minutes, which is 15 minutes shorter than 10th & Alberta. As for Robson Creek and Hyland Creek, the time of rise is delayed for the former by a difference of only 7 minutes. The computed time of rise seems to correlate quite strongly to the state of development of the watersheds. However, caution should also be excercised in interpreting these values as watershed size will have a significant effect on the time of rise. Hence, the constraint here lies with the unequal watershed sizes.

Despite having the highest value for the length of the longest channel (Table 2.5), and being an unusually elongated watershed, 10th & Alberta has a shorter time of rise compared to the basins in Surrey. The writer mentioned the above parameters, i.e. length of the longest channel and watershed shape, because their significance lies in the distance the storm runoff has to travel to the outlet. Additionally, the time for peak discharge to occur will, to a very large extent, depend on these 2 parameters. One possibility besides the degree of imperviousness in explaining the shorter time of rise would be the efficient sewerage system that 10th & Alberta is equipped with compared to the partial use of open ditches and watercourses of the Surrey watersheds.

4.3.2 Unit hydrograph widths

The unit hydrograph widths are widths at 0, 50 and 75 percent of the peak discharge (Fig. 4.6). Accordingly, it may be represented as W0 (also known as the time base), W50 and W75. They are expressed in minutes. The characteristic widths for a given unit of duration is a function of the unit peak discharge (Corps of Engrs. 1948). The widths are determined largely by the geometry and type of watershed, which are therefore described by the shape of the unit hydrograph. Another way of looking at it is that if the storage between watersheds does not vary significantly then a strong correlation among the widths would be the result.

From Table 4.6, the distribution of unit hydrograph widths are plotted in Fig. 4.7 against the maximum peak flow per unit area. A best fit line is drawn through the W0, W50 and W75 accordingly. A number of observations can be obtained from Fig. 4.7.

Firstly, the correlation of data for W0 is the weakest compared to the other 2 width values. This is because base widths essentially depend on 2 factors: the unit duration or duration of rainfall excess and, the size of the watersheds. A longer unit duration will result in longer base widths, as will those from larger watersheds.

Secondly, the data scatter for W50 and W75 varies, but quite consistently with the magnitude of peak discharge. The relationship is inverse between peak discharge per unit area and widths. This characteristic is attributed to the fact that the area of the graph is constant (1 cm of rainfall excess), and that the shape is fundamentally triangular.

Thirdly, there is a distinct difference in the scatter of data between the City of Vancouver watersheds and Surrey watersheds. Unlike the rest, Robson Creek records a relatively longer base width but with very narrow widths at W50 and W75. The narrow widths at W50 and W75 seems to suggest the rapidity at which

Watershed	Storm No.	W_o (mins.)	W_{50} (mins.)	W_{75} (mins.)
12th & MacKenzie	1	60	37	20
	3	70	34	19
	4	90	38	22
10th & Alberta	1	105	32	17
	4	105	39	20
	5	90	35	23
Robson Creek	1	210	47	29
	2	225	48	27
Hyland Creek	1	210	127	83
	3	240	126	76

Table 4.6 30-minute unit hydrograph widths

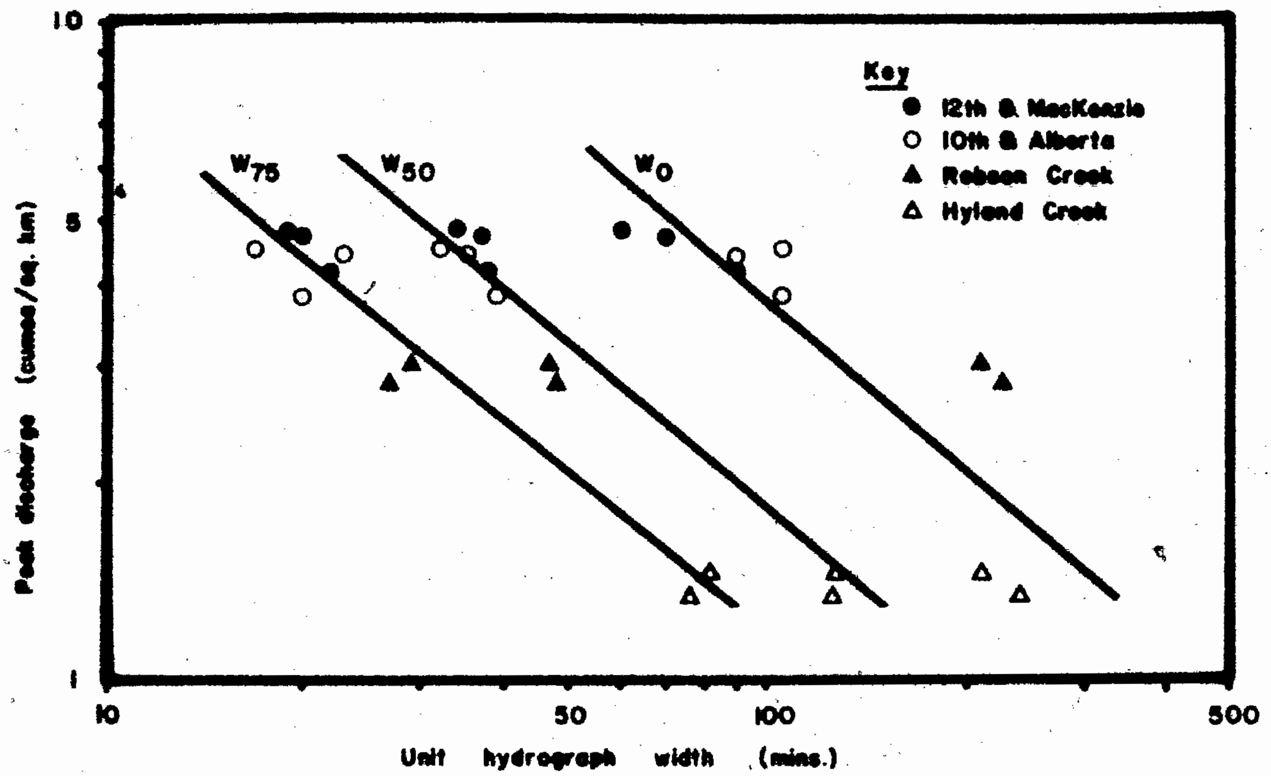


Fig. 4.7 Relationship between unit hydrograph widths and unit peak discharge.

storm runoff is conveyed to the outlet. This has actually been revealed by the shape of the hydrograph as stated earlier. On the other hand, Hyland Creek records relatively broader W50 and W75 widths, which seems to suggest a higher storage capacity within the watersheds.

Lastly, the correlation weakens as we descend to widths at W50 and W0 for all data points. The reason is perhaps not hard to find. Since the widths are another interpretation of the shape of the unit hydrograph, it is not suprising that this factor, i.e. shape, is the principal determinant of the width values. To a very large extent, the shape depends on the amount of storage that occurs. This is to say that a hydrograph with . broad widths at W0, W50 and W75 will result. Therefore, because the storage capacity varies for the watersheds, this is reflected by the wider variations of data points at W0 and W50.

4.4 Runoff coefficients

The determination of rainfall excess or effective rainfall will yield, in turn, the direct runoff or, indirectly, the surface loss for a given storm.

Generally, the types of losses that occur in urban and natural watersheds are the same except that one may be more dominant than the other. Three principal types of losses can be recognised; i) depression storage; ii) infiltraticn into the subsurface soil and; iii) interception by vegetal cover.

a) Depression storage - This is water stored in natural depressions as well as those that are man-made. Man-made depressions include partly depressed sidewalks, vacant lots and carefully graded lawns. The water trapped is ultimately disposed by evaporation and infiltration. Some investigators have assumed overland flow to commence only when all depressions are filled, but it is more reasonable to assume this only if the larger depressions are grouped near the downstream end of all overland flow strips (Tholin and Keifer 1960).

b) Infiltration loss - This is the water that infiltrates into the subsurface soil. Infiltration losses will depend largely on the type of soil within the area. Infiltration rates increase from fine to coarse textured soils. Generally however, the initial infiltration capacity is high, and is gradually lowered until it reaches a constant rate as the rain continues. The differences in and within watersheds have been assumed to be minimal because of the distribution of the surficial deposits (see Fig. 2.2 and Table 2.1). Additionally, it may be interesting to note that infiltration capacities are actually being modified in urban areas because of frequent disturbances from well maintained lawns to areas used by heavy machineries. Hence, only the open spaces which are left undisturbed could a reliable range of infiltration for a particular soil texture be estimated and used.

c) Interception loss - This is storage and retention by and on vegetation. Of course interception for bare ground and impervious surfaces is zero. Therefore, generally speaking, interception will depend on the amount of vegetation left in the urban watersheds.

In the event of a storm, the loss rate will depend on the storm characteristics, while the actual volume lost will essentially be dependent on the nature of the watersheds in terms of storage capacity, topography, land use, vegetation and seasonality. For urban watersheds, it is reasonable to assume a lesser role played by infiltration and interception in storage capacity because of the presence of roads, parking lots buildings and the like, which, as mentioned earlier, increases the degree of imperviousness of the drainage basins. Conversely, depression storages assume a dominant role in the storage capacity, thus representing the major form of loss in urban basins. This is normally assumed because the modifications in the initial landscape often result in greater areas for pondage.

Despite such losses one would expect that for a given storm, greater storm runoff volumes from urban watersheds compared to rural or natural ones because of the extent of imperviousness rendered as the affected area undergoes development. The volume lost could be expressed as:

Losses = Rainfall - Rainfall excess

or, it may be written as,

Runoff coefficient = Runoff/rainfall (expressed in a ratio or %)

Table 4.7 shows the computed runoff coefficient for the individual storm events. The mean runoff coefficient for 12th & MacKenzie is 19.1%; 10th & Alberta - 16.5%; Robson Creek - 12.7%; Hyland Creek - 9.1%. This means that the losses from storms are greater for areas with less impervious cover (Fig. 4.8). This is logical as most of the rain would be expected to infiltrate or be intercepted and subsequently lost. Somewhat suprising, 12th & Mackenzie has a lesser loss than 10th & Alberta, despite having a smaller proportion of impervious cover. One possibility is that the depression storage component may be higher in 10th & Alberta as a result of having more paved areas around.

Watershed	Storm No.	Losses (mm)	Runoff Rainfall (%)	Mean (%)
(1)	(2)	(3)	(4)	(5)
12th & MacKenzie	1	1.13	11.0	
	2	1.16	8.7	
	3	1.00	21.3	19.1
	4	0.65	36.3	
	5	1.87	18.3	
10th & Alberta	1	4.21	7.9	
	2	2.51	17.7	
	3	1.67	17.7	16.5
	4	0.82	19.6	
	5	1.63	19.7	
Robson Creek	1	1.82	20.5	
	2	9.77	6.1	12.7
	3	4.50	11.4	
Hyland Creek	1	9.08	3.4	
	2	16.43	14.9	
	3	8.79	3.8	9.1
	4	9.11	18.5	
	5	9.20	4.7	

Table 4.7 Runoff-coefficients

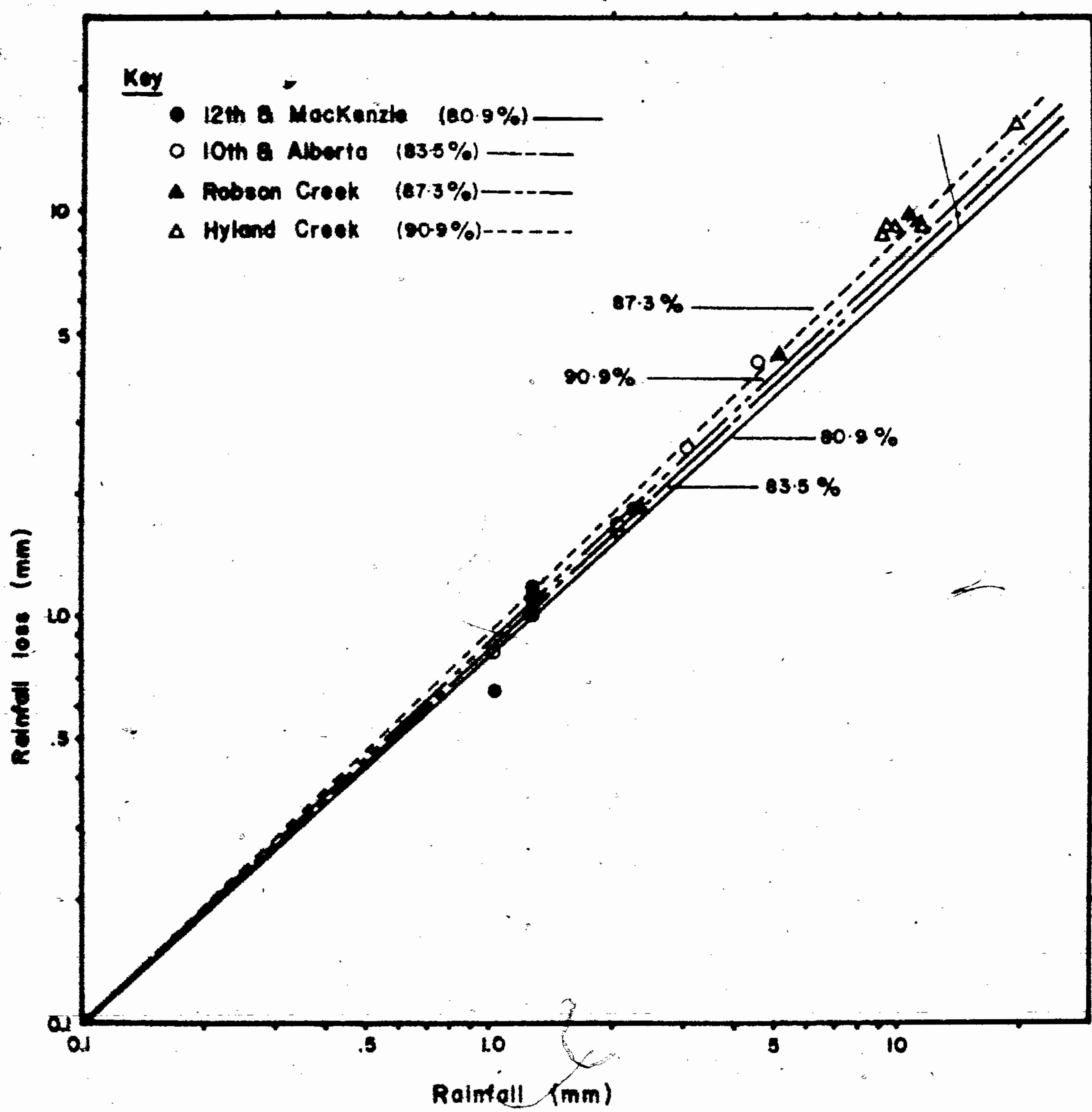


Fig. 4.8 Relationship between rainfall and rainfall losses.

It was also observed that the range of the runoff coefficients from individual storms is wide for all watersheds (12th & MacKenzie - from 8.7% to 36.3%; 10th & Alberta - from 7.95% to 19.7%; Robson Creek - from 6.1% to 20.5% and; Hyland Creek - from 3.4% to 8.5%). This variation may be attributable to the differences in antecedent conditions; that is, drier or wetter antecedent conditions are possible even though care was taken to choose isolated events. As mentioned earlier, because of the type of recording rain gauges used (see Section 3.1.1), it is possible that very light drizzles were not recorded prior to the occurrence of a much heavier storm.

Again, from Fig. 4.8, notice that there is a clear indication that rainfall losses have a direct relationship with storm size. It is most probable that even as the storm progresses, infiltration and depression storage capacities are not fulfilled but continue as the storms chosen are predominantly of short durations. Nevertheless, the results also infer that the rainfall excess increases with storm size. Therefore, even though most investigators have professed the 'significance' of depression storages being filled before rainfall excess can occur, it could be argued here that the time taken to fulfil the depression storages will depend on the amount of such storages, the storm size, duration and intensity.

4.5 Summary

Indeed, from the results of the three unit hydrograph characteritics; i) peak flows; ii) time of rise and; iii) unit hydrographs, some implications of the effect of urban land use on peak flows can be arrived at. However, it must be emphasised that despite the fact that the watersheds vary in land use intensity, they also vary physiographically. Such dissimilarities will also account for variations in peak flows. The runoff coefficients show quite clearly the relationship with the degree of urbanization, though minor variations between individual storms do occur.

CHAPTER FIVE

DISCUSSION

5.1 Introduction

In the preceding chapter, the results from the unit hydrograph, distribution graph and runoff coefficient analyses have been presented. The results obtained are consistent with the general assumption that peak flows will vary directly with urbanization. However, it would be naive to ignore the important influences of the geomorphologic characteristics on the peak flows. This is because some control is exercised on stormflows by the geomorphology for a given basin, which, it is believed to be important in the present study. Therefore, in order to accomodate a complete discussion, this chapter is divided into two main sections: i) land use and; ii) geomorphology.

5.2 Land use

Land use can have the most significant impact on hydrological relationships. Throughout this thesis, the proportion of impervious cover of watersheds has been termed the urbanization index. To a very large extent, the results indicate that the magnitude of unit peak discharge is directly related to the urbanization index. However, as shown earlier, a number of the results do appear to be rather abnormal. It was therefore felt necessary to have a closer examination of land use. Two aspects will be dealt with separately, namely, the degree of urbanization and urban land use pattern.

5.2.1 Degree of urbanization

One of the considerations in this study is that the impervious portions of the watersheds will contribute a certain percentage of rainfall to direct runoff. It is also within this context that the peak flows will be largely influenced by the amount of storm runoff. It is felt that this is where the runoff coefficients are important when assessing the role of the impervious cover on the hydrology of the basins.

From Fig. 5.1, there appears to be a general trend whereby the runoff coefficient correlates positively with the extent of impervious cover. However, it should be noted that the data points have a rather wide scatter for individual storms (Fig. 5.1).

Although several possibilities exist for explaining the variations of runoff coefficient values have been made (see Section 4.4), a more comprehensive assessment of the runoff

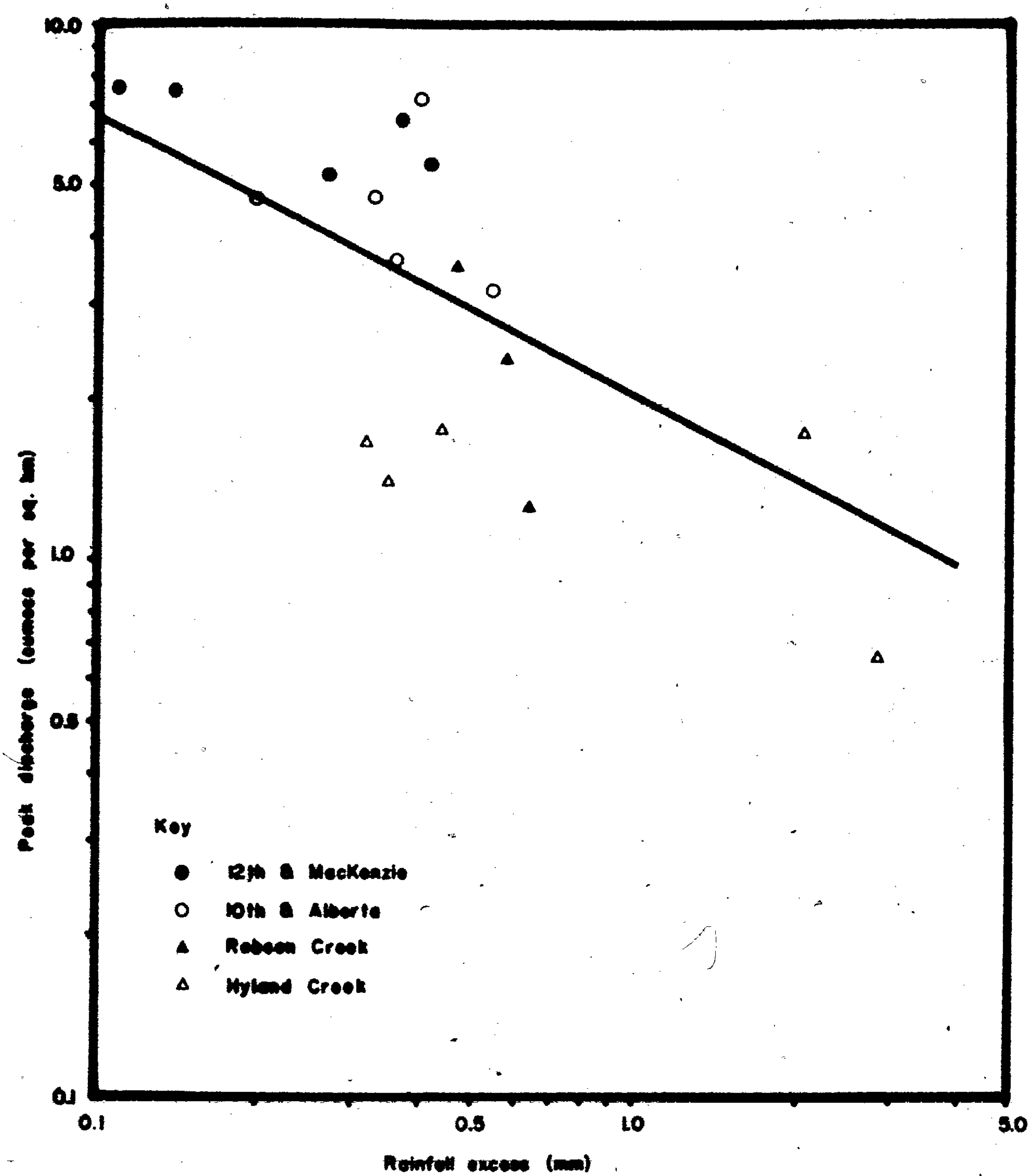


Fig. 5.1 Relationship between rainfall excess and peak discharge per unit area.

coefficients will be made. This is done by comparing the runoff ratios for storms in the studies carried out by Eagleson (1962) in Louisville, Kentucky, and Beard and Chang (1979) in Austin, Texas (Fig. 5.2) compared with the results obtained in this study.

While the comparison of runoff ratios between these three study areas may have been obscured by differences in soil types, the wide scatter of data does not reflect a strong correlation between the impervious cover and runoff ratios. One explanation may be that the effectiveness of retention and depression storages is expected to vary from one place to another despite the proportion of impervious cover. Such a result and its extent will depend on the precise nature of urban development: for instance, large depression storage values may result from parking lots and huge vacant spaces. It is suggested that such storages may be more significant in 10th & Alberta compared to the counterpart, 12th & MacKenzie in the the present study. This is so since the computed mean runoff coefficient value for the former is lower than that for the latter.

Similarly, even though the relationship between rainfall excess and unit peak flows correlates (though not highly) with the proportion of impervious cover (with the consistent exception of 10th & Alberta), the individual basin response does not seem

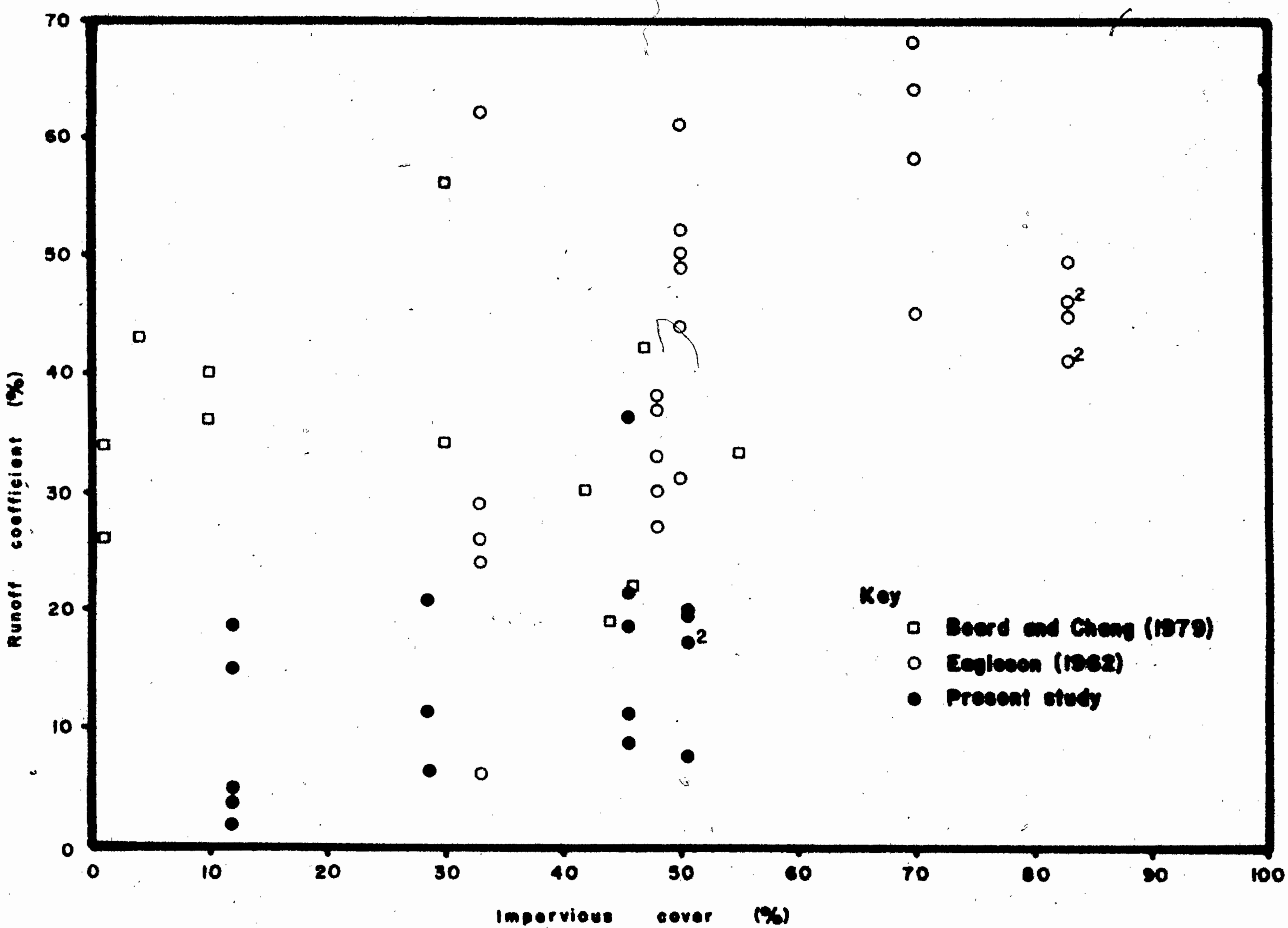


Fig. 5.2 Relationship between runoff coefficients and impervious cover: a comparison with urban watersheds in Louisville, Kentucky and Austin, Texas.

logical (Fig. 5.1): instead of a direct relationship as one would expect it to be, the results tend to be the indirect. It may be that the variations of rainfall excess resulting from a given storm and the poor relationship between rainfall excess and unit peak flow are functions of the storm characteristics.

5.2.2 Urban land use patterns

From earlier hydrograph studies, it was pointed out that urban land use patterns can influence hydrograph shape. Before demonstrating this influence, it may be useful to present a hypothetical situation first.

For this hypothetical situation (Fig. 5.3), three areas for urban development in the watershed have been chosen to demonstrate the effect of development on the hydrograph. In case A (Fig. 5.3A) where development takes place downstream, one would expect a very quick time of rise in the hydrograph because rainfall excess from the impervious areas (principally), aided by the installed storm sewers, takes a shorter time and distance to travel to the outlet. Storm runoff from further upstream will take a longer time to do so. As a result, the hydrograph recorded is one which is skewed to the left. The gentle sloping recessional limb indicates the gradual inflow of storm runoff from upstream. In case B (Fig. 5.3B),

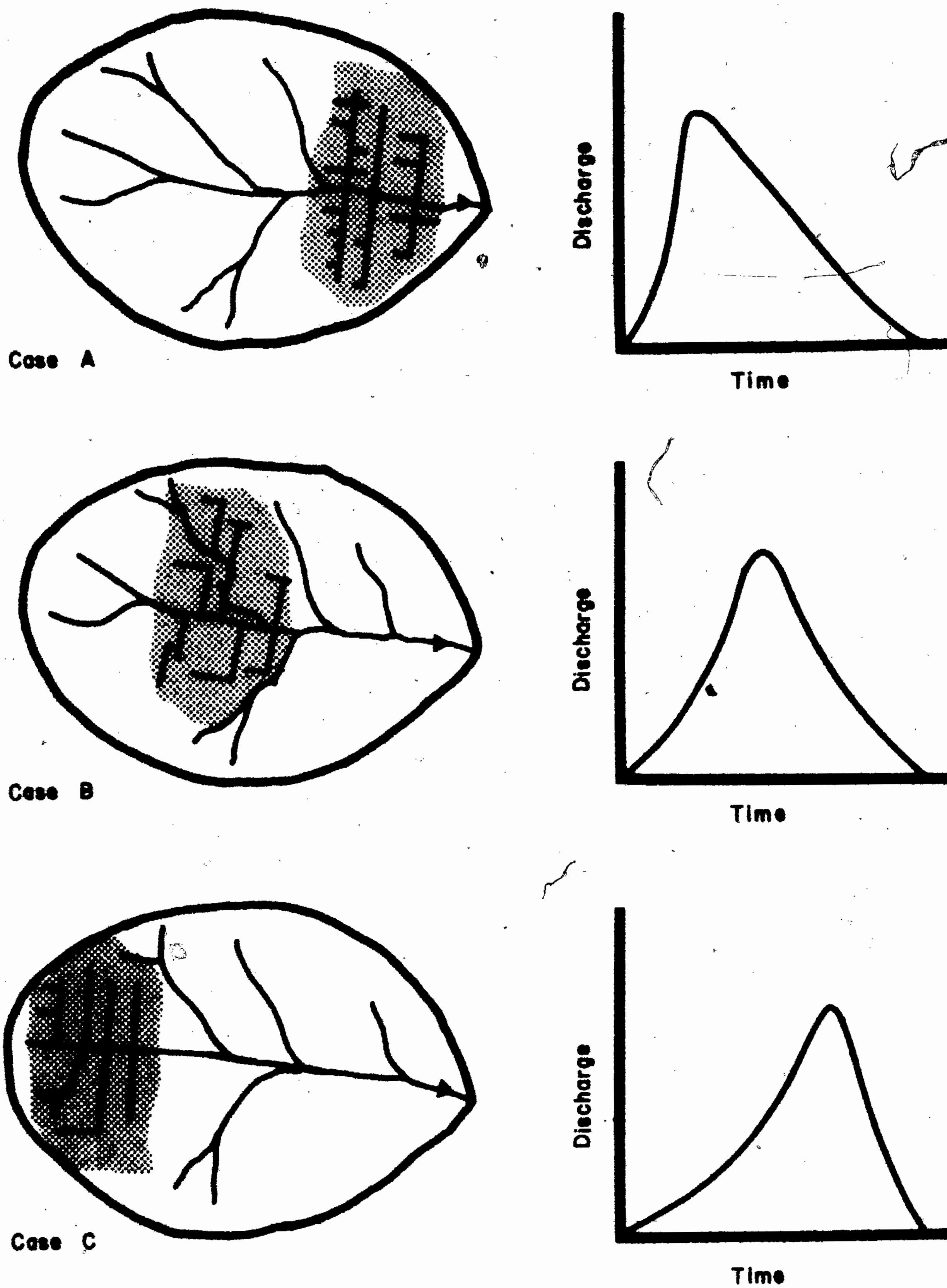


Fig. 5.3 Hypothetical development in a watershed with schematic hydrographs.

where development is assumed to have taken place in midstream, the resultant hydrograph peak may appear to be symmetrical with a longer time of rise. In case C, (Fig. 5.3C), the reverse of case A results when development occurs upstream. The resultant hydrograph is one which is skewed to the right.

Hyland Creek in particular, records a somewhat typical hydrograph similar to case A in the hyphothetical situation. Urban development is far more intensive downstream and portions of the midstream, e.g. along 66, 67, 68, 72 Avenues and 128 St. (Fig. 2.9). As a result, storm runoff reaches the outlet rather rapidly while those further upstream will take a longer time to do so. This explains partly for the relatively fast rise for its basin size and still a proportion of 67.5% of runoff volume after the peak (Fig. 4.4).

An analagous effect on the hydrograph will take place if storm movements are taken into consideration. This has been demonstrated by a physical model study by Roberts and Klingeman 1970. Depending on the location of a storm in a drainage basin, the hydrograph response will be similar to that generated by land use patterns. In other words, a similar outcome but resulting from very different processes.

5.3 Geomorphological characteristics

It is not possible within the scope of this study to explain and account for the role played by all the watershed characteristics on the temporal distribution of runoff. However, it is hypothesized that drainage network characteristics and basin shapes of the watersheds, in this study, are major influences on the unit area hydrograph.

5.3.1 Drainage networks: natural and man-made

Other things remaining constant, the rate at which water is conveyed by sewers for a given distance is much faster than the open ditches and watercourses. This is a result of the hydraulic efficiency of sewers in transporting water from one point to the other.

In the Surrey watersheds, the open ditches and watercourses (the difference between an open ditch and watercourses is that the former is man-made while the latter is a natural stream channel) are mostly vegetated. In a field survey, it was noted that the upper reaches of the watercourses are more heavily vegetated than the lower reaches. For the open ditches however, the presence of vegetation varies from one place to another. Vegetation in channels impedes the flow of water which results in a less efficient drainage system. An inspection of the open ditches also reveals an interesting characteristic - ponding. Parts of the ditches, especially those with depressions are excellent detention storage features (Fig. 2.8a): the water trapped does not contribute to direct runoff. Instead, it represents a loss thereby reducing storm runoff volumes.

To a very large extent the storm sewers are responsible for a faster supply of storm runoff (storm sewers form part of the drainage system for Robson Creek--51.4% and; Hyland Creek--41.5% The percentages refer to the proportion of the total drainage network in sewers). From Figs. 4.1 and 4.2, it is shown that the recessional limb of the hydrograph falls quite sharply but then rather gradually towards the end. One suggestion is the possible effect of vegetation in impeding the rate of flow so that even after the peak, water is supplied to the outlet very slowly. Three reasons have been suggested for this characterstic: firstly, a relatively large proportion of the storm runoff has been delivered out of the watershed at approximately the same period of time. This is largely due to the shape factor, which is discussed in section 5.3.3; secondly, the storm sewers may have increased the rate of flow thereby decreasing the time of rise and; thirdly, the retardant effect of the vegetation is manifested in the second half of the hydrograph as the flow is still being maintained but at a very low level. Similarly, Hyland Creek has a gradual but rather uniform recessional limb. A higher proportion of storm runoff (67.5%) is conveyed after the occurrence of peak discharge peak as indicated by the distribution graphs (Fig. 4.4). It is reasoned that, owing partly to the slower rate of flow, much of the storm runoff is distributed in a seemingly constant discharge, which not only results in a lower peak flow but a relatively broader W0, W50 and W75 of the unit hydrographs.

The significant differences of flow rates in open channels compared to sewers has been recognised by many investigators. Espey, et al. (1966) in a study in Austin, Texas, formulated a phi-index (ϕ) based on the amount of vegetation in the channels for their runoff equations. It was defined to account for the efficiency of the conveyance system of the drainage network.

On the other hand, from Figs. 4.1 and 4.2, hydrographs of the completely sewered watersheds (12th & MacKenzie and 10th & Alberta) appears to have very rapid falling limbs. This can be attributed mainly to the efficient sewerage system where almost all of the direct runoff is conveyed out of the watershed very quickly. Therefore, other things remaining constant, a completely sewered watershed will not only have a relatively much shorter time of rise, higher peak flows but also narrower W0, W50 and W75 unit hydrographs.

5.3.2 Drainage density

Drainage density has often been recognised to be very closely related to peak discharge (Langbein, and others 1947; Gregory and Walling 1968) though there have been some contradictory results. Moreover, it has been one of the most commonly used parameters in studies relating hydrologic and geomorphic characteristics of regions. An attempt was made to relate this parameter with peak flow in this study. The measured drainage density for the watersheds is highest for 10th & Alberta (15.7 km/sq.km) followed by 12th & MacKenzie (12.4 km/sq. km), Robson Creek (8.6 km/sq. km) and Hyland Creek (5.1 km/sq/ km) (Table 2.5). With reference to Fig. 4\2, it appears that a higher drainage density is not strongly associated with the peak flows. This is so for the case of 10th & Alberta. It appears therefore, other geomorphological characteristics may be controlling the peak flows, and this outcome will be discussed in the following subsection.

5.3.3 Watershed shape

Other things being equal, the rate at which water is conveyed to the main channel and out of the watershed is partly governed by the shape factor. This assertion can be modelled by the hypothetical basins shown in Fig. 5.4. Basin A is an elongated watershed compared to basin B and C which are about rectangular and fan-shaped respectively. The effects of such shapes on peak flows, assuming rainfall and other controls to be the same throughout are suggested by the schematic hydrographs. Theoretically, the hydrograph of a watershed is significantly correlated with the shape of the watershed (Seyhan 1976). The shape of a watershed influences the timing of the arrival of storm runoff to the mouth of the basin.

Quantitative descriptions or expressions of basin shapes have been formulated by various investigators and have been used with varying success. Examples are Horton's form factor (Rf), Miller's circulatory ratio, (Rc), and Schumm's elongation ratio, (Re) (Table 5.1). Determination of basin shapes were done using all three methods but the Rc index was found to be the better basin shape measurement (Table 5.2). The Rf and Re indices require the length of the main channel, Lb, in the computation. The criticism of these measures here lies with the fact that the length of the main channel is either man-made as in the Vancouver watershed or partly aligned as for the case of Surrey watersheds. The point is that the storm sewers are conveniently installed alongside the roads, consequently, the

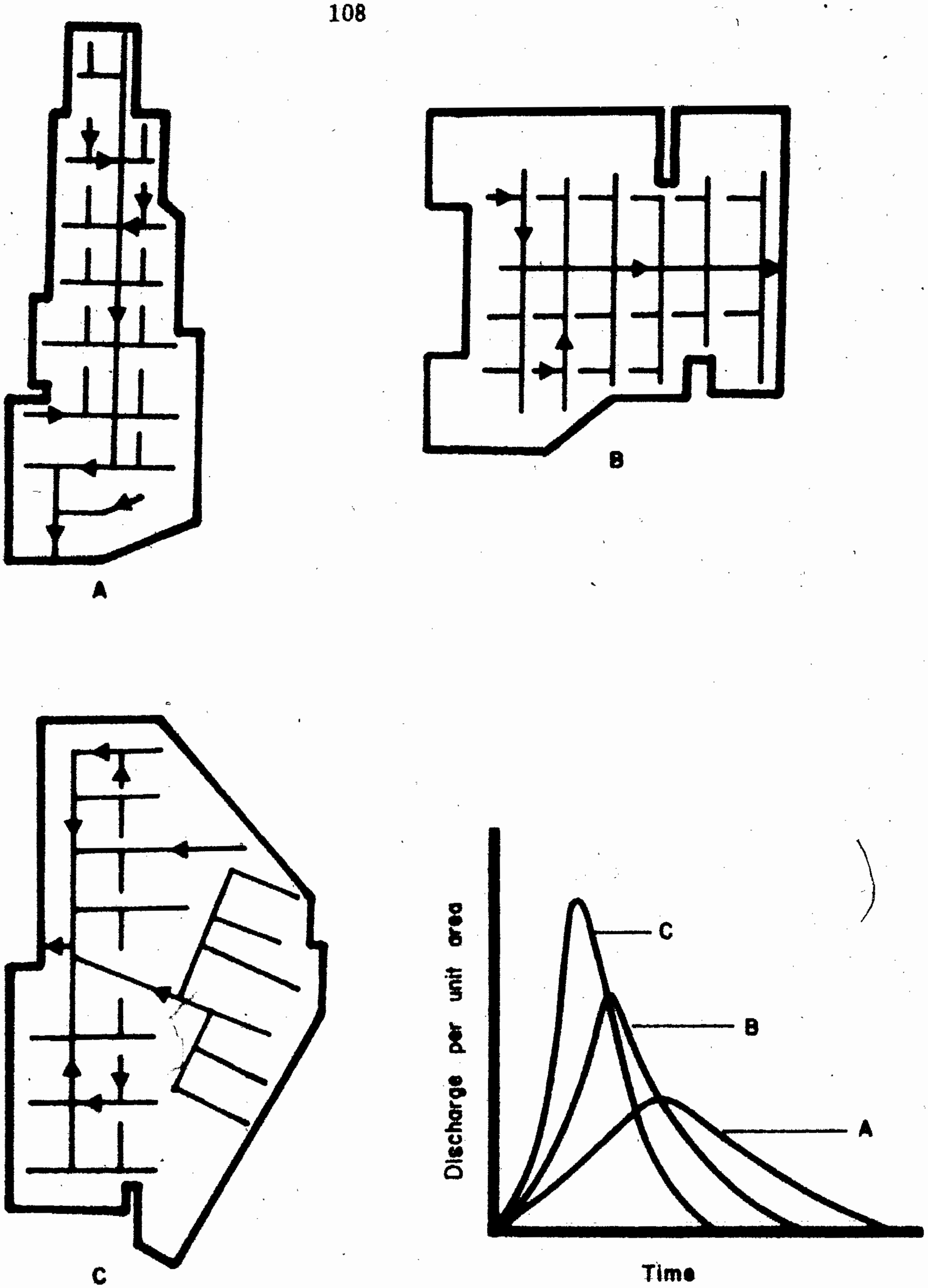


Fig. 5.4 Hypothetical urban watersheds of different shape with schematic hydrographs.

Shape Measurement	Equation	Definition
1) Horton's (1945) form factor	$Rf = \frac{A}{L_b^2}$	Rf = form factor A = area of watershed L_b = length of main channel
2) Miller's (1953) circulatory ratio	$R_c = \frac{A}{A_c}$	R_c = circulatory ratio A = area of watershed A_c = area of a circle having the same perimeter as the watershed
3) Schumm's (1959) elongation ratio	$R_e = \frac{D}{L_b}$	R_e = elongation ratio D = diameter of a circle having the same area as the watershed L_b = length of main channel

Table 5.1 Definition of shape measurements by: Horton (Rf); Miller (R_c) and; Schumm (R_e)

Shape Measurement (1)	12th & MacKenzie (2)	10th & Alberta (3)	Robson Creek (4)	Hyland Creek (5)
R_f	0.35	0.13	4.96	2.74
R_c	0.77	0.34	0.41	0.52
R_e	0.67	0.41	2.51	1.87

Table 5.2 Watershed shape measurement using R_f, R_c and R_e

total main channel length is artificially increased so that it is longer than what one would expect from a natural basin. This possibly explains the irregular results for formulas, Rf and Re. Secondly, Rc is calculated by using the watershed area and area of a circle having the same perimeter as the watersheds, which, to a very large extent, portrays a better quantitative shape expression than Rf and Re. Long and narrow watersheds will have low ratios and vice versa. A general statement can be made here. Because urban watersheds have been changed so much in terms of the original physiography, caution should be exercised when using quantitative measurements once designed for natural watersheds.

The distribution of surface runoff temporal characteristic is shown quite clearly by the distribution graphs (Fig. 4.4). Despite having a 50.5% impervious cover and an efficient drainage system, the peak volume for 10th & Alberta constitutes only 26.4%. However, this value is slightly lower due to the time interval chosen: peak discharge occurs at the 53rd minute while the peak at the distribution graph is at the 60th minute) of the total surface runoff. This is attributable mainly to its long and narrow drainage area (Rc - 0.34) with the longest sewer measured at 2.38 km (see Table 2.6). Since storm runoff will take a longer time to travel to the outlet, the peak is significantly reduced as most of the stormflow in the lower parts would have already been drained before the arrival of those from the upper sections. Robson Creek has an accounted peak flow of 24.8% which is quite close to 10th & Alberta's in spite of its urbanization level and type of drainage system. It is reasoned that because of its somewhat elongated fan-shape, water is supplied from adjacent sides, i.e. northerly and southerly directions at approximately the same time period. As a result, the peak volume is very much increased even though the time of rise is slower during the very early stages of a storm.

The other two watersheds, i.e. 12th & MacKenzie and Hyland Creek are approximately rectangular in shape. With the installation of an efficient, high density sewer system, the time of rise is reduced which partly explains the higher unit peak discharge for 12th & MacKenzie. Conversely, Hyland Creek's lower drainage efficiency and nature of land use explains the recorded lower peak discharge. Therefore, it is important to consider the significance of basin shape in this particular study because of its effect on the time distribution of runoff. This influences the rate of flow, affects the time of rise and determines the shape of the unit hydrograph.

In an attempt to explain the significant relationship between watershed characteristics (apart from impervious cover) and hydrographs, a comparison is made with a study in Louisville District, Kentucky, by Eagleson (1962) (Fig. 5.5). Ten minute unit hydrographs were derived in his study.

Though the soil conditions may be assumed to be dissimilar, the differences in the results due to this circumstance may be kept minimal since the same general principles of the unit hydrograph were applied. From Fig. 5.5, it is apparent that unit peak flows have a direct relationship with the degree of impervious cover (the best fit line for the peak discarges in Eagleson's study appears very much higher in comparison to watersheds in this study because of shorter unit durations used). However, even though watershed No. 4 has an impervious cover of 70%, which is the second highest, appears well below the 'average'. It was explained that the the main cause lies with the mean basin slope. It has the lowest slope value of 0.0012 among other watersheds in the study area. Conversely, even though 10th & Alberta has the highest mean basin slope of 0.03 among the watersheds in this study (see Table 2.5), the main cause for the lower unit peak flow suggested is the basin shape. The constraints imposed by the interrelationship between basin parameters on the hydrology are basically because they are closely knitted and complex.

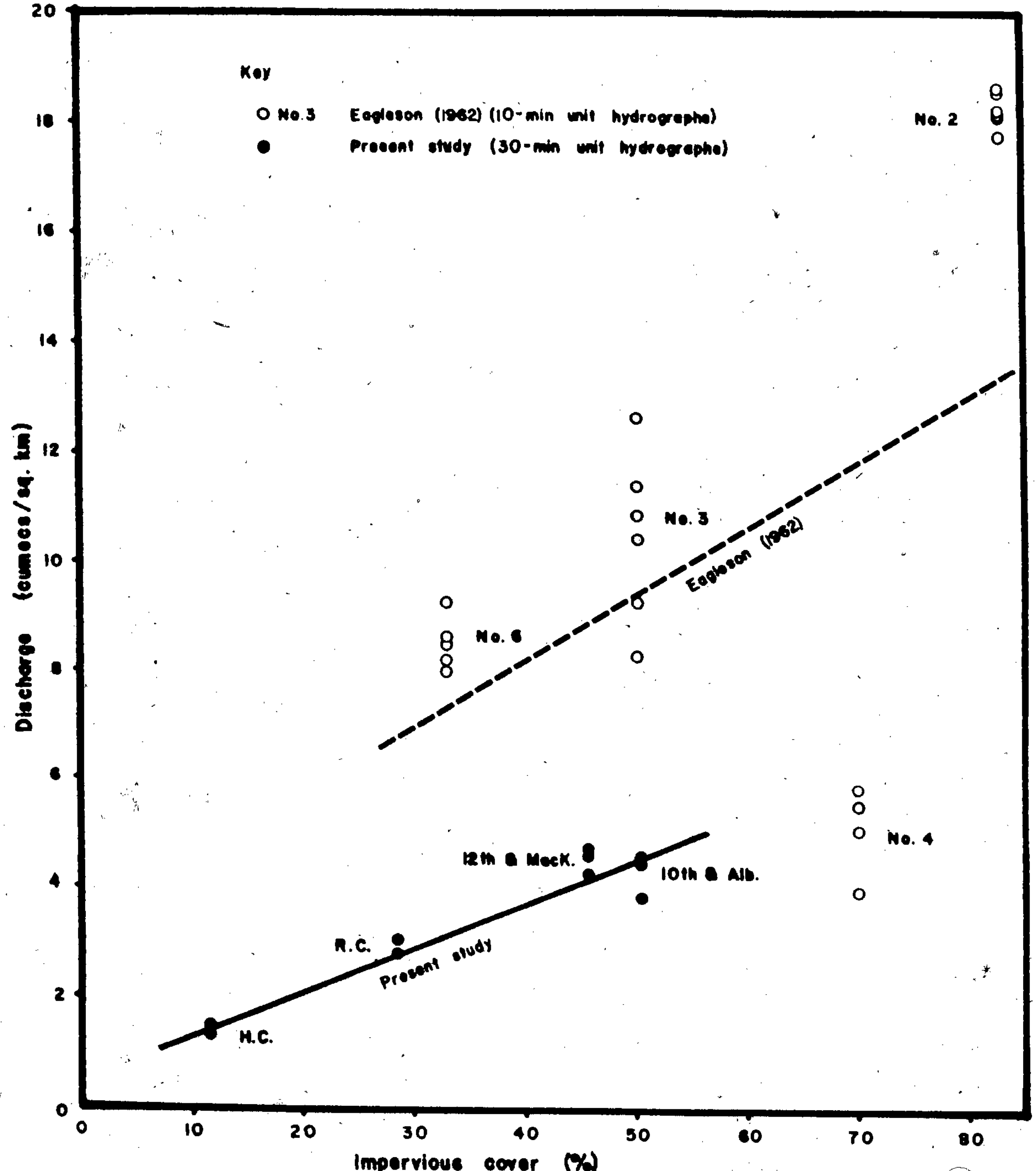


Fig. 5.5. Relationship between peak discharge per unit area and impervious cover: a comparison with urban watersheds in Louisville, Kentucky.

5.4 Summary

This chapter has presented the discussion and the complex interrelationships between the elements of the hydrological regime. While it is not possible within the scope of this study to account for every basin parameter, only the ones which are believed to be the major influence were dealt with. Although comparisons with Eagleson's (1962) and, Beard and Chang's (1979) studies in the United States show relative consistency in terms of the results obtained in this study, there are variations. Additionally, even though it is apparent that urbanization does have a significant effect in determining the unit peak flows, one must be careful not to totally disregard the intimate relationships between the surface properties, watershed geometry and storm characteristics.

CHAPTER SIX

CONCLUSION

6.1 Introduction

The impact of urbanization on peak flows has been examined in four watersheds in the Lower Mainland of British Columbia. The findings are in reasonably good agreement with published results of similar studies by Eagleson (1962), and Beard and Chang (1979). It should be noted, however, that these findings are subject to several important limitations: i) the small sample size of watersheds; ii) the size variation of watersheds and; iii) the uniqueness of watersheds in terms of their geomorphologic and morphologic characteristics. Clearly, care must be exercised when making generalizations from these results. The major findings are summarized below.

6.2 Findings

The principal general conclusion is that, other things constant, urbanization does have a considerable impact on peak flows in the study area.

In order to demonstrate the urban hydrological characteristics, however, it is necessary to control for basin area. It has been shown that peak flow is directly related to basin size. The findings related to urbanization are therefore based on unit yields. The major findings are as follows:

1. Pattern of urbanization - development within the watershed is a partial determinant on the shape of the hydrograph. This is manifested by the pattern of development in Hyland Creek. Development downstream and parts of midstream has resulted in a relatively early peak and a gradual and uniform recessional limb.

2. Imperviousness - in the present set of data, these relationships are complicated by geomorphologic and morphologic variations between the watersheds. As such, several analyses, though descriptive, were undertaken to explain the relationships. The findings are:

a) individual watershed characteristics are important in characterising the shape of the unit hydrographs; hence the resulting peak flows. In this study, for example, the basin shape of 10th & Alberta has, to a very large extent, reduced the magnitude of peak flow and conversely, the basin shape has caused a relatively more concentrated peak for Robson Creek.

b) Antecedent conditions, surface and depression storages affect the rainfall excess for a given storm. Although there is a general pattern whereby rainfall loss correlates positively with the extent of impervious cover, the important factor lies with the effectiveness of impervious area in influencing storm runoff.

3. Drainage characteristics - from field examinations and studies on the unit hydrographs, it appears that open ditches may be quite effective in reducing peak flows. Firstly, ponding results in depressions within the ditches, and secondly, the vegetation has a retardant effect on flows. On the other hand, storm sewers partially account for higher magnitudes of peak flows because of their rapid delivery of water.

4. Storm runoff - it was found that greater volumes of storm runoff do not necessarily mean that a higher peak flow will result. Other conditions remaining the same, peak flows will depend on storm characteristics such as storm intensity and duration.

6.3 Suggestions for further research

This study is limited by a small data set. Additionally, the cross comparison undertaken suffers from dissimilarities in terms of watershed characteristics.

Nevertheless, this study has provided some insights into the complexities involved especially when the geomorphologic and morphologic characteristics are different between watersheds.

It is recommended that more data be collected as they may improve the reliability of comparisons. Another topic given little attention in this study is the comparison of seasonal variations on the hydrographs within and between watersheds. It is felt that this comparison may improve the understanding of the effectiveness of urbanization on storm runoff characteristics.

Appendix A

Impervious cover data

Watershed: 12th & MacKenzie
Map No. 29
Scale: 1" to 200'
Year: 1976
Source: City Planning Dept., Vancouver, B.C.

Map No.	Total Area	No. of Dots Total Impervious Area: Buildings	Pavements + Roads
29	7060	1423	1783
Percentage		20.2%	25.2%
Total Percentage	100%	45.5%	

Watershed: 10th & Alberta
Map. No.: 32
Scale: 1" to 200'
Year 1976
Source: City Planning Dept., Vancouver, B.C.

Map No.	Total Area	No. of Dots Total Impervious Area Buildings	Pavements + Roads
32	20020	4305	5797
Percentage		21.5%	29.0%
Total Percentage	100%	50.5%	

Watershed: Robson Creek
Scale: 1" to 400'
Year: 1977
Source: Engineering Division, Surrey Municipal Hall, B.C.

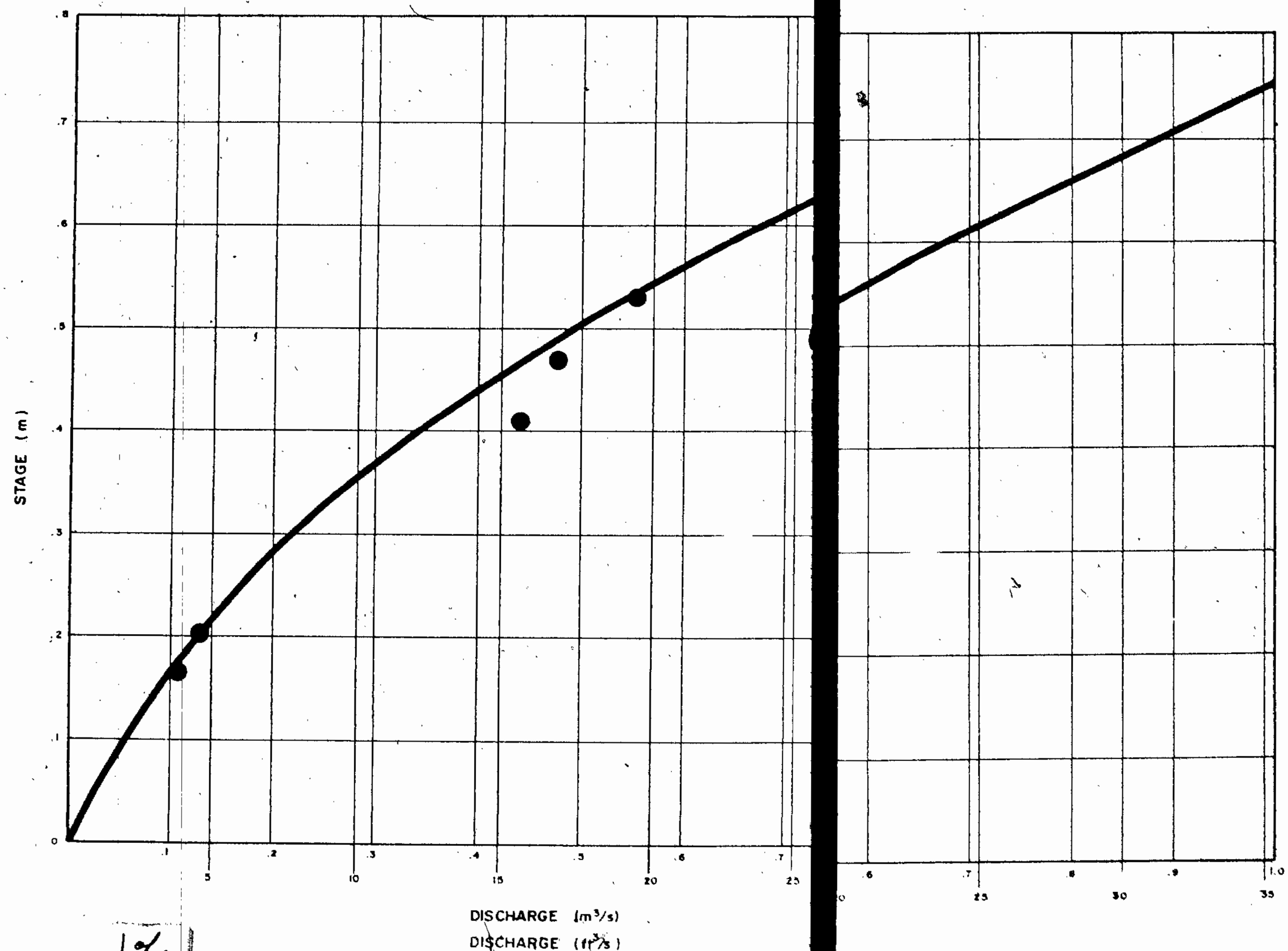
Map No.	Total Area	No. of Dots Total Impervious Area — Buildings	No. of Dots Total Impervious Area — Pavements + Roads
21	6455	775	1086
30	1670	144	185
31	400	46	22
11	410	14	66
10	100	0	12
20	5315	755	980
	14350	1734	2351
Grand Total	14350	4085	
Percentage		12.1%	16.4%
Total Percentage	100%	28.5%	

Watershed: Hyland Creek
Scale: 1" to 400'
Year: 1977
Source: Engineering Division, Surrey Municipal Hall, B.C.

Map No.	Total Area	No. of Dots / Total Impervious Area: Buildings	Pavements + Roads
50	1605	23	50
51	5620	250	445
52	2005	107	245
60	985	36	98
61	17080	717	1252
62	6450	271	401
		1404	2491
Grand Total	33745	3895	
Percentage		4.2%	7.4%
Total Percentage	100%	11.6%	

Appendix B

Stage-discharge curves



1 of

APPENDIX B.1

STAGE-DISCHARGE CURVE
ROBSON CREEK

LEGEND:

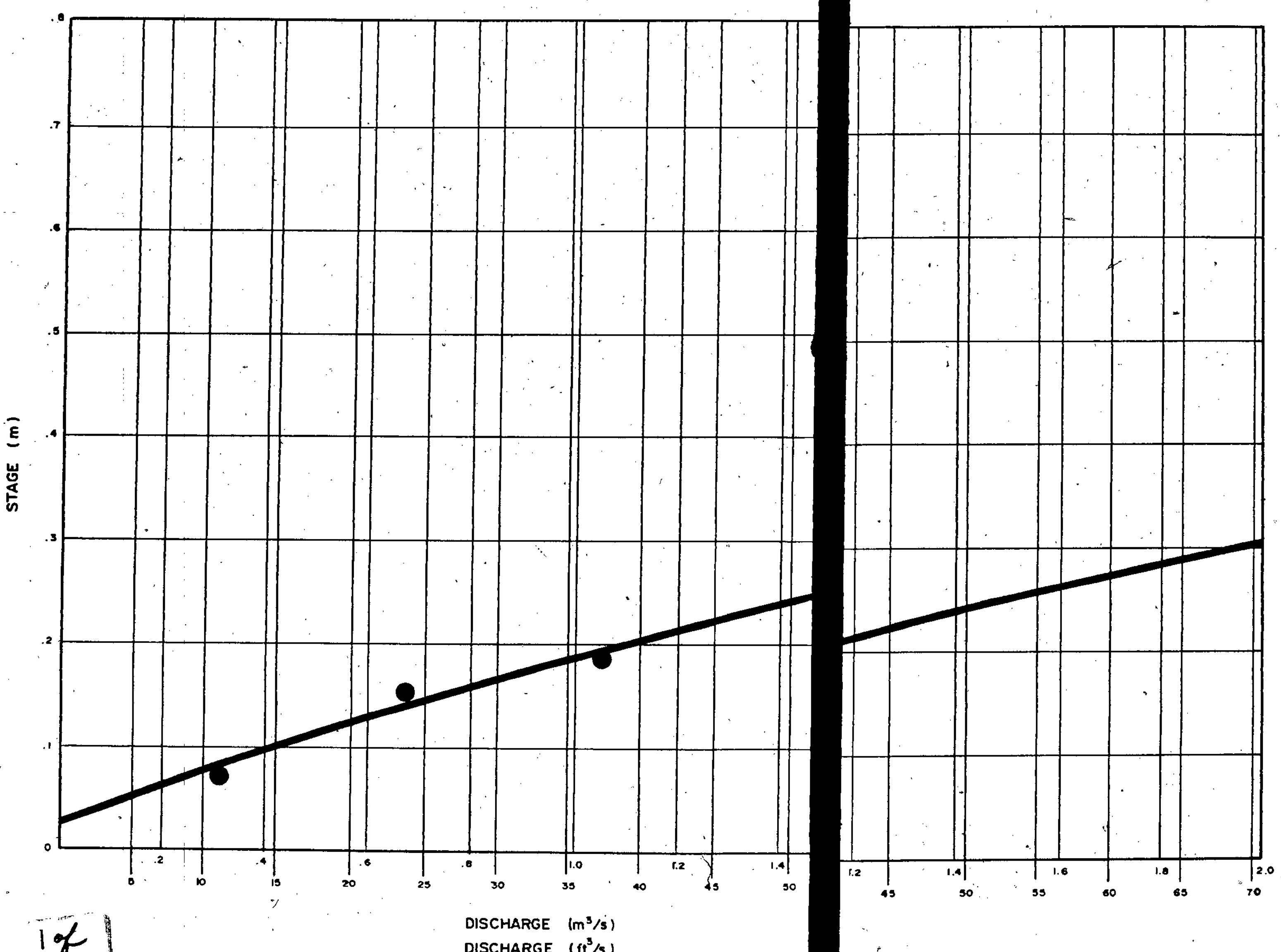
● Points for which current metered flows have been obtained

2 of 2

APPENDIX B.2

STAGE-DISCHARGE CURVE

HYLAND CREEK



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Appendix C

Unit hydrograph data

Watershed: 12th & MacKenzie
Date: 27th February, 1976
Unit Duration: 15 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.027	0.027	0.000	0.000
15	0.027	0.027	0.000	0.000
30	0.054	0.028	0.026	1.734
45	0.040	0.029	0.011	0.734
60	0.032	0.030	0.002	0.133
75	0.028	0.028	0.00	0.00

Date: 16th February, 1977
Unit Duration: 15 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.027	0.027	0.000	0.000
15	0.064	0.027	0.037	1.317
30	0.060	0.027	0.033	1.171
45	0.041	0.027	0.014	0.486
60	0.034	0.028	0.006	0.228
70	0.028	0.028	0.000	0.000

Date: 25th March, 1977
Unit Duration: 15 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.023	0.023	0.000	0.000
15	0.086	0.024	0.063	1.419
30	0.059	0.024	0.035	0.795
45	0.041	0.025	0.017	0.375
60	0.032	0.025	0.007	0.159
75	0.029	0.026	0.003	0.068
90	0.027	0.027	0.000	0.000
105	0.023			

Date: 27th March, 1977
Unit Duration: 15 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.032	0.032	0.000	0.000
15	0.055	0.032	0.023	0.520
30	0.091	0.033	0.059	1.340
45	0.055	0.033	0.022	0.490
60	0.046	0.033	0.013	0.290
75	0.041	0.034	0.008	0.170
90	0.036	0.034	0.002	0.050
105	0.034	0.034	0.000	0.000

Date: 17th March, 1976
Unit Duration: 30 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.019	0.000	0.000	0.000
15	0.019	0.000	0.000	0.000
30	0.027	0.019	0.008	0.660
45	0.041	0.020	0.021	1.830
60	0.023	0.021	0.002	0.330
75	0.022	0.022	0.000	0.000

Watershed: 10th & Alberta
Date: 4th February, 1977
Unit Duration: 30 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.069	0.069	0.000	0.000
15	0.069	0.069	0.000	0.000
30	0.092	0.069	0.023	0.960
45	0.138	0.070	0.068	2.840
60	0.121	0.072	0.051	2.130
75	0.104	0.071	0.033	1.380
90	0.094	0.072	0.022	0.920
105	0.081	0.073	0.008	0.330
120	0.075	0.075	0.000	0.000

Date: 7th November, 1976
Unit Duration: 30 minutes

00	0.012	0.012	0.000	0.000
15	0.012	0.012	0.000	0.000
30	0.013	0.012	0.010	0.280
45	0.106	0.016	0.100	3.340
60	0.109	0.018	0.090	2.500
75	0.013	0.019	0.060	1.670
90	0.000	0.020	0.030	0.830
105	0.011	0.022	0.010	0.280
120	0.013	0.023	0.000	0.000

Date: 26th March, 1977
Unit Duration: 15 minutes

00	0.069	0.069	0.000	0.000
15	0.069	0.069	0.000	0.000
30	0.069	0.069	0.000	0.000
45	0.115	0.070	0.045	1.049
60	0.282	0.073	0.209	4.870
75	0.150	0.076	0.074	1.724
90	0.104	0.079	0.025	0.583
105	0.092	0.081	0.011	0.256
120	0.085	0.085	0.000	0.000

Date: 18th November, 1976
Unit duration: 15 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit hydrograph (1 cm of rain-fall excess)
00	0.042	0.042	0.000	0.000
15	0.096	0.025	0.016	0.850
30	0.132	0.104	0.028	1.500
45	0.192	0.146	0.046	2.480
60	0.168	0.121	0.047	2.550
75	0.132	0.106	0.026	1.400
90	0.120	0.105	0.014	0.750
105	0.108	0.104	0.004	0.320
120	0.096	0.096	0.000	0.000

Date: 20th December, 1977
Unit duration: 30 minutes

00	0.048	0.048	0.000	0.000
15	0.053	0.048	0.005	0.170
30	0.058	0.053	0.010	0.330
45	0.060	0.057	0.023	0.810
60	0.084	0.061	0.069	2.500
75	0.144	0.068	0.076	2.720
90	0.156	0.070	0.058	2.090
105	0.120	0.072	0.039	1.400
120	0.108	0.075	0.025	0.900
135	0.091	0.077	0.014	0.500
150	0.084	0.078	0.006	0.200
165	0.084	0.084	0.000	0.000

Watershed: Robson Creek
Date: 23rd March, 1977
Unit Duration: 15 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.07			
15	0.07	0.07	0.00	0.00
30	0.11	0.07	0.04	0.77
45	0.35	0.07	0.28	5.53
60	0.39	0.08	0.31	6.70
75	0.29	0.08	0.21	4.23
90	0.19	0.08	0.11	2.20
105	0.15	0.08	0.07	1.36
120	0.13	0.08	0.05	0.93
135	0.12	0.09	0.03	0.69
150	0.12	0.09	0.03	0.56
165	0.11	0.09	0.02	0.43
180	0.11	0.09	0.01	0.30
195	0.10	0.09	0.01	0.16
210	0.09	0.09	0.00	0.00

Date: 26th March, 1977
Unit Duration: 30 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.06	0.06	0.00	0.00
15	0.07	0.06	0.01	0.13
30	0.37	0.06	0.30	4.50
45	0.45	0.06	0.39	5.75
60	0.33	0.07	0.26	3.94
75	0.24	0.07	0.17	2.58
90	0.20	0.07	0.13	1.89
75	0.17	0.07	0.10	1.40
90	0.13	0.07	0.06	0.88
105	0.12	0.07	0.04	0.63
120	0.10	0.07	0.03	0.39
135	0.10	0.08	0.02	0.29
150	0.09	0.08	0.01	0.12
165	0.08	0.08	0.01	0.07
180	0.08	0.08	0.00	0.00

Date: 25th October, 1977
Unit Duration: 45 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rainfall excess)
00	0.07	0.07	0.00	0.00
15	0.10	0.07	0.02	0.38
30	0.35	0.08	0.27	4.57
45	0.35	0.08	0.27	4.65
60	0.29	0.08	0.21	3.55
75	0.25	0.08	0.17	2.83
90	0.21	0.09	0.13	2.11
105	0.18	0.09	0.09	1.48
120	0.15	0.09	0.06	1.01
135	0.14	0.09	0.04	0.72
150	0.13	0.10	0.03	0.56
165	0.12	0.10	0.03	0.46
180	0.11	0.10	0.01	0.17
195	0.11	0.10	0.01	0.09
210	0.00	0.11	0.00	0.00

Watershed: Hyland Creek
Date: 25th December, 1976
Unit Duration: 30 mins.

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.16	0.16	0.00	0.00
15	0.27	0.17	0.10	3.22
30	0.32	0.17	0.15	4.57
45	0.37	0.18	0.19	5.92
60	0.41	0.19	0.22	6.95
75	0.41	0.20	0.22	6.73
90	0.41	0.20	0.21	6.51
105	0.40	0.21	0.19	6.01
120	0.39	0.22	0.17	5.29
135	0.36	0.22	0.14	4.29
150	0.34	0.23	0.11	3.35
165	0.31	0.24	0.07	2.25
180	0.29	0.25	0.04	1.25
195	0.28	0.25	0.02	0.63
210	0.27	0.27	0.00	0.00

Date: 11th February, 1977
Unit Duration: 150 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.19	0.19	0.00	0.00
15	0.19	0.19	0.00	0.00
30	0.19	0.19	0.00	0.00
45	0.22	0.20	0.02	0.07
60	0.23	0.20	0.03	0.11
75	0.27	0.20	0.07	0.24
90	0.30	0.21	0.09	0.33
105	0.33	0.21	0.12	0.44
120	0.36	0.21	0.15	0.53
135	0.40	0.21	0.19	0.66
150	0.47	0.22	0.25	0.88
165	0.51	0.22	0.29	1.02
180	0.59	0.22	0.37	1.30
195	0.67	0.22	0.45	1.57
210	0.78	0.23	0.56	1.95
225	0.85	0.23	0.62	2.18
240	1.01	0.23	0.78	2.73
255	1.08	0.23	0.85	2.97
270	1.08	0.24	0.85	2.96
285	p.08	0.24	0.84	2.95
300	1.08	0.24	0.84	2.94
315	1.06	0.24	0.82	2.86
330	0.94	0.25	0.69	2.43
345	0.93	0.25	0.69	2.39
360	0.88	0.25	0.63	2.19
375	0.84	0.25	0.59	2.06
390	0.77	0.26	0.52	1.80

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduce Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
405	0.75	0.26	0.49	1.72
420	0.71	0.26	0.45	1.58
435	0.69	0.26	0.43	1.50
450	0.64	0.27	0.38	1.31
465	0.60	0.27	0.33	1.16
480	0.58	0.27	0.31	1.09
495	0.53	0.27	0.26	0.90
510	0.51	0.28	0.24	0.82
525	0.50	0.28	0.22	0.76
540	0.45	0.28	0.17	0.60
555	0.42	0.28	0.14	0.48
570	0.41	0.29	0.13	0.44
585	0.39	0.29	0.10	0.36
600	0.36	0.29	0.07	0.24
615	0.36	0.29	0.07	0.23
630	0.33	0.30	0.04	0.12
645	0.31	0.30	0.01	0.03
660	0.30	0.30	0.00	0.01
675	0.30	0.30	0.00	0.00

Date: 28th February, 1977
Unit Duration: 30 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.30			
15	0.30			
30	0.30	0.30	0.00	0.00
45	0.37	0.30	0.07	1.97
60	0.43	0.30	0.12	3.52
75	0.48	0.30	0.18	5.06
90	0.53	0.30	0.23	6.46
105	0.53	0.31	0.23	6.44
120	0.53	0.31	0.22	6.29
135	0.51	0.31	0.20	5.75
150	0.48	0.31	0.17	4.77
165	0.44	0.31	0.13	3.65
180	0.43	0.31	0.12	3.29
195	0.41	0.31	0.10	2.83
210	0.39	0.31	0.07	2.09
225	0.36	0.31	0.05	1.34
240	0.33	0.31	0.02	0.46
255	0.33	0.32	0.01	0.29
270	0.32	0.32	0.00	0.00

Date: 4th April, 1978
Unit Duration: 135 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rainfall excess)
00				
15				
30	0.27	0.27	0.00	0.00
45	1.11	0.27	0.83	3.99
60	1.49	0.28	1.21	5.82
75	1.79	0.28	1.51	7.24
90	1.97	0.29	1.68	8.08
105	1.79	0.29	1.50	7.20
120	1.32	0.29	1.03	4.92
135	1.24	0.30	0.94	4.52
150	1.02	0.30	0.72	3.45
165	0.88	0.31	0.57	2.73
180	0.76	0.31	0.45	2.14
195	0.67	0.31	0.36	1.71
210	0.58	0.32	0.26	1.26
225	0.48	0.32	0.15	0.73
240	0.39	0.33	0.06	0.28
255	0.36	0.33	0.03	0.14
270	0.34	0.34	0.00	0.00

Date: 20th October, 1978
Unit Duration: 75 minutes

Time (mins.)	Discharge Ordinate (cumecs.)	Baseflow Ordinate (cumecs.)	Reduced Ordinate (cumecs.)	Unit Hydrograph (1 cm of rain-fall excess)
00	0.30	0.30	0.00	0.00
15	0.36	0.30	0.06	1.33
30	0.60	0.30	0.30	6.66
45	0.67	0.30	0.37	8.21
60	0.67	0.30	0.37	8.22
75	0.67	0.30	0.37	8.23
90	0.67	0.30	0.37	8.21
105	0.60	0.30	0.30	6.51
120	0.41	0.30	0.11	2.33
135	0.39	0.30	0.09	1.89
150	0.33	0.30	0.03	0.56
165	0.31	0.30	0.01	0.16
180	0.30	0.30	0.00	0.00

Appendix D

30-minute unit hydrograph data

by summation method

Unit hydrographs for the watersheds were derived from the procedures outlined in Chapter three. As caution was taken in choosing the appropriate events (see Section 3.4) only a small number of events were selected (Table 4.1). Unit hydrographs were later derived from these storm events (Table 4.2). However, while proceeding into the averaging procedure of the 30-minute unit hydrograph by the summation method. Further consideration was undertaken to examine the unit hydrograph before doing so. Elimination of events considered not appropriate was therefore done. The eliminated events are those which are caused by uneven rainfall intensities. It is detected by the observation of the rising limbs. Either the initial stages of the rising limb inclines steeply and tapers gradually as it peaks or a very slowly rising limb at the initial stages and a steep inclination as it peaks are two forms recognised. The first case implies a high intensity storm at the early stages while the second case implies one which has a late high intensity. Consequently, the events which are believed to be the result of uniform intensities are chosen for averaging the 30-minute unit hydrographs for the respective watersheds. Although the number of cases are limited, the data are considered to be of good quality because they are well-defined and consistent.

Watershed: 12th & MacKenzie

Date: 27th Feb. 1976	Time (mins.)	Ordinates of 15-min. unit hydrographs (cumecs.)		Sum (cumecs.)	30-min. Unit Hydrograph (cumecs.)
	0	0.00		0.00	0.00
	15	1.73	0.00	1.73	0.67
	30	0.73	1.73	2.47	1.23
	45	0.13	0.73	0.87	0.43
	60	0.00	0.13	0.13	0.07
	75		0.00	0.00	0.00
16th Feb. 1977	0	0.00		0.00	0.00
	15	1.32	0.00	1.32	0.66
	30	1.17	1.32	2.49	1.24
	45	0.49	1.17	1.66	0.83
	60	0.23	0.49	0.71	0.36
	75	0.00	0.23	0.23	0.11
	90		0.00	0.00	0.00
25th Mar. 1977	0	0.00		0.00	0.00
	15	1.42	0.00	1.42	0.71
	30	0.79	1.42	2.21	1.11
	45	0.38	0.79	1.17	0.59
	60	0.16	0.38	0.53	0.27
	75	0.07	0.16	0.23	0.11
	90	0.00	0.07	0.07	0.03
	105		0.00		0.00

Watershed: Robson Creek

Date: 23rd Mar. 1977	Time (mins.)	Ordinates of 15-min. unit hydrographs (cumecs.)		Sum (cumecs.)	30-min. Unit Hydrograph (cumecs.)
	0	0.00		0.00	0.00
	15	0.77	0.00	0.77	0.39
	30	5.53	0.77	6.30	3.15
	45	6.70	5.53	12.23	6.12
	60	4.23	6.70	10.93	5.47
	75	2.20	4.23	6.43	3.22
	90	1.36	2.20	3.56	1.78
	105	0.93	1.36	2.29	1.15
	120	0.69	0.93	1.62	0.81
	135	0.56	0.69	1.25	0.63
	150	0.43	0.56	0.99	0.50
	165	0.30	0.43	0.73	0.37
	180	0.16	0.30	0.46	0.23
	195	0.00	0.16	0.16	0.08
	210		0.00	0.00	0.00

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