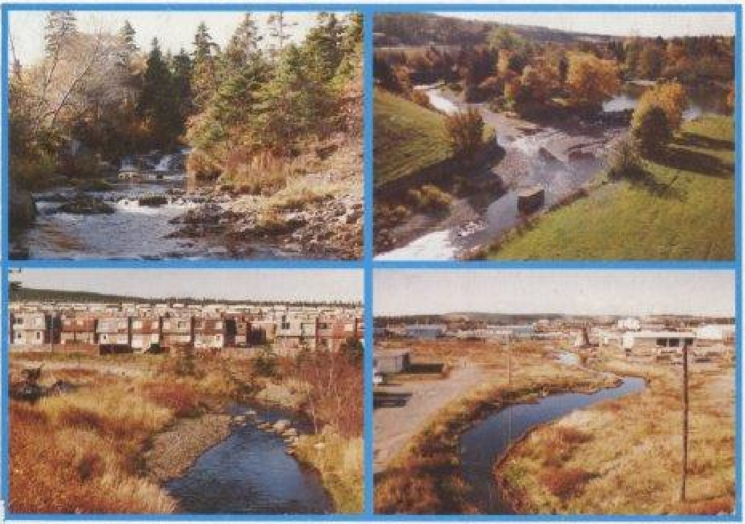
Government of Newfoundland and Labrador
Department of Environment
Water Resources Division
St. John's, Newfoundland

Government of Canada
Environment Canada
Inland Waters Directorate
Dartmouth, Nova Scotia
National Water Research Institute
Burlington, Ontario

STORM RUNOFF STUDY OF NEWTOWN URBAN CATCHMENT



Urban Hydrology Study of the Waterford River Basin

TECHNICAL REPORT No.
UHS-WRB 1.5

STORM RUNOFF STUDY OF

NEWTOWN URBAN CATCHMENT

by

J. Marsalek[1], U. Panu[2] and H.Y.F. Ng[1]

[2] Water Resources Division
Department of Environment
Government of Newfoundland
and Labrador
St. John's, Newfoundland

[1] Environmental Hydraulics Section
Hydraulics Division
National Water Research Institute
Canada Centre for Inland Waters
Burlington, Ontario, Canada

March 1985

Environment Canada / Environnement Canada

Canada Centre for Inland Waters
867 Lakeshore Road
P.O. Box 5050
Burlington, Ontario, Canada
L7R 4A6

Your file / Votre référence

Our file / Notre référence

September 20, 1986

Dr. Wasi Ullah, Chairman
Technical Committee
Waterford River Basin Study
Newfoundland Department of Environment
P.O. Box 4750
St. John's, Newfoundland
A1C 5T7

Dear Dr. Ullah:

On behalf of the study team investigating urban runoff in the Waterford River Basin, I am pleased to submit herewith the final version of our report entitled "Storm Runoff Study of Newtown Urban Catchment" which was approved for publication during the meeting of the Technical Committee in August, 1986.

Yours sincerely,

Jiri Marsalek

Jiri Marsalek
Hydraulics Division
National Water Research Institute

encl.

:ns

ABSTRACT

A study of urban runoff in the St. John's area was conducted as part of comprehensive investigations of the effects of urbanization on water resources in the Waterford River Basin, Newfoundland. The main objective of this study, which included both field and analytical investigations, was to develop principles of hydrological design of urban drainage in Newfoundland. In field investigations, rainfall, runoff and its composition were monitored in a 13 ha residential subdivision in the St. John's area. The collected data were used to evaluate three runoff computational procedures - the rational method, the Illinois Urban Drainage Simulator (ILLUDAS) and the Storm Water Management Model (SWMM). Each of these procedures produced good results within their operational limits. The rational method somewhat underestimated the observed or extrapolated runoff peak flows. Satisfactory results were obtained by applying this method in conjunction with locally-derived composite runoff coefficients and inlet times, and a partly-contributing catchment area. Both runoff hydrograph models, the ILLUDAS and SWMM, reproduced the observed runoff hydrographs fairly well. The modelling results were particularly good for intermediate storms during which practically all runoff was generated on impervious elements. Only for a few most severe storms it was necessary to improve the modelling results by accounting for antecedent moisture conditions. Observed characteristics of runoff quality indicated pollution arising from urban activities. Such indications were particularly strong for zinc and lead whose concentrations exceeded the safe guidelines for aquatic life. Recommendations for addressing runoff problems in future urban developments were given. These included various runoff management strategies and technical guidelines for drainage design.

RÉSUMÉ

Dans le cadre des enquêtes exhaustives sur les effets de l'urbanisation sur l'alimentation du bassin de la rivière Waterford (Terre-Neuve), on a effectué une Etude sur l'écoulement urbain dans la région de St. John's. Elle comporte deux volets: enquête sur le terrain et analyse de données. Son principal objectif est d'élaborer des principes destinés à régir la planification hydrologique ou le drainage en zone urbaine à Terre-Neuve. Au cours des enquêtes sur le terrain, on a surveillé l'écoulement et la composition des eaux pluviales à l'exutoire de drainage d'une zone résidentielle de 13 hectares à St. John's. Les données récoltées ont servi à évaluer la valeur de trois méthodes de calcul de l'écoulement: la méthode rationnelle, le Illinois Urban Drainge Simulator ou ILLUDAS (simulateur de drainage urbain mis au point en Illinois) et le modèle de gestion des eaux pluviales (SWMM). Les trois méthodes ont produit des résultats satisfaisants, compte tenu de leurs limites d'application. La méthode rationnelle sous-estimait quelque peu les débits de pointe observés ou calculés par extrapolation. Les meilleurs résultats ont été atteint en utilisant des coefficients d'écoulement composés et des temps d'entrée basés sur des données locales, ainsi qu'une partie des caractérisques du bassin d'alimentation. Les deux modèles d'écoulement, l'ILLUDAS et le SWMM, ont reproduit assez fidèlement les caractéristiques hydrographiques de l'écoulement. Les résultats ont été particulièrement bons dans le cas d'orages intermédiaires pour lesquels pratiquement tout l'écoulement provenait de surfaces imperméables. Ce n'est que dans le cas de quelques orages violents qu'il a été nécessaire de modifier les résultats en tenant compte de conditions antérieures d'humidité. La composition de l'écoulement indique que les activités urbains sont source de pollution. On a relevé des concentrations particulièrement élevées de zonc et de plomb excédant les normes de sécurité et menaçant la flore et la faune aquatiques. En dernier lieu, on propose des solutions pour éliminer les problèmes d'écoulement dans de futurs projets de développement urbain. Entre autres, on suggère de mettre au point des stratégies de planification de l'écoulement et d'établir des directives pour la conception technique des systèmes de drainage.

PREFACE

The Waterford River Basin Urban Hydrology Study, developed as a cooperative effort between the Governments of Canada and the Province of Newfoundland, was proposed by the Newfoundland Department of Environment in response to watershed management problems that had resulted from urbanization of the Waterford River Basin. Among such problems, negative effects of urbanization on both water quality and quantity were found to be so serious that the Newfoundland Department of the Environment identified the Waterford River Basin as a high priority area.

The five-year study, begun in 1980, was completed in March, 1985. The primary objectives of the study were to develop environmentally acceptable criteria for urban development in Newfoundland and to utilize the study results directly in the urban planning processes in the Province. The specific objectives of the study, as outlined in the report "Waterford River Basin - Urban Hydrology Study Plan", were as follows:

(1) To examine the processes leading to changes in the hydrologic regime of the Waterford River watershed. This should include evaluation and monitoring of major hydrologic changes caused by urbanization, the study of precipitation-runoff processes, and the study of various forms of pollution originating in the urban areas of the watershed.

(2) To provide a hierarchy of mathematical models describing hydrologic processes in the watershed. Such models should deal with both water quantity and quality, and should be capable of simulating the impact of urbanization on the water resources in the studied basin.

(3) To recommend solutions to specific water management problems in the studied basin and to develop guidelines for implementation of similar solutions elsewhere in Newfoundland. Furthermore, planning and management criteria should be developed for those aspects of the urban development which related to the environmental protection of the affected water resources.

The complexity of the study called for a comprehensive approach which included hydrometric surveys, hydrological modelling, groundwater studies, biological surveys, water quality assessment, investigations of flooding, land use, and socio-economic analyses.

The study was administered by a Steering Committee appointed by the governments of Newfoundland and Canada. To implement the study plan, a Technical Committee consisting of three representatives, one from each participating agency, was established. Subsequently, the Technical Committee appointed sub-committees and working groups to prepare and carry out the work plans for the various components of the Study.

The report that follows deals with one such component.

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

LIST OF TABLES

List of Tables (continued)

LIST OF FIGURES

1.0 INTRODUCION

Urbanization of the Waterford River Basin is adversely affecting water resources in the basin (6). The reported adverse effects of urbanization include increased runoff and incidence of flooding, the lowering of groundwater level and deterioration of surface water quality. It is conceivable that further development of the basin will aggravate the conditions of water resources in the basin, unless proper control measures are adopted for future urban developments.

In response to the concerns about the state of water resources in the Waterford River Basin, the Department of Environment of the Province of Newfoundland proposed to study the hydrology of the basin (6). This proposal was modified in cooperation with Environment Canada and, eventually, a joint hydrologic study of the basin was started in 1980.

One of the components of this joint study was to investigate urban runoff processes in the St. John's area. Such investigations had a number of objectives which can be summarized as follows:

1) evaluate the applicability of various runoff computational procedures to the studied area using rainfall, runoff and water quality data observed in an experimental test catchment in the St. John's area; and
2) recommend basic principles for hydrologic design of urban drainage in the studied area.

The execution of the study of urban runoff required contributions of many staff members from several federal and provincial government agencies. In particular, the contributions of the following agencies and their staff are gratefully acknowledged.

Department of Environment, the Government of Newfoundland and Labrador, St. John's, Nfld. - Selection of the test catchment, catchment documentation and surveys, the collection and processing of rainfall

data and runoff samples, water quality analysis , and preparation of the study report.

Water Planning and Management Branch, Environment Canada, Dartmouth, N.S. - Collection of some physiographic data and detailed reviews of draft reports.

Water Resources Branch, Environment Canada, Dartmouth, N.S. - Design of the flow monitoring station and the collection of runoff data.

Water Quality Branch, Environment Canada, Moncton, N.B. - Analysis of runoff samples.

Hydraulics Division, National Water Research Institute, Environment Canada, Burlington, Ont. - Overall study coordination, processing of runoff charts, applications of computational methods, and the preparation of the study report.

The material presented in the report that follows starts with a description of the study approach, followed by catchment characteristics and instrumentation, field data, runoff computations, runoff quality, recommendations for drainage design, and study conclusions and recommendations.

2.0 STUDY APPROACH

Extensive field investigations of urban runoff have been undertaken in various parts of Canada during the last 15 years. In spite of valuable results produced, there was concern that such results would not be applicable to the conditions in Newfoundland which differ from those studied earlier as far as the climate, local geology, construction practices and sewer ordinances are concerned. It was therefore decided that any such studies of urban runoff in the St. John's area should include field observations of rainfall, runoff, and runoff composition. It was further decided that the interpretation of field data should be enhanced by applications of various computational procedures (6). Among such procedures, it was desirable to include the conventional empirical approaches as well as hydrologic synthesis by runoff models. Further details of the approaches taken follow.

2.1 Field Investigations

Field observations of flows and their composition provide the best estimates of runoff and its water quality for drainage design or analysis. Because it is impractical and sometimes even impossible to observe runoff at every point of interest, various computational procedures are used to produce such information. Local field data are used for calibration and verification of such computational procedures. Although the calibration data are sometimes transposed from other catchments, the hydrometeorological and physiographic conditions found in the basin studied were substantially different from those reported in other Canadian test catchments (14). It was, therefore, necessary to obtain local calibration data. Such data were collected in a typical modern urban test catchment which is located in the Waterford River Basin and referred to as the Newtown Test Catchment.The Newtown data were used to calibrate runoff computational procedures and it was proposed to transpose the calibrated computational parameters, after appropriate modifications, to other similar catchments in Newfoundland. Such calibrated parameters are further discussed in Chapters 6 and 7.

2.2 Runoff Computations

The development of runoff computational procedures has been greatly advanced during the last 15 years. It became apparent during this development, that the main criterion for the selection of a computational procedure is not the procedure complexity, which is sometimes perceived as synonymous to accuracy, but its technical correctness and feasiblity of application with the available supporting data (17). It was also recognized that the design of urban drainage presents various classes of design problems and, that the simplest design procedures, appropriate for a particular class, will yield satisfactory results. Thus considering the whole spectrum of drainage design problems, the designer should have a spectrum of corresponding procedures. For the simplest design task dealing with pipe sizing in a small drainage area, a simple empirical design procedure, such as the rational method, may yield fully satisfactory results, if applied properly. As the drainage area and sewer sizes increase, it may become desirable to use a hydrologic model for runoff computations, in order to obtain a better understanding of flows in various drainage network segments and to consider runoff controls, such as storage. An example of the modelling tool appropriate for this application is the ILLUDAS model. Finally, in complex cases with considerations of surcharged sewers, runoff quality and runoff control, it is necessary to use complex hydrologic and hydraulic models, such as the SWMM model. Consequently, the sections on runoff computations (Chapters 6 and 7) includes several approaches with various levels of complexity.

The above choice of three computational procedures does not exclude the use of other equivalent procedures. At this time, however, the rational method and the ILLUDAS and SWMM models appear to be among the most popular procedures used in urban drainage design. It should be also noted that, as discussed in Chapter 9, drainage design involves many other considerations than just runoff computations.

Field studies of urban runoff are described in the next two chapters, followed by descriptions of runoff computational procedures.

3.0 NEWTOWN URBAN TEST CATCHMENT

The Newtown urban test catchment was established for the purpose of monitoring urban rainfall and runoff processes. In the following, the procedures for the selection of the test catchment and its characteristics are described.

3.1 Catchment Selection

The selection of urban test catchments is a fairly complex process which, if executed correctly, enhances the probability of the study success (1). The catchment to be monitored must be viewed as a representative sample of the hydrologic, geographic, terrestrial, and demographic characteristics of the study area. Before proceeding with the actual selection process, it is necessary to establish the selection criteria and some rating system for these criteria. The ratings and comparisons of potential sites are fairly subjective and experience from other studies is invaluable in this regard.

In the Waterford River Basin study, the following nine criteria were used in the selection of the urban test catchment:

1) Type and state of urban development and type of drainage.
2) Catchment size.
3) Drainage outfall condition.
4) Monitoring feasibility.
5) Access.
6) Sewer system condition.
7) Availability of documentation.
8) Availability of electricity.
9) Susceptibility to vandalism.

The type of urban development describes land use, the level of completion of development, and the type of sewerage. It was desirable to select a catchment with residential land use, because future development is expected to be primarily in this category. The catchment should be served by separate sewers and fully developed to avoid any disturbances or changes in the runoff regime caused by an ongoing development.

The test catchment size should be representative of catchment sizes in the study area. Generally, the typical urban catchment sizes are smaller than 40 ha and larger than 5 ha. Both smaller and larger catchments raise some doubts about the representativeness of such catchments. In the case of very small catchments (<5 ha), additional problems arising from the fast hydrologic response may be encountered.

Monitoring stations are most conveniently located at the drainage outfall. Thus it is desirable to select a catchment with a free outfall into the receiving water body. Such an outfall should not be flooded by high water conditions in the receiving waters. If the above conditions are met, the outfall can be easily instrumented for discharge measurements and sampling. At the same time, such a monitoring station is easily accessible.

The connectivity of sewers and their physical condition are also considered in the catchment selection process. Generally, flow measurement is simplified in tree-type sewer networks without loop connections and sewer surcharging.

The availability of documentation for the catchment reduces the costs associated with establishing the catchment physiographic characteristics and eliminates costly field surveys.

The availability of electricity is important for operation of various instruments and equipment and reduces the cost of establishing the monitoring site.

Finally, the susceptibility to vandalism is another important consideration. Installations in areas with high occurrences of vandalism require extensive protection and repeated vandalism of the installation may result in abandoning the site.

Using the above selection criteria, nine potential test catchments in the study area were scrutinized. Eventually, the Newtown test catchment was selected as the best site in terms of its representativeness and suitability for instrumentation. A description of this catchment follows.

3.2 Catchment Characteristics

The Newtown test catchment (see Fig. 1) is a residential subdivision of about 13.2 ha which is located in Mount Pearl, Newfoundland. This subdivision was built during the period from 1976 to 1980 and, at the start of the Waterford River Basin study, it was fully developed. The catchment boundary was determined from a drainage map, sewer layout, and field inspection of drainage patterns.

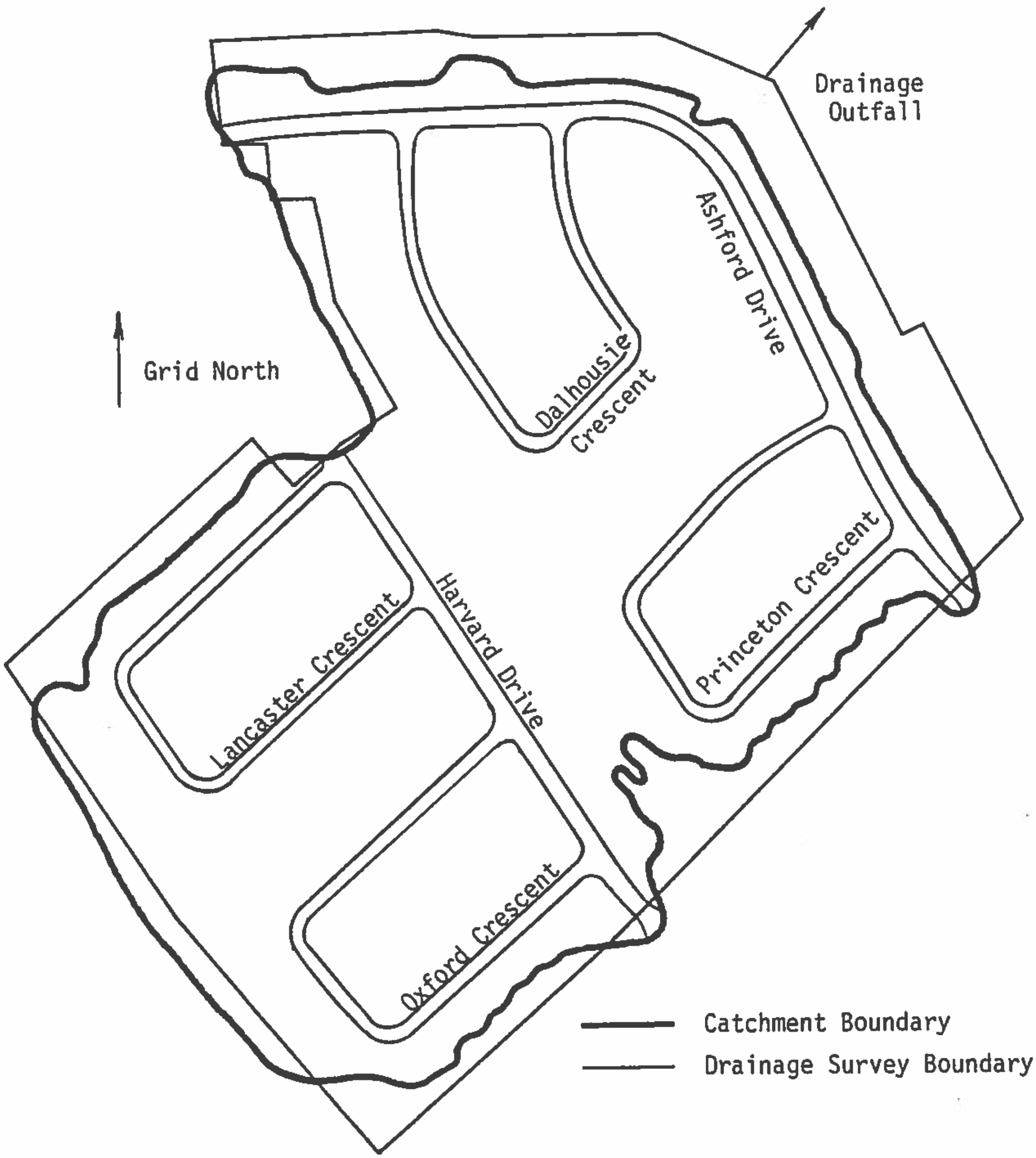


Fig.1. Newtown Urban Test Catchment

3.2.1 Topography

The catchment is sloping in the north-easterly direction with average slopes varying from 0.025 m/m to 0.040 m/m. The highest elevation in the catchment, which is about 162 m above the sea level (ASL), is found along the northwest boundary of the catchment. The lowest elevation of 143 m ASL is found in the northeast corner of the catchment close to the drainage outfall.

Local slopes depend on lot gradings. Typically, front yards slope towards streets with slopes varying from .02 m/m to .08 m/m. Backyards slope gently (.01 m/m - .03 m/m) away from houses towards drainage swales running along the back property lines. Furthermore, the lots of houses on streets running in the southwest-northeast direction also have lateral slopes in the same direction as streets.

The slope of streets in the catchment follows closely the slope of the terrain. The mildest slopes were found in the case of streets running in the NNW direction - from 0.007 m/m to 0.03 m/m. The streets running in the NE direction had slopes from 0.020 m/m to 0.066 m/m.

3.2.2 Land use

The whole test catchment is zoned as single-family residential land. Most of the 203 houses in the catchment were built during the period from 1978 to 1980. Thus all the properties are fairly new and well-maintained. The average rooftop area of houses in the catchment was estimated as 120 m^2 and the average lot size was determined as 15 m x 30 m (width x depth). Besides residential properties, there is about 0.3 ha of open land which serves as a park and playground.

3.2.3 Catchment surface cover

For runoff calculations, two basic types of catchment surfaces are distinguished - impervious and pervious. As detailed later, both these types may be subdivided into several subtypes depending on their role in runoff generation.

To evaluate the extent of various surface types, a detailed survey of the catchment area was conducted. In this survey, every lot was inspected and evaluated in terms of areas with various surface covers and drainage patterns. The results of this survey are summarized below.

From the point of view of urban runoff generation, the most important type of catchment surface is the effective impervious area (also sometimes referred to as the directly-connected impervious area) which comprises those impervious surfaces that are hydraulically connected to the drainage system (19). Examples of such surfaces are paved streets with curbs and gutters, paved driveways draining towards streets, and roofs draining into storm sewers. Other types of impervious areas are the so-called supplemental impervious areas which comprise impervious areas draining onto pervious areas. Among examples of supplemental impervious areas are roofs or driveways draining onto lawns. The sum of the effective and supplemental impervious areas equals the total impervious area.

The extent of pervious areas in the catchment was also determined for both front yards and backyards. In this evaluation, only the pervious areas potentially contributing runoff to the sewer system were considered. The total runoff contributing area of the catchment was established as 13.23 ha.

A summary of areas of various surfaces in the Newtown catchment is given in Table 1.

The total imperviousness of the Newtown catchment can be determined from Table 1 as 38% but the effective imperviousness is only

Table 1. Newtown Catchment Surface Cover Characteristics

Type of Surface	Area (ha)
Road surface area	1.75
Roof and driveway areas hydraulically connected	0.86
Effective impervious area	2.62
Supplemental impervious area	2.44
Total impervious area	5.05
Contributing pervious area	8.18
Total catchment area	13.23

20%. These values will be further verified in runoff simulations described in Chapter 7. It appears that the catchment has a fairly low effective imperviousness and this should result in relatively low runoff peaks per unit area (i.e., peaks expressed in $m^3/s/ha$).

The pervious parts of the catchment comprise grassed areas around residential properties. It was noted that the layer of topsoil was relatively thin and that from the hydrologic point of view the predominant soil could be classified as type B according to the classification system of the U.S. Soil Conservation Service (20). The classification of soils in the Newtown catchment is further discussed in Chapter 7.

3.3 Sewer System

The Newtown catchment is served by a tree-type, converging storm sewer system. All the sewers are made of corrugated steel pipes whose roughness was characterized by Manning's n = 0.024. The sewer system is fairly new and in good condition. Upon completion, the system was pressure tested and found watertight.

The sewer pipe diameters vary from the minimum size of 0.305 m in the upper reaches to 0.915 m at the drainage outfall. The sewer system comprises 37 pipe sections of lengths varying from 20 m to 120 m and with slopes varying from 0.006 m/m to 0.062 m/m. The layout of the sewer system is shown in Fig. 2. The pipe diameters, lengths, slopes, capacities, full-bore velocities and the corresponding times of travel are listed in Table 2.

Runoff observations at the drainage outfall indicated the presence of baseflow, particularly during storms of long duration (more than 12 hours). This baseflow was gradually increasing during the storm and persisted long after the surface runoff stopped. Two possible sources of baseflow were considered - foundation drains and infiltration of groundwater into the sewers. The former source is probably more significant because the sewer system is fairly new and was fairly watertight when constructed.

4.0 CATCHMENT INSTRUMENTATION

The Newtown catchment was fully instrumented for the purpose of field observations of rainfall and runoff processes. Towards this end, precipitation, runoff flow rate and runoff quality were monitored in the catchment. Observations of precipitation and runoff have been made since September, 1981. The most recent data included in this report are for September, 1984. Thus, when writing this report, three years of rainfall and runoff data were available. The data collected after September 1984 will be presented in future reports.

Observations of runoff quality were conducted by means of collection of stormwater samples at the catchment drainage outfall. The collection of samples was delayed until numerous illicit connections of sanitary sewage to the storm sewers were disconnected. Thus the collection of stormwater samples started only in 1983 and was continued from then on. During this period, there was some sanitary sewage discharged from one house whose owner refused to have this problem fully

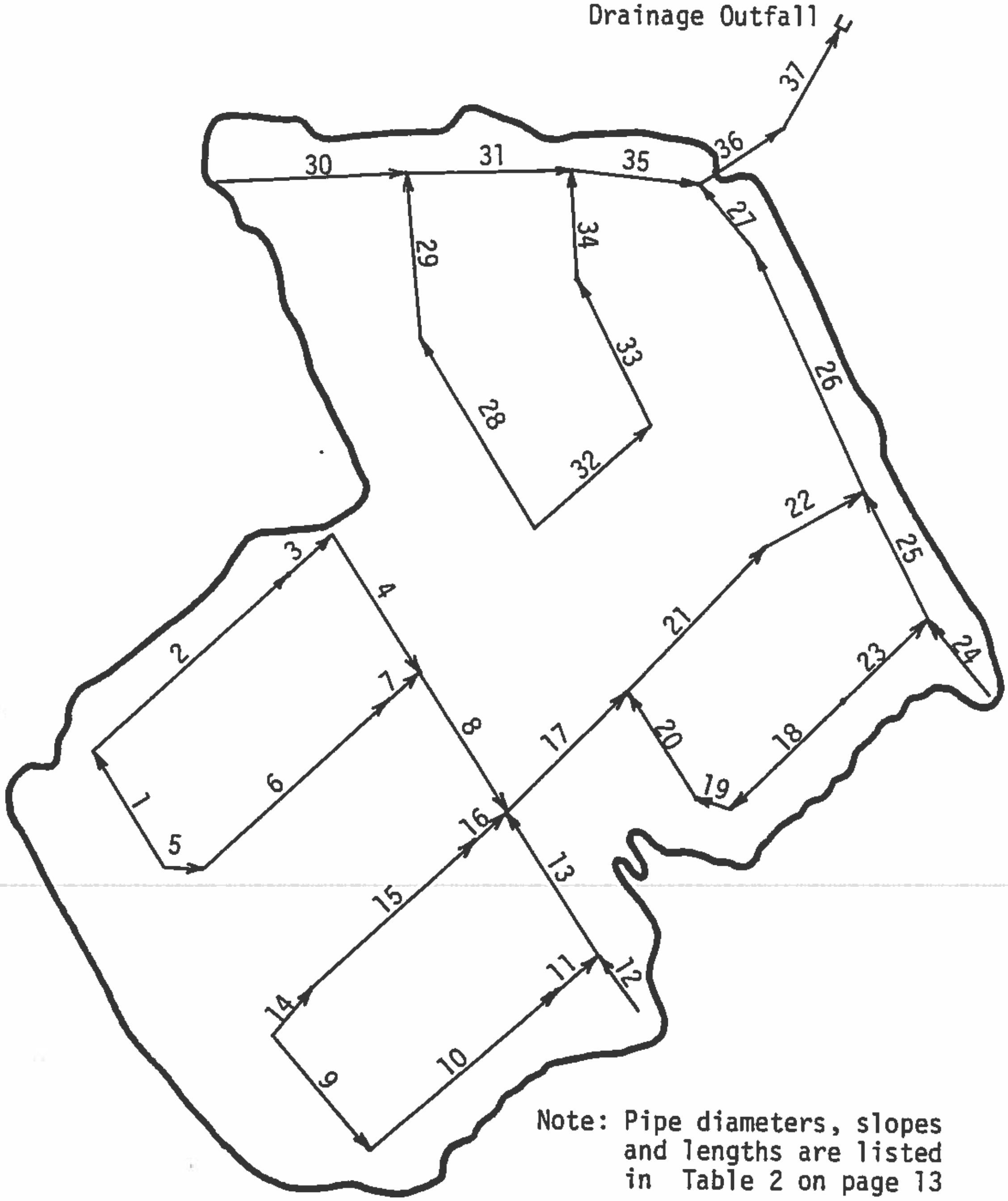


Fig.2. Newtown Storm Sewer System

Table 2. Characteristics of Newtown Storm Sewer System

Pipe No.	Diameter (m)	Length (m)	Slope (m/m)	Capacity (m^3/s)	Full-Bore Velocity (m/s)	Full-Bore Time of Travel (s)
1	0.305	67.1	0.0064	0.044	0.60	112
2	0.305	122.0	0.0508	0.124	1.69	72
3	0.381	28.1	0.0140	0.118	1.03	27
4	0.534	75.6	0.0053	0.178	0.79	95
5	0.305	19.5	0.0352	0.103	1.41	14
6	0.305	111.9	0.0598	0.134	1.84	61
7	0.305	19.2	0.0175	0.073	0.99	19
8	0.610	75.3	0.0077	0.305	1.05	72
9	0.305	71.7	0.0181	0.074	1.01	71
10	0.305	115.9	0.0484	0.121	1.65	70
11	0.381	22.6	0.0108	0.103	0.91	25
12	0.305	25.3	0.0050	0.039	0.53	48
13	0.458	74.7	0.0078	0.143	0.87	86
14	0.305	27.1	0.0360	0.104	1.42	19
15	0.305	104.3	0.0620	0.136	1.87	56
16	0.381	18.3	0.0217	0.146	1.28	14
17	0.610	74.1	0.0099	0.347	1.19	62
18	0.305	64.1	0.0100	0.055	0.75	85
19	0.305	20.7	0.0103	0.056	0.76	27
20	0.305	61.9	0.0172	0.072	0.99	63
21	0.610	100.0	0.0152	0.429	1.47	68
22	0.610	48.8	0.0163	0.444	1.52	32
23	0.305	58.3	0.0262	0.089	1.22	48
24	0.305	40.9	0.0097	0.054	0.74	55
25	0.305	64.7	0.0184	0.074	1.02	63
26	0.610	121.1	0.0265	0.567	1.94	62
27	0.763	41.2	0.0082	0.572	1.25	33
28	0.305	105.2	0.0107	0.057	0.78	135
29	0.381	75.6	0.0165	0.128	1.12	68
30	0.305	89.4	0.0444	0.115	1.58	56
31	0.381	79.3	0.0223	0.148	1.30	61
32	0.305	72.0	0.0203	0.078	1.07	67
33	0.305	72.6	0.0235	0.084	1.15	63
34	0.381	51.9	0.0135	0.115	1.01	51
35	0.610	63.1	0.0058	0.265	0.91	70
36	0.915	43.3	0.0120	1.125	1.71	25
37	0.915	59.5	0.0065	1.249	1.40	43

corrected. It was felt that such small discharges would not significantly affect the compositon of high volumes of stormwater in the system and the monitoring of stormwater quality was started.

A description of catchment instrumentation and measurement procedures follows.

4.1 Precipitation

Precipitation was measured in the Newtown catchment by means of a tipping-bucket recording rain gauge and by a standard rain gauge. Both gauges were serviced and operated by locally hired observers and this led to some changes in gauge locations during the study.

The tipping-bucket rain gauge was the standard instrument used by the Atmospheric Environment Service (AES). The tipping bucket has a capacity of 0.2 mm per tip and thus the total rainfall depth can be read in increments of 0.2 mm. The recorder chart speed was 15 mm/hr and this permitted the discretization of the rainfall records into two-minute intervals. This detailed discretization was initially used for short storms of medium-to-high intensity. For longer storms, a five-minute discretization was used. As discussed later in Chapter 7, the relatively slow response of the Newtown catchment made it possible to use five-minute rainfall intensities in final runoff simulations for storms of all characteristics.

The tipping-bucket rain gauge was operated throughout the whole year, though no attempts were made to use it to measure snowfall. Instrument malfunctions resulted in losses of data for some events. In such cases, the rainfall data from the St. John's West station, located at the Canada Department of Agriculture (CDA) farm, were used. The rainfall records from this station, which is located about 1.8 km east of the Newtown catchment, somewhat differed from Newtown records and, consequently, it was not feasible to substitute the St. John's West data for Newtown data without introducing additional uncertainties.

At least one standard rain gauge, with a collector diameter of 112.5 mm, was operated in or near the catchment throughout the study period. The gauge was read daily at times which slightly differed depending on the observer. During periods of snowfall, estimates of 24-hour snowfall were also produced. The standard rain gauge data were used to verify the rainfall depths recorded by the tipping-bucket gauges.

The locations of rain gauges used in this study are listed in Table 3 and shown in Fig. 3.

Table 3. Locations of Rain Gauges From Which Data Were Used in the Study

Time Period	Rain Gauge Location: Tipping - Bucket	Rain Gauge Location: Standard Gauge
Before March, 1982	No suitable data collected in Newtown, all data originated at St. John's West (CDA)	
March-July, 1982	29 Lancaster Crescent	
August, 1982 -	14 Dalhousie Crescent	
February-May, 1981		29 Lancaster Crescent and 19 Princeton Crescent
June-July, 1982		29 Lancaster Crescent
August, 1981-May, 1983		22 McGill Crescent
July, 1983-November, 1983		10 Princeton Crescent and 13 Lancaster Crescent
December, 1983 -		13 Lancaster Crescent

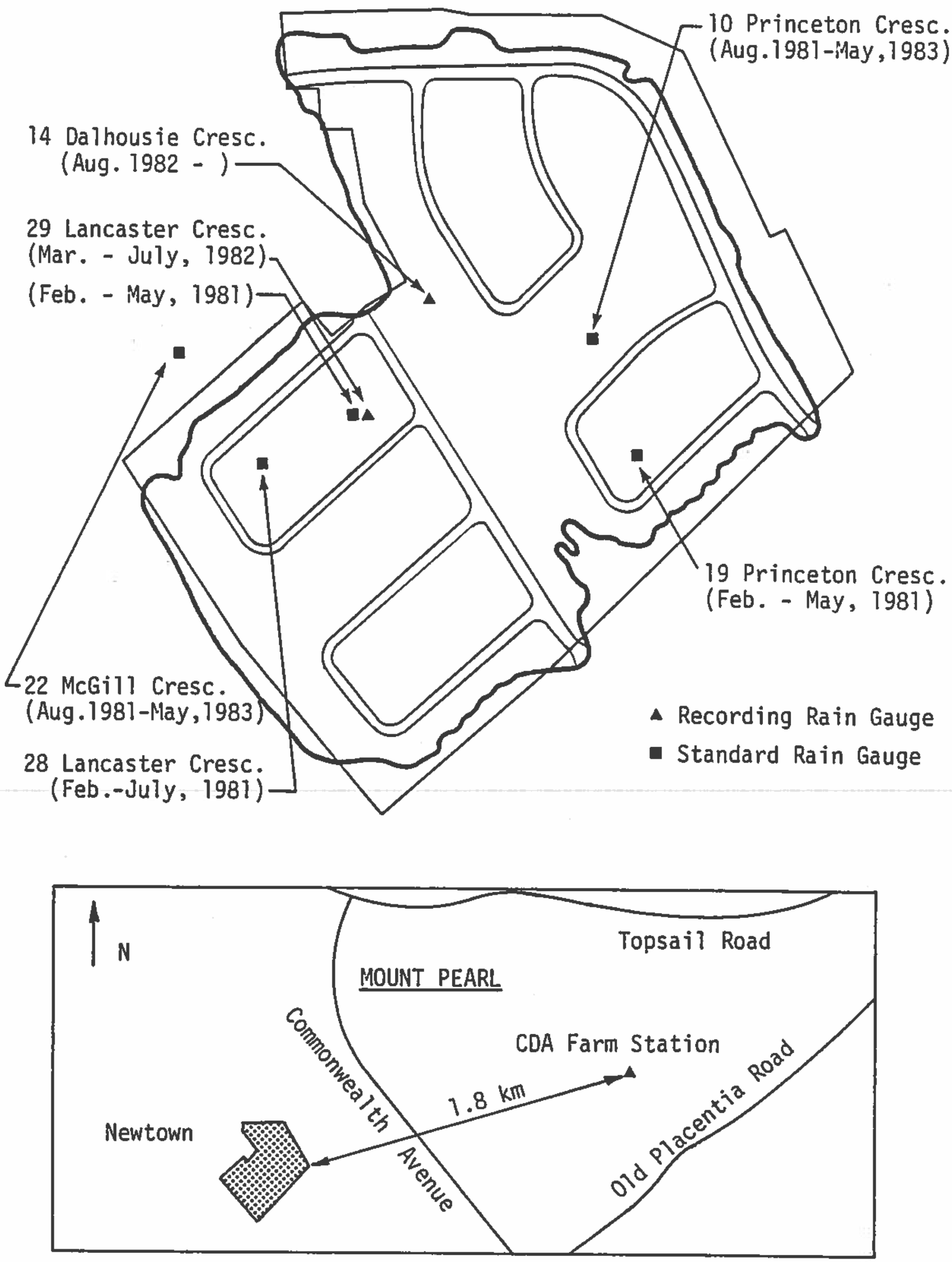


Fig.3. Location Map of Rain Gauges

4.2 Runoff

Runoff was monitored continuously at the outfall from the sewer system. For this purpose, a wooden weir box was built at the outfall. In the initial stage, a V-notch weir was used to measure the discharge and later it was replaced by a rectangular weir devised to reduce the rise in the water level at the outlet and the backwater effects in the outlet sewer.

A sketch of the weir box is shown in Fig. 4. Because this installation of the measuring weir at the outlet is unconventional, it was felt that the standard weir rating equations would not be applicable and the installation should be calibrated. Such a calibration was done using a scale model of the installation in the Hydraulics Laboratory of NWRI.

A model of the Newtown weir was constructed in a scale 1:6.06 and installed in the laboratory. The model weir was then calibrated for a wide range of weir heads which corresponded to the range from 0.06 m to 0.60 m in the prototype. The measured heads and flows were then approximated by the Kindsvatter equation (4) in the following form:

$$Q = \frac{2}{3} C_e \sqrt{2g}\, b_e H_e^{\;1.5} \qquad (1)$$

where Q is the weir discharge (m^3/s), C_e is the effective coefficient of discharge, b_e is the effective length of the weir crest, H_e is the effective weir head (H_e = H + 0.001, where H is the observed weir head in metres), and g is the acceleration due to gravity.

Using the observed weir discharges and weir heads, the following expressions for b_e and C_e were fitted to the experimental data:

b_e = b + 0.0035 H, where b = 1.1571 m is the actual length of the weir crest

C_e = 0.602 + 0.6097 H for $H \leq 0.342$ m, and

C_e = 0.2791 + 1.5538 H for $0.342 \leq H \leq 0.60$ m.

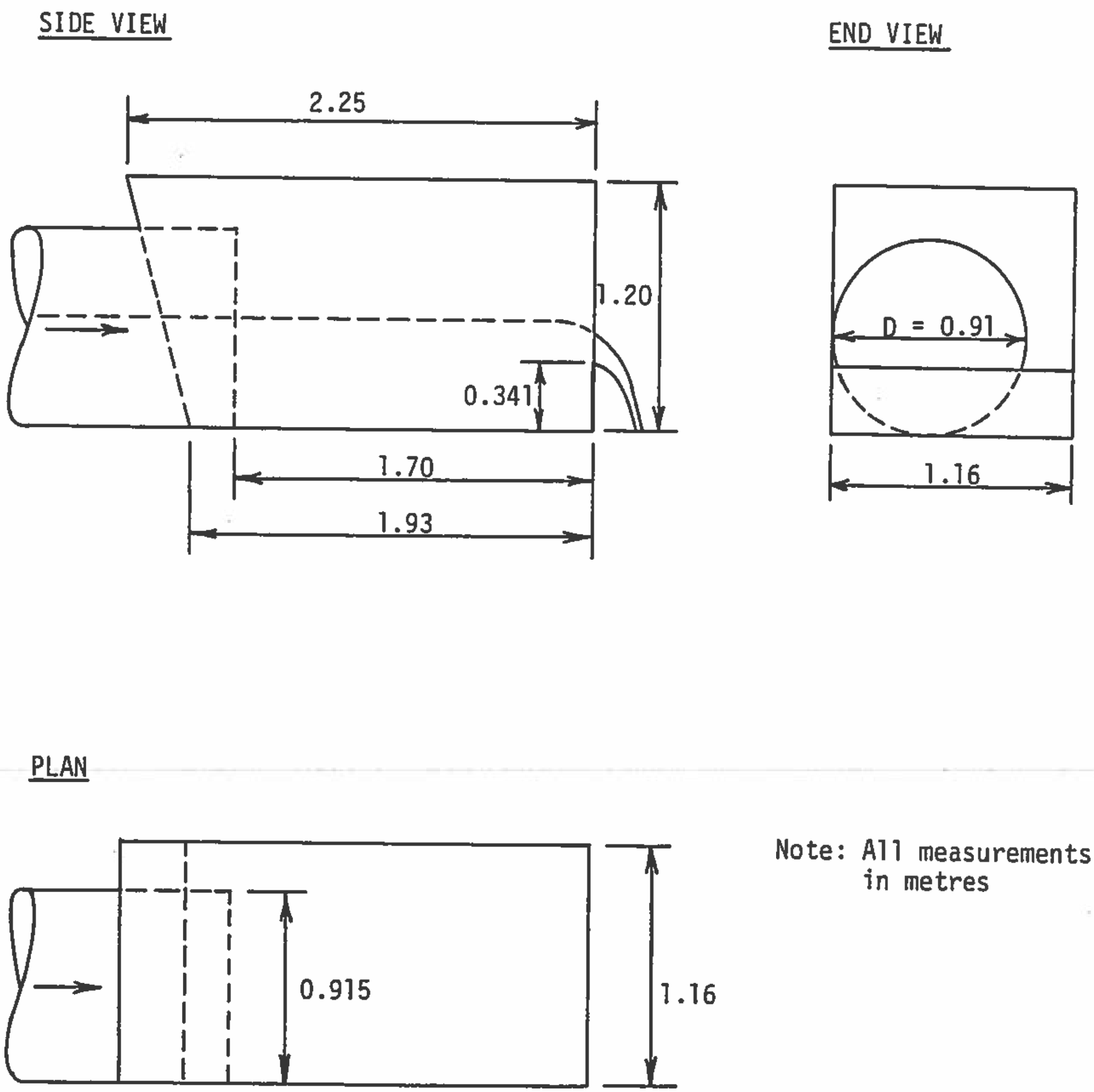


Fig.4. Newtown Measuring Weir Installation

Because all the weir heads observed for the Newtown weir were smaller than 0.342 m, only one weir rating curve was used for data processing in the following form:

$$Q = 2.953(0.602+0.6097H)(1.1571+0.0035H)(H+0.001)^{1.5} \qquad (2)$$

A somewhat different expression would be obtained for weir heads greater than 0.342 m.

The rating curve for the Newtown weir is shown in Fig. 5 which includes the fitted rating curve as well as the observed points (scaled-up from the model). It was of interest to evaluate the accuracy of the fitted rating curve. For this purpose, the mean absolute value e of the relative deviations of calculated from observed discharges was determined as

$$e = \sum_{i=1}^{N} \left(\frac{\left| Q_{obs_i} - Q_{calc_i} \right|}{Q_{obs_i}} \right) / N \qquad (3)$$

where Q is the discharge, subscripts 'obs' and 'calc' refer to observations and calculations, respectively, N is the total number of experimental points, and the subscript i refers to individual points.

For weir heads smaller than 0.324 m, which covers all the measurements taken at the Newtown site so far, the mean value of e was ±3.1%. Such an accuracy is quite acceptable. For heads greater than 0.324 m, the error described by eq. (3) increased to ±5.2%, which would be still within the acceptable limits. It was felt that at higher heads and flows, the ability of the weir box to act as a flow control device with a uniquely defined flow rate vs the head relationship was somewhat limited. This led to an increased scatter in experimental data.

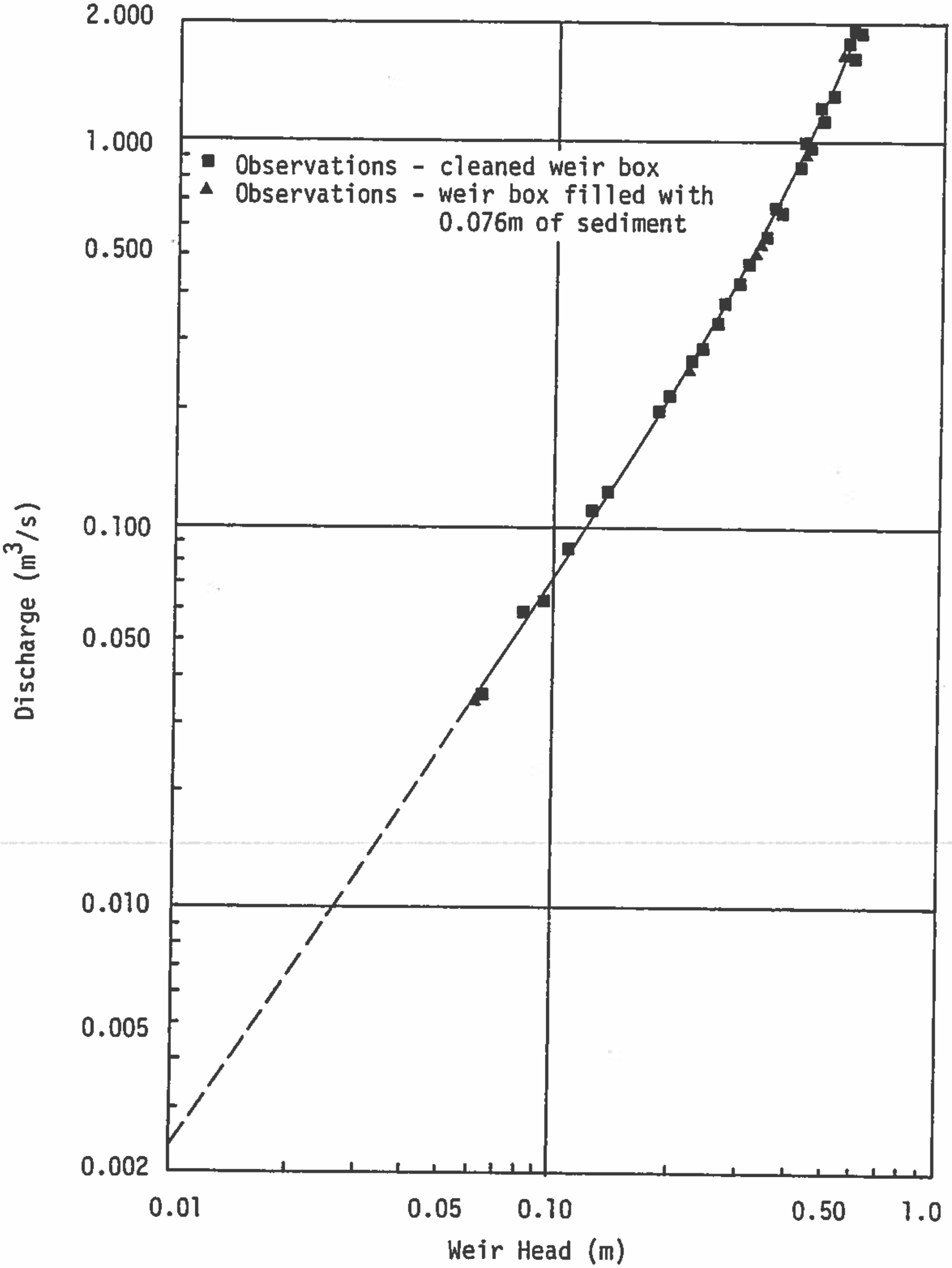


Fig.5. Newtown Weir Rating Curve

During the operation of the Newtown weir, it was noticed that small quantities of sediment accumulated in the weir box. Because of concerns that such deposits might affect the weir rating curve, the effects of sediment deposits on the rating curve were investigated in a special experimental series. In this case, the consolidated sediment deposits were approximated by a wooden insert, 12.5 mm thick, which was placed on the box bottom. Such an insert represented a sediment layer of 0.076 m in the prototye, which is certainly thicker than any accumulations actually observed at the site. The presence of the insert decreased the relative weir height. It would be expected that such a change will not affect the weir rating curve at small weir heads, but could have a noticeable effect at higher relative heads.

The observations made for the weir box with a layer of material on the bottom were also plotted in Fig. 5. It appears from this plot that the presence of the fill had no significant effect on the weir rating curve, especially in the range of weir heads observed so far in the study. Application of eq. (3) to experimental data collected for the weir box with the insert (sediment accumulation) yielded an error estimate $e = \pm 3.2\%$ which is almost identical to that reported earlier for the weir box without any sediment and the prototype weir heads smaller than 0.342 m. Thus it was concluded that minor sediment accumulations in the weir box ($\leq$ 0.076 m) do not significantly affect the flow measurements at the Newtown site.

4.3 Runoff Quality

The compositon of urban runoff was monitored by collecting and analyzing runoff samples. The samples were collected by an automatic water sampler, the Sirco Model B/VS. The start of runoff sampling (September, 1983) was delayed because of problems with illicit sanitary sewage discharges into the storm sewer system. A description of the sampler and sampling procedures follows.

The Sirco sampler B/VS is a stationary automatic sampler which uses the pressure/vacuum method for sample collection. The sampling operation starts with a purge period during which air is forced through the sampler intake tube and purges all liquid from the tube. Following that, a vacuum pump is used to draw the sample into a metering chamber. When the chamber is filled to a preset volume, its content is drained into a sample bottle and the sample distributor arm advances to the next bottle to be filled.

Discrete sequential grab samples were collected at intervals varying from 5 to 10 minutes. Up to 24 0.5-litre samples were collected during one sampling cycle. When the whole cycle had been completed, the samples were transferred from the sampler bottles into laboratory bottles and submitted for analysis. The sampler was then reset for the next sampling cycle.

During individual events, the sampler was activated automatically by a water-level sensitive actuator and from there on, the samples were collected at constant time intervals.

Collected samples were shipped to the water quality laboratories of the Water Quality Branch, Moncton, New Brunswick, the Environmental Protection Service, St. John's, Newfoundland, and Newfoundland Department of Environment, St. John's, Newfoundland. The samples were analyzed for the following groups of parameters:

physical parameters
major ions
selected metals, and nutrients.

Furthermore, special grab samples were collected manually and analyzed for bacteria by the Newfoundland Public Health Laboratory in St. John's.

Details of individual parameter groupings, references for sample preservation and analysis, and analytical results are given in Chapter 8.

5.0 FIELD DATA

The field data on precipitation, runoff and water quality are discussed briefly in this chapter. Such a discussion comprises the data collected by the Waterford River Basin Study personnel as well as data obtained from other sources.

5.1 Rainfall Data

Rainfall data pertinent to the study area were available from a special data collection program instituted for this study and from the AES stations at St. John's airport and the CDA farm.

5.1.1 Observed rainfall data

The rainfall data series was discretized into individual storm events which were separated by inter-event periods of no rainfall of three hours or longer. Rainfall data were obtained for individual storm events and converted into storm hyetographs to be used in runoff simulations. Whenever available, the data from the Newtown recording rain gauge were used for simulations of urban runoff. During some events, the Newtown gauge was inoperative and it became necessary to use the rainfall records from the St. John's-West station at the experimental farm of CDA which is located about 1.8 km east of the Newtown catchment. Although the annual precipitation at the CDA site should be comparable to that at Newtown, distributions of intensities during individual events may differ. Consequently, runoff hydrographs simulated for CDA hyetographs may not closely reproduce the runoff hydrographs observed at Newtown.

The rainfall/runoff events to be studied in detail were determined from runoff records as the events with the peak runoff weir head greater than 0.18 m. For all such events, the rainfall records were obtained and discretized in short time intervals.

It was further of interest to evaluate the severity of the observed storms in terms of rainfall intensities. For this purpose, storm hyetographs were screened for peak intensities of durations varying from 5 to 30 minutes and then ranked according to the mean rank obtained from the following expression.

$$R = (R_5 + R_{10} + R_{15} + R_{30})/4 \qquad (4)$$

where R is the rank and the subscript refers to the duration. Using this procedure, the ranking of the top twelve storms is given in Table 4. Another ranking of storms was done according to the observed peak flows. Again, the top twelve storms were listed in Table 4.

Table 4. Ranking of Rainfall/Runoff Events

Rank	Storm Number Ranked According to the Peak Intensities of Durations from 5 to 30 minutes	Storm Number Ranked According to the Observed Peak Flow
1	73	76
2	69	73
3	63	19
4	246	63
5	152	246
6	14	142
7	76	152
8	168	253
9	142	69
10	173	14
11	253	193
12	239	198

It was of interest to note that altogether there are 15 storms listed in Table 4. Out of these, nine storms appear in both rankings according to the peak intensities as well as peak flows, though their ranks do not correspond closely in all cases. Among the storms ranked according to intensity, the following did not appear in the listing of storms ranked according to the peak flows (storms are identified by chronologically assigned numbers and the rainfall ranks are in the brackets):

168 (8), 173 (10), and 239 (12).

The runoff hydrographs of these events were further checked and found to contain runoff peaks with ranks from 17 to 20. The rainfall record for storm No. 168 was obtained from St. John's-West, so for this event, the difference in rainfall and runoff rankings is not surprising.

On the other hand, the following storms ranked according to the observed peak flow, did not appear in the intensity ranking (flow ranks in the bracket):

19 (3), 193 (11), and 198 (12).

For all these events, no rainfall data were available for Newtown and, furthermore, events 193 and 198 occurred in the winter and probably reflected special catchment conditions.

Thus, it appears from the above discussion that there is a fair agreement between the top high-intensity storms and the top runoff peak producing storms. The most frequent source of disagreement seems to follow from the lack of rainfall measurements in the Newtown catchment (events 19, 168, 193 and 198). Other sources of deviations in rankings include the antecedent catchment conditions and the rainfall distribution during the storms which is not fully described by peak intensity ranking. It was also of interest to compare the peak intensities of the observed storms against those obtained from the local

Intensity-Duration-Frequency (IDF) curves. The nearest station for which such curves were available was the St. John's Airport station which is located about 13.2 km northeast of Newtown. These curves which are shown in Fig. 6 and listed in Table 5 were produced from a 24-year rainfall record for the period from 1949 to 1983. Such a length of record justifies the preparation of IDF curves with return periods up to 50 years. Although the 100-year curve is also shown, it represents a somewhat unreliable estimate which should be used with caution. In general, the IDF curves are expressed as

$$i = at^b \qquad (5)$$

where i is the rainfall intensity (mm/hr) and t is the corresponding duration (hr). Parameters a and b are listed in Table 5 for various return periods.

Table 5. IDF Curves for St. John's Airport Station

Return Period (yr)	IDF Curve Parameters a and b	
	a	b
2	15.833	-0.530
5	20.898	-0.555
10	24.241	-0.565
25	28.445	-0.575

The observed rainfall hyetographs were further analyzed by estimating the return periods of their peak intensities from the IDF curves for the St. John's airport. Such estimates were produced for five durations from 5 to 60 minutes and presented in Table 6.

The data in Table 6 indicate fairly frequent occurrences of intensities with return periods in the range from 2 to 25 years in the 3-year data record. Such occurrences indicate that either the study

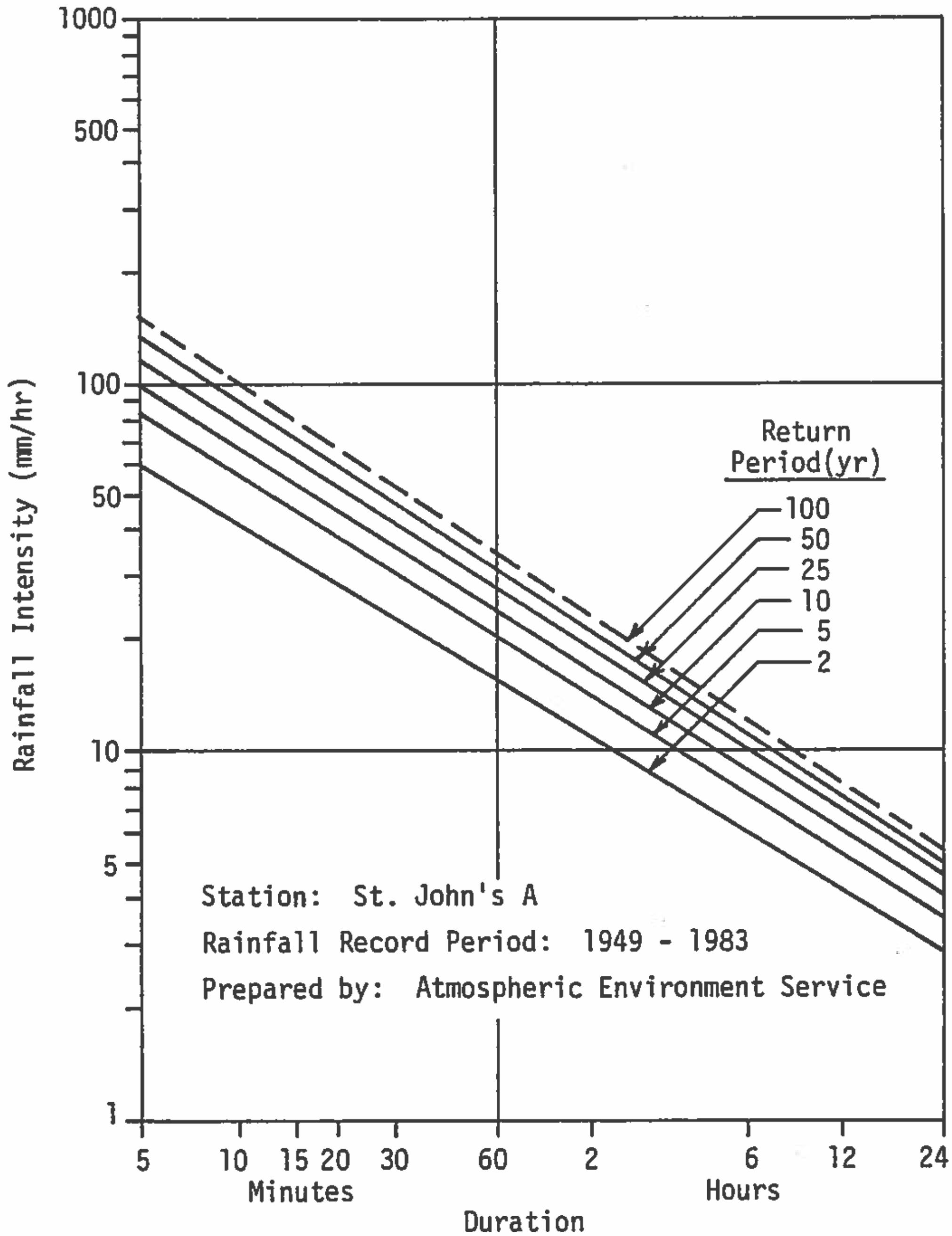


Fig.6. Rainfall Intensity-Duration-Frequency
Curves for St. John's Airport

Table 6. Estimated Return Periods of Peak Intensities Observed at Newtown, CDA and St. John's Airport

Storm Number	5 A[1]	5 B[2]	10 A	10 B	15 A	15 B	30 A	30 B	60 A	60 B
	Return Periods (yr) of Observed Peak Intensitites (As Determined from IDF Curves) — Duration (minutes)									
73	25	<2	17	2	10	<2	10	5	6	10
69	4	<2	4	<2	5	<2	5	<2	2	<2
63	3	2	3.5	4	4.5	9	9	8	4	5
152[3]	<2	<2	3	<2	2.5	<2	3	<2	<2	<2
14[3]	<2	<2	2	<2	2.5	<2	2	<2	<2	<2
246	<2	2	<2	6	2.5	9	3.5	25	2	23
76	<2	<2	<2	<2	2	<2	3	<2	4	<2
168[3]	2	<2	<2	<2	<2	<2	<2	<2	<2	<2

[1] A = Data collected at Newtown or CDA (see footnote 3)
[2] B = Data collected at the St. John's Airport
[3] Data collected at CDA Farm (St. John's West)

period contained an unusually high number of intense storms or the Newtown intensities are greater than those at the airport. The latter point cannot be fully substantiated with the limited data available for the Newtown station and should be further addressed in the future as more data become available for both the Newton and CDA stations. It was noted, however, that for the events listed in Table 6, short-duration intensities at Newtown generally exceeded those at the St. John's airport.

5.1.2 Design storms

Additional rainfall data for the St. John's area were obtained from AES (8) and Hydrotek, Inc. (10). The former data represent typical temporal distributions of 1-hour rainfall for actual storms observed at the airport. Such distributions were tentatively recommended by AES for use as urban design storms, particularly the 30% exceedance and mean distributions. The applications of these storm distributions are currently under review and, for this reason, they were not used in this study. The distributions are listed in the Appendix.

The second type of design storms is that developed by Hydrotek (10) under contract with AES. The Hydrotek design storm has not yet been officially approved for distribution and, for that reason, it has not been used in this study, but simply presented in the Appendix for completeness.

Both types of design storms, the AES and Hydrotek storms, are based on local data and, when finalized and officially released, should be evaluated for use in urban drainage design in Newfoundland. It was noted that the Hydrotek distribution possessed some advantages arising from its wider applicability in terms of storm return periods and durations.

5.2 Runoff Data

Runoff flow rates were recorded continuously in the form of weir heads which were then converted into flow rates using eq. (2). The runoff flow record available for analysis was only three years long. In spite of this short record length, it was desirable to attempt to estimate frequencies of the observed runoff peaks. For this purpose, two approaches were considered. The first one was a frequency analysis. The second approach was to develop a synthetic runoff record which would be subject to frequency analysis and the results would be then used to estimate frequencies of observed runoff peaks. The former approach is relatively simple, but the reliability of results may be questioned because of the short record length. The latter approach involves large uncertainties arising from the selection of rainfall data (i.e., design storms or actual storms, both from a relatively distant location) and modelling bias. For the sake of simplicity, the former approach based on frequency analysis was adopted.

Although the uncertainties inherent to frequency analysis of a short record are obvious, it should be emphasized that they may be acceptable in storm sewer design which generally involves appreciable safety. Such safety arises from two sources - the use of nominal pipe sizes with rounding off upward and the design of sewers for open channel flow. Regarding the first point, the calculated sewer pipes are rounded off upward to the nearest nominal size. For the sewer sizes found in the Newtown catchment, every step in the nominal size series represents a capacity increase from 26% to 81%. Thus, this procedure introduces a considerable safety margin into the design. Regarding the second point, it should be noted that sewers are generally designed for open-channel flow. If the design flow is exceeded, the system temporarily surcharges and, for limited surcharge heads, no damages occur. Thus, the sewer systems are usually capable of conveying flows higher than the design flow without any damages.

Finally, it should be recognized that minor drainage design is done for relatively short return periods (as short as one or two years) and this fact somewhat relaxes the requirements on the actual data record length. Keeping in mind the above limitations, the observed runoff data were analyzed as described below.

The first step in flow data processing was to identify flow peaks for individual events and to subject these peaks to frequency analysis. For this purpose, a partial duration analysis was performed using all events with the peak weir head greater than 0.180 m. The resulting set of data included 18 events. The results of frequency analysis based on the use of the Weibull's plotting position formula (5) are shown in Table 7 together with the month of storm occurrence. The flow peaks used in this analysis were adjusted by subtracting the baseflow using the procedure explained in Chapter 7.

The results of frequency analysis in Table 7 were then plotted in Fig. 7. In this figure, a simple regression line was fitted through the data to facilitate interpolation and extrapolation of discharges for various standard return periods. Although the short record length

Table 7. Frequency Analysis of Runoff Peaks Observed* at the Newtown Storm Sewer Outfall

Rank	Storm Number	Peak Discharge (m^3/s)	Return Period $T = (N + 1)/R$ (yr)	Month of Storm Occurrence
1	76	0.391	4	9
2	73	0.368	2	9
3	19	0.330	1.33	11
4	63	0.328	1.00	8
5	246	0.314	0.80	8
6	142	0.301	0.67	7
7	152	0.293	0.57	8
8	253	0.263	0.50	9
9	69	0.242	0.44	9
10	14	0.218	0.40	11
11	140	0.198	0.36	7
12	193	0.196	0.33	1
13	157	0.193	0.31	9
14	239	0.192	0.29	7
15	251	0.177	0.27	8
16	80	0.164	0.25	10
17	198	0.156	0.24	2
18	173	0.149	0.22	10

* Corrected by subtracting baseflow

limits the reliability of discharges estimated for various return periods, the experimental frequency curve provides some guidance for drainage design in similar developments. Using the data in Fig. 7, runoff peak flows were determined for various return periods common in drainage design, expressed as flows per unit catchment area, and presented in Table 8.

It was also of interest to investigate when runoff events with large peaks occur during the year, because such information would be helpful in assessing the initial catchment conditions in runoff computations. The month of occurrence of each of the large runoff events is also listed in Table 7, and plotted in Fig. 8.

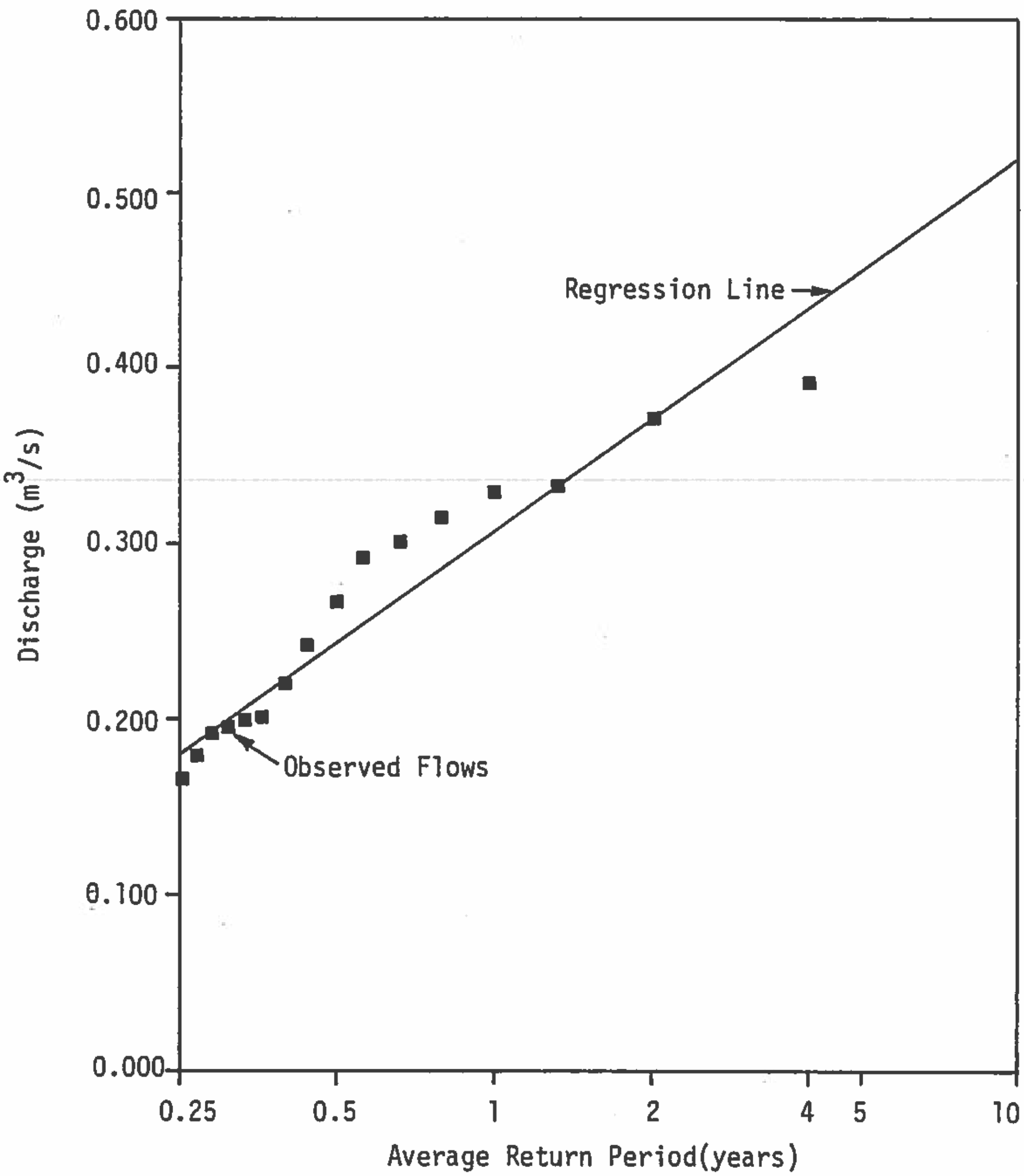


Fig.7. Newtown Runoff Peak Frequency Curve

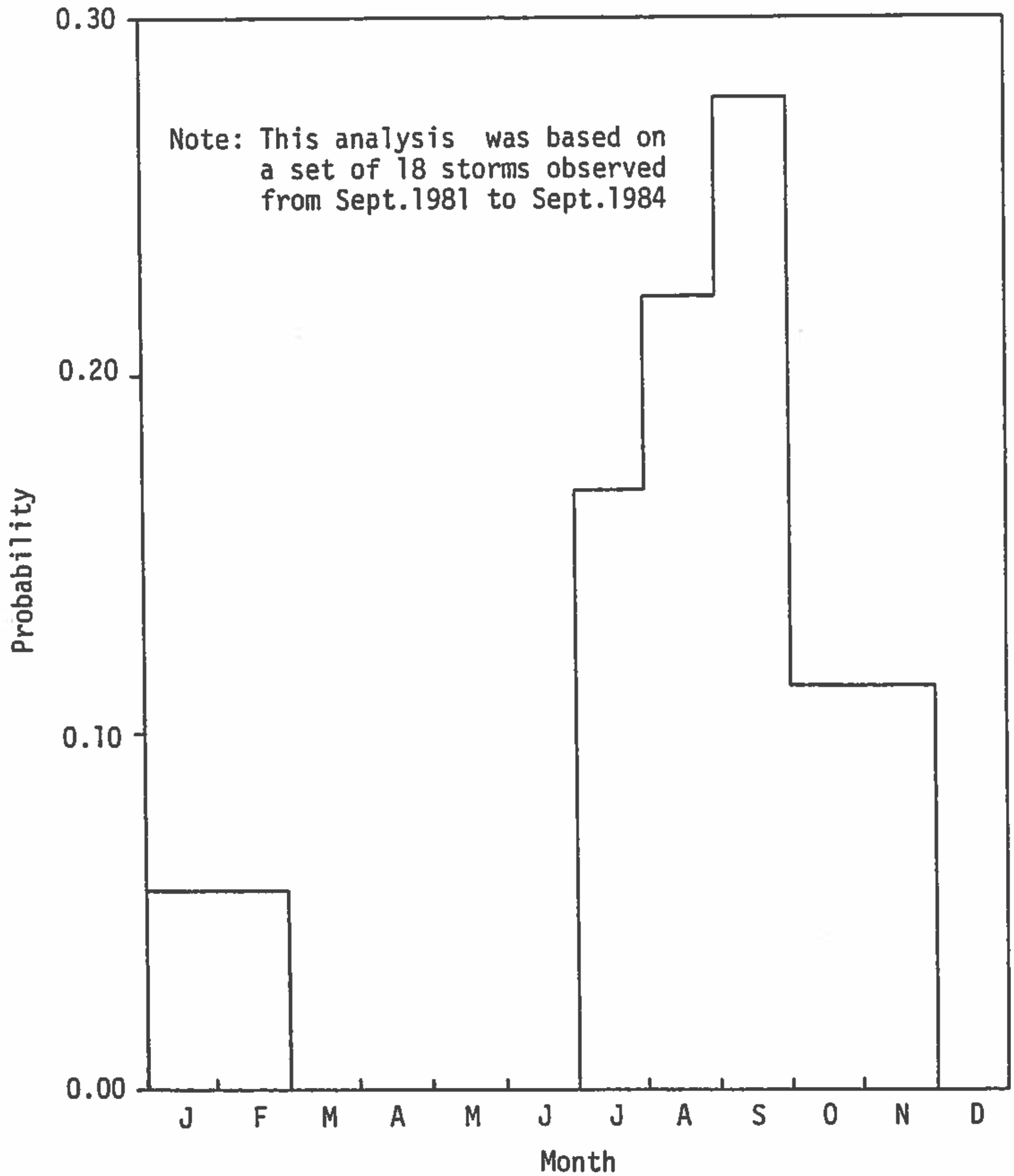


Fig.8. Monthly Distribution of 18 Top-Ranked Runoff Peaks During the Study Period

Table 8. Estimates of Runoff Peaks per Unit Catchment Area for Various Return Periods

	Return Period (yrs)			
	1	2	5	10
Runoff Peak Flow per Unit Area ($m^3/s/ha$)	0.023	0.028	0.034	0.039

It was of interest to note in Fig. 8 that the storms producing the highest runoff peaks occur most frequently (in 67% of all cases) from July to September. For the 18 events included in the analysis, almost 90% of all events occurred during the period with no significant snow cover on the ground or frozen ground in the catchment. If similar considerations are limited just to the top ten ranked events, with return periods from 0.4 to four years, none of the events falls into the period with frozen ground and snow cover on the ground. Such considerations are important for runoff calculations by the rational method. Runoff coefficients selected for the study area should correspond to the non-frozen ground without snow. The assumption of frozen ground, which is sometimes made in drainage design, would greatly increase the return period of the design event. Since winter runoff events did not produce any of the top-ranked runoff peaks, it is recommended to design minor drainage systems in the study area for rainfall events and catchment conditions occurring from April to November.

5.3 Urban Runoff Quality

A general discussion and interpretation of quality of urban runoff in the Newtown catchment are given in Chapter 8.

6.0 APPLICATION OF THE RATIONAL METHOD

One of the early approaches to runoff calculations was the rational method. In its simplest form, the runoff peak flow is expressed as (21):

$$Q = \alpha C\ i_{av} A \tag{6}$$

where Q is the runoff rate at equilibrium (m^3/s), C is the runoff coefficient (dimensionless), α is a unit conversion coefficient (0.00278), i_{av} is the average rainfall intensity (mm/hr) corresponding to the time of concentration t_c, and A is the contributing area (ha). The time of concentration was traditionally taken as the total travel time from the most remote point (in time) in the catchment to the point where runoff is calculated. The total travel time is the sum of the overland travel time plus the time of travel in the transport network. The practice of using the time of concentration of the whole area may lead to underestimation of calculated flows (21) as shown later. In other words, greater peak flows may be calculated from eq. (6) if only a part of the catchment is contributing, but the concentration time is shorter and the corresponding rainfall intensity is higher.Further discussion of individual parameters in eq. (6) follows.

The runoff coefficient in eq. (6) generally depends on the type of catchment surface (pervious or impervious) and its state (i.e., wet or dry), the average rainfall intensity, and the antecedent moisture conditions. In design practice, such considerations are considerably simplified by specifying the runoff coefficients for various types of urban surfaces and the design rainfall intensities in drainage design criteria. If such criteria are not available, the designer should consult various handbooks (11, 21) which list runoff coefficients either for typical surfaces (e.g., pavements, grassed areas) or land use (residential, industrial, etc.). Runoff coefficients for typical surfaces are better transferable than those for typical land use whose specific characteristics may differ from one municipality to another.

Using the surface-type runoff coefficients, the catchment runoff coefficient can be derived as a composite coefficient C_c from the following formula:

$$C_c = \sum_{i=1}^{m} C_i A_i / \sum_{i=1}^{m} A_i \tag{7}$$

where C_i is the runoff coefficient corresponding to the subarea A_i, and i denotes the type of surface.

Eq. (7) was used to derive the runoff coefficient for the Newtown catchment considering the catchment surfaces and runoff coefficients listed below:

Type of Surface	Runoff Coefficient
Effective impervious (A_{ei})	0.95
Supplemental impervious (A_{si})	0.95
Pervious (grassed) (A_{fp})	0.15, 0.18, 0.22

The above listed runoff coefficients were adopted from a design handbook (21). Further comments on their selection follow. The runoff coefficient for the effective impervious area was selected at the upper limit of the recommended values (21) to maintain consistency with the modelling approaches described later. In such approaches, depression storage is the only abstraction considered for impervious areas and, consequently, 95% or even more of rainfall is converted into runoff under design conditions. The coefficient for the supplemental impervious area was also taken as 0.95. Because this coefficient just reflects the rate of inflow to pervious areas, where most of this inflow infiltrates into the ground, the calculated flows are barely sensitive to the variations in the coefficient. This point is further elaborated on in the discussion below.

The selection of the runoff coefficient for pervious areas is particularly difficult. Such a coefficient depends on the physical properties of the soil described by the soil type, surface cover (e.g., grassed or bare) and slope; antecedent moisture; and, rainfall intensity. Three coefficients were used in calculations for Newtown, 0.15, 0.18, and 0.22. These values correspond to grassed areas with moderately heavy soils and a range of surface slopes from 2% to 7% (21). In an attempt to reflect the effect of rainfall intensities on the runoff coefficient, in initial calculations, the lowest coefficient value was used for the 2-year storm, the next one for the 5-year storm, and the highest one for the 10-year storm.

The composite runoff coefficient for the Newtown catchment was calculated in two ways - firstly by considering the effective impervious areas as the only impervious parts of the catchment and taking the rest of the catchment as pervious. In the second approach, three types of areas were considered - effective impervious, supplemental impervious and pervious. Supplemental impervious areas represent parts of some rooftops and/or paved driveways which drain onto lawns. Travel times for these areas are in the order of 10-20 seconds and, therefore, negligible in relation to both inlet times and the times of travel on pervious areas. For that reason, the rainfall intensities considered in both approaches are identical. Both approaches are further detailed below assuming steady-state conditions.

(1) Supplemental impervious areas taken as pervious areas:

$$Q = \alpha C_c i_{av} A \qquad (8)$$

where $C_c = \dfrac{C_i A_{ei} + C_p A_p}{A}$, is the composite runoff coefficient, C_i is the runoff coefficient for effective impervious areas (taken here as 0.95), C_p is the runoff coefficient for pervious areas, A_{ei} is the effective impervious area, A_p is the pervious area (which in this case includes the supplemental impervious area, i.e., $A_p = A_{fp} + A_{si}$), A is the total contributing area ($A = A_{ei} + A_p$) and i_{av} is the average rainfall intensity. After substituting for C_c and simplifying, the following expression is obtained.

$$Q = \alpha(C_i A_{ei} + C_p A_p) i_{av} \tag{9}$$

(2) Supplemental impervious areas (A_{si}) taken as impervious areas which drain instantaneously onto the fully pervious area A_{fp}.

In this case, the runoff from the supplemental impervious areas can be considered as supplemental rainfall applied to the fully pervious area and the expression for the peak flow can be written as

$$Q = \alpha[C_i A_{ei}\ i_{av} + C_p A_{fp}(i_{av} + i_s)] \tag{10}$$

where $A_{fp} = A_p - A_{si}$, and i_s is the supplemental rainfall intensity which can be calculated by uniformly distributing runoff from the supplemental impervious area over the fully pervious area:

$$i_s = \frac{C_i\ A_{si}\ i_{av}}{A_{fp}} \tag{11}$$

After substituting for i_s, one obtains the following expression:

$$Q = \alpha[C_i A_{ei}\ i_{av} + C_p\ A_{fp}\ i_{av} + C_p C_i\ A_{si}\ i_{av}], \tag{12}$$

and after rearranging

$$Q = \alpha[C_i\ A_{ei} + C_p\ (A_{fp} + C_i\ A_{si})] i_{av} \tag{13}$$

Recognizing that $A_p = (A_{fp} + A_{si})$ and C_i (= 0.95) is close to unity, it is obvious that eqs. (9) and (13) yield almost identical results. Even for a very low C_i (= 0.70), eq. (13) produced peak flows only 4% smaller than those calculated from eq. (9).

Thus, it can be concluded that in the applications of the rational method, the supplemental impervious areas which drain onto pervious areas may be taken as a part of the pervious area. It is, therefore, sufficient to consider only two types of surfaces in the calculation of the composite runoff coefficient - the effective impervious areas and the pervious areas. Consequently, the combined runoff coefficient for the Newtown catchment was taken as $C = (C_i A_{ei} + C_p A_p)/A$.

The rainfall intensity i_{av} is defined as the constant rainfall intensity corresponding to the time of concentration t_c. When equilibrium is reached, the runoff rate reaches its maximum for a particular contributing area which at that time may be smaller than the total catchment area. Thus to find the maximum discharge, it is necessary to consider various contributing parts of the catchment and their equilibrium runoff flows and then select the highest one as the maximum runoff. Note that as the contributing area increases, so does the travel time of runoff from the most remote point of the contributing area and the corresponding average intensity obtained from the IDF curves is reduced.

To define the average intensity used in the rational method, one needs to select the contributing area and determine the time of concentration of this area. Because this time of concentration varies with rainfall intensity, the final step in this calculation has to be done by iterations.

It was of interest to evaluate the effect of various partial contributing areas on the runoff peaks calculated for the Newtown catchment. From the theoretical point of view, such partial areas would be best defined by using isochrones. Such a refinement, however, would be clearly outside of the conventional definition of the rational method

(21) and it would also make the application of the method much more tedious. For these reasons, various contributing areas were selected as easily defined parts of the Newtown catchment and, for consistency, their times of concentration were taken as the maximum times of travel.

Three different contributing areas were considered in calculations for the Newtown catchment:

(a) Effective impervious areas only.

(b) Effective impervious areas plus front yards sloping towards streets (i.e., more or less areas within 15 metres of roads).

(c) Entire catchment area.

The above choices were based on drainage patterns in the catchment and differences in time of concentration. The fastest response was expected for the effective impervious area, because in this case, runoff travels on smooth surfaces, such as road gutters, with fair slopes. In case (b), the response is slowed by runoff travel over grassed surfaces of limited lengths. Finally, in case (c), the slowest response resulting from an extended travel over grassed surfaces is expected.

In case (a), runoff flows mostly in street gutters which can be taken as shallow triangular channels. The velocity of flow in such channels can be expressed from a modified Manning's equation used in road drainage in the following form (12):

$$V = 0.96\, n^{-0.75}\, s^{0.375}\, S_x^{\,0.25}\, Q_g^{\,0.25} \qquad (14)$$

where V[m/s] is the mean flow velocity, n is the Manning's roughness coefficient (taken here as $n = 0.013$), s is the gutter longitudinal slope, S_x is the gutter lateral slope (taken here as $S_x = 0.125$ m/m; the other sidewall is vertical), and Q_g is the gutter discharge (m^3/s). The time of travel in gutter is then obtained as $t_{tr} = L/V$ where L is the gutter length.

Using the above expressions, travel times were estimated for effective impervious areas contributing to 14 inlets which were considered in these calculations (for locations, see Fig. 9). For each inlet, the length of travel L and the corresponding slope s were determined from the map, and the discharge Q_g was prorated from the frequency curve in Fig. 7 taking the inlet contributing area as the proportionality factor.

The calculated inlet times varied from 0.8 min. to 6.8 min. depending on the route travelled and the discharge conveyed. In accord with the current practice and for simplicity, it was desirable to replace the 14 different calculated travel times by a single value which would be applicable anywhere in the Newtown catchment. Such a value was selected as t_{tr} = 4 minutes. For this inlet time, 95% of the entire effective impervious area was contributing runoff and, furthermore, this value was applicable for all return periods. In later calculations, however, it was assumed that the whole effective impervious area was contributing runoff. Thus, the calculations for case (a) can be summarized as follows:

Return Period (yr)	Inlet Time (min)	Contributing Effective Impervious Area (ha)
2 - 10	4	2.62

For cases (b) and (c), it was necessary to calculate the time of travel on the overland flow plane. Among various formulas for calculations of times of travel, the kinematic wave formula (11) was considered the best one because of its wide acceptance and completeness in consideration of various factors. In metric units, this formula can be written as

$$t_{tr} = 6.9 \frac{L^{.6} \, n^{.6}}{i^{.4} \, s^{.3}} \qquad (15)$$

where t_{tr} is the time of travel (min.), L is the travelled length (m), n is the Manning's coefficient describing the surface roughness, i is the rainfall intensity (mm/hr), and s is the slope.

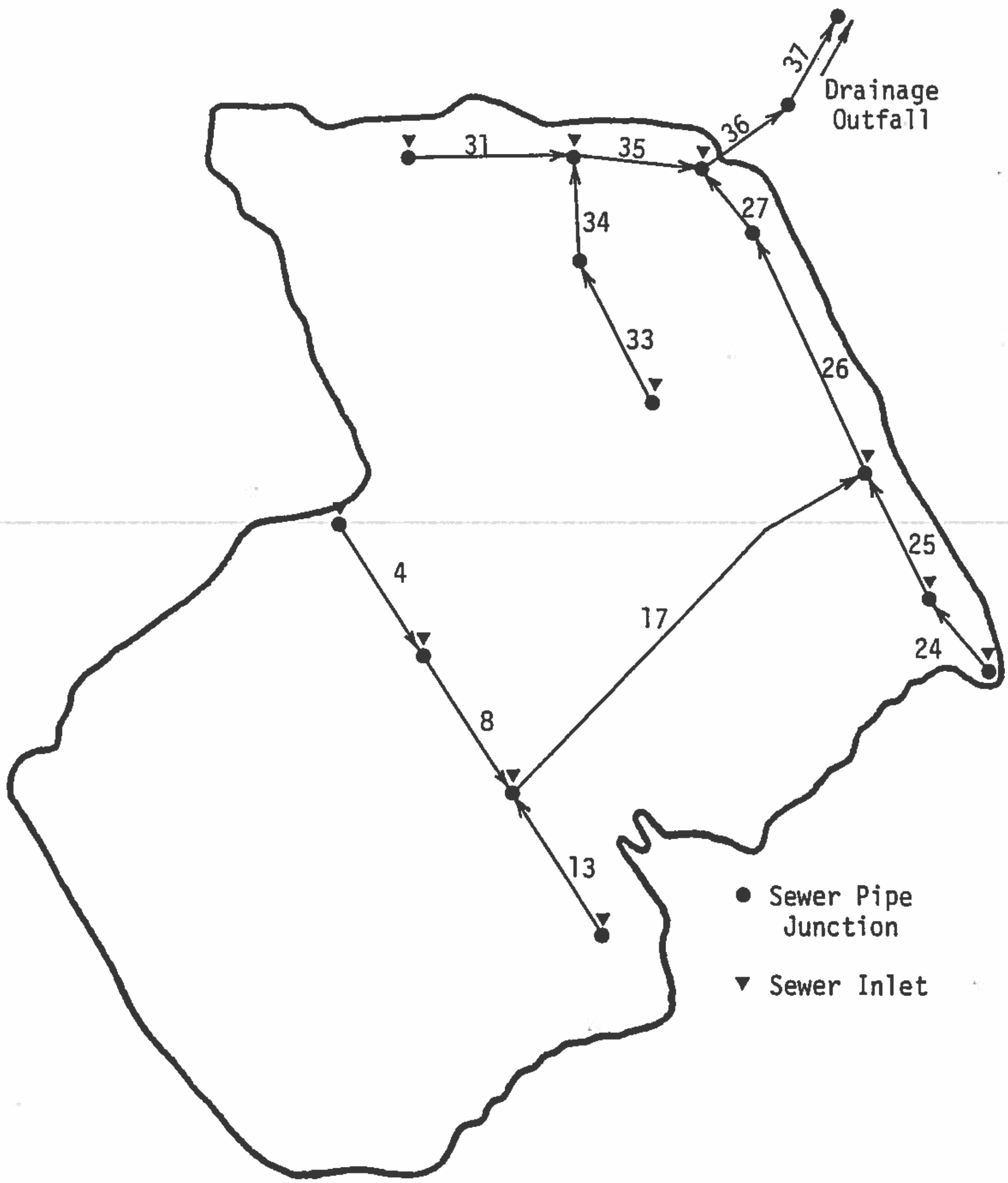


Fig.9. Newtown Sewer Network Used in Applications
of the Rational Method

The rainfall intensities read from the IDF curves for the St. John's airport station were used in the calculations. As shown in Table 5, the rainfall intensity can be expressed as $i = at^b$ and after setting the duration $t = t_{tr}$, the expression for i can be substituted into eq. (15) and the modified equation is solved for the time of travel:

$$t_{tr} = \left(6.9 \frac{L^{.6}\, n^{.6}}{a^{.4}\, s^{.3}}\right)^{\frac{1}{(1+0.4b)}} \tag{16}$$

where a and b are the IDF curve parameters corresponding to a particular return period and the duration time is expressed in minutes.

In case (b), runoff was generated not only on effective impervious elements, but also on areas sloping toward streets. Such areas were generally front yards of residential lots, within about 15 m of streets. In this case, the inlet time was taken as the time of travel on pervious parts (L = 15 m; n = 0.2; and, s = .05) plus the earlier calculated travel time for impervious parts (i.e., case a). From eq. (15), the time of travel on the pervious parts is obtained as $t_{tr} = 32.8/i^{0.4}$ and, for various return periods, the following data are obtained:

Return Period (yr)	Travel Time: Grassed Areas (within 15 m) (min)	Travel Time: Impervious Areas (min)	Total Inlet Time (min)	Total Contributing Area (ha)
2	7	4	11	8.21
5	6	4	10	8.21
10	5	4	9	8.21

Finally, in case (c), the entire catchment was contributing runoff. The times of travel were much longer than in the earlier discussed cases, because of delayed runoff from backyards. In backyards, runoff was draining away from streets towards swales running along the property backlines. The time of travel was calculated as the overland flow time (eq. 15) plus the time of travel in drainage swales. The velocity of flow in swales was calculated from eq. (14). For the longest swales (over 200 m), the calculated times of travel were as long as 22 minutes. Because of these large times and the fact that swales drained onto streets at intermediate points between inlets (not at upstream drainage area ends), the inlet times were taken as the sum of the overland travel time and the travel time in swales. The results of such calculations are listed below.

Return Period (yr)	Travel Time (min) Overland Flow	Swale Flow	Inlet Time (min)	Total Contributing Area (ha)
2	7	22	29	13.23
5	6	21	27	13.23
10	5	21	26	13.23

The calculations of runoff flows throughout the Newtown sewer network were done for the above three cases using the existing storm sewers as the transport network. The concentration times at individual points in this network were taken as the inlet times plus the times of travel in the sewer network. The results of such calculations are shown in Table 9. It should be added for clarity that the flows were calculated for the St. John's airport IDF curves and the pervious area runoff coefficients of 0.15, 0.18 and 0.22 for the return periods of 2, 5 and 10 years, respectively. A brief discussion of these results follows.

The peak flows calculated for various return periods were plotted together with observed flows in Fig. 10. It is apparent from this figure that the calculated flows were significantly smaller than the corresponded observed flows, by about 16%. Such deviations cannot

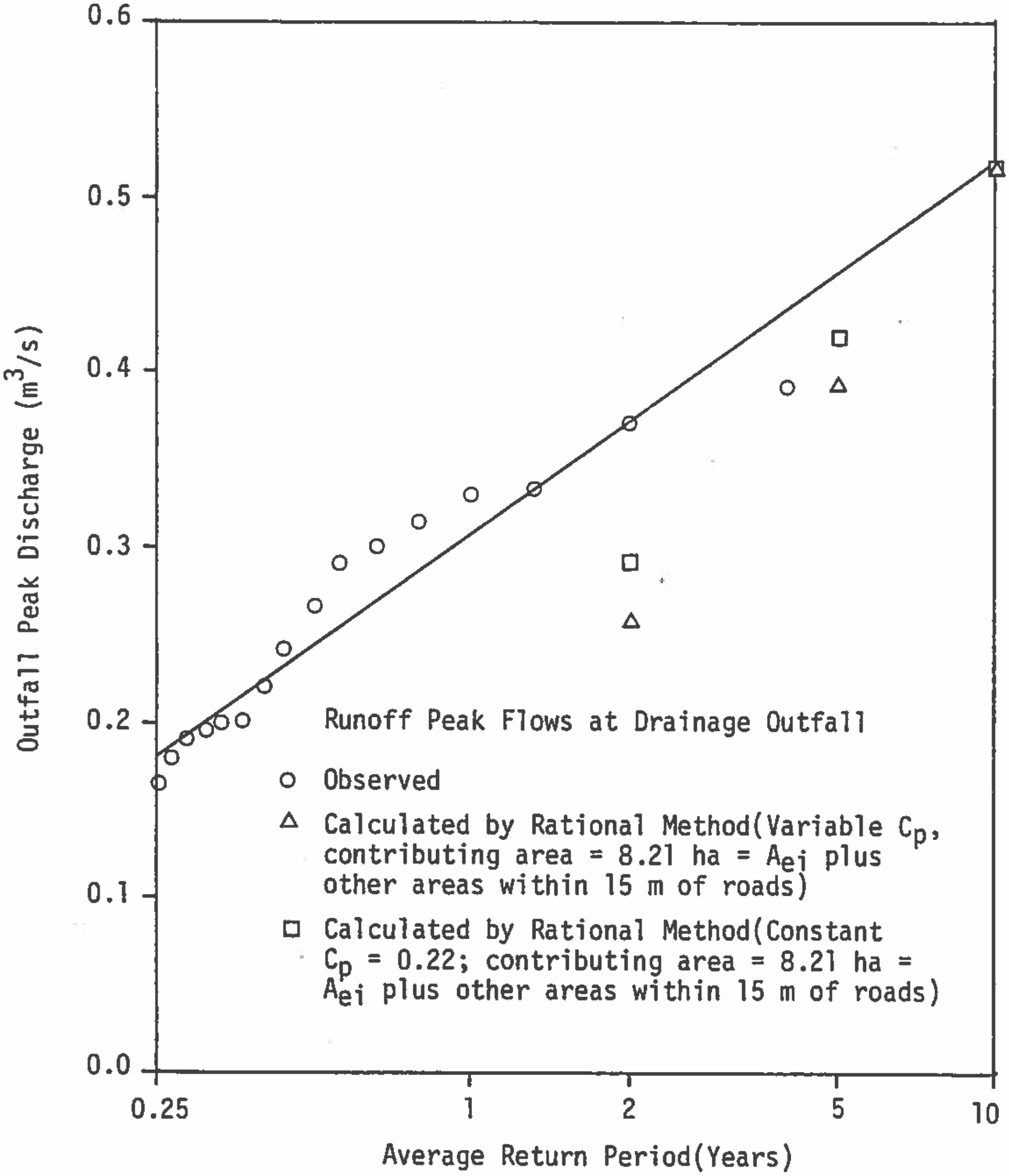


Fig.10. Newtown Runoff Peak Flows: Observed and Calculated from the Rational Method

Table 9. Runoff Peaks Calculated from the Rational Method for Various Return Periods and Contributing Areas

Contributing Area	Runoff Peak Flows (m^3/s)		
	Return Period (yr)		
	2	5	10
1. Varying Runoff Coefficient for Pervious Parts (C_p = 0.15, 0.18 and 0.22 for return periods of 2, 5 and 10 years, respectively			
(a) Effective impervious area A_{ei} only (2.62 ha)	0.246	0.350	0.420
(b) A_{ei} plus other areas within 15 m of roads (8.21 ha)	0.259	0.392	0.516
(c) Whole catchment area (13.23 ha)	0.224	0.337	0.442
2. Constant Runoff Coeffficients (C_i = 0.95, C_p = 0.22)			
(d) A_{ei} plus other areas within 15 m of roads (8.21 ha)	0.292	0.419	0.516
(e) Whole catchment area (13.23 ha)	0.266	0.371	0.442

be fully attributed to the rational method calculations, because they may be also caused by uncertainties in the frequency curve which was produced from a short runoff data record. These uncertainties would be particularly significant in the region of return periods greater than two years. Noting the earlier discussed frequent occurrences of high-intensity storms during the study period, the frequencies assigned to observed flows and plotted in Fig. 10 may be underestimated. Under such circumstances, the discharges plotted in Fig. 10 may be overestimated by as much as 15-20%.

Notwithstanding the uncertainties in the runoff frequency curve, it was noted that the calculted runoff peaks were particularly low for shorter return periods (two and five years) for which low runoff coefficients were used for pervious areas (C_p = 0.15 and 0.18, respectively). To reduce the deviations of the calculated values from the frequency curve and also for simplicity, a constant runoff coefficient (C_p = 0.22) was adopted for all return periods and the newly calculated peak discharges (Tables 10 and 11) were also plotted in Fig. 10. On the average, these peaks were about 11% smaller than those indicated by the frequency curve. Such values appeared to be within the uncertainty bounds of the frequency curve and, therefore, no further adjustments were made in rational method calculations.

Comparison of peak discharges in Tables 10 and 11 indicated that case (b), for which the runoff contributing area comprises A_{ei} and pervious areas within 15 m of streets, produced the highest runoff peaks at the outlet and all the intermediate points with the total contributing areas greater than one-quarter of the total catchment area. For the remaining sections of the sewer system, which represent the upstream reaches, cases (a) or (c) produced slightly higher discharges. Such exceedances were characterized by the mean of 0.007 m^3/s and the maximum of 0.015 m^3/s. When examining the sewer system sized for case (b), all the slightly higher discharges found for cases (a) and (c) were conveyed by the sewers without exceeding their design capacity. It appears from Tables 10 and 11 that, in actual sewer design, the effects of partial contributing areas should be considered for all the sewers in the system.

Newtown sewer system pipes were sized to convey the calculated 5-year peak flows. For 15 pipes with significant stormwater inflows, the existing as well as newly designed diameters are listed in Table 12. The remaining 22 pipes have the minimum diameter of 0.305 m. Except for the pipes with the minimum diameter, the newly proposed pipes are generally one or two sizes smaller than the existing ones.

The costs of the existing and newly-proposed sewer systems are compared in Table 13. For this purpose, only the costs of sewer pipes and their installation were considered, because only such costs would

Table 10. Calculation of Runoff Peaks from the Rational Method (Contributing Area = Effective Impervious Areas Plus Any Other Areas Within 15 m of Roads; $C_i = 0.95$; $C_p = 0.22$)

Pipe No.	Contributing Area (ha)			2-Year Storm			5-Year Storm			10-Year Storm		
	Effective Impervious	Other Within 15 m	Total	t_c (min)	i (mm/hr)	Q_p (m^3/s)	t_c (min)	i (mm/hr)	Q_p (m^3/s)	t_c (min)	i (mm/hr)	Q_p (m^3/s)
4	0.26	0.51	0.77	11	39	0.038	10	56	0.056	9	71	0.070
8	0.53	1.13	1.66	13	36	0.074	12	51	0.107	11	64	0.133
17	1.19	2.62	3.81	15	34	0.158	13	48	0.229	12	60	0.284
21	1.19	2.62	3.81	16	32	0.152	14	46	0.220	13	58	0.272
22	1.19	2.62	3.81	17	31	0.146	15	44	0.210	14	55	0.260
26	1.68	3.74	5.42	17	30	0.202	16	44	0.291	14	54	0.360
27	1.68	3.74	5.42	19	29	0.196	17	42	0.281	15	52	0.347
36	2.62	5.59	8.21	19	29	0.296	18	41	0.425	16	51	0.524
37	2.62	5.59	8.21	20	28	0.292	18	41	0.419	16	50	0.516
13	0.30	0.64	0.94	11	39	0.046	10	56	0.067	9	71	0.084
25	0.18	0.32	0.50	11	39	0.026	10	56	0.038	9	71	0.047
31	0.33	0.65	0.98	11	39	0.046	10	56	0.067	9	71	0.084
33	0.08	0.17	0.25	11	39	0.012	10	56	0.018	9	71	0.022
34	0.08	0.17	0.25	13	36	0.011	11	53	0.017	10	66	0.021
35	0.65	1.20	1.85	14	34	0.083	13	50	0.120	11	62	0.150

Table 11. Runoff Peak Flows Calculated from the Rational Method for Various Contributing Areas (C_i = .95, C_p = .22)

Pipe Number	Contributing Area = Effective Impervious Areas (A_{ei}) Only			Contributing Area = Whole Catchment		
	Return Period (Years)			Return Period (Years)		
	2 - Yr	5 - Yr	10 - Yr	2 - Yr	5 - Yr	10 - Yr
4	0.045	0.064	0.076	0.025	0.035	0.042
8	0.075	0.107	0.128	0.054	0.076	0.091
17	0.149	0.213	0.255	0.129	0.180	0.215
21	0.139	0.198	0.238	0.127	0.177	0.211
22	0.129	0.184	0.221	0.124	0.173	0.206
26	0.176	0.250	0.300	0.167	0.233	0.278
27	0.166	0.236	0.284	0.164	0.229	0.273
36	0.251	0.358	0.430	0.268	0.374	0.446
37	0.246	0.350	0.420	0.266	0.371	0.442
13	0.053	0.074	0.089	0.029	0.041	0.049
25	0.032	0.045	0.054	0.018	0.025	0.030
31	0.058	0.082	0.097	0.041	0.057	0.068
33	0.014	0.020	0.024	0.021	0.030	0.036
34	0.012	0.017	0.020	0.007	0.010	0.012
35	0.085	0.121	0.145	0.072	0.101	0.120

Table 12. Sizing of Storm Sewers Based on the Rational Method and a 5-Year Rainfall IDF Curve

Pipe No.[1]	Design Flow (m^3/s)	Required Pipe Diameter (in)	Required Pipe Diameter (m)	Selected Pipe Diameter (in)	Selected Pipe Diameter (m)	Actual Pipe Diameter (in)	Actual Pipe Diameter (m)
4	0.064	14.3	0.345	15	0.381	21	0.533
8	0.107	16.2	0.412	18	0.457	24	0.610
17	0.229	20.6	0.525	21	0.533	24	0.610
21	0.220	18.6	0.472	21	0.533	24	0.610
22	0.210	18.0	0.458	18	0.533	24	0.610
26	0.291	18.6	0.473	21	0.533	24	0.610
27	0.281	23.0	0.585	24	0.610	30	0.762
36	0.425	25.0	0.644	27	0.686	36	0.914
37	0.419	27.9	0.709	30	0.762	36	0.914
13	0.074	14.1	0.344	15	0.381	18	0.457
25	0.045	9.9	0.236	12	0.305	12	0.305
31	0.082	12.0	0.282	12	0.305	12	0.305
33	0.030	8.1	0.171	12	0.305	12	0.305
34	0.017	7.3	0.186	12	0.305	15	0.381
35	0.121	18.0	0.453	18	0.457	24	0.610

[1] Nineteen pipes (Nos. 1, 2, 3, 5, 6, 7, 9, 10, 11, 14, 15, 16, 18, 19, 20, 23, 28, 29 and 32) which have no sewer inlets are considered as minimum-diameter pipes (D = 0.305 m). Three pipes (Nos. 12, 24 and 30) receive a very small inflow of storm water and are also considered as minimum-diameter pipes.

Table 13. Index Costs of Sewer Installations for the Existing and Two Newly Proposed Sewer Systems

		Existing Design		New Design, Minimum Sewer Size 0.305 m		New Design, Minimum Sewer Size 0.203 m	
Pipe No.	Length (m)	Diameter (m)	Index Costs ($)	Diameter (m)	Index Costs ($)	Diameter (m)	Index Costs ($)
1	67.1	0.305	2,684	0.305	2,684	0.203	1,880
2	122.0	0.305	4,880	0.305	4,880	0.203	3,417
3	28.1	0.381	1,375	0.305	1,122	0.203	786
4	75.6	0.534	5,068	0.381	3,706	0.381	3,706
5	19.5	0.305	781	0.305	781	0.203	547
6	111.9	0.305	4,477	0.305	4,477	0.203	3,136
7	19.2	0.305	769	0.305	769	0.203	538
8	75.3	0.610	5,725	0.458	4,369	0.458	4,369
9	71.7	0.305	2,867	0.305	2,867	0.203	2,008
10	115.9	0.305	4,636	0.305	4,636	0.203	3,247
11	22.6	0.381	1,106	0.305	903	0.203	632
12	25.3	0.305	1,013	0.305	1,013	0.305	1,013
13	74.7	0.458	4,334	0.381	3,662	0.381	3,662
14	27.1	0.305	1,086	0.305	1,086	0.203	760
15	104.3	0.305	4,172	0.305	4,172	0.203	2,922
16	18.3	0.381	897	0.305	732	0.203	513
17	74.1	0.610	5,633	0.534	4,966	0.534	4,966
18	64.1	0.305	2,562	0.305	2,562	0.203	1,794
19	20.7	0.305	830	0.305	830	0.203	581
20	61.9	0.305	2,477	0.305	2,477	0.203	1,734
21	100.0	0.610	7,603	0.534	6,703	0.534	6,703
22	48.8	0.610	3,709	0.458	2,831	0.534	3,270
23	58.3	0.305	2,330	0.305	2,330	0.203	1,632
24	40.9	0.305	1,635	0.305	1,635	0.305	1,635
25	64.7	0.305	2,586	0.305	2,586	0.305	2,586
26	121.1	0.610	9,202	0.534	8,113	0.534	8,113
27	41.2	0.763	3,870	0.610	3,129	0.610	3,129
28	105.2	0.305	4,209	0.305	4,209	0.203	2,948
29	75.6	0.381	3,706	0.305	3,026	0.203	2,119
30	89.4	0.305	3,575	0.305	3,575	0.305	3,575
31	79.3	0.381	3,886	0.305	3,172	0.305	3,172
32	72.0	0.305	2,879	0.305	2,879	0.203	2,016
33	72.6	0.305	2,904	0.305	2,904	0.305	2,904
34	51.9	0.381	2,541	0.305	2,074	0.305	2,074
35	63.1	0.610	4,798	0.458	3,662	0.458	3,662
36	43.3	0.915	4,851	0.686	3,681	0.686	3,681
37	59.5	0.915	6,661	0.763	5,591	0.763	5,591
Total Index Costs			$128,300		$114,800		$101,000

change as a result of changes in sewer diameter. Furthermore, the interest is limited to a relative cost comparison. Under such circumstances, it was possible to adopt a sewer installation cost formula from the literature (15) in the following form:

$$T = 10.98\ D + 0.8\ H - 5.98 \tag{17}$$

where T = the sewer installation costs (dollars per linear ft), D = the sewer diameter (ft), and H = the average sewer invert depth below the surface. This empirical formula was recommended for $D \leq 3$ ft and $H \leq 10$ ft. After conversion to metric units, eq. (17) will take the following form:

$$T = 118.02\ D + 8.60\ H - 19.60 \tag{18}$$

where the units are as follows: T [$/m], D[m], H[m] and the ranges of validity are $D \leq .915$ m and $H \leq 3.05$ m.

It should be emphasized that eq. (18) refers to the early 1970's costs. While the costs calculated from the formula are clearly outdated, they can be used as cost indices for comparing various design variants.

A brief inspection of sewer plans indicated that the average depth of sewer inverts below the ground surface was about 2.75 m and this value was used in all calculations.

The calculation of sewer costs was done for three cases - firstly, for the existing system (as built), secondly, for a newly designed system with a minimum pipe diameter of 0.305 m and, thirdly, for the newly designed system and two smallest pipe sizes - 0.305 m for pipes with some stormwater inflow and 0.203 m for pipes serving for foundation and basement drainage only. All the calculated costs are shown in Table 13.

Several interesting findings can be inferred from Table 13. The first one is that almost 40% of the total costs are directly attributable to the foundation and basement drainage. The dimensions of the sewers serving for foundation and basement drainage are not affected by the hydrological design, because they are simply designed as minimum-diameter sewers.

Using field observations of rainfall and runoff from the Newton catchment, it would be possible to design a new system, for a 5-year storm, at somewhat lower costs - about 90% of the costs of the existing system. The new system would be designed for the same conditions as the existing system - free-flowing sewers and the minimum pipe diameter of 0.305 m. By reducing the minimum size of sewers serving just for foundation and basement drainage from 0.305 m to 0.203 m, the new system costs would be reduced to about 79% of the existing system costs.

The findings regarding the application of the rational method to the Newtown catchment are summarized below.

Field observations of rainfall and runoff in the Newtown catchment can be used to further improve applications of the rational method to sewer design in the St. John's area. During the study period, the rainfall data from Newtown exhibited somewhat higher intensities than those observed at the St. John's airport station. It is therefore recommended to develop reliable IDF curves for Newtown which according to the AES recommendations would require at least seven years of rainfall data. Thus the present rainfall monitoring program should be continued at least until 1988.

The runoff peak flows calculated from the rational method approximated fairly the observed flows provided that the following steps were taken in the calculations.

(a) The St. John's airport IDF curves were used.
(b) Runoff travel times were calculated from a modified Manning's equation for triangular channels (eq. 14) and from the kinematic wave equation (eqs. 15, 16) using the lengths and slopes measured from the map, the rainfall intensities from the airport IDF curves,

and taking the overland flow plane roughness coefficients as $n = 0.013$ and 0.20 for impervious and pervious areas, respectively. Using such procedures, the calculated inlet times varied from 4 to 29 minutes depending on the return period and the contributing area.

(c) The highest runoff flow at the outlet was obtained for the case where only 62% of the total catchment area was contributing runoff. This contributing area represented all effective impervious areas plus other contributing areas within 15 m of streets (i.e., both supplemental impervious and pervious areas, mostly front yards). For this particular case, the combination of the contributing area, the composite runoff coefficient and the relatively high rainfall intensity corresponding to the fairly fast response of the partial contributing area yielded the highest runoff peak. The runoff coefficients used in these calculations were constant for all return periods (2-10 years) and their values were 0.95 and 0.22 for impervious and pervious areas, respectively.The corresponding composite runoff coefficient was $C_c = 0.45$ ($A = 8.21$ ha). It was further noted that for the purpose of runoff peak calculations, the runoff coefficient for supplemental impervious areas, which drain onto pervious areas, should be taken as that for pervious areas.

(d) Using the procedures summarized above, the calculated runoff peak flows were comparable to those corresponding to a frequency curve fitted to observations. As noted earlier, both sets of data contain significant uncertainties.

(e) Runoff peaks at the outlet calculated for any other contributing areas, full or partial, different from those described in item (c) above, were always smaller. For the whole catchment contributing runoff and return periods from 2 to 10 years, the highest calculated peaks represented only 86% of the maximum values. This was caused by long travel times on pervious parts and the corresponding reductions in mean rainfall intensities. The composite runoff coefficient, for the entire catchment, was $C_c = 0.36$.

Peak calculations for the effective impervious areas only yielded peaks ranging from 81% to 84% of the maximum values at the outlet. Thus in applications of the rational method, it is recommended to make several calculations for full and partial contributing areas, in order to closely approximate the maximum values. The consideration of three cases should be sufficient to get close to the maximum values - the effective impervious areas only, the effective impervious areas plus any other contributing areas within say 10-30 m of the effective impervious areas (i.e., more or less front yards), and finally the entire catchment area contributing. From the theoretical point of view, the partial contributing areas would be best defined by isochrone curves. Such a procedure has not been adopted here because it would have greatly increased the tediousness of calculations and would have deviated from the conventional application of the rational method (21).

7.0 HYDROLOGIC MODELLING OF URBAN RUNOFF

The use of mathematical models in water resources design is guided by numerous considerations which extend beyond the scientific evaluation of the correctness of the model formulation and its applicability to the problem on hand. Past experience shows that the search for the "best" model, defined in terms of its ability to reproduce observed data, is illusory. Because of inherent uncertainties in observations, model formulations and application methods, a number of models may perform equally well in a given situation. The selection of the most appropriate model is then governed by such considerations as the availability of the model and its documentation, effort required for its application, and the model acceptance in practice. In urban drainage design, it is generally recommended to use non-proprietary, widely-used, well-documented models which are continuously maintained and updated by some agencies. Furthermore, efforts and costs required for model application should not be excessive. In particular, applications of the model should not require an excessive level of expertise and the input data requirements should be controlled by the user depending on the complexity of the application. The model should run on standard computer hardware, including microcomputers.

On the basis of the above desired model features and an evaluation of the existing models given elsewhere (17), two hydrological models were selected for simulations of urban runoff in the Newtown test catchment - the ILLUDAS model of the State Water Survey of Illinois (19) and the SWMM model of the U.S. Environmental Protection Agency (9). Both models are widely used in engineering practice and continuously maintained and updated by several agencies. For details of these models, a reference is made to the existing documentation (9,19). Various aspects of both models are described in this section only in connection with description and interpretation of results obtained for the Newtown catchment.

7.1 ILLUDAS Application

7.1.1 Model overview

The ILLUDAS model was developed for design of urban drainage in the early 1970's. It is a deterministic model which calculates runoff for three types of surface areas - directly-connected impervious areas, supplemental paved areas, and pervious areas. For directly-connected impervious areas (also referred to as effective impervious areas), which drain directly into the transport elements, the only abstraction considered is the surface detention storage. Supplemental paved areas behave similarly to directly-connected areas, but they drain onto pervious areas. On pervious areas, two types of abstractions are considered - surface detention storage and infiltration. For infiltration, standard curves have been devised for soils of the four hydrologic groups proposed by the U.S. Soil Conservation Service. These four groups can be described as follows (19):

A - Low runoff potential, high infiltration rates (consist of sand and gravel).

B - Moderate infiltration rates and moderately well drained.

C - Slow infiltration rates (may have a layer that impedes downward movement of water).

D - High runoff potential, very slow infiltration rates (consist of clays with a permanent high water table and a high swelling potential).

For each of these groups, a standard infiltration curve was devised, following the Horton's equation, and built into the model.

Representative values of the available storage capacity were established for four types of antecedent moisture conditions (AMC) which are defined below.

ILLUDAS AMC Number	Soil Moisture Description	Total Rainfall During 5-Days Preceeding the Storm (mm)
1	Bone Dry	0
2	Rather Dry	0-12.7
3	Rather Wet	12.7-25.4
4	Saturated	over 25.4

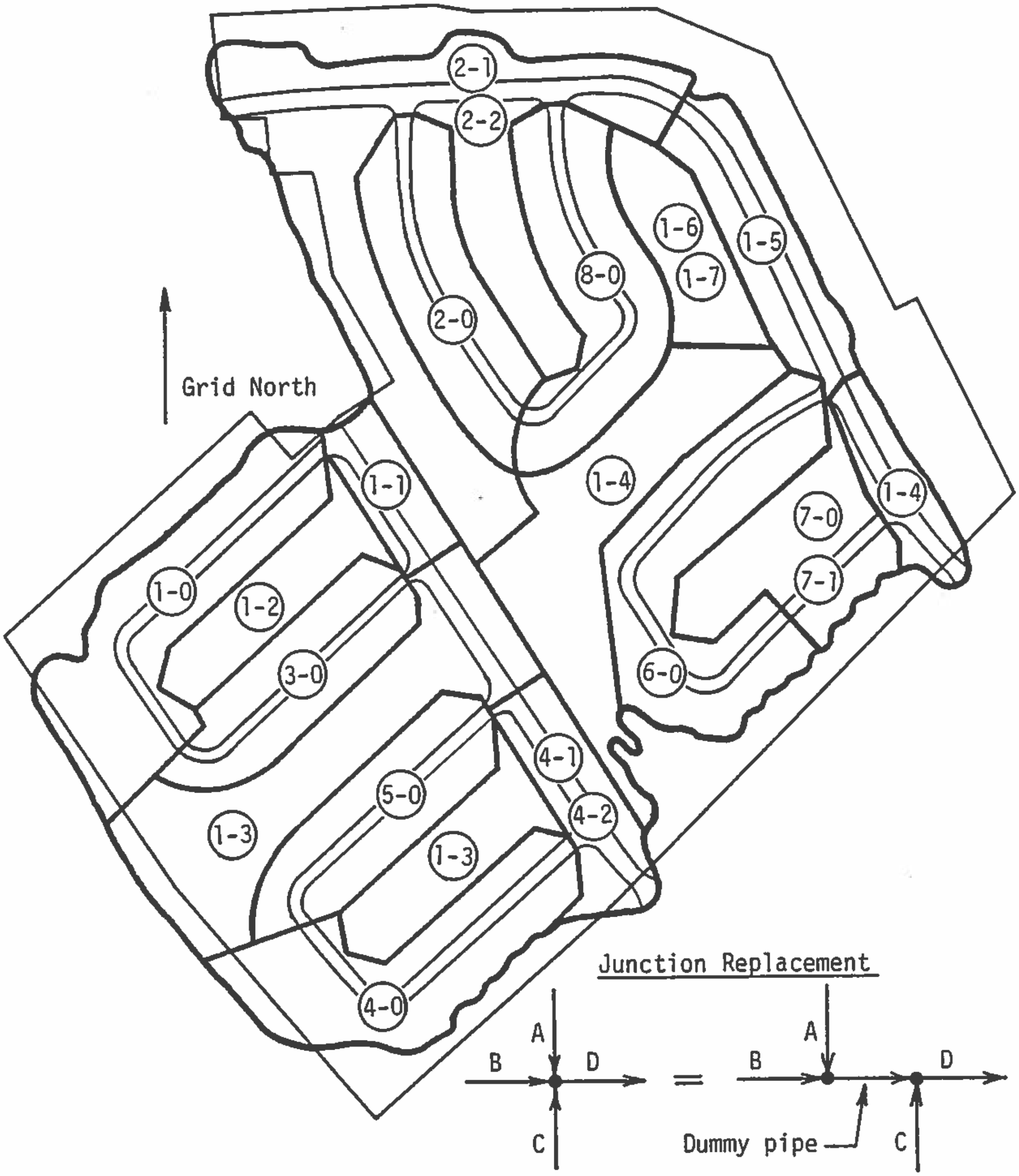
Using the rainfall excesses (supply rates) for all three types of areas, inlet hydrographs are developed from the supply rates and time vs contributing area curves. The inlet hydrographs are then combined into one and routed through the sewer network by using either a time-lag procedure, or a storage routing method. The former procedure was used in this study because of numerical instabilities encountered when attempting to use the storage routing procedure. In the following, the applicaton of the ILLUDAS model to the Newtown catchment is briefly described.

7.1.2 Application of ILLUDAS to the Newtown Catchment

The ILLUDAS model applied was the 1979 version with the time-lag routing option. For proper model application, the catchment needs to be divided into a number of subcatchments which correspond to the number of pipes in the transport system. Thus the transport system (a network of sewers and gutters) was established first and then the contributing subcatchments for individual pipes/channels were established from maps and drainage patterns.

For the purpose of runoff modelling, the Newtown catchment was discretized into 20 subcatchments which were drained by 20 transport elements (see Fig. 11). Detailed characteristics of subcatchments and transport elements are given in Tables 14 and 15. Most parameters in these tables are self-explanatory; additional comments follow.

In Table 14, all transport elements are listed. Such elements are either sewer pipes, or street gutters. Gutters were modelled inside all crescents (Lancaster, Oxford, Princeton and Dalhousie) in sections without sewer inlets. Street gutters were taken as shallow (d = 0.15 m) triangular channels of a relatively low roughness (n = .015). Sewer pipes in Newtown are made of corrugated steel pipes with the roughness characterized by n = 0.024. Two further explanations are made. When selecting the sewer pipes to be modelled, the pipe No. 1-3 (the first digit is the branch number, the second digit is the reach number) replaced three pipes of identical diameter (0.61 m) but different slopes. The equivalent slope used for the single pipe 1-3 preserved the time of travel through this section. Furthermore, in the ILLUDAS model only two inflowing and one outflowing pipes can be joined at a junction.For that reason, the junctions of three incoming pipes were replaced by two junctions with two incoming pipes each and a short connecting pipe (L = 3 m; see pipes 1-7, 4-2, and 7-1). This procedure is further explained in Fig. 11.



Note: In ILLUDAS runs, four-pipe junctions had to be replaced by two three-pipe junctions connected by short dummy pipes. To maintain an equal number of pipes and subcatchments, four dummy subcatchments (1-7, 2-2, 4-2, and 7-1) with negligible areas were introduced.

Fig.11. Discretization of the Newtown Catchment for Application of the ILLUDAS Model

Table 14. Transport Elements Used in ILLUDAS Simulations

Number	Branch and Reach	Length (m)	Slope (%)	Pipe	Gutter[1]	Diameter (m)	n
1	1 - 0	107	5.2		•		0.015
2	1 - 1	76	0.5	•		0.53	0.024
3	3 - 0	76	6.0		•		0.015
4	4 - 0	98	5.0		•		0.015
5	4 - 1	72	0.8	•		0.46	0.024
6	5 - 0	76	6.2		•		0.015
7	1 - 2	75	0.8	•		0.61	0.024
8	4 - 2	3	0.8	•		0.46	0.024
9	7 - 0	62	1.8	•		0.30	0.024
10	6 - 0	131	1.6		•		0.015
11	1 - 3	223	1.3	•		0.61	0.024
12	7 - 1	3	1.8	•		0.30	0.024
13	2 - 0	76	2.7		•		0.015
14	2 - 1	79	2.2	•		0.38	0.024
15	8 - 0	124	1.6	•		0.38	0.024
16	1 - 4	121	2.7	•		0.61	0.024
17	1 - 5	41	0.8	•		0.76	0.024
18	2 - 2	63	0.6	•		0.61	0.024
19	1 - 6	103	0.9	•		0.91	0.024
20	1 - 7	3	0.9	•		0.91	0.024

[1] Gutters were taken as open channels, with a triangular cross-section. The curb side wall was vertical and the road side wall had a slope of 1:8.

Table 15. Subcatchments Used in ILLUDAS Simulations

Sub-catchment Number	Branch and Reach for Drainage	Total Contributing Area (ha)	Directly Connected Paved Areas: Area (ha)	Directly Connected Paved Areas: Max. Path (m)	Directly Connected Paved Areas: Slope (%)	Supplemental Paved Area (ha)	Pervious Areas: Area (ha)	Pervious Areas: Max. Path (m)	Pervious Areas: Slope (%)
1	1 - 0	0.902	0.259	54	0.5	0.277	0.416	47	3.0
2	1 - 1	0.235	0.085	72	0.5	0.057	0.093	24	3.0
3	3 - 0	0.656	0.186	137	6.0	0.231	0.239	19	3.0
4	4 - 0	0.975	0.267	84	2.6	0.214	0.494	21	3.0
5	4 - 1	0.340	0.113	52	0.5	0.061	0.166	24	3.0
6	5 - 0	0.647	0.198	58	6.0	0.214	0.235	18	3.0
7	1 - 2	0.461	0.000	137	6.0	0.000	0.461	139	5.0
8	4 - 2	0.004	0.000	-	-	0.000	0.004	24	3.0
9	7 - 0	0.664	0.089	52	4.9	0.077	0.498	128	4.0
10	6 - 0	1.016	0.304	129	1.0	0.409	0.303	21	2.0
11	1 - 3	1.594	0.081	67	0.5	0.077	1.436	198	4.0
12	7 - 1	0.004	0.000	-	-	0.000	0.004	128	4.0
13	2 - 0	0.672	0.227	84	1.0	0.219	0.226	21	3.0
14	2 - 1	2.019	0.267	78	3.8	0.235	1.517	183	2.0
15	8 - 0	0.757	0.239	64	2.4	0.206	0.312	21	3.0
16	1 - 4	1.178	0.093	31	1.4	0.000	1.085	214	2.0
17	1 - 5	0.704	0.210	50	1.4	0.219	0.275	23	3.0
18	2 - 2	0.004	0.000	-	-	0.000	0.004	21	3.0
19	1 - 6	0.393	0.000	-	-	0.000	0.393	122	2.0
20	1 - 7	0.004	0.000	-	-	0.000	0.004	122	2.0

The discretization of the test catchment into 20 subcatchments more or less followed the transport element network described earlier. For each transport element, it was necessary to determine the total contributing area and its division into the directly connected impervious areas (mostly street pavement, paved driveways, some rooftops draining onto paved driveways), supplemental paved areas (i.e., impervious areas draining onto pervious areas), and pervious areas. Furthermore, for both directly-connected impervious and pervious areas, it was necessary to specify the maximum length of runoff travel and the slope along such paths. Alternately, one could specify the time of travel (i.e., the inlet time) for each subcatchment. All the data given in Table 15 were determined from detailed field surveys of the test catchment.

Finally, the following model parameters common to all subcatchments were also selected on the basis of literature data:

Paved area abstraction	- 1.52 mm (corresponds to the generally recommended value, ref. (9)).
Grassed area abstraction	- 5.1 mm (recommended in the model manual (19))
Predominant soil group	- B (follows from field surveys)

The time increment for rainfall data which equals the computational time step was selected as five minutes. This increment was found sufficient for the relatively slowly responding test catchment and simplified the processing of rainfall data and the running of the model.

The results of ILLUDAS simulations are presented and discussed later in this chapter.

7.2 SWMM Application

In relatively small catchments with simple converging sewer networks without special structures, runoff simulations can be made using only the RUNOFF Block of the SWMM model (9). This approach was used for the Newtown catchment.

The requirements for the application of the SWMM model are similar to those described earlier for ILLUDAS. A brief description of the model structure follows.

In the RUNOFF Block of the SWMM model, runoff is generated on two types of surfaces - impervious and pervious areas. The impervious areas more or less correspond to the directly-connected paved areas in the ILLUDAS model. The abstraction applicable to impervious areas is the surface detention storage, which can be taken as a fixed storage depth or a function of the area slope. Both approaches were used in this study.

For the pervious areas, two abstractions are considered - the surface storage (its recommended value is 4.7 mm (9)) and infiltration. In the model version employed, infiltration was calculated from the Horton equation for user-selected values of three equation parameters. The rainfall excess is routed on overland flow planes to produce the inlet hydrographs. These hydrographs are then routed through the sewer system using a storage routing procedure. In conventional applications of the SWMM model, no accounting for the antecedent moisture is done. However, it would be possible to account for antecedent moisture by somewhat adjusting the infiltration equation parameters.

For the purpose of application of the SWMM model to the Newtown catchment, two discretization approaches were used. In both cases, the catchment was subdivided into 22 subcatchments, but the transport network differed. In the first case, all sewers in the network were considered, even those inside crescents without sewer inlets. In the second approach (see Fig. 12), the transport of runoff from crescents was modelled by means of street gutters and these drained

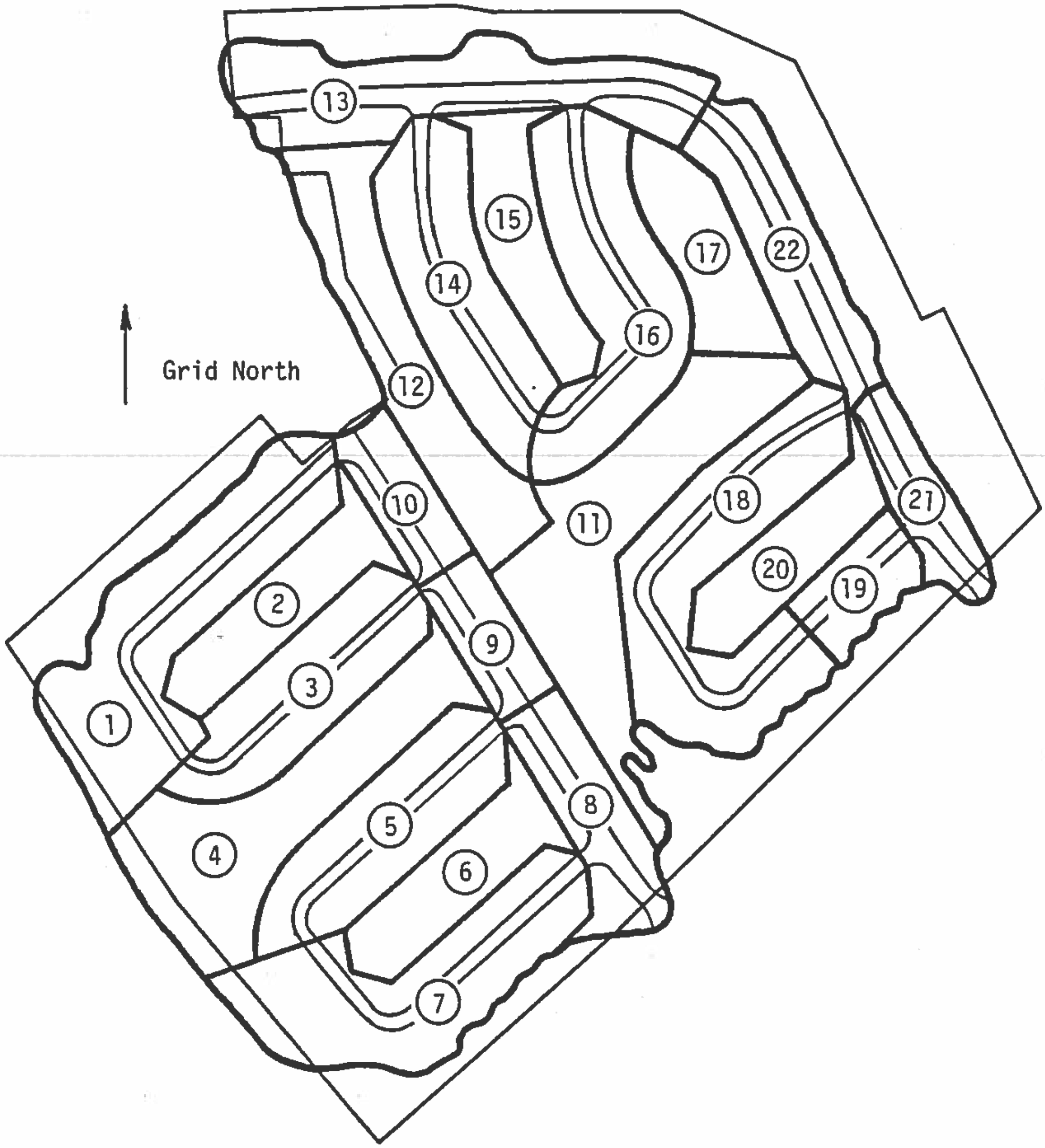


Fig.12. Discretization of the Newtown Catchment
for Application of the SWMM Model

into sewer pipes at sewer inlet points. This second approach led to a much lower number of transport elements. As discussed later, no significant differences between both approaches were found.

Transport elements used in both approaches are listed in Tables 16 and 17.

Basic characteristics of the 22 subcatchments used are listed in Table 18. This table includes the so-called subcatchment widths which represent the physical widths of the overland flow plane. Generally, such widths were taken as the catchment area divided by twice the drainage element length. All the other subcatchment parameters were determined from maps and field surveys.

Finally, it should be added that, according to the SWMM manual (9), the following parameter values were also selected for simulation runs:

Surface slope	3%
Roughness - impervious areas	n = 0.013
- pervious areas	n = 0.250
Surface detention storage	
- impervious areas	1.6 mm (in the first series of runs)
	$.77/\sqrt{S}$ (in subsequent runs) where S = slope in percent
- pervious areas	4.7 mm
Infiltration rates	
- initial	51 - 76 mm/hr
- final	7.6 - 13.2 mm/hr
- decay rate	$.00115\ s^{-1}$

Both the rainfall data interval and the computational time step were taken as five minutes. The results of SWMM simulations are presented later in this chapter.

Table 16. Extended Transport Network Used in SWMM Simulations

Sewer Number	Drains Into	Diameter (m)	Length (m)	Slope (%)	Drains Subcatchment Number
1	2	0.31	67	0.64	-
2	3	0.31	122	5.08	1
3	4	0.38	28	1.40	-
4	8	0.53	76	0.53	2,10
5	6	0.31	20	3.52	-
6	7	0.31	112	5.98	3
7	8	0.31	19	1.75	-
8	17	0.61	75	0.77	4,9
9	10	0.31	72	1.81	-
10	11	0.31	116	4.84	7
11	13	0.38	23	1.08	-
12	13	0.31	25	0.50	-
13	17	0.46	75	0.78	6,8
14	15	0.31	27	3.60	-
15	16	0.31	104	6.20	5
16	17	0.38	18	2.17	-
17	21	0.61	74	0.99	-
18	19	0.31	64	1.00	-
19	20	0.31	21	1.03	-
20	21	0.31	62	1.72	-
21	22	0.61	100	1.52	18
22	26	0.61	49	1.63	-
23	25	0.31	58	2.62	19
24	25	0.31	41	0.97	-
25	26	0.31	65	1.84	20,21
26	27	0.61	121	2.65	11
27	36	0.76	41	0.82	22
28	29	0.31	105	1.07	-
29	31	0.38	76	1.65	14
30	31	0.31	89	4.44	12
31	35	0.38	79	2.23	13,15
32	33	0.31	72	2.03	-
33	34	0.31	73	2.35	16
34	35	0.38	52	1.35	-
35	36	0.61	63	0.58	-
36	37	0.91	43	1.20	17
37	-	0.91	59	0.65	-

Note: Pipe roughness, n = 0.024.

Table 17. Reduced Transport Network Used in SWMM Simulations

Sewer Number	Drains Into	Drains Subcatchment Number	Diameter (m)	Length (m)	Slope (%)	n
1	2	10	0.53	76	0.5	0.024
2	4	2	0.61	75	0.8	0.024
3	4	4,8	0.46	75	0.8	0.024
4	5	6,9	0.61	74	1.0	0.024
5	6	-	0.61	100	1.5	0.024
6	8	-	0.61	49	1.6	0.024
7	8	19,20	0.31	65	1.8	0.024
8	9	11,21	0.61	121	2.7	0.024
9	14	22	0.76	41	0.8	0.024
10	13	12,13,15	0.38	79	2.2	0.024
11	12	16	0.31	73	2.4	0.024
12	13	-	0.38	52	1.4	0.024
13	14	-	0.61	63	0.6	0.024
14	15	17	0.92	43	1.2	0.024
15	-	-	0.92	59	0.7	0.024
16	1	1	*	107	5.2	0.015
17	2	3	*	76	6.0	0.015
18	4	5	*	76	6.2	0.015
19	3	7	*	98	5.0	0.015
20	10	14	*	76	2.7	0.015
21	8	18	*	131	1.6	0.015

* Triangular channel, the curb side wall is vertical and the other side has a slope of 1:8.

Table 18. Characteristics of Subcatchments Used in SWMM Simulations

Subcatchment Number	Drains Into Sewer Number	Area (ha)	Subcatchment Width (m)	Imperviousness (%)	Surface Detention Storage (mm)*
1	16	0.901	397	28.8	0.6
2	2	0.462	55	0	1.6
3	17	0.656	313	28.4	0.4
4	3	0.911	37	0	1.6
5	18	0.646	310	30.9	0.4
6	4	0.457	43	0	1.6
7	19	0.976	392	27.3	0.4
8	8	0.345	134	32.5	0.9
9	4	0.226	78	35.0	0.8
10	1	0.233	82	36.6	0.8
11	8	0.970	40	0	1.6
12	10	0.785	52	0	1.6
13	10	0.792	445	33.5	0.5
14	20	0.671	315	33.6	0.8
15	10	0.447	43	0	1.6
16	11	0.755	384	31.7	0.6
17	14	0.397	37	0	1.6
18	21	1.020	519	30.0	0.6
19	7	0.278	134	31.6	0.4
20	7	0.390	37	0	1.6
21	8	0.209	189	43.7	0.6
22	9	0.703	323	30.0	0.6

* In the discretization with 37 pipes, the surface detention storage for all subcatchments was 1.6 mm.

7.3 Simulation Results

The presentation of results starts with a brief description of rainfall data used in simulations, followed by the presentation of simulations of runoff volumes and peak flows. The timing of runoff peaks was not analyzed for two reasons - the simulation of timing is unimportant in drainage design and the synchronization of rainfall and runoff records was not sufficiently accurate to justify such an analysis.

7.3.1 Storms used in simulations

The rainfall/runoff events used in simulations were observed during the period from November 7, 1981, to September 19, 1984. More observations have been collected since then and these will be analyzed in a later stage of the study.

During the above-noted period, almost 260 runoff events have been observed in the test catchment. Most of these were rather minor and, therefore, of no interest in this study dealing primarily with drainage design. Consequently, only the events with the highest runoff peaks were selected. In a preliminary simulation series, the top ranked 47 events (i.e., ranked according to the observed peak flows) were used. After detailed scrutiny, this set was further reduced because of various limitations of the data. The characteristics of these 47 events are summarized in Table 19.

It is apparent from the data in Table 19 that not all the selected events are suitable for detailed runoff simulations and evaluation of simulation models. Consequently, a somewhat reduced set of events was used in further work. For evaluation of runoff volumes, 23 events were eliminated from the list presented in Table 19. In particular, all the events with qualifying comments in the table, or with rainfall measurements from the CDA station were left out. The remaining storms used for analysis of simulated volumes are denoted by

Table 19. Basic Characteristics of Storms Considered for Runoff Simulations

Number	Storm Number	Date	Rainfall Depth (mm)	Source of Rainfall Data	Data Used in Simulations	Comment
1	14	7/11/81	61.4	CDA[1]	Q[3]	)
2	17	18/11/81	19.4	CDA		)
3	19	25/11/81	83.2	CDA	Q	)
4	24	15/12/81	18.4	CDA		) Inaccurate runoff
5	38	7/04/82	30.2	N[2]		) volume measure-
6	39	14/04/82	11.8	N		) ments - slow
7	42	21/04/82	10.2	N		) recorder chart
8	46	12/05/82	40.0	N		) speed
9	51	15/06/82	52.4	CDA	Q	)
10	52	20/06/82	61.8	CDA	Q	)
11	61	14/08/82	6.0	N	Q	)
12	62	19/08/82	28.2	N	V[4],Q	
13	63	24/08/82	31.2	N	V,Q	
14	64	25/08/82	27.2	N	V,Q	
15	69	2/09/82	16.0	N	V,Q	
16	73	16/09/82	70.0	N	V,Q	
17	76	21/09/82	28.2	N	V,Q	
18	80	2/10/82	45.6	N	Q	Poor synchronization
19	86	23/11/82	25.6	N	V,Q	
20	96	19/12/82	26.4	CDA		) Winter months,
21	101	6/01/83	30.0	CDA		) possible snowmelt
22	108	2/03/83	42.8	CDA		) difficulties with
23	113	20/03/83	13.6	N		) rainfall measure-
24	116	25/03/83	20.0	N		) ments

[1] Agriculture Canada station outside of the catchment.
[2] Newtown station, within the catchment boundaries.
[3] Q = Used for analysis of simulated runoff peaks.
[4] V = Used for simulation of simulated runoff volumes.

(continued)..

Table 19 (Continued). Basic Characteristics of Storms Considered for Runoff Simulations

Number	Storm Number	Date	Rainfall Depth (mm)	Source of Rainfall Data	Data Used in Simulations	Comment
25	123	19/04/83	15.6	N	V,Q	
26	131	10/06/83	24.4	N	V	
27	135	2/07/83	3.6	N	V	
28	139	15/07/83	8.2	N	V,Q	
29	140	17/07/83	10.0	N	V,Q	
30	141	22/07/83	17.6	N	V	
31	142	25/07/83	15.8	N	V,Q	
32	152	16/08/83	33.6	CDA	Q	
33	156	27/08/83	17.6	N	V	
34	157	2/09/83	41.6	N	V	
35	159	10/09/83	16.6	N	V	
36	168	14/10/83	20.0	CDA	Q	
37	173	25/10/83	60.8	N	V,Q	
38	198	6/02/84	47.0	N	Q	Winter event
39	227	9/06/84	15.6	N		Flow recorder problems
40	239	17/07/84	9.6	N	V,Q	
41	246	18/08/84	39.0	N	V,Q	
42	247	19/08/84	13.0	N	V,Q	
43	248	20/08/84	12.4	N	V,Q	
44	251	24/08/84	32.2	N	V,Q	
45	252	6/09/84	20.4	N	Q	Poor synchronization
46	253	7/09/84	11.4	N	V,Q	
47	258	19/09/84	33.2	N	Q	Flow recorder problems

code V. Similarly, the storms used for analysis of simulated peak flows are denoted by code Q. It is obvious that even some storms unsuitable for analysis of volumes were analyzed for peaks. This was necessary to keep all significant events in the data set for frequency analysis. Although all 47 events were used in the initial simulations and analysis of peaks, this data set was later reduced to the top-ranked 29 events in order to reduce the costs of detailed simulations.

For all the storms listed in Table 19, the field records had to be fully processed and analyzed. The processing of rainfall data was particularly tedious as it initially involved discretization into 2-minute intervals. Further analysis showed that this fine interval was not needed and only 5-minute rainfall data were used in final simulations.

The measurement of rainfall in urban areas is a difficult task because of problems with finding good locations for rain gauges. Such problems were also encountered in this study and it is believed that some variations in simulation results were caused by variations in observed data. To obtain some appreciation for variations in observed rainfall within the test catchment, the total storm rainfalls from the Newtown tipping-bucket records were compared against the data from the standard gauge installed in the Newtown catchment. The data available allowed this type of analysis for 19 events listed in Table 20.

It is apparent from data in Table 20 that, on the average, the tipping bucket gauge recorded somewhat less rainfall, by about 5%. The mean difference between the standard and tipping-bucket gauge recorded rainfalls was 1.8 mm with the standard deviation of 6 mm. The 95% confidence limits for this mean difference were -1.0 mm and 4.6 mm. Thus it is apparent that the differences between the rainfalls measured by the standard and tipping-bucket gauges (at various locations) can be significant. Both data sets were further examined using linear regression analysis. In spite of a good correlation between both sets of data (r^2 = 0.97), in 15 out of 19 cases, the deviations between both recorded rainfalls were greater than 10%. Although such deviations

Table 20. Storm Rainfalls Measured at Newtown by Standard and Tipping Bucket Rain Gauges

Storm Number	Rainfall Standard Gauge (mm)	Rainfall Tipping Bucket (mm)	$H_{S.G.} - H_{T.B.}$ (mm)
19	73.600	83.200	-9.600
38	37.600	31.000	6.600
39	15.600	11.800	3.800
46	51.800	40.000	11.800
61	7.600	6.000	1.600
62	23.500	28.800	-5.300
69	25.200	29.400	-4.200
73	78.800	91.400	-12.600
76	55.400	53.600	1.800
113	17.200	15.500	1.700
123	20.300	15.000	5.300
140	14.000	10.000	4.000
141	24.000	17.900	6.100
157	51.000	41.200	9.800
159	21.400	16.200	5.200
173	62.100	60.400	1.700
198	44.800	47.800	-3.000
227	23.200	15.600	7.600
239	11.200	9.400	1.800
Mean	34.65	32.85	1.80
St. Deviation	21.80	24.90	6.44

would be fully acceptable in the modelling work for large rural watersheds, they may have a significant influence on runoff simulations for a small urban catchment.

For peak flow simulations, spatial variations in rainfall intensities over the catchment are also of interest. However, no data for assessing this problem were available.

Flow records from the Newtown drainage outfall required some further processing before they could be used for assessment of simulation results. In particular, it was noted that during wet weather periods, there was a significant baseflow in the outfall, long after the surface runoff ceased. Two sources of such flow were identified - firstly, foundation drains serving houses in the area and, secondly, groundwater infiltration into the sewer system. Although the baseflow rates were relatively small, the total volume of baseflow during runoff events was highly significant. The significance of baseflow volumes was further increased by the character of runoff events in the Newtown catchment. Such events were generally characterized by long duration (sometimes spanning over several days) and relatively low flow rates.

For studies of runoff volumes, it was necessary to separate the baseflow from the observed hydrographs and to make the corresponding corrections of the hydrograph volumes and peak flows. To make these corrections, it was assumed that the runoff events commenced with the start of the rainfall and finished 0.5 hours after the end of the rainfall. Such start and end points were identified on the hydrograph, connected by a straight line, and the hydrograph segment above this line was taken as representing the surface runoff hydrograph. The hydrograph portion below the cutoff line was assumed to represent the baseflow. The magnitude of these corrections ranged from 10% to 60% of the total hydrograph volume, with the average value of 31.5% (540 m^3, and a standard deviation of 16.6%). The highest corrections were found for long-duration storms with very low rainfall intensities.

The corrections of peak flows were relatively smaller, the average correction being only 0.014 m^3/s or 9% of the mean hydrograph peak.

It is obvious from the magnitude of the above corrections that they will contribute to uncertainties in comparisons of observed and simulated values, particularly in the case of volumes. On the other hand, it was necessary to make these corrections, because without them, the simulations results could not be evaluated at all.

7.3.2 Simulation Runs

For the events described in the preceding section, nine series of simulation runs with the SWMM and ILLUDAS models were made. These runs are further characterized in Table 21 and briefly described below.

Table 21. Characteristics of Runoff Simulation Runs for the Newtown Catchment

Simulation Series Number	Code	Model	Discretization Number of Subcatchments and Pipes	Infiltration Characteristics Max./min. rate (mm/hr)	Antecedent Moisture Conditions
1	S1	SWMM	22/37	76/13	Not considered
2	S2	SWMM	22/21	76/13	Not considered
3	S3	SWMM	22/21	51/8	Not considered
4	I1	ILLUDAS	20/20	Soil B	Rather dry
5	I2	ILLUDAS	20/20	Soil C	Rather dry
6	I3	ILLUDAS	20/20	Soil B	) Varied, accord-
7	I4	ILLUDAS	20/20	Soil C	) ing to the ante-
8	I5	ILLUDAS	20/20	Soil A	) cedent rainfall
9	I6	ILLUDAS	20/20	Soil B	Similar as in series 6-8, but slightly modified

The selection of the simulation runs was guided by the sensitivity analyses which were reported elsewhere for the simulation models used (18,19). For that reason, little attention was paid to those parameters which barely affect the simulation results within the practical range of values (e.g., roughness of surface elements, overland flow slopes, etc.), or which were determined fairly accurately from field surveys (e.g., the area of impervious elements). A brief description of individual simulation series follows.

Series S1 - This was the first series done with the SWMM model. As discussed in Chapter 6, storm sewers in crescents in the test catchment serve only for foundation drainage. The runoff is conveyed by gutters which drain into sewer inlets located at the bottom of crescents. For simplicity, street gutters were neglected in this series and the runoff was allowed to enter into sewers in the crescents, always upstream of the actual inlets. Characteristics of all catchment surfaces were determined according to the findings of field surveys. The infiltration rates were taken equal to the SWMM default values - the initial rate being 76 mm/hr and the final rate of 13 mm/hr. No consideration of antecedent moisture conditions was made.

Series S2 - In this series, the transport network was changed to properly simulate runoff flow in street gutters in crescents and flow entry into sewers at the bottom of crescents. Other parameters were the same as in the series S1.

Series S3 - In this series, it was attempted to increase the runoff contribution from pervious parts of the catchment by reducing,somewhat arbitrarily, the infiltration rates to 51 mm/hr and 7.6 mm/hr for the initial and final (minimum) rates, respectively. The remaining parameters were the same as in the series S2.

Series I1 - This was the first series with the ILLUDAS model. The catchment discretization was comparable to that used for the SWMM model, with the main difference arising from consideration of the supplemental impervious areas (i.e., the paved segments draining onto pervious parts). Infiltration rates were adopted as those corresponding to the soil group B and rather dry antecedent moisture conditions (AMC = 2; less than 12.7 mm of rainfall during five days before the storm).

Series I2 - This series differed from the preceding one just by the choice of the predominant soil group - Group C. This was done to test the uncertainty in the identification of the predominant soil group and the possible effects of soil layer thickness on soil type classification.

Series I3 - This series was designed to test the effects of the antecedent moisture conditions. Instead of assuming constant (dry) antecedent conditions for all events as done in two preceding series, the conditions were varied for individual storms according to the antecedent rainfall. The B soil group was used.

Series I4 - This series was identical to I3 except for the soil type - the group C was assumed.

Series I5 - This series was identical to I3 except for the soil type - the group A was assumed to further test the sensitivity of results to infiltration rates. This series was used only for the evaluation of simulated volumes.

Series I6 - This series represents a minor modification of I3. In particular, the antecedent conditions for individual storms were closely scrutinized and, in some cases, modifications of AMC were made. This series was used only for the evaluation of simulated runoff peak flows.

Characteristics of all simulation runs are summarized in Table 21.

7.3.3 Simulated runoff volumes

In the assessment of simulations of runoff volumes, 24 events with the most reliable data were used. These events are listed in Table 19. It can be inferred from this table that no events with questionable records or rainfall data outside of the Newtown catchment were used in runoff volume analyses. Furthermore, winter-month events were also excluded because of questionable records from the tipping-bucket rain gauge operated in cold weather and possible contributions of snowmelt to the runoff. The observed depths of runoff ranged from 0.75 mm to 23.56 mm, thus covering a wide range of runoff events.

It was attempted to reproduce the observed runoff volumes by eight series of simulation runs with the SWMM and ILLUDAS models. Runoff volumes simulated for the selected 24 events are listed in Table 22. In this table, all runoff volumes were expressed as the depth of runoff distributed over the total catchment area, i.e., $H = V_{runoff}/A$.

Table 22 also contains basic statistics of individual data sets - the total runoff depth for all 24 events, the mean depth and its standard deviation. It was noticed that the statistical parameters of individual event sets were strongly influenced by the two largest events -, storms No. 73 and 173. For this reason, the statistical parameters of the event set without storms 73 and 173 were also evaluated and included in Table 22.

It is obvious from simulation results that the observed runoff volumes of the two largest events could not be reproduced well by simulations. This was particularly obvious for storm No. 173 in which case the closest simulated volume represented only 75% of the observed one. Furthermore, if the model parameters used to obtain this best reproduction (i.e., Series I2 and I4) were applied consistently to all other events in the set (as done in series I2 and I4), the simulated runoff volumes would generally somewhat exceed the observed ones. For smaller events, much closer reproduction was achieved. In most runs, the cumulative runoff volume for all 22 events ranged from 100% to 110% of the observed value. Only one series (I4) seriously overestimated the observed total volume.

Further evaluations of the simulated volumes were done by examining linear correlation between the observed and simulated volumes, and examining the deviations of simulated volumes from the observed ones, i.e., $D = H_{obs} - H_{sim}$. The pertinent statistics are given in Table 23.

Table 22. Observed and Simulated Runoff Event Volumes

	Newtown Runoff Event Volumes H (mm)								
		SWMM Simulations			ILLUDAS Simulations				
Storm No.	Observed	S1	S2	S3	I1	I2	I3	I4	I5
62	3.98	5.39	5.53	5.53	5.26	5.39	5.26	5.39	5.26
63	5.08	6.45	6.48	9.29	6.49	8.81	6.49	8.81	6.10
64	4.61	5.76	5.90	5.82	5.59	5.74	5.77	9.23	5.59
69	2.49	3.30	3.46	3.45	3.21	3.67	3.96	6.00	3.43
73	23.56	14.59	15.04	26.44	17.50	28.74	15.07	20.97	14.10
76	7.01	5.53	5.67	6.59	5.51	6.62	8.86	15.03	6.37
86	4.06	4.46	4.61	4.61	4.43	4.43	4.43	4.43	4.43
123	3.05	2.87	3.02	3.02	2.86	2.86	2.86	2.86	2.86
131	5.66	4.64	4.79	4.79	4.50	4.50	4.50	4.50	4.50
135	0.75	0.86	1.01	1.01	0.61	0.61	0.61	0.61	0.61
139	1.68	1.80	1.92	1.92	1.69	1.69	1.69	1.69	1.69
140	2.24	3.34	3.49	3.46	3.30	3.33	3.31	3.73	3.30
141	3.82	3.23	3.38	3.38	3.24	3.24	3.24	3.24	3.24
142	3.59	2.92	3.08	3.06	2.74	2.78	2.81	3.75	2.74
156	1.83	3.76	3.91	3.91	3.64	3.64	3.64	3.64	3.64
157	10.31	7.71	7.86	7.87	7.65	9.25	7.65	9.25	7.65
159	3.83	3.10	3.25	3.25	3.64	3.16	3.16	3.16	3.16
173	20.97	11.77	11.92	11.92	11.90	15.65	11.90	15.65	11.65
239	1.59	1.71	1.86	1.86	1.59	1.59	1.59	1.59	1.59
246	7.11	8.00	7.92	8.31	7.67	8.88	7.66	7.84	7.66
247	3.02	2.40	2.55	2.55	2.36	2.36	2.48	4.39	2.36
248	2.35	2.48	2.63	2.63	2.21	2.21	2.27	3.42	2.21
251	5.95	6.45	6.61	6.49	6.29	7.13	6.67	8.01	6.31
253	2.34	2.02	2.18	2.18	1.90	1.90	2.15	3.46	1.98
ΣH(n=24)	130.88	114.54	118.07	133.34	115.78	138.18	118.03	150.65	112.43
Mean	5.45	4.77	4.92	5.56	4.82	5.76	4.92	6.28	4.68
St. Dev.	5.62	3.24	3.27	5.18	3.68	5.93	3.42	4.95	3.19

Data set statistics without events Nos. 73 and 173

ΣH (n=22)	86.35	88.18	91.11	94.98	86.38	93.79	91.06	114.03	86.68
Mean	3.93	4.01	4.14	4.32	3.93	4.26	4.14	5.18	3.94
St. Dev.	2.25	1.99	1.96	2.26	1.98	2.51	2.23	3.33	2.00

Table 23. Statistics Used for Evaluation of Simulations of Runoff Volumes

Number of Events		S1	S2	S3	I1	I2	I3	I4	I5
		Simulation Series							
	Correlation coefficient r^2 - H_{sim} vs. H_{obs}	0.89	0.90	0.82	0.91	0.89	0.86	0.79	0.89
	ΣD (mm)	16.3	12.8	-2.5	15.1	-7.3	12.9	-19.8	18.5
24	$\bar{D}$ (mm)	0.7	0.5	-0.1	0.6	-0.3	0.5	-0.8	0.8
	σ_D	2.9	2.9	2.5	2.5	2.0	2.9	2.7	2.9
	95% confidence limits for $\bar{D}$ - lower	-0.5	-0.6	-1.1	-0.4	-1.1	-0.6	-1.9	-0.4
	- upper	1.9	1.7	0.9	1.6	0.5	1.7	0.3	2.0
	Correlation coefficient r^2 - H_{sim} vs. H_{obs}	0.78	0.78	0.70	0.79	0.77	0.78	0.59	0.81
	ΣD (mm)	-1.8	-4.8	-8.6	0.0	-7.4	-4.7	-27.7	-0.3
22	$\bar{D}$ (mm)	-0.1	-0.2	-0.4	0.0	-0.3	-0.2	-1.3	0.0
	σ_D	1.1	1.1	1.3	1.0	1.2	1.1	2.2	1.0
	95% confidence limits for $\bar{D}$ - lower	-0.6	-0.7	-1.0	-0.5	-0.9	-0.7	-2.2	-0.5
	- upper	0.4	0.2	0.2	0.5	0.2	0.3	-0.3	0.4

Legend: D = $H_{obs} - H_{sim}$ = Observed runoff volume (mm) - Simulated runoff volume (mm).

$\bar{D}$ = Mean D

σ_D = Standard deviation of D about the mean.

It can be inferred from Table 23 that, with the exception of series I4, all the series yielded comparable results in terms of correlations and statistics of deviations of simulated volumes from the observed ones. This is particularly apparent for the reduced set of 22 events. Further interpretation of simulation results is presented later in the section dealing with discussion of results.

From the drainage point of view, the simulation accuracy for runoff volumes is of secondary importance in drainage systems without storage. However, because the models used first establish runoff volumes and their distribution in time to obtain the runoff hydrograph, large errors in volume simulations would be reflected in simulated peak flows which are of primary interest in drainage design.

7.3.4 Simulated runoff peak flows

Using the simulation procedures described earlier, the simulated runoff peak flows were determined for 29 events indicated in Table 19 and further analyzed.

An early review of results revealed difficulties with simulation of the runoff peak for event No. 73. For this event, the record from the Newtown tipping-bucket rain gauge indicated very high peak intensities which produced high simulated peak flows exceeding significantly the observed peaks. To investigate this problem further, additional rainfall records for event No. 73 were obtained from the Newtown standard rain gauge and the CDA rain gauge. A summary of such rainfalls, recorded at various stations, is given in Table 24.

It can be inferred from Table 24 that storm No. 73, also referred to in some records as a hurricane, had a very non-uniform spatial distribution of rainfall. In particular, there is hardly any similarity between the data from Newtown and CDA, although the latter station is located only 1.8 km east of the Newtown catchment. On the day with the highest rainfall, September 19, 1982, the tipping-bucket recorded 15% more rainfall than the standard rain gauge located only

Table 24. Rainfalls Recorded at Three Stations for Storm Number 73

Date	24-Hour Rainfalls (Read at 1600 NST, in mm)		
	Newtown Standard Rain Gauge	Newton Tipping Bucket Rain Gauge	CDA Tipping Bucket Rain Gauge
15/09/82	0.0	0.0	0.0
16/09/82	11.0	12.2	2.0
17/09/82	14.6	15.2	4.0
18/09/82	3.0	2.8	0.6
19/09/82	61.2	72.0	45.0
20/09/82	0.0	0.0	0.0
Σ	89.8	102.2	51.6

250 m west of the tipping bucket. It was therefore concluded that because of the non-uniform spatial distribution of intensities during storm No. 73, it was desirable to adjust the Newtown rainfall intensities using a coefficient 0.85 which was obtained as the ratio of the standard rain gauge catch to that of the Newtown tipping bucket. Although other storm records producing large deviations between the observed and simulated data were also checked, none of the other events justified similar adjustments as done for event No. 73.

For the 29 events selected, the runoff peaks obtained in seven series of simulations are presented in Table 25 together with the observed peaks corrected for baseflow. Observed peaks varied from 0.076 m^3/s to 0.391 m^3/s, with the mean value of 0.195 m^3/s. Significantly larger variations were noticed for simulated peaks - from 0.165 m^3/s to 0.796 m^3/s and the series means varied from 0.165 m^3/s to 0.245 m^3/s. Table 25 also contains results for simulation series I6. In this series, which is similar to I3, AMC parameters of four events were

Table 25. Observed and Simulated Runoff Peaks at the Newtown Outfall

Storm No.	Observed and Simulated Runoff Peak Flows $Q(m^3/s)$								
		SWMM Simulations			ILLUDAS Simulations				
	Observed	S1	S2	S3	I1	I2	I3	I4	I6*
14	0.218	0.231	0.232	0.271	0.311	0.476	0.261	0.379	0.261
19	0.330	0.103	0.104	0.145	0.125	0.241	0.156	0.295	0.173
51	0.081	0.115	0.116	0.116	0.119	0.119	0.119	0.119	0.119
52	0.139	0.093	0.093	0.093	0.093	0.150	0.096	0.181	0.096
61	0.121	0.123	0.145	0.145	0.120	0.125	0.125	0.125	0.125
62	0.103	0.130	0.131	0.131	0.140	0.136	0.136	0.136	0.136
63	0.328	0.315	0.310	0.568	0.309	0.459	0.300	0.320	0.300
64	0.093	0.107	0.107	0.107	0.110	0.108	0.110	0.195	0.110
69	0.242	0.314	0.313	0.313	0.337	0.351	0.357	0.453	0.337
73	0.368	0.387	0.387	0.491	0.507	0.607	0.459	0.475	0.459
76	0.391	0.248	0.251	0.279	0.292	0.320	0.433	0.657	0.433
80	0.164	0.081	0.081	0.081	0.090	0.159	0.113	0.200	0.113
86	0.093	0.096	0.096	0.096	0.100	0.105	0.102	0.102	0.102
123	0.076	0.065	0.065	0.065	0.065	0.065	0.065	0.065	0.065
139	0.114	0.116	0.116	0.116	0.139	0.139	0.139	0.139	0.139
140	0.198	0.146	0.146	0.136	0.139	0.139	0.139	0.176	0.139
142	0.301	0.230	0.231	0.231	0.238	0.244	0.244	0.266	0.269
152	0.293	0.270	0.271	0.311	0.284	0.362	0.351	0.530	0.284
168	0.176	0.167	0.171	0.171	0.184	0.190	0.201	0.218	0.201
173	0.149	0.120	0.121	0.121	0.130	0.178	0.130	0.178	0.130
198	0.156	0.088	0.088	0.088	0.119	0.127	0.119	0.125	0.119
239	0.192	0.163	0.173	0.173	0.184	0.184	0.184	0.184	0.184
246	0.314	0.284	0.284	0.294	0.277	0.343	0.277	0.283	0.277
247	0.171	0.124	0.124	0.124	0.132	0.133	0.136	0.224	0.136
248	0.165	0.115	0.115	0.115	0.108	0.108	0.110	0.181	0.110
251	0.177	0.162	0.163	0.162	0.175	0.204	0.195	0.229	0.195
252	0.064	0.079	0.079	0.079	0.079	0.079	0.079	0.127	0.079
253	0.263	0.175	0.176	0.176	0.195	0.195	0.204	0.272	0.204
258	0.169	0.117	0.117	0.103	0.125	0.133	0.125	0.133	0.125
Mean $\bar{Q}$	0.195	0.165	0.166	0.186	0.183	0.217	0.195	0.245	0.188
St.Dev. σ_Q	0.093	0.085	0.084	0.125	0.104	0.138	0.124	0.156	0.104

* In this series, some minor adjustments in the AMC parameter were made on the basis of detailed analysis of antecedent precipitation (for details,see Section 7.3.4).

changed on the basis of detailed scrutiny of the antecedent rainfall, rather than mechanically applying the ILLUDAS criteria for evaluation of AMC conditions. For example, in series I6, an event with 2 mm of rainfall on the 5th day before the storm and no rainfall for the 1st to 4th days would be assigned AMC = 1 rather than AMC = 2, as would follow from the AMC criteria (series I3).

In evaluations of simulated peaks, correlations between the simulated and observed peaks were investigated and the statistics of the deviations of simulated peaks from the observed ones were also determined. All the pertinent results are summarized in Table 26.

The data in Table 26 indicate good correlation between observed and simulated peaks (r^2 = .80 to .87). The mean differences between the observed and simulated peaks varied from -0.048 m^3/s to 0.030 m^3/s. When comparing individual simulation series, it became obvious that the least satisfactory results were obtained for series I4, in which the simulated peaks over-estimated the observed ones by a significant margin. The best agreement was obtained for series I6 and I3, with the remaining series being almost equal.

Furthermore, it was of interest to compare the simulation results among themselves. For this purpose, only five simulation series were used; two SWMM series and three ILLUDAS series yielding the best results: S2, S3, I1, I3 and I6. The results of these comparisons are shown in Table 27.

It can be inferred from Table 27 that the agreement between runoff peaks simulated in different model series is somewhat better than that between simulated and observed data. This was particularly true for comparisons between series S2 and all the ILLUDAS simulations series.

As a final means of evaluation of simulated peaks, the observed and simulated peaks were subject to frequency analysis. The results of this analysis are shown in Table 28 and Fig. 13. The frequency curves in Fig. 13 show better agreement between the observed and simulated peaks than the earlier analysis given in Table 26.

Table 26. **Statistics Used for Evaluation of Simulation of Runoff Peaks**

Number of Events	Statistics	Simulation Series							
		S1	S2	S3	I1	I2	I3	I4	I6
	Correlation coefficient r^2 for Q_{sim} vs. Q_{obs}	0.80	0.80	0.80	0.80	0.83	0.83	0.81	0.87
29	$\bar{D}$ $(D = Q_{obs} - Q_{sim})^1$	0.030	0.029	0.009	0.012	-0.022	0.000	0.048	0.007
	$\sigma_{\bar{D}}$ [1]	0.057	0.057	0.075	0.063	0.080	0.069	0.099	0.052
	95% confidence limits for $\bar{D}$-lower[1]	0.008	0.007	-0.020	-0.012	-0.053	-0.027	-0.085	-0.012
	-upper[1]	0.052	0.051	0.037	0.036	0.008	0.027	0.010	0.027

[1] Units = m^3/s

Table 27. **Comparison of Runoff Peaks Simulated by SWMM and ILLUDAS Models for the Newtown Catchment Outfall**

SWMM Simulation Series	Statistics	ILLUDAS Simulation Series		
		I1	I3	I6
S2 29 Events	Correlation coefficient r^2	0.97	0.86	0.93
	$\bar{D}$ $(D = Q^{1}_{S_2} - Q^{2}_{I})$	0.004	-0.003	-0.001
	σ_D	0.052	0.056	0.056
	95% confidence			
	limits for $\bar{D}$ - lower	-0.016	-0.024	-0.022
	- upper	0.024	0.018	0.020
S3 29 Events	Correlation coefficient r^2	0.91	0.84	0.90
	$\bar{D}$ $(D = Q_{S_3} - Q_I)$	-0.017	-0.029	-0.021
	σ_D	0.031	0.067	0.039
	95% confidence			
	limits for $\bar{D}$ - lower	-0.029	-0.055	-0.036
	- upper	-0.005	-0.003	-0.006

[1] Q_S = Runoff peak from SWMM simulations
[2] Q_I = Runoff peak from ILLUDAS simulations

Table 28. Frequency Analysis of Observed* and Simulated Runoff Peaks

Rank	Return Period (yrs)	Runoff Peak Flows $Q(m^3/s)$								
		Observed	S1	S2	S3	I1	I2	I3	I4	I6
1	4	0.391	0.387	0.387	0.568	0.507	0.476	0.459	0.657	0.459
2	2	0.368	0.315	0.313	0.491	0.337	0.459	0.443	0.530	0.453
3	1.33	0.330	0.314	0.310	0.313	0.311	0.447	0.357	0.475	0.337
4	1.00	0.328	0.284	0.284	0.311	0.309	0.362	0.351	0.453	0.300
5	0.80	0.314	0.270	0.271	0.294	0.292	0.351	0.300	0.379	0.284
6	0.67	0.301	0.248	0.251	0.279	0.284	0.343	0.277	0.320	0.277
7	0.57	0.293	0.231	0.232	0.271	0.277	0.320	0.261	0.295	0.269
8	0.50	0.263	0.230	0.231	0.231	0.238	0.244	0.244	0.283	0.261
9	0.44	0.242	0.175	0.176	0.176	0.195	0.241	0.204	0.272	0.204
10	0.40	0.218	0.167	0.173	0.173	0.184	0.204	0.201	0.266	0.201
11	0.36	0.198	0.163	0.171	0.171	0.183	0.195	0.195	0.229	0.195
12	0.33	0.196	0.162	0.163	0.162	0.175	0.190	0.184	0.224	0.184
$\bar{D}$ ($D = Q_{obs} - Q_{sim}$)			0.041	0.040	0.000	0.012	-0.033	-0.003	-0.079	0.001
σ_D			0.018	0.018	0.072	0.041	0.045	0.037	0.079	0.039
95% confidence limits										
- lower			0.030	0.029	-0.046	-0.014	-0.062	-0.026	-0.129	-0.024
- upper			0.052	0.051	0.046	0.038	-0.004	0.020	-0.029	0.026

* Corrected by subtracting baseflow

Schedule A - Culvert

Project Description
(Please complete one Schedule for each crossing)

Location

Site Name/No/Civic Address: Thorburn Road / Rainbow Gully Road

Please mark location on a copy of a topographic map (preferably at 1:50,000 scale) or Google Earth Image and include as a separate attachment with the application.

If including a 1:50,000 Topographic Map, Please Provide:

Map No: Refer to Civil IFT Drawing C100

Or, UTM Coordinates:

N	E	NAD	ZONE
5270 403	359 921	83	22T
5270 369	359 844	83	22T

Design Refer to Drainage Area Plan Figure 12054-F007A

Drainage Area Profile:

Drainage Area: ________ km^2

Main Channel Length: ________ km

Slope of Drainage Area: ________ %

Drainage Area Classification:

Forest: ________ %

Barren: ________ %

Wetland: ________ %

Urban: ________ %

Hydrologic Details:

Return Period: 1: 100 years

Estimation Method: ☐ Rational ☐ TR55 ☐ RFFA ☒ Other HEC HMS

Maximum Flow: 24.2 m^3/s Design Flow: 25.7 m^3/s

Description of Estimation:
Please show calculation(s) below or attach separate sheets, if required.

Please refer to the following reports, prepared by Pinnacle Engineering Limited:

- Flood Risk Analysis, dated 01 Nov. 2015
- Final Supplemental Flood Risk Analysis, dated 29 May 2015.

Design (cont'd)

Culvert Shape: ☐ Round ☐ Pipe-arch ☑ Other *Open Box Culvert*

Culvert Material: ☐ Steel ☑ Concrete ☐ Aluminum ☐ Other ________

Headwall Type: ☑ Concrete ☐ Wood ☐ Rip-rap ☐ Other ________

Number of Pipes: *1* Length: *29* metres

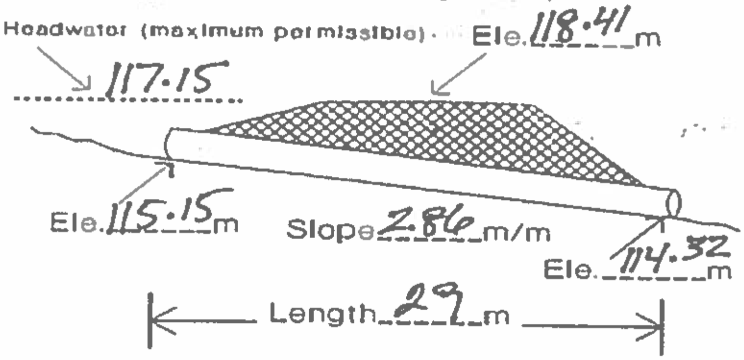
Width: *6.0* ~~mm~~ *M* Height: *2.0* ~~mm~~ *M* (for arch-pipe)

Diameter: *N/A* mm (for round pipe)

Max Velocity: *2.14* m/s Min Velocity: *0.60* m/s

If the undertaking involves replacing an existing culvert, please provide dimensions of existing culvert and reasoning behind the replacement (undersized, collapse, etc.): ________

Complete the diagram below and check inlet/outlet type:



☐ Projecting

☐ Mitred

☑ Headwall

Construction

Equipment to be used: ________

Proposed dewatering method: ________

Briefly describe how erosion control and stabilization will be carried out:

Please refer to the Environmental Control Plan for installation of Open Bottom Concrete Box Culvert by Stantec Consulting, dated 29 April/2015, FILE No. 121413487

Briefly describe how site restoration will be carried out:

Please refer to Civil IFT Drawing C109.

Technical guidelines, departmental policies and application forms are available at:
http://www.env.gov.nl.ca/env/waterres/regulations/appforms/index.html

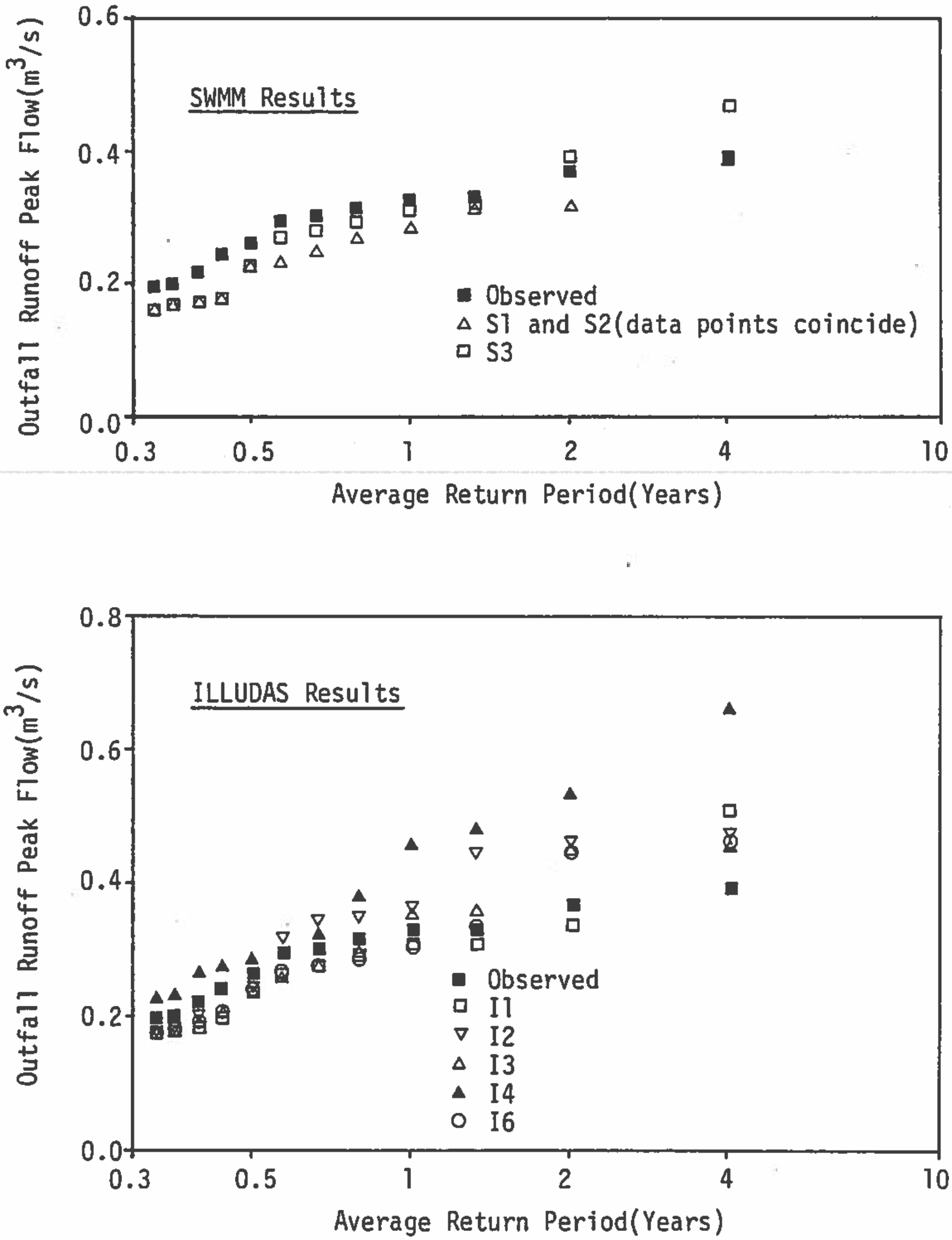


Fig.13. Runoff Peak Flow Frequency Curves: Observed and Simulated

Similarly, the statistics of differences between the ranked observed and simulated peaks (i.e., Table 28) show a significant improvement compared to the results given for non-ranked data sets in Table 26.

7.3.5 Discussion of runoff modelling results

The discussion presented in this section concentrates on the modelling of runoff volumes and peaks without consideration of the timing of the peaks. Before proceeding with the discussion, a brief assessment of input and calibration data used in runoff modelling is given.

In the assessment of catchment physiographic data, it appears that good quality, detailed data available for the study area were sufficient for runoff modelling. On the other hand, the quality of hydrometeorological data, particularly the rainfall data, could be improved in the future. For a number of events, rainfall data from Newtown were not available because of malfunctions of the tipping-bucket rain gauge. Other difficulties were encountered in the synchronization of rainfall and runoff records. It is desirable to improve on this in the future by operator training and closer supervision, and/or by adding another tipping bucket to the observation network.

Flow data seemed to be fairly good with rare exceptions of recorder malfunctions. The zero-reading of the weir should be checked more often than once a season. It also became obvious that the catchment outlet hydrographs contained significant baseflow which made it difficult to determine runoff volumes. Recognizing the uncertainties in the hydrometeorological data, it is possible to proceed with the discussion of simulation results, starting with runoff volumes.

Simulations of runoff volumes were adversely affected by the presence of baseflow which introduced significant uncertainties into the calibration and evaluation of models. Notwithstanding these problems, volumes of runoff from smaller storms (rainfall less than 50 mm) were reproduced by both models, SWMM and ILLUDAS, with an acceptable accuracy. Considering the mean event runoff as 4 mm, with the exception of series I4, the simulated mean runoff volumes were within 10% of this mean value. The difference between the observed and simulated volumes varied from -25% to +13% at the 95% level of confidence .

It became apparent that for smaller events, runoff was primarily generated on impervious areas with hardly any contributions from pervious areas. Under such circumstances, both models performed equally well which follows from similar formulations of losses in both models. The only exception was the series I4 which overestimated runoff volumes even for small events. Series I4 assumed soil type C and used actual AMC conditions.

When considering all 24 events, wider discrepancies in runoff volume simulations became apparent. It is however difficult to judge to what extent such discrepancies were caused by the uncertainties in hydrometeorological data as opposed to the ability of models to reproduce the observed events. As discussed earlier, there were some problems found with the rainfall spatial distribution during the event No. 73 and the tipping-bucket data did not necessarily represent the catchment rainfall. For event No. 173, however, there was a close agreement between the rainfall data measured by the standard and tipping-bucket gauge. For this event, the ILLUDAS model, applied with an appropriately adjusted AMC parameter and the assumption of soil group C, produced better results than the SWMM model.

It appears that in simulations of runoff volumes from large storms, the infiltration rates need to be calibrated. The commonly used rates, such as those used in runs S1, S2 and I1, were too small. For adjustment of the infiltration rates, the ILLUDAS model offered more convenience and flexibility, because the user can select the appropriate rates by specifying the soil type and the antecedent moisture conditions for the 5-day antecedent rainfall. It should be emphasized that the same effect could be achieved with the SWMM model by calculating infiltration rates for various AMC conditions and then substituting them into the model.

The following observations on the results of the modelling of runoff volumes can be made:

1. Directly-connected (effective) impervious areas, as determined from maps and surveys (area = 2.62 ha), and the impervious surface detention storage in the range from 0.4 mm to 1.6 mm provide a good reproduction of runoff volumes for rainfall events with less than 50 mm of rainfall. This finding follows from the results of simulation series S1, S2 and I1 with surface detention storage values of 1.2, 0.4, and 1.6 mm, respectively.
2. For pervious areas, satisfactory results were obtained for soil type B with appropriate antecedent moisture conditions, or by lowering the default infiltration rates in the SWMM model to the values of 51 mm/hr and 7.6 mm/hr as the initial and terminal rates, respectively. Such conditions were met in series S3 and I3. It was felt that series S1, S2, I1 and I5 would possibly underestimate observed values; and series I2 and I4 would overestimate observed values, particularly for large events. It was desirable to provide for runoff contributions from pervious areas by accounting for both the infiltration characteristics of the soil and the antecedent moisture conditions, especially for the most severe storms. For smaller storms (rainfall < 50 mm, average antecedent moisture conditions), such considerations were unimportant.

For evaluation of runoff peak simulations, the set of top-ranked events was different from that used in runoff volume evaluations. Several storms with relatively minor rainfall but high intensities produced relatively high peak flows. As stated earlier, uncertainties in runoff volume simulations were also reflected in simulated runoff peak flows.

The simulated runoff peaks can be effectively evaluated by plotting peak frequency graphs (Fig. 13). It became immediately apparent that simulations with very low infiltration rates, such as S3, I2 and

I4, consistently over-estimated runoff peaks for return periods greater than 0.6 years. The best results were obtained for two almost identical ILLUDAS series I6 and I3, followed by S1, S2, and I1. The SWMM series S1 and S2 were practically identical and although they consistently underestimated the observed flows, such an under-estimation was not excessive (0.030-0.052 m^3/s).

Good agreement between series S1 and S2 indicates that a detailed modelling of transport elements within subcatchments is unimportant as long as the correct length of travel is preserved. In this regard, it should be reiterated that series S1 used only sewers as transport elements and series S2 also employed street gutters which fed into sewers.

ILLUDAS series I1, I3, and I6 produced fairly good results with slight overestimations for the two largest peaks. Such peaks could be somewhat reduced by using the storage routing procedure instead of the time lag procedure. Numerical instabilities in the model prevented further investigations of such routing effects (19).

In the overall assessment, both models performed fairly well in reproducing runoff volumes and peaks for small to medium events (say with rainfall less than 50 mm). In this regard series S1, S2, I1, I3 and I6 produced comparable results. For events with large rainfalls and runoffs, it appeared desirable to account for increased contributions from pervious areas. Although this was best achieved in series I3 and I6 similar results could be obtained with SWMM as well. No further calibration runs were done with the SWMM model because they would be applicable only to two largest runoff events and no verification would be possible. When comparing both models, the ILLUDAS model seemed to be more convenient for considerations of infiltration and antecedent moisture conditions.

8.0 STORMWATER QUALITY

The compositon of stormwater discharging from the Newtown drainage outfall was monitored by collecting grab samples at the outfall and analyzing them for 25 parameters. Although the ultimate goal of these investigations was to model stormwater quality, this could not be done at this time, because of the lack of necessary supporting data. Consequently, only a brief summary and interpretation of the currently available data is presented below.

8.1 Parameters Studied

The selection of parameters to be studied followed the recommendations of the Water Quality Subcommittee established under the Waterford River Basin Study. Such parameters could be grouped into the following groups:

a) Physical parameters
b) Major ions
c) Selected metals
d) Nutrients
e) Bacteria.

The list of parameters shown in Table 29 is very comprehensive. It contains all the parameters which are usually cited as major causes of pollution by urban runoff and also some more general parameters which may provide links between urban runoff and the streamflow quality. Among the parameters which are particularly important for characterization of urban runoff, the following should be named: Suspended solids, sodium chloride, lead, zinc, nitrogen, phosphorus and coliforms. These parameters will be discussed in the following sections in more detail than the others.

Table 29. **Water Quality Parameters Studied**

Parameter Group	Parameters
1. Physical Parameters	Water colour, temperature, dissolved oxygen, pH, specific conductance, turbidity and suspended solids.
2. Major Ions	Sodium, chloride, magnesium, calcium, alkalinity or bicarbonate, sulphate, potassium and silica.
3. Selected Extractable Metals	Iron, manganese, copper, lead and zinc.
4. Nutrients	Organic carbon (dissolved and total), inorganic carbon, nitrogen (dissolved and total), phosphorus (dissolved and total).
5. Bacteria	Total coliform, and fecal coliform.

8.2 Methods

Sampling methods were described earlier in Chapter 4 on catchment instrumentation. Sample preservation methods and analytical methods are described in detail in manuals which are used by the Water Quality Branch, IWD, Moncton (2,22,23).

8.3 Results and Discussion

Results of sample analyses are summarized in a series of tables giving the basic statistics of such data sets - means, standard deviations and ranges. The discussion concentrates on the seven parameters of special interest.

8.3.1 Physical parameters

A summary of results of physical parameter analyses performed on stormwater samples is given in Table 30. The readings of water colour, in Hazen units, varied from less than 5 to 40, with most readings between 5 and 20, and the mean value of 13.3. Such values appear to be somewhat lower than those observed in surface waters in the basin (3).

The readings of water temperature, in-situ, were relatively limited (33 in total). Such readings are affected by the season and have no particular significance with regard to water quality in the studied case. Runoff temperatures may be of interest when modelling stormwater ponds, because thermal and density effects will influence currents in such facilities.

Table 30. Summary of Physical Parameter Data

Parameter	Number of Samples	Mean	Standard Deviation	Range
Water colour (Hazen units)	123	13.3	7.3	<5 - 40
Water temperature (°C)	33	7.3	3.2	0.6 - 12.5
Dissolved oxygen (mg/L)	19	10.0	1.6	7.2 - 12.6
pH (field)	32	6.6	0.22	6.2 - 7.0
pH (laboratory)	199	6.1	0.72	3.2 - 7.4
Specific conductance, laboratory (μS/cm)	172	283.3	415	19 - 3600
Specific conductance, field (μS/cm)	33	279.9	455	70 - 2070
Turbidity (JTU)	183	22.2	19.7	0.4 - 125
Total Suspended Solids (non-filterable residue, 105°C, mg/L)	207	52.4	57.7	2 - 366

Dissolved oxygen values were relatively high, varying from 7.2 to 12.0 mg/L, with the mean value of 10.0 mg/L. Such fairly high values indicate that stormwater discharges studied do not induce any stress on dissolved oxygen in the receiving waters. Note that the critical low value for aquatic organisms is 4.0 mg/L (3).

Two types of pH readings were taken - in the field and in the laboratory. The former readings indicated slightly acidic to neutral conditions (pH 6.2-7.0) and were well within values reported for insular Newfoundland (3).

Specific conductance reflects the ability of the sampled liquid to conduct an electrical current and, as such, it depends on ionic concentrations and temperature. Two sets of data were produced - laboratory and in situ readings. The means of both data sets were about 280 uS/cm, with the range from 19 to 360 uS/cm. Such readings are elevated in comparison to those from background (rural) surface water stations in the basin and reflect the effects of urban activities and pollution. The highest values were observed during the winter months and reflect road salting operations.

Suspended particles in stormwater were quantified by means of two parameters - turbidity and total suspended solids. The former parameter is less commonly used for stormwater, it is more frequently used for natural waters. Turbidity of stormwater varied from 0.4 to 125 JTU, with the mean value of 22.2 JTU. Such values are relatively low and compare well with readings taken in the Waterford River (3).

Total suspended solids (at 105°C) varied from 2 to 366 mg/L, with the mean value of 52.4 mg/L. Such a range of values is relatively low and indicates a somewhat lower production of suspended solids than is usually encountered in residential areas.

8.3.2 Major ions

Among major ions, eight parameters were studied - dissolved chloride, sodium, sulphate, potassium, calcium and magnesium; reactive silica; and, alkalinity. With the exception of chloride and sodium, major ions are of little interest in studies of stormwater quality and this is reflected in the relative brevity of the following discussion. Characteristic values of major ions in Newtown catchment stormwater are summarized in Table 31.

Among major ions, chloride and sodium occurred in the highest concentrations. The mean concentrations of sodium and chloride were 76 mg/L and 139 mg/L, respectively. Although these ions originate from both marine aerosol deposits and road salting, the highest reported values were clearly associated with road salting in winter months and exceeded significantly the highest values reported for surface waters in the basin (3).

Both calcium and sulphate concentrations correlated quite well with sodium chloride concentrations. Conceivably, the road salt used in the catchment contained some calcium sulphate and this would explain such correlations (3).

The remaining major ions occurred in fairly low concentrations as can be seen in Table 31.

Table 31. Summary of Major Ion Data

Parameter	Number of Samples	Mean	Standard Deviation	Range
Dissolved chloride (mg/L)	76	138.9	208.7	10.3 - 1180
Dissolved sodium (mg/L)	99	76.2	121.7	7.6 - 690
Dissolved sulphate (mg/L)	56	16.1	13.2	6.8 - 83.0
Dissolved potassium (mg/L)	100	1.5	0.74	0.6 - 2.5
Dissolved calcium (mg/L)	74	9.6	5.3	4.5 - 35.0
Dissolved magnesium (mg/L)	73	1.7	0.8	0.4 - 4.5
Reactive silica (mg/L)	101	3.6	2.1	1.0 - 8.4
Total alkalinity	97	11.1	3.5	3.6 - 18.1
Gran alkalinity (mg/L)	85	6.6	2.9	0.2 - 17.8

8.3.3 Metals

The stormwater samples collected during this study were analyzed for extractable metals only. In such analyses, the samples were acidified and the measured concentrations reflect not only dissolved metals but also metals which were originally adsorbed on the particulate matter and not readily available to the aquatic biota. The limitations of the extractable metal data to reflect their toxic potential for aquatic life should be borne in mind throughout this section.

Concentrations of five extractable metals in stormwater are summarized in Table 32. Compared to the other data for surface waters in the basin, the concentrations in stormwater are higher (3).

Although the mean concentrations of iron and manganese appear to be elevated, this seems to be caused by a small number of samples with very high concentrations of these metals. From the water quality point of view, the observed concentrations of iron and manganese are of no special concern.

Copper concentrations were found to be rather low with the mean value of 0.008 mg/L. Lead had a mean concentration of 0.044 mg/L.

Table 32. Summary of Selected Extractable Metal Data

Parameter	Number of Samples	Mean	Standard Deviation	Range
Iron (mg/L)	135	0.95	1.20	0.03 - 5.20
Manganese (mg/L)	134	0.26	0.25	0.09 - 1.90
Copper (mg/L)	120	0.008	0.009	0.002 - 0.050
Lead (mg/L)	167	0.044	0.066	0.002 - 0.330
Zinc (mg/L)	187	0.130	0.060	0.05 - 0.53

Although lead concentrations were relatively low compared to runoff quality data from other catchments (13), they exceeded the guideline for protection of freshwater aquatic life, 0.03 mg/L (3), in about 30% of all samples. Thus the lead concentrations described here would potentially be harmful to aquatic life in the receiving waters. The main sources of lead in the stormwater are traffic byproducts.

Finally, zinc, which had a mean concentration of 0.130 mg/L, was found in concentrations (0.05 mg/L-0.53 mg/L) typical for urban runoff. Such concentrations are somewhat elevated and exceed the guideline for protection of aquatic life - 0.03 mg/L (3). Among various sources of zinc in urban areas (tires, galvanized metals, terrestrial sources), the galvanized corrugated-steel sewer pipes seem to be the most important in the case of Newtown stormwater.

8.3.4 Nutrients

Six forms of nutrients were studied - dissolved nitrites and nitrates (as N), total nitrogen, dissolved phosphorus, total phosphorus, dissolved organic carbon and dissolved inorganic carbon. Nutrient data are summarized in Table 33. Further comments on nutrient data follow.

Table 33. Summary of Nutrient Data

Parameter	Number of Samples	Mean	Standard Deviation	Range
NO_3/NO_2 as Nitrogen (mg/L)	78	1.30	0.75	0.25 - 4.00
Total nitrogen	132	1.14	0.74	0.25 - 4.60
Dissolved phosphorus (mg/L)	26	0.011	0.013	0.001 - 0.051
Total phosphorus (mg/L)	56	0.087	0.122	0.013 - 0.600
Dissolved organic carbon (mg/L)	122	2.10	1.26	0.8 - 9.9
Dissolved inorganic carbon (mg/L)	45	3.20	1.36	1.0 - 8.8

Dissolved nitrites and nitrates occurred with concentrations typical for urban runoff. Their mean concentration of 1.30 mg/L corresponds fairly well to the mean concentration of 0.94 mg/L reported for urban runoff elsewhere (13). Total nitrogen, however, appears to be somewhat lower than reported elsewhere. The mean value found for Newtown data was 1.14 mg/L which is significantly lower than the mean concentrations reported for other areas (2.0 mg/L, ref. 13).

The remaining nutrients, phosphorus and carbon, were also found in relatively low concentrations. The mean concentrations of dissolved and total phosphorus were 0.011 and 0.087 mg/L, respectively. By comparison, the typical mean concentration of total phosphorus in stormwater is about 0.3 mg/L, or about 3.6 times higher.

For Newtown stormwater, dissolved organic carbon had a mean concentration of 2.10 mg/L, as compared to 7.63 mg/L reported elsewhere (13). Finally, the mean concentration of dissolved inorganic carbon was 3.20 mg/L. The lower values of total nitrogen, phosphorus and carbon in Newtown may be explained by the lower fertility of the Newtown soil in comparison to that of soils in the above referenced areas (13).

8.3.5 Bacteria

Two types of bacteriological counts were done for Newtown stormwater samples - total coliforms and fecal coliforms. These organisms are used as pathogenic bacterial indicators. Fecal coliforms then indicate that portion of the total coliforms that is associated with feces of warm-blooded animals.

The Newtown bacteria counts are summarized in Table 34 together with data from several other urban catchments in Ontario (7).

Total coliform densities (MPN/100 ml) in Newtown stormwater varied from 6.4×10^2 to 7.6×10^4, with the geometric mean of 6.2×10^3. The fecal coliform densities varied from 0.9×10^2 to 4.5×10^4, with the geometric mean of 1.1×10^3. Both ranges indicate significant bacteriological pollution which is probably caused, to large extent, by

Table 34. Bacteria Counts in Stormwater from Various Urban Test Catchments

Stormwater Origin	Total Coliform (MPN/100 mL)[1] Range	Total Coliform Geometric Mean	Fecal Coliform (MPN/100 mL) Range	Fecal Coliform Geometric Mean
Newtown (Mount Pearl), Nfld.	$6.4 \times 10^{2} - 7.6 \times 10^{4}$	6.0×10^{3}	$0.9 \times 10^{2} - 4.5 \times 10^{4}$	1.1×10^{3}
Aldershot (Burlington), Ont.[2]	$1.9 \times 10^{4} - 1.8 \times 10^{7}$	1.5×10^{5}	$5.2 \times 10^{2} - 5.1 \times 10^{4}$	7.4×10^{4}
Barrington (Toronto), Ont.[2]	$3.0 \times 10^{3} - 1.19 \times 10^{6}$	9.6×10^{4}	$2.2 \times 10^{2} - 5.6 \times 10^{5}$	1.5×10^{4}
Brucewood (Toronto), Ont.[2]	$2.8 \times 10^{3} - 3.5 \times 10^{4}$	1.1×10^{4}	$1.0 \times 10^{3} - 1.9 \times 10^{4}$	3.9×10^{3}
Malvern (Burlington), Ont.[2]	$1.4 \times 10^{3} - 5.6 \times 10^{6}$	1.5×10^{4}	$1.0 \times 10^{2} - 3.3 \times 10^{5}$	3.6×10^{3}

[1] Bacteria count expressed as the most probable number of bacteria per 100 mL of sampled water.

[2] After Ref. 7

the remaining connection of sanitary sewage to storm sewers in the drainage system. Compared to other urban runoff data, however, the coliform densities in Newtown stormwater are not excessive and, as such, represent the lower end of the range of densities reported elsewhere (7).

8.4 Overview of Stormwater Quality

Stormwater quality in the Newtown catchment shows typical signs of pollution associated with urban runoff. In comparison to data from other areas, the Newtown stormwater shows a relatively low degree of pollution. This may follow from the relatively low density of urban development in the area, a low level of land-use activities in the catchment, and the absence of large industrial complexes in the vicinity of the catchment. It is conceivable that some improvement in the Newtown stormwater quality would be achieved by disconnecting the remaining source of sanitary sewage from the storm sewer system. While this single connection does not represent a particularly strong source which would continuously affect the composition of stormwater, it may contribute to some very high concentrations of nitrogen and bacteria occasionally observed at the outfall. Brief comments on selected individual parameters follow.

Suspended solids were found in concentrations typical for clean urban catchments (about 50 mg/L). Among major ions, dissolved sodium, chloride, sulphate and calcium were found in somewhat elevated concentrations. The main sources of sodium and chloride were marine aerosol deposits and road salt, with the former source prevailing as indicated by the ratio Na(mg/L) : Cl(mg/L) = 76 : 139 $<$ 2 : 3. The road salt was thought to be the main source of sulphate and calcium. Among metals, only zinc and lead occurred in concentrations which may cause some concerns because they exceeded safe guidelines for aquatic life. The true toxic potential of these metals, however, could not be properly evaluated, because the data produced in this study represented extractable metals and thus included those metal quantities which were originally adsorbed on the particulate matter and not readily available to the aquatic biota as dissolved species. The main sources of zinc include galvanized sewer pipes, tires, and soils. Lead concentrations in Newtown were relatively low compared to other urban areas - as a result of low traffic density in the area. Nutrient levels in Newtown stormwater were also fairly low and did not cause any special concerns. Finally, although the bacteria densities were also low, they indicated a presence of significant sources of bacteria.

9.0 RECOMMENDATIONS FOR URBAN DRAINAGE DESIGN

9.1 Design Concepts

Conceptual design of urban drainage has been greatly improved during the last 10 years. The advent of computer models for urban runoff and increased awareness of water resources problems caused by drainage led to the formulation of new drainage policies designed to alleviate such problems. Some of the new concepts earlier proposed in Ontario (16) and recommended for application in Newfoundland are summarized below.

1. Master Drainage Plans:

Master plans should be developed for all watersheds within the municipal boundary. Such plans are particularly important for rapidly developing municipalities so that drainage systems can be developed in an orderly manner compatible with watershed needs. Furthermore, this approach makes it possible to identify existing water quality and flooding problems, and to avoid future problems caused by urban development.

2. Pollution Control Strategies:

As drainage effluents may contribute to pollution of receiving waters, it is desirable to develop a comprehensive pollution control strategy for the municipality considering both wet and dry-weather pollution sources. In the first stage, all pollution sources and their contributions are identified. In the second stage, for the receiving water objectives, a comprehensive pollution control strategy is developed. With regard to urban drainage, one would need to evaluate stormwater and combined sewer overflows (if they exist, or sewage treatment plant bypasses) as wet-weather sources. Such sources are then considered in the development of a pollution control strategy.

3. Minor-Major Drainage Concept:

Drainage systems in new developments should recognize the minor-major drainage concept. In this concept, the minor drainage provides convenience during minor (high frequency) runoff events and the major drainage provides for reduction of flood damages and life protection during major (rare) runoff events. The minor drainage system typically consists of road gutters and sewers and should be designed for return periods from 2 to 25 years. The lower end of the range, say 2 to 5 years, is used in residential areas and the upper end, up to 25 years, might be used in downtown districts where any disruption of normal activities is to be avoided. The major drainage represents the route followed by runoff water during severe storms. Such a system may consist of streets, drainage swales and local streams. This major system is then designed (or checked) for return periods up to 100 years.

4. Runoff Control Measures for New Developments:

Proponents of new developments should indicate the effects of the proposed development on the watershed and receiving waters. Should the proposed development have a serious impact on the watershed and receiving waters, e.g., in the form of hydrologic changes and pollutional effects, remedial measures for controlling such effects should be requested by the design approval agency. Under severe conditions, it may also be necessary to implement an erosion and sediment control program during the construction stage of development.

9.2 Technical Aspects of Drainage Design

Besides the above general concepts for urban drainage, specific recommendations for urban drainage design should be developed for individual municipalities with drainage collection. Some of the findings from this study with a direct bearing on such design work are summarized below.

1. Design Rainfall Data

The sources of rainfall data for drainage design in the St. John's area are the stations operated at the airport, CDA, and Newtown. In general, the airport data should be used because they represent the longest record (24 years) in the area. As more data are collected at Newtown and the CDA data are fully analyzed, it should be possible to develop new low return period IDF curves for these two stations and, should they differ from the airport curves, to use them for drainage design in the vicinity of Mount Pearl.

The IDF curves from the airport station can be used directly with the rational method. For runoff modelling, it is recommended to use either historical storms from this study or after some verifications, the newly proposed 1-hour design storm (presented in the Appendix).

2. Runoff Computations

Computations of runoff for drainage design can be generally accomplished by two types of procedures - empirical formulas for peak discharge, or by urban runoff models.

When designing drainage for small areas with no need for runoff storage and water quality considerations, empirical formulas for runoff peaks, such as the rational method, are adequate for this type of design work. In applications of the rational method in the St. John's area, it is recommended to consider the runoff coefficient for the type of surface under consideration and summer conditions of the ground and moisture, because the highest runoff rates were observed in the test catchment during summer months. For these conditions and 5- to 10-year return periods, the runoff coefficients should be considered in the following ranges:

Character of Surface			Range of Runoff Coefficients
Pavement			0.8 - 0.95
Rooftops			0.75 - 0.95
Lawns	- sandy soil	- flat slope (2%)	0.05 - 0.10
		- average slope (2-7%)	0.10 - 0.15
		- steep slope (>7%)	0.15 - 0.20
	- heavy soils	- flat slope (2%)	0.13 - 0.17
		average slope (2-7%)	0.18 - 0.22
		steep slope (>7%)	0.25 - 0.35

Using the above runoff coefficients, it is possible to develop composite runoff coefficients for various types of urban development (e.g., residential areas, industrial areas, etc.).

Close attention should be paid to the selection of the time of concentration and the contributing area in runoff calculations. As shown earlier in this report, the assumption of runoff contribution from the whole catchment area leads to very long times of concentrations, low rainfall intensities, and the resulting peak flows smaller than those calculated for some parts of the catchment with fast response. Thus, it is recommended to repeat the whole calculation for various parts of the drainage area contributing runoff. As done in this report, the highest runoff discharge was found in the case when all directly connected imperious areas and the areas within 15 m of the directly connected areas were contributing runoff. This finding may vary depending on the drainage patterns in the area as well as on the shape of the IDF curve. Applications which include the use of IDF curves with fast decay may be very sensitive to the variation in the contributing area and the time of concentration.

For estimating the overland runoff time of travel, the kinematic wave formula used in Chapter 6 was found satisfactory. The use of this formula yielded travel times from 4 to 7 minutes, depending on the rainfall intensity and the contributing area.

In runoff calculations for larger areas (say greater than 5-10 ha), and where the entire runoff hydrograph or pollutograph are of interest, it may become necessary to use an urban runoff model. In the Canadian drainage practice, the SWMM and ILLUDAS models are most widespread and both models have been found satisfactory in this study. Consequently, it is recommended to use SWMM, ILLUDAS, or equivalent models for drainage design. Both models are non-proprietary and widely available. For quantity calculations in small-to-intermediate catchments similar to the Newtown catchment, both models should perform equally well. The SWMM model has some additional features which are of interest when dealing with runoff quality, storage and treatment, and when analyzing receiving waters.

In detailed modelling work, the catchment discretization is not particularly important, as long as the subcatchments are not excessively large (say 0.5-1 ha) and their layout follows the actual drainage pattern. Among the physiographic data, the delineation of the directly-connected (effective) impervious areas is probably the most important task and requires close attention. The surface detention storage on these areas should be taken between 0.5-1.6 mm. For pervious areas, it is important to determine soil infiltration rates. For smaller rainfalls (say less than 50 mm), it was found to be sufficient to correctly identify the soil type and the corresponding infiltration rates. Errors in the identification of the soil group by one class up or down were not significant. For very heavy rainfalls (> 50 mm), not only the soil group is important, but also the antecedent moisture conditions. In this regard, the consideration of the antecedent moisture conditions was easier with the ILLUDAS model (which accepts the AMC parameter as one of the inputs) than the single-event SWMM model.

Further attention should be paid to the utilization of sewers in various parts of the catchment. As discussed in Chapter 6, storm sewers in several crescents in the test catchment do not convey any stormwater, only discharges of basement and foundation drains. Under such circumstances, the use of the sewer pipes with the minimum diameter of 0.305 m seems excessive and should be reviewed. Foundation drain flow could be conveyed by considerably smaller pipes (e.g., 0.15 m in diameter). Because of the purity of such water, there are no concerns about possible clogging of these small pipes. There are other alternatives to basement drainage - e.g., using sumps and sump pumps for individual houses. Such alternatives may be, however, less convenient and desirable from the homeowners point of view. The use of small pipes for foundation and basement drainage would significantly reduce drainage costs.

Finally, there are many other technical details which need to be also specified in drainage design criteria. Such details include specifications for various drainage structures and appurtenances. The development of such specifications is beyond the scope of this study.

9.3 Transposition of Study Results

The hydrological study of the Newtown catchment demonstrated applicability of the rational method and two urban runoff models to the hydrological design of urban drainage. Because all these design tools represent more or less physically-based approaches to hydrological computations, they are transferable to other areas provided that appropriate physiographic data and model parameters are used. Since the collection of physiographic data is a straight forward task, the transposition of study results is reduced to the question of transposition of various process parameters used in this study. Such a problem is discussed below starting with the rational method and followed by the runoff models.

9.3.1 Rational method

The rational method is an empirical method used widely for design of minor urban drainage. In most cases, the parameters used in applications of the method have been established through local experience and produce satisfactory results. From the theoretical point of view, the validity of the rational method cannot be experimentally proven because the runoff equilibrium conditions are never observed in urban catchments. Thus, only an indirect verification of the rational method is possible by comparing, for selected return periods, the calculated and observed peak flows. Such comparisons are made difficult by the fact that the observed peak flows contain uncertainties inherent to the frequency analysis and such uncertainties increase in the case of short records which have to be extrapolated. Notwithstanding the above difficulties, it is possible to standardize the use of the rational method and to ensure its consistent use. With that goal in mind, a discussion of individual method parameters follows.

Runoff coefficient - Runoff coefficient attempts to synthesize the effects of numerous hydrological processes occurring in urban catchments. Because of large variations in such processes from one catchment to another, it is not recommended to transpose composite runoff coefficients. Instead, only the runoff coefficients developed for individual surface types should be transposed and used to develop site-specific composite runoff coefficients. In this sense, the runoff coefficients used here for the impervious and pervious areas could be transposed to other similar catchments.

Rainfall intensity - The design rainfall intensity is selected from local IDF curves for a given return period and the time of concentration of the contributing area. The time of concentration is defined (21) as the time of travel from the hydraulically most remote point within the contributing area. Such a time is difficult to determine, because it depends on the length, slope and roughness of the travelled route, and the rainfall intensity. Generally, times of concentration are not transposable from one catchment to another, if their physiographic characteristics differ. To simplify calculations of the time of concentration, the task is sometimes divided into two parts - the determination of the inlet time, or the time of travel of the overland flow, and the determination of the time of travel in sewers or other transport elements. Again, it is preferable to calculate the local inlet times rather than to transpose them from elsewhere.

When calculations produce long inlet times, it is possible that the highest runoff peak is obtained when only the fast-responding parts of the catchment are contributing runoff, because with the increasing time of concentration, the corresponding intensity read from the IDF curves diminishes. This possibility should be checked in calculations.

9.3.2 Urban runoff models

Applications of urban runoff models require three types of data - hydrometeorological data, physiographic data, and hydrological process parameters. Hydrometeorological data (mostly rainfall data) are transferable within the local range of their validity. On the other hand, physiographic data describe catchment characteristics and, therefore, are not transferable. The transferability of hydrological process parameters is a complex problem which is further discussed below.

Hydrological process parameters are defined as the numerical values used in various relationships to quantify the movement and storage of water in a catchment. Only some of these parameters are important in urban hydrology; namely, overland flow parameters, detention storage and infiltration. The transposition of such parameters is discussed below.

In applications of the SWMM and ILLUDAS models, three parameters related to overland flow computations need to be established:

(a) The overland flow plane roughness (in both models).
(b) The overland flow plane width (in SWMM).
(c) The inlet time (ILLUDAS).

With regard to the first parameter, the sensitivity analysis (18) indicates that the results are barely sensitive to the overland flow plane roughness, within the practical range of values. The commonly used values of n_i = 0.013 (impervious areas) and n_p = 0.20 - 0.25 (pervious areas), which were also used in the Newtown catchment, may be transposed to other catchments.

The width of the overland flow plane is in principle a physiographic parameter defined as the subcatchment area divided by twice the length of the sewers in the subcatchment (for more details, see the SWMM manual (9)). This parameter is, however, sometimes used as a calibration parameter. In the absence of calibration data, such a practice is discouraged.

The specification of an inlet time is one of two options in overland flow calculations with the ILLUDAS model. It is preferable to avoid this option and specify the length, slope and roughness of the flow route and let the model calculate the inlet time internally.

Another group of hydrological process parameters relates to surface detention storage. The values used in this study ($d_{imp.}$ = 1.5 mm; $d_{per.}$ = 5 mm) followed the recommendations in the SWMM manual (9). While the value for impervious areas was tested and found satisfactory in the Newtown catchment, the pervious area value was not adequately tested. Detention storage on pervious areas becomes important only during severe storms. Only two such storms were observed during the study period and, therefore, did not provide a sufficient data base for verifying the detention storage values for the pervious areas.

Infiltration parameters - It became obvious from the modelling results that for storms with low to intermediate rainfall depths, the best reproductions of observed hydrographs were obtained when practically all runoff originated on impervious areas. In this case, the soil infiltration characteristics corresponded approximately to those assigned to the soil group B (in the ILLUDAS Model) which was also reported in field surveys. For such storms, the consideration of the antecedent moisture was not important. The antecedent moisture became important only for the two most severe storms (rainfall greater than 50 mm). Such results are transposable to similar catchments with the same soil group (B), soil cover (grass), and range of slopes (2%-7%). In other cases, the model infiltration parameters may have to be adjusted. It should be noted, however, that for storms whose runoff originates mostly on impervious areas, the choice of infiltration parameters is not critical.

9.3.3 Comparison of study recommendations with drainage design standards

It is desirable to compare the Newtown study recommendations for drainage design with the existing design standards. Such standards were prepared for Environment Newfoundland in January, 1974. The section which is of particular interest is Chapter VIII dealing with storm sewers.

Chapter VIII deals with general aspects of sewer design, design factors, and appurtenances. The only section which may be affected by the Newtown study recommendations is the section on design factors. Such factors include the design return period and runoff computations.

Design Return Period

This particular section should be further expanded in the existing standards. It should be mentioned in this section that there are two drainage systems, the minor and the major, and if provisions for both systems are made in the design, then the discussion in the existing standards is appropriate for minor drainage. It would be desirable to recommend minor drainage system design return periods for various types of developments, as suggested below:

Land Use	Design Return Period (Years)
Residential	2 - 5
Commercial	5 - 10
Industrial	2 - 10

Where flooding would cause serious problems, longer design return periods should be considered.

Runoff Computations

Runoff computations should be accomplished by the rational method or hydrological models similar to those used in this study (ILLUDAS or SWMM). The modelling results obtained with properly applied well-established models should be accepted even if they differ from the results obtained from the rational method. The rational method should be presented in metric units, because the input data (e.g., rainfall intensity and areas) are available in metric units. The appropriate metric form of the rational method is given by Eq. (6).

The standards recommend composite runoff coefficients for certain types of land use. As discussed in section 9.2 of this report, it is preferable to derive the composite runoff coefficient for each area under design by considering runoff coefficients for individual types of catchment surfaces (i.e., using Eq. 7 and the data on page 106). The composite runoff coefficients given in the standards more or less follow the lower limits given in a standard design handbook (21). Should the composite coefficients be retained, then it would be desirable to list their full ranges:

Residential Areas	Runoff Coefficient	Inlet Time (min)
Single-family units	0.3 - 0.5	15 - 30
Multi-units	0.4 - 0.7	10 - 20
Apartments	0.5 - 0.7	10 - 20
Business Areas		
Downtown	0.7 - 0.95	5 - 10
Neighbourhood	0.5 - 0.7	10 - 15
Industrial Areas	0.5 - 0.9	-----

Finally, the inlet times also need to be established. Although their best estimates may be obtained from Eqs. (14-16), such procedures may be too tedious. If it is desirable to specify inlet times, their full ranges should be given, as shown above. The lower values should be considered for longer return periods, high surface drainage density (in the form of open channels), and fair slopes. The higher values are appropriate for shorter return periods, low drainage density, and mild slopes. In case of industrial developments, inlet times should be calculated because of wide variations in drainage design for such areas.

It should be emphasized that the selection of values within the recommended ranges should be guided by considerations of drainage characteristics of the areas under design. Arbitrary choices may lead to wide variations in runoff estimates as demonstrated below in an example. In this example, the first line represents the calculations made in this study, the second line follows the existing design standards, and lines three and four represent the worst combinations of runoff coefficients and inlet times selected from the table above.

Sample Calculations of Runoff Peaks by the Rational Method

Area (ha)	Runoff Coeffi- cient	Inlet Time (min)	Sewer Tra- vel Time (min)	Time of Con- centration t_c (min)	5-Yr Rainfall Intensity for t_c (mm/hr)	Dis- charge (m /s)
13.23	0.36	27	9	36	28	0.370
13.23	0.3	20	9	29	31	0.350
13.23	0.3	30	9	39	27	0.290
13.23	0.5	15	9	24	35	0.640

10.0 CONCLUSIONS

1. The data collection system operated in the Newtown catchment provided good data for analysis of rainfall/runoff processes during relatively frequent storms. Only a few heavy storms have been fully monitored in the catchment so far. The extension of the above analysis to such infrequent storms would require continuing the data collection. The data collected indicate that rainfall intensities in Newtown may differ from those at the airport. This should be considered in future analysis of design rainfall. The observed runoff flows were subject to frequency analysis and, on the basis of the limited data available, the design flows for the Newtown catchment are estimated as follows:

	1-year	2-years	5-years	10-years
Flow rate (m /s/1 ha of developed area)	.023	.028	.034	.039

The flow rates for 5 and 10-year return periods have been extrapolated. Because of the short duration of the runoff record available, the above rates contain uncertainties which increase with an increasing return period. Relatively frequent occurrences of high intensity storms during the study period led to a speculation that the 10-year flow rate may be somewhat overestimated. Further revisions of these rates will be possible as more data become available.

2. The above listed observed or extrapolated flow rates were only slightly underestimated by those calculated from the rational method provided that the following steps were taken in such calculations:

    (a) The St. John's Airport IDF curves were used.

    (b) Runoff travel times and inlet times were calculated from the kinematic wave equation using the slopes and lengths measured in the field or from maps, rainfall intensities read from the IDF curves, and taking the overland flow roughness as n = 0.013 and 0.20 for impervious and pervious areas, respectively. The calculated inlet times for effective impervious areas was taken as 4 minutes regardless of the return period.

    (c) The highest runoff flow at the outlet was obtained for the case where only 62% of the catchment area was contributing runoff. This area represented the entire effective impervious area and pervious or noneffective impervious areas within 15 m of streets. In this case, the combination of the contributing area, the composite runoff coefficient $C_c = 0.45$ ($C_{impervious} = 0.95$, $C_{pervious} = 0.22$, for all return periods studied), and the relatively high rainfall intensity corresponding to short inlet times yielded the highest runoff discharge. Using the above procedures, the peak flows calculated from the rational method were about 11% below the observed (or extrapolated observed) flows which may be overestimated.

For any other combinations of contributing areas, including the case of the whole catchment contributing, the calculated peak flows at the outlet were smaller, because of increased inlet times and correspondingly reduced rainfall intensities and runoff coefficients.

(d) In applications of the rational method, it is recommended to check flow calculations for various partial contributing areas, in order to find the maximum runoff peak. It appears that calculations for three contributing areas should be sufficient to obtain a peak close to the maximum. Such three combinations of contributing areas could be selected as follows:

- Effective impervious areas only.
- Effective impervious areas plus any other areas draining directy towards streets (those are generally areas within say 10-30 m of the transport channels - street gutters).
- The entire catchment area.

3. Runoff events observed in the Newtown catchment were successfully reproduced by both models used - the ILLUDAS and SWMM urban runoff models. For the purpose of runoff modelling, the Newtown catchment was subdivided into about 20 subcatchments which were drained by 20 sewer pipes. The rainfall input data were discretized in 5-minute intervals. Using such data and subcatchment physiographic data from maps and field surveys, the observed runoff events were reproduced fairly well by both models. For smaller storms (rainfall <50 mm), the contributions of pervious areas were rather small and without much calibration, both models reproduced the mean observed runoff volume within 10% of the mean value. For individual events, the differences between observed and simulated volumes varied from -25% to 13%, at the 95% level of confidence.

When heavy storms (with rainfall >50 mm) were included in the test data, it became desirable to properly simulate runoff contributions from pervious areas. Although this can be done by both models, the ILLUDAS model, which accepts the soil group and antecedent moisture conditions as input, was found to be easier to apply. The accuracy of runoff volume simulations was adversely affected by the presence of baseflow in Newtown sewers. Such a baseflow was contributed by foundation drains and infiltration of groundwater into the sewers. The required separation of baseflow from flow hydrographs did not eliminate all uncertainties in the observed runoff volumes.

Runoff peak flows were reproduced fairly well by both models. The mean value of the difference between observed and simulated peaks varied from .000 to 0.009 m^3/s for the best three simulation series (I3, I6 and S3). The 95% confidence limits for these means were about -0.020 and +0.030 m^3/s.

When considering both runoff volumes and peaks, the simulation series S1, S2, I1, I3 and I6 produced comparable results for small to medium events. For events with large rainfall and runoff, it was desirable to account for an increased contribution of pervious areas. This was best achieved in runs I3 and I6, assuming the soil group B and the antecedent moisture conditions based on the 5-day antecedent rainfall.

In summary, the following model parameters are recommended:

(a) Effective impervious areas - their area should be determined from maps and field surveys (in Newtown, A_{ei} = 2.62 ha). The surface detention storage values in the range from 0.4 mm to 1.6 mm provided good results. The surface roughness coefficient may be taken as n = 0.013.

(b) For pervious areas, the best results were obtained by assuming the soil group B and the antecedent moisture conditions derived from the 5-day antecedent rainfall. For the SWMM model, the initial and terminal infiltration rates were set at 51 mm/hr and 7.6 mm/hr, respectively. Suitable values for the surface detention storage and surface roughness coefficient are 5 mm and n = 0.20, respectively.

4. The composition of Newtown stormwater shows typical signs of urban pollution arising from characteristic land-use activities. In comparison to other urban areas, the Newtown stormwater is relatively unpolluted and this may follow from a relatively low density of the development, low levels of urban activities within the catchment (e.g., light traffic), and absence of industrial complexes in the immediate vicinity. When examining individual groups of parameters, some concerns may be raised by the elevated concentrations of sodium, chloride, zinc and lead. The first two elements most likely are connected with road-salting operations in the catchment. Two main sources of zinc were assumed to be galvanized sewer pipes and automobile tires. The main sources of lead are automobile exhausts.The mean observed concentrations of zinc and lead exceeded the safe guidelines established for aquatic life.

11.0 RECOMMENDATIONS

Based on this study, the following three recommendations are made for future activities:

1. Continue the collection of rainfall and runoff data in the Newtown catchment. With minimal additional effort and expenses, it should be possible to extend the field rainfall and runoff records past the current length of three years. This would permit more accurate estimates of the statistics of rainfall and runoff peaks especially for the common design periods from two to 10 years. Future efforts

should concentrate only on heavy storms producing large runoff volumes and peak flows. For future data collection, the flow monitoring station does not require any modifications. However, closer attention should be paid to the monitoring of rainfall. The field observers should be further trained and more closely supervised. The feasibility of installing a second recording gauge in the catchment should be investigated.

2. The rainfall/runoff data to be collected in the future should be used to produce new rainfall IDF curves and to revise the modelling results given earlier in this report. These extended data records may also warrant further modelling work with newer versions of the models used.

3. In consultations with municipal and consulting engineers, the principal findings and recommendations of this report should be implemented in urban drainage design in the Province of Newfoundland.

12.0 REFERENCES

1. Alley, W.M. (Ed.). Guide for Collection, Analysis and Use of Urban Stormwater Data. American Society of Civil Engineers, Urban Water Resources Research Council, New York, N.Y., 1977.
2. American Public Health Association. Standard Methods for Examination of Water and Waste Water. 15th Edition, 1980.
3. Arseneault, R.A., T.L. Pollock, S. Ratnam, and D. Hansen. Surface Water Quality Study in the Waterford River Basin, Newfoundland. Technical Report, Waterford River Basin Study, St. John's, Nfld., 1985.
4. Brater, E.F. and King, H.W. Handbook of Hydraulics. 6th Edition, McGraw-Hill Book Company, New York, N.Y., 1976.

5. Chow, V.T. Handbook of Applied Hydrology. McGraw-Hill, New York, N.Y., 1964.
6. Department of Consumer Affairs and Environment. Waterford River Basin Urban Hydrology Study Plan. Environmental Management and Control Division, St. John's, Newfoundland, January 1978.
7. Dutka, B.J. I. Rybakowski, A.A. Qureshi, and S. Tobin. Microbiological Characteristics of Urban Storm Water Runoffs in Central Ontario. Res. Report No. 87, Canada-Ontario Agreement on Great Lakes Water Quality, Environment Canada, Ottawa, 1978.
8. Hogg, W.D. Distribution of Rainfall With Time: Design Considerations. Paper presented at the American Geophysical Union Chapman Conference on Rainfall Rates, Urbana, Illinois, April 27-29, 1982.
9. Huber, W.C., J.P. Heaney, S.J. Nix, R.E. Dickinson, and D.J. Polman. Storm Water Management Model User Manual - Version III. Dept. of Environmental Engineering Sciences, University of Florida, Gainesville, Florida, 1982.
10. Hydrotek, Inc. Urban Design Storms for Canada. Report to the Atmospheric Environment Service, Environment Canada, Toronto, Ontario, March, 1985.
11. Kibler, D.F. (Editor). Urban Stormwater Hydrology. Water Resources Monograph 7, American Geophysical Union, Washington, D.C., 1982.
12. Larson, C.L. Experiments on Flow Through Inlet Gratings for Street Gutters. Research Report 6-B, Highway Research Board, Washington, D.C., 1948.
13. Marsalek, J. Caractérisation du ruissellement de surface issu d'une zone urbaine commerciale. Sciences et Techniques de l'Eau, Vol. 17, No. 2, May, 1984, pp. 163-167.
14. Marsalek, J. Urban Hydrological Modelling and Catchment Research in Canada. Technical Bulletin No. 98, Inland Waters Directorate, Environment Canada, Ottawa, Ont., 1976.

15. Meredith, D.D. Dynamic Programming With Case Study on Planning and Design of Urban Water Facilities. Treatise on Urban Water Systems, Colorado State University, Fort Collins, Colo., Jan., 1972.
16. Ministry of the Environment and Environment Canada. Proposed Model Policies for Urban Drainage Management. Research Report 102, Canada-Ontario Agreement, Environment Canada, Ottawa, Ont., 1980.
17. Panu, U. Evaluation of Urban Hydrologic Models for Application in the Waterford River Basin Urban Hydrology Study. Department of Environment, Province of Newfoundland, St. John's, Feb., 1982.
18. Proctor & Redfern, Ltd. and James F. MacLaren, Ltd. Storm Water Management Model Study Vol. I. Res. Report No. 47, Canada-Ontario Agreement on Great Lakes Water Quality, Environment Canada, Ottawa, Ont., 1976.
19. Terstriep, M.L. and J.B. Stall. The Illinois Urban Drainage Area Simulator: ILLUDAS. Bulletin No. 58, Illinois State Water Survey, Urbana, Ill., 1974.
20. U.S. Department of Agriculture. Urban Hydrology for Small Watersheds. Technical Release 55, Soil Conservation Service, Washington, D.C., Jan., 1975.
21. Water Pollution Control Federation. Design and Construction of Sanitary and Storm Sewers. WPCF Manual of Practice No. 9, WPCF, Washington, D.C., 1970.
22. Water Quality Branch. Analytical Methods Manual. Water Quality Branch, Environment Canada, Ottawa, 1979.
23. Water Quality Branch. Manual Sampling for Water Quality. Water Quality Branch, Environment Canada, Ottawa, 1983.

APPENDIX

DESIGN STORMS FOR ST. JOHN'S, NEWFOUNDLAND

Hydrologic synthesis of urban runoff hydrographs requires the use of time-variable rainfall data. For design purposes, such data are specified in the form of the so-called design storms which, in their simplest form, are characterized by the storm return period, duration, total rainfall, and temporal rainfall distribution. Design storms are typically established by special analysis of local rainfall patterns. In the case of St. John's, two types of design storms were reported in the literature (8,10). A brief summary of characteristics of these design storms is given below.

Design storms for sizing of storm sewers in small drainage systems without storage facilities are fully characterized by the above-mentioned four parameters which are further discussed below.

a) Return period - the return period of a design storm is selected on the basis of considerations presented earlier in Section 9.3.3. Typically, such return periods vary from 2 to 10 years.
b) Storm duration - design storm duration depends on both the nature of local storms and the response of the catchment under consideration. For small urban catchments (A = 20 ha), a storm duration of one hour is appropriate and should be used in design.
c) Total storm rainfall - for a given storm return period and duration, the total storm rainfall can be determined from local IDF curves. In the case under consideration, the St. John's Airport IDF curves would be appropriate to use.
d) Temporal rainfall distribution - for the area considered, two types of temporal rainfall distributions were found in the literature. The first one represents the AES temporal distributions (8) and the second one is the empirical distribution developed by Hydrotek for AES (10).

AES temporal rainfall distributions for various probabilities of exceedance are given in Table 1. Such distributions are given as cumulative percentages of the total storm rainfall for 12, 5-minute, intervals. For example, the value of "81" for 10% exceedance in the table below means that there is a 10% probability that 81% (or more) of the total storm rainfall will occur during the first 20 minutes of the 1-hour storm.

Table 1. AES 1-Hour Rainfall Distributions for St. John's Airport (8)

Probability of exceedance	Cumulative Rainfall (in percents)											
	Time (minutes)											
	5	10	15	20	25	30	35	40	45	50	55	60
10%	15	53	77	81	91	94	94	98	100	100	100	100
20%	13	33	56	61	80	88	92	96	97	100	100	100
30%	12	24	40	54	67	77	86	92	96	98	100	100
40%	10	21	35	46	61	71	82	89	92	97	99	100
50%	7	18	29	42	52	64	78	86	91	95	98	100
60%	5	13	23	34	49	60	69	78	87	92	97	100
70%	5	11	20	29	45	53	65	73	85	91	96	100
80%	4	9	15	23	32	48	48	64	75	87	94	100
90%	3	8	11	19	27	38	45	62	71	79	89	100
Mean Distribution	1	5	14	31	60	75	85	92	96	98	99	100

For drainage design, AES recommended to use the 30% or mean distributions (8).

The Hydrotek temporal distribution was developed by extensive analysis of St. John's rainfall data. This distribution is independent of the storm return period for return periods from 2 to 15 years and may be used for storm durations up to three hours. The Hydrotek temporal rainfall distribution is given below in Table 2.

Table 2. Hydrotek 1-Hour Rainfall Distribution for St. John's (10)

	Time (minutes)											
	5	10	15	20	25	30	35	40	45	50	55	60
Cumulative Rainfall %	2	10	22	39	61	83	93	97	99	100	100	100

Although neither of the above temporal distribution has been yet fully evaluated and tested, the Hydrotek distribution seems to have some advantages arising from its greater applicability in terms of storm return periods and durations.

To demonstrate the use of temporal rainfall distributions, two design storms are developed in the example below.

Problem: Develop Hydrotek and AES design storms for St. John's

Given Data: Return period = 2 years; Storm duration = 1 hour; Temporal distribution - both Hydrotek and AES distribution are given. For AES, use the mean distribution. St. John's Airport IDF curves are given.

Procedure: From the IDF curves (Fig. 6), the total storm rainfall is determined as 16 mm. This rainfall is then distributed in time as shown in Table 3.

Table 3. Design Storm Hyetographs for St. John's (2-year, 1-hour storms)

	Hydrotek Storm			AES Storm (mean distr.)		
Time (min)	Interval (%)	Rainfall (mm)	Rainfall Intensity (mm/hr)	Interval (%)	Rainfall (mm)	Rainfall Intensity (mm/hr)
5	2	.32	4	1	.16	2
10	8	1.28	15	4	.64	8
15	12	1.92	23	9	1.44	17
20	17	2.72	33	17	2.72	33
25	22	3.52	42	29	4.64	56
30	22	3.52	42	15	2.40	29
35	10	1.60	19	10	1.60	19
40	4	.64	8	7	1.12	13
45	2	.32	4	4	.64	8
50	1	.16	2	2	.32	4
55	0	0	0	1	.16	2
60	0	0	0	1	.16	2