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NWRI Unpublished Manuscript

URBAN RUNOFF PEAK FREQUENCY CURVES

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Marsalek (56)

Study 82-310

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January 1983

ABSTRACT

Runoff peak frequency curves are derived from the runoff flows observed in an urban test catchment and from the runoff flows simulated for two rainfall records. The first record, of a shorter length, has been recorded in the same test catchment. The second rainfall record, of a greater length, has been recorded at a nearby station outside of the catchment.

Comparison of the three runoff peak frequency curves produced are used to verify the proposed technique for deriving the runoff peak frequency curve from single-event runoff simulations and also for evaluation of the simulation model - the SWMM model of the U.S. Environmental Protection Agency.

RÉSUMÉ

Des courbes des fréquences des pointes de l'écoulement sont dérivées des écoulements observés dans un bassin versant urbain expérimental ainsi que des écoulements simulés pour deux enregistrements de hauteur de précipitations. Le premier de ces enregistrements, d'une durée plus courte, a été obtenu dans le même bassin versant expérimental. Le deuxième enregistrement de la hauteur de précipitations, portant sur une période de plus longue durée, a été obtenu à une station voisine à l'extérieur du bassin versant.

Les comparaisons des trois courbes des pointes de l'écoulement obtenues sont utilisées pour la vérification d'une méthode proposée d'obtention de la courbe des fréquences de l'écoulement à partir de simulations d'écoulements résultant de cas individuels ainsi que pour l'évaluation d'un modèle de simulation, le SWMM de l'Environmental protection Agency américaine.

MANAGEMENT PERSPECTIVE

Determination of runoff flows in the urban areas is the most important task of urban hyrology. This report offers a methodology for deriving design runoff flows by mathematical modelling of runoff from a series of actual rainfall events. Such a methodology is then verified by comparing the computed and observed flows. The good agreement which was obtained further proved the validity of the mathematical model used.

The methodology can be used in those urban catchments where the generation of runoff peaks is controlled by impervious elements. Such cases include catchments with medium - to - high imperviousness and catchments with well-drained soils.

T. Milne Dick, Chief
Hydraulics Division
May 31, 1983

PERSPECTIVE DE GESTION

En hydrologie urbaine la tâche la plus importante consiste à déterminer les écoulements en milieu urbain. Ce rapport présente une méthode permettant de dériver les écoulements nomimaux par simulation mathématique des écoulements pour un ensemble de cas observés de précipitations. Cette méthode est ensuite vérifiée par comparaison des écoulements calculés et observés. La concordance entre les résultats obtenus a confirmé la validité du modèle mathématique utilisé.

La méthode peut être utilisée dans les bassins versants urbains où les pointes de l'écoulement dépendent d'éléments imperméables, incluant les bassins versants d'une imperméabilité moyenne à élevée et les bassins versants où les sols sont bien drainés.

T. Milne Dick
Chef de la Divison de l'hydraulique
31 main 1983

1.0 INTRODUCTION

Determination of urban runoff peak frequency curves is one of the most important tasks of urban hydrology. Such curves serve for the design of various components of urban drainage systems.

One of the tools used for determination of runoff peak frequency curves is simulation of rainfall/runoff processes for selected rainfall inputs and catchment conditions. Ideally, a continuous simulation runoff model is used to produce a simulated runoff record which is then subject to frequency analysis to derive the runoff peak frequency curves. As an alternative to continuous simulation, surrogate continuous simulation has been proposed (6, 9). In this procedure the rainfall record is screened for events with high runoff potential, runoff hydrograph peaks are simulated for these events by means of an event model and then subjected to frequency ananlysis. Advantages of surrogate continuous simulation arise from lower input data requirements, lower computer costs, and the possibility of using common design runoff models.

Acceptability of discrete even runoff simulation (as opposed to continuous simulation) has been discussed by various researchers at some length (6, 9). Much of the criticism of discrete event simulation has been transposed from studies of natural catchments where the catchment flood potential is undoubtedly controlled by the antecedent moisture conditions. The runoff controlling processes are quite different in urban catchments. For relative short return periods which are used in minor drainage design (say 2-5 years), the generation of runoff peaks is primarily controlled by the impervious parts of the catchment. This is particularly true for catchments with certain minimum imperviousness (say 15%) and well drained soils. Consequently, the conditions pertaining to the pervious part of the catchment, such as the antecedent moisture conditions, become of secondary importance and may be approximated in discrete events simulations without any significant loss of reliability of results.

In the paper that follows, the runoff peak frequency curves are derived from simulated runoff events and compared to the curves derived from runoff peaks which were observed in a test catchment. Such comparisons serve to confirm the viability of using discrete event simulation to derive runoff peak frequency curves and also indicate the reliability of the runoff model employed to reproduce runoff events of widely varying return periods.

2.0 TEST CATCHMENT DESCRIPTION

Runoff simulation were undertaken for the Malvern test catchment which is located in Burlington. The catchment has been described in detail elsewhere (4,5) and only a brief description is included below for completeness.

The Malvern urban test catchment (see Figure 1) is a residential subdivision of 23.31 ha which is drained by storm sewers. The catchment was documented and instrumented in 1973. From 1973 to 1977, numerous rainfall runoff events were observed in the catchment and the results of such observations were reported earlier (4, 5).

The Malvern catchment is gently sloping from the north boundary line towards the drainage outfall in the southwest corner. The overall catchment slope is about 1%. Local slopes, however, depend on lot gradings. Front yards typically slope toward streets with slopes varying from 2 to 10%. Backyards are gently sloping away from streets toward drainage swales which run along the back line of lots.

The total area of impervious parts of the catchment was estimated as 7.85 ha, thus yielding the catchment imperviousness of 34%. Impervious parts of the catchment include roofs, roads, driveways and sidewalks. With the exception of sidewalks (0.66 ha), all the impervious parts are clearly directly connected to the storm sewers. The sidewalks drain either on driveways (directly connected) or on a narrow grass strip which separates them from streets.

The pervious, grass-covered parts of the catchment amount to 15.46 ha. The soil in these parts is the Fox sandy loam, shallow phase, and well-drained. Limited point measurements of infiltration yielded the initial infiltration values of about 120 mm/hr.

The Malvern catchment is served by a tree type, converging storm sewer system (see Figure 2). The sizes of sewers vary from 0.25 m in the upper reaches to 0.838 m at the outfall. All sewers are made of standard concrete pipes which are in relatively good condition. Assuming the pipe roughness as n = 0.013, the outfall pipe can convey a 2-year runoff peak without surcharge.

The Malvern catchment served for observations of rainfall/runoff events. For this purpose, a recording rain gauge and a measuring weir were installed at the outfall. The rain gauge was a standard tipping-bucket with the capacity of 0.25 mm. The measuring rectangular weir was installed in a weir box which was attached to the outfall. The weir remained operational even when the outfall pipe was surcharged. The accuracy of flow measurements was estimated as ±5% (4).

3.0 RAINFALL-RUNOFF DATA BASE

The rainfall/runoff data used in this study consisted of rainfall/runoff data which were observed at the Malvern catchment and of rainfall data from the meteorological station at the Royal Botanical Gardens (RBG) in Hamilton. The RBG station is about 10.7 km west of the Malvern catchment. The interest in the RBG data followed from the greater length of the RBG rainfall record - 15 years as opposed to five years in the case of the Malvern station. Although one would expect great differences in individual hyetographs observed at both stations, the general rainfall characteristics which are rather conservative should be similar at both stations. For example, it has been noted that the Intensity-Duration-Frequency curves at the RBG station and at the Oakville OWRC station which is 11.6 km east of the Malvern catchment are practically identical. It would appear the the maximum rainfall intensities at the Malvern catchment may be characterized by the IDF curves from the RBG station.

3.1 Malvern Runoff Peak Flows

Runoff flows were monitored at the outfall from the Malvern catchment from 1973 to 1977, inclusive. Such monitoring was continuous during the field season which spanned from April to December. High runoff peaks in the area under study are typically caused by summer thunderstorms (e.g., the top five peaks were observed from July to September) and it may be therefore assumed that the seasonal records contain top-ranked events.

During the monitoring period, about 300 rainfall/runoff events were monitored. Most of these events were rather minor. For the purpose of this study, only 13 events with top-ranked runoff peaks were selected for further analysis. A list of the selected events is presented in Table 1 below.

TABLE 1. Top-Ranked Historical Events Observed in the Malvern Test Catchment

Rank	Storm No.	Q_p [m^3/s]	Storm Date	T [years]
1	12	1.744	10.7.75	6
2	3	1.608	31.7.73	3
3	13	1.218	20.7.75	2
4	31	1.212	16.8.77	1.5
5	26	1.202	17.9.76	1.2
6	27	1.099	16.6.77	1.0
7	15	1.079	3.8.75	.86
8	4	1.031	2.10.73	.75
9	19	1.021	2.11.75	.67
10	17	.993	13.8.75	.60
11	14	.947	24.8.75	.55
12	1	.907	22.9.73	.50
13	6	.904	31.5.74	.46

In Table 1, the return periods T were calculated from the Weibull's formula as T = (N+1)/R where N is the record length in years and R is the rank of the runoff peak flow Q_p.

For storms listed in Table 1, rainfall hyetographs were prepared for runoff simulations. Some of these hyetographs were truncated at the end of the storm in order to reduce computer costs. All the storms were characterized by the total rainfall in the high-intensity burst(s) and by maximum rainfall intensities for durations of 5, 10, and 15 minutes.

3.2 Rainfall Data From the RBG Station

A 15-year rainfall record was available for the RBG (Hamilton) station which is operated by the Atmospheric Environment Service. This record was analyzed to identify storms that would be mostly likely to produce high runoff peaks.

Selection of actual storms that would be most likely to produce high runoff peak flows was facilitated by screening the rainfall record to segregate all storms with either a total rainfall depth larger than 1.25-cm or a ten-minute intensity larger than 1.5-cm/hr. A total of 54 storms met one or both of these criteria. Next, the top 20 storm depths were identified for durations of 5-, 10-, 15-, 30- and 60-minutes. Because a number of the storms contained multiple maxima, the segregation process yielded only 27 storms that met all the selection criteria. For the purpose of establishing the frequency of occurrence of runoff peaks on the catchments studied, these 27 storms were regarded as a suitable replacement for the 15-year rainfall record. The basic characteristics of the 27 selected storms are summarized in Table 2.

In the segregation of storms, the minimum inter-event time was taken as three hours. That is, a storm event was defined as one where at least three hours without rainfall occurred before and after the event. On this basis, the average total rainfall depth was about 33-mm and the average storm duration was about six hours for the storms selected, Table 2.

TABLE 2. Characteristics of Top-Ranked Actual Storms (Royal Botanical Gardens, Hamilton)

Storm Number	Total Rainfall, mm	Duration hours	Antecedent Dry Weather Period, Days	5-Day Antecedent Precipitation, mm
44	37.8	0.5	8	0.8
2	57.7	10.3	2	46.2
46	31.2	1.5	2	10.9
10	14.2	5.4	6	15.2
25	44.7	4.8	3	4.8
36	20.8	1.0	1	18.8
47	15.3	1.3	1	8.9
20	46.5	6.5	3	19.1
23	22.9	0.6	1	3.0
6	28.7	6.3	6	8.4
1	30.0	9.2	3	16.3
8	30.7	0.7	1	17.5
39	17.0	4.5	3	19.8
54	78.5	18.4	8	0.5
31	27.7	2.4	0	21.3
29	26.4	3.4	10	0.5
37	24.9	1.9	1	13.7
22	32.8	5.6	7	0.3
35	24.4	5.6	2	13.5
11	80.3	19.5	4	5.3
15	26.4	3.8	4	5.8
53	27.2	6.6	5	3.6
17	20.6	9.1	6	0.3
9	25.9	2.9	8	0
32	27.9	5.2	18	0
43	37.3	14.1	2	36.3
26	23.4	6.2	0	18.5
Means:	32.6	5.8	4	11.5

Of interest is the relationship between the antecedent dry-weather period and the antecedent five-day precipitation of these heavy storms. Because the values of these parameters indicated that catchments in the area studied would have been fairly dry at the beginning of heavy storms, neglecting the effects of antecedent precipitation on runoff from the associated storms appeared to be a safe approximation. This observation contradicts to some extent one of the objections to the use of design storms but at the same time removes a possible limitation from the results to be presented.

4.0 RUNOFF SIMULATIONS

Simulations of urban runoff in the Malvern test catchment were done by means of the Stormwater Management Model of U.S. Environmental Protection Agency, Version III, dated September, 1981. The SWMM model has been described in detail elsewhere (2,8) and, consequently, the discussion here is limited to a few important model features.

The SWMM model consists of a number of blocks which can be used in various combinations, depending on the nature of the problem under investigation. The generation of runoff and runoff routing through simple sewer networks without surcharging or special hydraulic structures is accomplished by the RUNOFF block. In more extensive sewer networks with special hydraulic features, but free flow, the sewer flow routing is accomplished by means of the TRANSPORT block. Finally, the sewer flow routing in surcharged systems with special hydraulic structures is accomplished by means of the EXTRAN model. As the sophistication of the routing model increases, so do the computer costs.

Earlier studies (4,5) indicated that, for free flow conditions, satisfactory simulations of runoff in the Malvern catchment can be obtained by using the RUNOFF block only. Consequently, the same approach was adopted here. For pressurized flow conditions, it was desirable to use a dynamic flow routing model and, consequently, the EXTRAN model was used to route inlet hydrographs which had been produced by the RUNOFF block.

4.1 Catchment Discretization

For modelling purposes, the Malvern catchment was subdivided into 20 paired subcatchments. Such discretization followed the earlier work with 10 subcatchments (4,5), which were further subdivided by separating backyards from the rest of the subcatchment area. Such an arrangement was deemed necessary to properly model runoff contributions from backyards. Such contributions should be relatively small (fully pervious area) and delayed because of the long flow route.

The general outline of subcatchment boundaries is shown in Figure 3; basic characteristics of subcatchments are given in Table 3.

Other subcatchment characteristics which were common for all the subcatchments were determined on the basis of earlier studies (4,5) as follows:

Slope: 0.03 - subcatchments with impervious segments
0.02 - fully pervious (backyard) subcatchments

Roughness (Manning's n): 0.013 - impervious segments
0.30 - pervious segments

Depression Storage: 0.5 mm - impervious segments
9.4 mm - pervious segments

Infiltration rates
(Hortons parameters): f_{max} = 127 mm/hr
f_{min} = 13.2 mm/hr
decay rate K = 0.00115 s^{-1}

Additional discussion of the subcatchment parameters follows.

Subcatchment areas were derived from a map of the Malvern catchment and from drainage patterns. it should be recognized that the imperviousness derived from maps contains some uncertainties arising from measurement and sampling errors. Furthermore, the connectivity of impervious elements is not always clear, because some of these elements may drain onto pervious elements and barely contribute to the catchment runoff. It was therefore desirable to verify the catchment imperviousness which was derived from the map against the value obtained from observed volumetric runoff coefficients. Such verification was undertaken for intermediate rainfall runoff events during which all the runoff volume was generated on impervious elements. This condition may be expressed as

$$V_{ru} = A_{imp} (h-d) \tag{1}$$

where V_{ru} is the runoff volume, A_{imp} is the total area of impervious elements, h is the rainfall depth, and d is the depression storage depth. By dividing both sides of Eq. (1) by the catchment area, A, one obtains

$$r = i (h-d) \tag{2}$$

where $r = V_{ru}/A$ is the runoff depth, and $i = A_{imp}/A$ is the catchment imperviousness. Thus by plotting r versus h for a number of events, one obtains a straight line whose slope represents the effective catchment imperviousness. Such a procedure was followed using 9 intermediate events from the 1975 data (see Figure 4). The slope of the regression line was 0.346. Such a value is within the range of values (0.31-0.35) which were determined from the map for the directly connected and total imperviousness, respectively.

The subcatchment width in the SWMM model represents the physical width of the overland flow. In accord with the SWMM manual (2), the widths of individual subcatchments were taken as twice the main sewer pipe length.

Slopes of subcatchments are not particularly important because the simulated runoff peaks are barely sensitive to the slope within practical limits (7). The chosen values were 0.03 and 0.02 for the subcatchments with impervious elements and backyard subcatchments, respectively. Such slopes reflect local slopes (lot grading, road and roof slopes) rather than just the overall catchment slope.

The roughness of subcatchment surfaces was characterized by the Manning's n. The values used here were adopted from the SWMM Model Manual (2).

Depression storage on impervious areas was determined in earlier studies (5). For pervious areas, a value slightly higher (by 3mm) than the earlier SWMM default value (4) was adopted to reflect low surface slopes and possible ponding in backyards.

Finally, the infiltration capacities were selected on the basis of limited field measurements and soil description (sandy loam) as f_{max} = 127 mm/hr and f_{min} = 13.2 mm/hr. It is believed that, for the storms studied, the integrated Horton's infiltration capacity equation used in the SWMM III model makes simulated runoff peaks less sensitive to the choice of Horton's parameters than the earlier used non-integrated form of the same equation.

4.2 Sewer Network

The Malvern sewer network was represented in two ways - in a simplified form adequate for open-channel flow routing in the RUNOFF block and in a comprehensive form which is required for pressurized flow routing in the EXTRAN model. A description of both forms follows.

4.2.1 Sewer network used in the runoff block

For runoff simulations, the Malvern sewer network was approximated by nineteen sewer pipes ranging in diameter from 0.305 m to 0.838 m. Inlets to the sewer system were placed close to the centroids of individual subcatchment areas. Pipes upstream of inlets which had small diameter (.305 m) were neglected. Such a loss of pipe storage volume was compensated by increasing the diameter of four pipes (Nos. 6, 13, 25, and 34) along the route from the inlet to the downstream subcatchment boundary. This was done to avoid sewer surchage which could result from allowing the entire subcatchment outflow to enter through the inlet. In the real system, the subcatchment runoff enters the sewer at a number of points along the pipe and only the sewer section at the downstream subcatchment boundary is designed to convey the entire subcatchment runoff. Thus, the maximum sewer diameter within each subcatchment was extended upstream to the subcatchment inlet.

Apart from the diameter, the sewer pipes were characterized by their length, slope and roughness. A summary of all sewer pipe characteristics is given in Table 4.

4.2.2 Sewer network used in the EXTRAN model

For pressurized flow simulations, the sewer system is described somewhat differently from the description given earlier for flow routing in open channels. Basically, the sewer system is defined as a set of nodes (sewer junction manholes) which are connected by links (sewer pipes).

The sewer system which was used in pressurized flow routing is shown schematically in Figure 5. In total, there are 21 nodal points and 20 connecting links.

Sewer junction data are listed in Table 5 which contains junction invert elevations, ground surface elevations (at the junction), and contributing subcatchment numbers.

Parameters of sewer pipes which were used in pressurized flow routing are given in Table 6. Compared to the earlier described sewer system for the RUNOFF block simulations, a number of modifications has been made and these are described below.

For pressurized flow routing, there was no need to be concerned about possible surcharging between the inlet and the downstream subcatchment boundary. Consequently, the actual sewer diameters were used through the system.

Other modifications were necessitated by numerical solutions which are employed in the EXTRAN model. In particular, the longest conduit should not exceed the shortest one by more than five times (8). It became necessary to shorten the pipe No. 125 by inserting a node and dividing this pipe into two.

In order to establish the computation time step, the time of travel of surface waves through individual conduits was calculated as

$$\Delta t_c = L/\sqrt{gD}$$

The computational time step should then be shorter than t_c's calculated for all conduits. A preliminary calculation indicated that a computational time step of t = 20s would be realistic for all conduits except Nos. 121, 136, and 142. Consequently, these conduits were replaced by their equivalents which are longer, but smoother to maintain the same time of travel. The roughness of the equivalent pipes was calculated from the following formula (8):

$$n_e = n_p \, L_p^{\frac{1}{2}}/L_e^{\frac{1}{2}}$$

where n is the Manning's conduit roughness, L is the pipe length, and the subscripts e and p refer to the equivalent and prototype conduits, respectively. Note that as recommended in the EXTRAN manual (8), the prototype conduit roughness was taken as $n_p = 0.014$.

The finalized conduit lengths and the corresponding times of travel T_c are listed in Table 6. The computational time step was finalized as t = 20 s.

For pressurized flow routing, it was desirable to account for head losses at sewer junctions. Although this cannot be done directly in the EXTRAN model, one can compensate for junction head losses through equivalent conduit roughness.

The junction head loss, h_j, can be expressed as

$$\Delta h_j = K \frac{v^2}{2g}$$

and the conduit friction head loss, h_c, can be expressed as

$$h_c = S_c L$$

where K is the loss coefficient, v is the mean flow velocity, S_c is the friction slope, and L is the conduit length. The total head loss, H, which is attributed to the conduit, can be written as

$$H = h_j + h_c = h_{ec}$$

where h_{ec} is the equivalent pipe loss. After substituting for S_e from the Manning equation, one obtains the following expression for the equivalent conduit roughness n_{ec}.

$$n_{ec} = n_p^{\,2} + 0.008 \frac{KD}{L}$$

where n_p is the prototype roughness, D is the conduit diameter, and both D and L are given in metres. Junction loss coefficients and the equivalent conduit roughness coefficients, which were calculated from the above equation, are listed in Table 6.

Finally, the pipe invert heights above the junction invert are also listed in Table 6.

The EXTRAN model is capable of simulating the behaviour of various special hydraulic structures in the sewer system. The only such structure in the Malvern system was the measuring weir at the outfall. This weir was included in simulations with the EXTRAN model. The basic weir parameters, its height, length and discharge coefficient, were specified as input data for simulations.

4.3 Simulation Procedures

Simulations of runoff from the Malvern catchment were undertaken for the selected Malvern and RBG storms. Such simulations were done with the SWMM III model which was operated in the single event mode. Considering the low antecedent rainfalls for the events studied, no adjustment of model infiltration parameters was deemed necessary and all the simulation runs were done for dry antecedent conditions.

The computational time step was taken as two minutes. Such a time step coincides with the rainfall discretization interval and was used successfully in earlier studies in the Malvern catchment.

Whenever sewer surcharging was detected in simulations with the RUNOFF block, the event was resimulated using both the RUNOFF block and the EXTRAN model, in that case, the RUNOFF block was used to produce inlet hydrographs which were then routed through the sewer system using the EXTRAN model. For flow routing, the time step was 20 s. The maximum number of iterations was selected as 30 and the surcharged flow tolerance was taken as 5%. Both these values are recommended in the EXTRAN manual (8).

5.0 RESULTS AND DISCUSSION OF RESULTS

The simulation results are presented in two parts - first the results for the Malvern events and then the results for the RBG events.

5.1 Malvern Events

Altogether, 13 rainfall/runoff events which had been observed in the Malvern catchment were considered in this study. This total number resulted from the objective to include all the five annual peaks in the analysis (the 1974 peak was ranked the 13th). Runoff hydrographs were simulated for all 13 events, and the simulated peak flows are listed in Table 7 and plotted in Figure 6.

In general, a fairly good agreement between the observed and simulated peaks was obtained. The individual peak flows were then used to produce peak frequency curves for both observations and simulations. The agreement between both frequency curves is better than for individual events (1), because random errors in individual peaks are smoothened out in the curve fitting procedure. Note also that in frequency analysis the ranks of observed and simulated peaks are not identical for individual events.

Initial simulations for the events which produced the three largest peaks indicated surcharging in the sewer system. Consequently, runoff simulations were repeated using the EXTRAN model for the pressurized flow routing. The agreement between the observed and simulated peaks was fairly good and fully comparable to that obtained for less intense events with open channel flow in the sewer system.

5.2 RBG Events

In order to extend runoff simulations to the region of lower frequencies, the RBG storms were also applied to the Malvern catchment. In this case, the top 10 events had to be processed by using the EXTRAN model for pressurized flow routing. The simulated peak flows are listed in Table 7 and plotted in Figure 6. A good agreement between peak flows simulated for the Malvern and RBG events is apparent from Figure 6.

It was of interest to note that the frequency curve for the RBG storms has a more or less constant slope for the whole range of flows from the open-channel flow conditions to pressurized flow conditions. It is expected that for higher return periods (15 years), the major drainage will convey a larger proportion of the total runoff and the minor drainage discharge will be less than indicated by the rainfall input. Under such circumstances, the slope of the frequency curve will be reduced in the region of low frequency flows.

The agreement between the frequency curves which were derived from simulations for the Malvern and RBG storms indicates that peak runoff producing characteristics of storms may be relatively conservative in space. Although both stations are about 10.6 km apart and significant differences between the rainfall data observed at both stations for the same storms are quite apparent, the general properties of the rainfall data seem to be fairly conservative and such properties then control the runoff frequency curves. Such a finding which is so far limited to the data discussed here indicates the feasibility of using a single rainfall record to develop historical or synthetic design storms for the entire municipality, provided that there are no orographic effects.

6.0 SUMMARY AND CONCLUSIONS

Runoff peak frequency curves were produced for an urbanized catchment from five years of observations and from runoff simulations for storms observed in the catchment and at another station 10.6 km away. The simulations were performed by means of the SWMM model and whenever sewer surcharging was encountered, the pressurized flow routing was accomplished by means of the EXTRAN model. The model was calibrated for the test catchment. The most important calibration parameter was the catchment imperviousness which was calibrated through regression analysis of observed rainfall and runoff volumes. Observed runoff hydrographs indicate that in the Malvern catchment, which can be characterized by an intermediate imperviousness and well-drained soils, the pervious areas barely contribute to the generation of runoff peaks with return periods up to around five years.

The study results indicate that with a calibrated model, runoff frequency curves can be derived from runoff simulations, for selected actual storms, with a better accuracy than that typically achieved for individual simulations. This follows from the fact that random errors in individual simulated peak flows are reduced in plotting and curve-fitting procedures.

The SWMM model reproduced the observed runoff peaks fairly well. The agreement between the observed and simulated peaks of return periods from one to five years was comparable to that reported earlier for fairly frequent events. Such an agreement was obtained for dry antecedent conditions which seem to represent the normal antecedent conditions in the study area. These results further confirm the feasibility of using design storms for establishing design runoff flows, provided that the normal antecedent conditions are specified. Note that these findings are limited to the catchments similar to the Malvern catchment. In catchments with low imperviousness and poorly drained soils, the soil infiltration would play a much more significant role.

The EXTRAN model performed satisfactorily in pressure flow routing. For good simulation of losses in the sewer network, head losses at sewer junctions were approximated by increasing the conduit roughness. Finally, it was noticed that the runoff peak frequency curves maintained more or less a constant slope throughout the full range of flows. In particular, no change in slope was observed in the pressurized flow region.

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TABLE 3. Subcatchment Characteristics

Subcatchment Number	Area (ha)	Width (m)	Percent Impervious
1	1.30	853	62.0
2	.98	161	0.0
3	1.59	1042	60.5
4	.93	153	0.0
5	1.25	821	61.8
6	.32	52	0.0
7	1.60	1052	60.9
8	.83	136	0.0
9	1.11	731	65.5
10	1.36	224	0.0
11	.80	523	58.9
12	.56	92	0.0
13	1.87	1225	62.5
14	1.97	323	0.0
15	1.35	888	63.5
16	1.33	218	0.0
17	1.30	853	64.8
18	2.00	328	0.0
19	0.78	513	62.7
20	.08	14	0.0

TABLE 4. **Malvern Sewer Network for Simulating with the RUNOFF Block**

Pipe No.	Diameter [m]	Length [m]	Slope [-]	Manning's n
103	.458	209	.0051	.013
106	.458	91	.0132	.013
107	.458	91	.0132	.013
110	.534	122	.0120	.013
111	.534	57	.0120	.013
113	.458	192	.0050	.013
115	.610	74	.0100	.013
118	.534	107	.0200	.013
119	.686	117	.0120	.013
140	.686	54	.0120	.013
121	.686	49	.0090	.013
142	.763	40	.0050	.013
122	.763	81	.0050	.013
125	.458	341	.0156	.013
130	.686	92	.0024	.013
141	.686	134	.0024	.013
134	.305	89	.0236	.013
135	.686	85	.0042	.013
136	.839	54	.0086	.013

TABLE 5. Junction Data

Junction Number	Invert Elevation [m]	Ground Elevation [m]	Runoff Inflow from Subcatchment Number
1	86.32	88.30	1,2
2	85.28	87.35	-
3	86.67	88.66	3,4
4	84.12	86.44	-
5	85.25	87.82	5,6
6	83.36	86.13	-
7	84.85	86.49	7,8
8	81.93	85.77	-
9	84.06	87.08	9,10
10	80.46	85.10	-
11	81.59	84.85	11,12
12	79.94	84.42	-
13	79.74	83.88	19,20
14	79.10	83.33	-
15	82.97	86.44	13,14
16	80.25	82.96	-
17	80.01	83.39	15,16
18	79.61	83.81	-
19	83.43	86.01	17,18
20	78.64	79.91	-
21	81.72	84.49	-

TABLE 6. Characteristics of Sewer Pipes Used in Pressurized Flow Routing

Pipe No.	D [m]	Length [m]	t_c [s]	Junction Head Loss Coefficient	Manning Coefficient	Invert Height Above Junction [m]	
						Upstream	Downstream
103	.458	209	98.6	1.2	.0146	0	0
106	.305	91	52.6	1.2	.0148	0	0.19
107	.458	91	42.9	1.4	.0155	0	0.11
110	.381	122	63.1	0.2	.0141	0	0.15
111	.534	57	24.9	0.4	.0148	0	0.08
113	.458	192	90.6	2.2	.0151	0	0.53
115	.610	74	30.3	1.2	.0162	0	0.70
118	.534	107	46.7	0.2	.0142	0	0.15
119	.686	117	45.1	1.4	.0146	0	0.06
140	.458	54	25.5	1.2	.0161	0	0.49
121	.686	57	22.0	0.2	.0136	0	0
142	.763	64	23.4	0.2	.0118	0	0
122	.763	81	29.6	1.4	.0171	0	0.24
125	.458	125	59.0	1.6	.0149	0	0.22
126	.458	125	59.0	0.8	.0144	0	0.22
130	.686	92	35.5	0.2	.0144	0	0
141	.686	134	51.7	1.2	.0155	0	0
134	.305	89	51.5	0.2	.0148	0	1.71
135	.686	85	32.8	0.6	.0152	0	0.15
136	.839	62	21.6	0.3	.0141	0	0

TABLE 7. Observed and Simulated Runoff Peaks for the Malvern Catchment

Storm No.	Observed $Q_p[m^3/s]$	Simulated $Q_p[m^3/s]$	Storm No.	Simulated Q_p for RBG Data
12	1.744	1.824	123	2.043
3	1.608	1.892	144	1.944
13	1.218	1.395	125	1.898
31	1.212	1.433	120	1.765
26	1.202	1.481	102	1.694
27	1.099	.798	146	1.646
15	1.079	1.109	108	1.623
4	1.031	.964	110	1.580
19	1.021	.676	139	1.419
17	.993	.813	147	1.402
14	.947	1.096	101	1.375
1	.907	1.064	136	1.368
6	.904	1.015	131	1.300
			106	1.269
			135	1.201
			131	1.177
			129	1.124
			154	1.080
			115	1.014