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ABSTRACT

Runoff hydrographs from nine urban test catchments of widely varying characteristics were simulated for two synthetic design storms as well as for a number of selected actual storms. The frequencies of occurrence of runoff events were determined and, for identical frequencies, the runoff peaks produced by both synthetic storms were compared to the peaks produced by actual storms.

RÉSUMÉ

Des hydrogrammes d'écoulement de neuf bassins hydrographiques urbains d'essai, aux caractéristiques très variées, ont été simulés pour deux averses nominales artificielles, de même que pour un certain nombre d'averses réelles choisies. La fréquence des phénomènes d'écoulement a été déterminée et, à des fréquences identiques, l'écoulement maximal produit par les deux averses artificielles a été comparé à l'écoulement maximal produit par les averses réelles.

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1.0 INTRODUCTION

During the past 15 years, a number of mathematical models have been developed for the calculation of runoff hydrographs from urban catchments. All these models use some form of rainfall data as one of the inputs, and the output obtained from these models is the resulting runoff hydrograph at a selected location. In applications of runoff models to the design of urban drainage, the following two types of rainfall input data are used:

(a) Synthetic design hyetographs

(b) Actual rainfall records (10-20 years long)

A design rainfall hyetograph completely describes the distribution of rainfall intensity during a storm of a known return period. Typically, such a hyetograph is derived by synthesizing a large number of historical rainfall events and serves as input for single-event runoff models.

Actual rainfall records are typically used for continuous runoff simulation which has so far gained a little acceptance in the design of urban draiange. Under special circumstances, continuous simulation can be successfully approximated by a multi-event simulation with single-event models.

In the following discussion, runoff peak flows simulated for synthetic as well as actual rainfall events are compared for a number of catchments which were patterned after some typical urban developments in Southern Ontario. Although the results obtained are only valid for the conditions studied, the comparisons give a general indication of the relationship between the synthetic and actual storms and demonstrate some shortcomings of the approach based on the design rainfall hyetograph. The analysis is restricted to runoff peak flows on small and intermediate catchments (less than 130 ha).

2.0 SYNTHETIC DESIGN STORMS

Traditionally, the design of urban drainage has been based on the design event concept which is well accepted by the engineering profession. The design event, a rainstorm, is characterized either by a block rainfall or, more recently, by a rainfall hyetograph. Such hyetographs are typically derived by synthesis and generalization of a large number of actual events. A probable frequency of occurrence of these synthetic design storms is estimated, and the runoff calculation proceeds under the assumption that the frequencies of occurrence of the design storm and of the calculated runoff peak are identical.

The concept of design storms and its application in urban drainage design is subject to considerable criticism. In particular, the attempts to assign mean frequencies of probable occurrence to storms of various intensities and durations are criticized, and the assumption of the identical frequencies of occurrence of the rainfall and runoff events is questioned because of the statistical non-homogeneity of rainfall and runoff data (McPherson, 1975). Although such criticism seems to be generally justified, the shortcomings of the design storm concept have never been demonstrated on actual rainfall data, or in conjunction with runoff calculations. Such an evaluation of the design storm concept was attempted in the following analysis which was limited to two typical examples of design storms, the Chicago storm and the storm proposed in the Manual of the Illinois State Water Survey (ISWS storm). The Chicago storm was selected because of its wide acceptance in the Canadian engineering practice. The ISWS storm was selected because it is closely based on the actually observed storms.

2.1 Chicago Design Storm

One of the first design storms, the Chicago storm, was recommended for the design of urban drainage more than 20 years ago (Keifer and Chu, 1957). Additional information on this storm was recently presented by Bandyopadhyay (1972), and Preul and Papadakis (1973). The Chicago storm has become fairly widespread in the North American practice, partly because the Chicago storm hyetograph can be easily derived from the existing rainfall intensity-duration-frequency curves, and partly because of the lack of other approaches. In recent years, several Canadian municipalities have adopted this type of a design storm in their design criteria for urban drainage.

In relation to the actual events, the Chicago design storm preserves the maximum volumes of water falling within the specified durations, the average amount of rainfall antecedent to the peak intensity, and the relative timing of the intensity peak.

To develop the Chicago storm hyetograph, one needs first to determine the dimensionless time of the peak intensity. This time, t_r, divides the hyetograph into two parts and is defined as:

$$t_r = t_p/T \tag{1}$$

where t_p is the time to the peak intensity measured from the beginning of the storm, and T is the total storm duration. Values of t_r are determined for a number of historical storms and a mean value is adopted for the design hyetograph. Both parts of the hyetograph, before and after the peak intensity, are derived from the rainfall intensity-duration-frequency curves expressed as:

$$i_{av} = \frac{a}{t_d^b + c} \tag{2}$$

where i_{av} is the average rainfall intensity over the duration t_d, and a, b, c are constants determined by fitting the above function to the observations. The total storm duration is typically selected from one to six hours; however, this duration does not affect the magnitude of the peak rainfall intensity of the storm, or the dimensionless time to peak.

The Chicago-type rainfall hyetographs of various return periods were derived for the area of interest by M. M. Dillon Ltd. (1977) from a 15-year rainfall record available for the station at the Royal Botanical Gardens in Hamilton. These hyetographs were adopted here and one of them is shown in Figure 1 as an example.

2.2 Illinois State Water Survey Storm

The Illinois State Water Survey (ISWS) developed a procedure for deriving a synthetic storm for the design of urban drainage (Terstriep and Stall, 1974). In this procedure, the maximum hourly rainfall depths are derived, from local data or the rainfall intensity-duration-frequency curves, for various return periods. These rainfall depths are then distributed in time following the technique used by Huff (1967) to analyze heavy rainstorms in Illinois. Actual storms are first divided into a number of groups according to the relative timing of the peak intensity. For the largest group, the distributions of rainfall in time are determined, and the median distribution is adopted for the design storm.

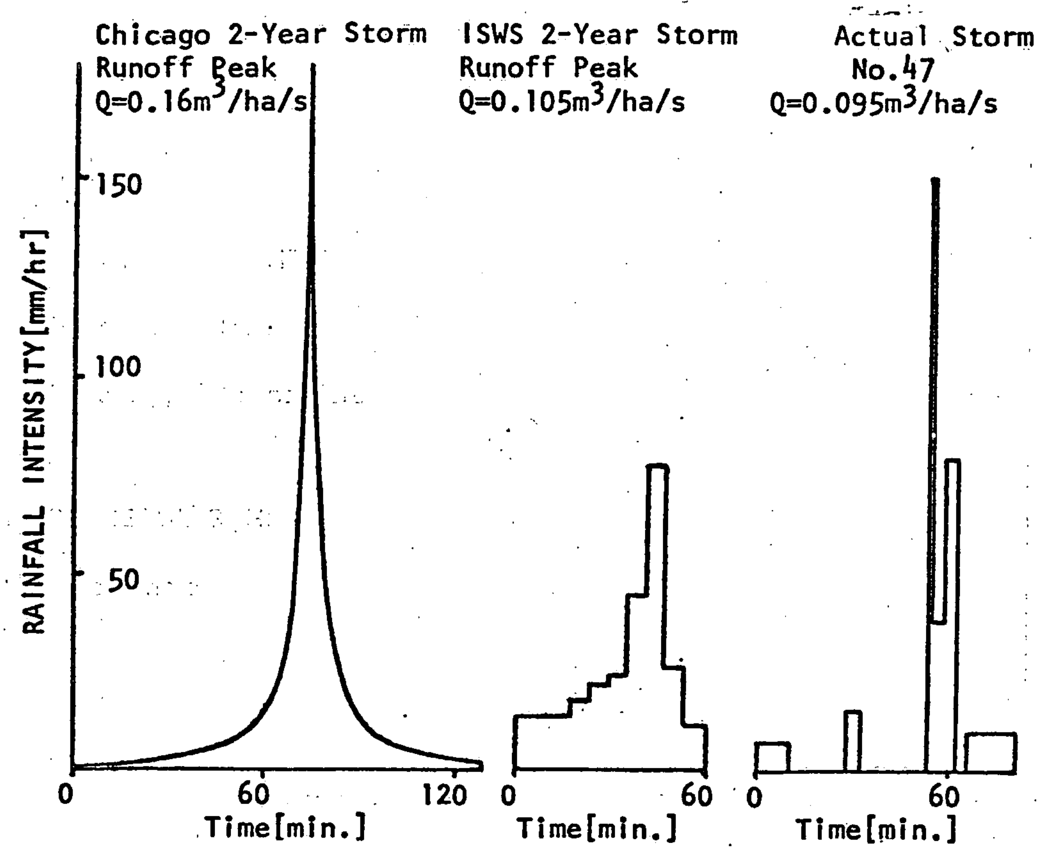

Fig.1. Actual and Synthetic Storm Hyetographs

For the rainfall record available, the maximum hourly rainfall depths were taken directly from the intensity-duration curves prepared by M. M. Dillon Ltd. (1977) for the return periods of 1, 2, 5, and 10-years (see Table 1).

Table 1. Maximum Hourly Rainfalls of Various Return Periods (Royal Botanical Gardens, Hamilton)

Return period [years]	1	2	5	20
Maximum hourly rainfall [mm]	22.1	26.0	33.0	38.6

To determine the temporal rainfall distribution, about 30 heavy actual storms, which are further described in the next section, were divided into three groups according to the part of the storm in which the peak intensity burst had occurred. The majority of storms had their peak intensity occurring in the last third of the storm duration. A median rainfall distribution was determined for this group and expressed as:

$$R_{cp} = f(T_{cp}) \tag{3}$$

where R_{cp} is the cumulative percent of rainfall and T_{cp} is the cumulative percent of storm time, and f is an empirical function. The numerical values of this distribution, which was adopted for the design hyetograph, are shown in Table 2.

Table 2. Median Rainfall Distribution of Predominant Storms

Cumulative % of storm time $-T_{cp}$	0	10	20	30	40	50	60	70	80	90	100
Cumulative % of rainfall $-R_{cp}$	0	5	10	15	22	30	39	56	86	96	100

An example of the ISWS design hyetograph with a two-year return period is shown in Figure 1.

3.0 ACTUAL STORMS

As an alternative to the use of synthetic design storms, several authors (Linsley and Crawford, 1974; McPherson, 1975) proposed to transform an actual rainfall record into a runoff record which could directly serve for the selection of design runoff flows. Typically, rainfall records are transformed into runoff records by means of continuous simulation models. Although this approach avoids the shortcomings of synthetic design storms, it has not gained much acceptance so far. Continuous simulation may prove expensive when sophisticated models are used, or inaccurate in the case of simplistic models. However, certain types of urban runoff problems, particularly those related to water quality, cannot be effectively analyzed by any tool other than continuous simulation.

In the design of urban drainage, most projects deal only with runoff quantities, and then for typical catchments with no runoff controls, continuous simulation may be approximated by a series of single-event simulations. Such simulations were performed in this study for the selected actual storms which were likely to cause high runoff peak flows on urban catchments. Whenever necessary, the antecedent conditions were taken into account by adjusting the parameters of the runoff model.

3.1 Selection of Actual Events

To select the actual storms which were likely to produce high runoff peak flows, the rainfall record was screened to identify all the storms having either the total rainfall depth larger than 1.25 cm or a ten-minute intensity larger than 1.5 cm/hr. In total, 54 storms meeting the selection criteria were found. Subsequently, the storms were ranked, according to their maximum 5, 10, 15, 30 and 60-minute rainfall intensities, to identify the top 20 storms for each duration. Because many storms were ranked among the top 20 storms in several categories, this selection process yielded only 27 storms meeting all the selection criteria. For the purpose of establishing the frequency of occurrence of runoff peaks on the catchments studied, these 27 storms effectively replace the 15-year rainfall record. The basic characteristics of the top 15 selected storms are summarized in Table 3.

Table 3. Characteristics of Top-Ranked Actual Storms

Number	Storm Number	Total Rainfall [mm]	Duration [hr]	Antecedent Dry Weather Period [days]	5-Day Antecedent Precipitation Index [mm]
1	44	37.8	0.5	8	0.5
2	2	57.7	10.3	2	11.5
3	46	31.2	1.5	2	4.3
4	10	14.2	5.4	6	10.8
5	25	44.7	4.8	3	1.5
6	36	20.8	1.0	1	7.5
7	47	15.3	1.3	1	6.3
8	20	46.5	6.5	3	4.3
9	23	22.9	0.6	1	2.2
10	6	28.7	6.3	6	0.4
11	1	30.0	9.2	3	3.5
12	8	30.7	0.7	1	10.5
13	39	17.0	4.5	3	2.4
14	54	78.5	18.4	8	0.4
15	31	27.7	2.4	0	11.8

A few observations regarding these storms are of interest. On average, the total rainfall depth was about 34 mm and the storm duration was five hours. Both these values are, however, affected by the definition of a storm event, i.e., the minimum inter-event time which separates the individual events. The minimum inter-event time was taken here as three hours.

The relationship between the antecedent dry weather period and the antecedent five-day precipitation of these heavy storms is rather interesting. Low observed values of these parameters indicate that catchments in the area studied are fairly dry at the beginning of heavy storms and that the effects of antecedent precipitation on runoff from design storms may be neglected. This somewhat contradicts the general criticism of design storms presented earlier.

4.0 RUNOFF SIMULATIONS

The analysis of rainfall data is only a preparatory step in drainage design, because eventually the designer needs to know the frequency of occurrence of runoff flows of various magnitude. Therefore, the rainfall data described in the previous two sections were transformed into runoff flows by means of hydrologic synthesis. Towards this end, the Storm Water Management Model (SWMM) of U.S. Environmental Protection Agency was used. SWMM is a single-event model which was specifically designed for simulation of urban runoff. A detailed description of the model was presented elsewhere (U.S. Environmental Protection Agency, 1971). The values of the SWMM hydrologic parameters were adopted from the runoff simulation studies undertaken for a test catchment in Burlington, Ontario (Marsalek, 1977).

Physical catchment parameters strongly influence runoff simulations and can to some extent influence the selection of rainfall input. Runoff flows were, therefore, simulated for a series of nine hypothetical catchments of widely varying characteristics. These catchments were patterned after some typical urban catchments in modern residential developments inOntario. Three catchment sizes were used; 26 ha, 52 ha, and 130 ha. In all three cases, the drainage density was maintained about the same. The catchment imperviousness was varied in three steps; 15%, 30%, and 45%. The last two values are typical for modern residential areas in Ontario.

Two types of rainfall inputs were used in runoff simulations for all the catchments. Firstly, runoff flows were simulated for two synthetic design storms, the Chicago and ISWS storms, of various frequencies of occurrence. The frequencies of the runoff peaks produced by these storms were assumed to be identifical to the frequencies attributed to the design storms.

Secondly, runoff flows were simulated for the selected actual storms. The frequencies of occurrence of the simulated runoff peaks had to be determined by frequency analysis. Towards this end, the peak flows were ranked and their recurrence intervals calculated from the Weibull plotting-position formula (Chow, 1964) as follows:

$$T = (N + 1)/m$$

where N is the number of items, m is the order of the items arranged in descending magnitude (thus m=1 for the largest item), and T is the recurrence interval (T=1/P, where P is the probability). Note that the choice of a plotting-position formula was not very important because only the middle section of the distribution, where all plotting-position formulas give practically the same results, was of a particular interest.

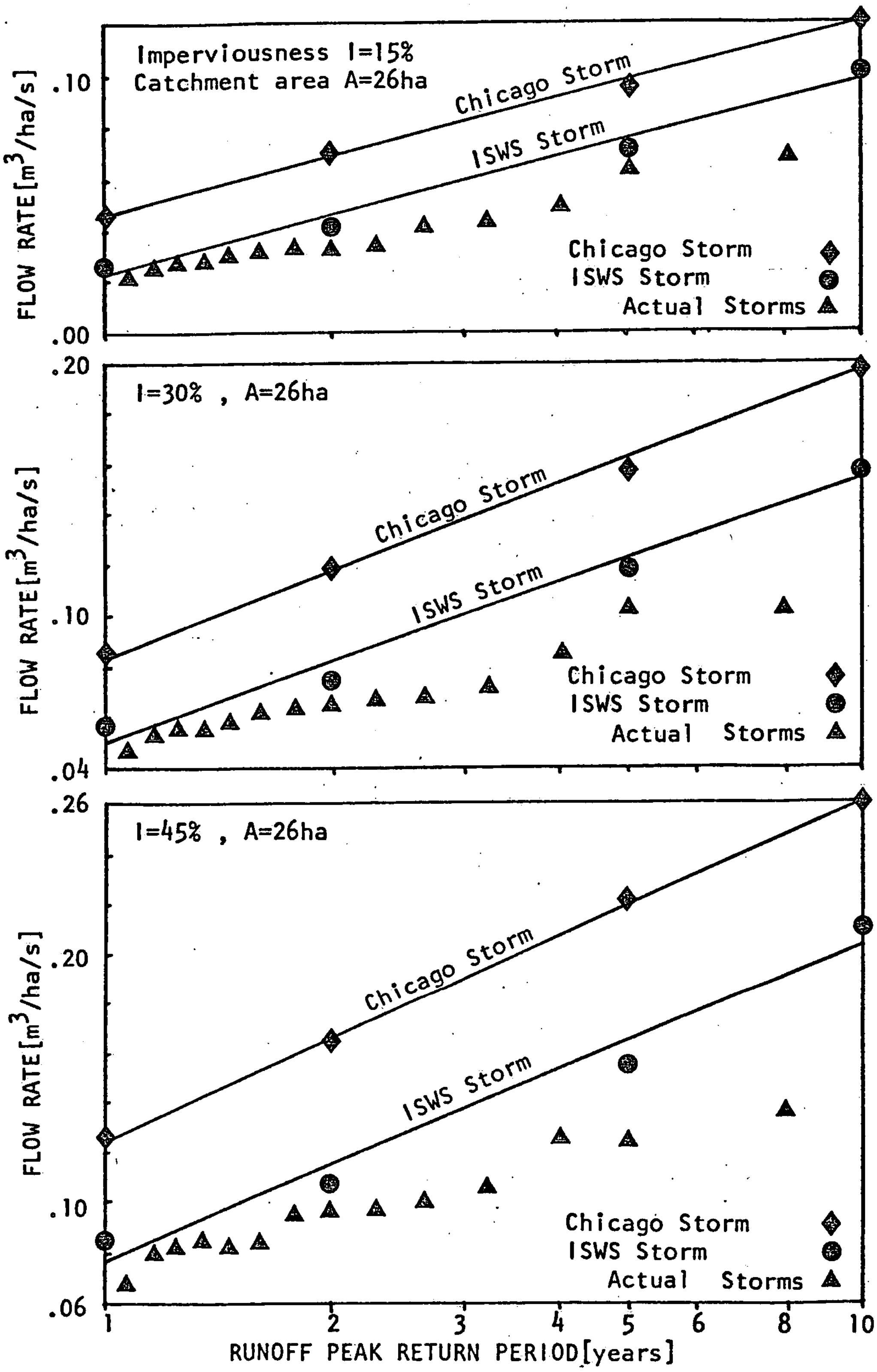

Fig.2. Return Periods of Runoff Peak Flows

5.0 RESULTS AND DISCUSSION

Return periods of runoff flows simulated for various actual and synthetic storms were plotted in Figure 2 for the smallest catchment studied and for three values of the catchment imperviousness. From this graph, one can readily compare the results obtained for the two synthetic design storms and the actual storms.

When studying the effect of the catchment size, the peak flows per unit area were found to be attenuated with an increasing area. This peak attenuation was fairly consistent and represented about a 13% reduction when comparing the smallest (26 ha) and the largest (130 ha) catchments of otherwise identical characteristics. It is conceivable that even larger differences could be encountered in the practice, depending on the relation of the concentration times of the catchments studied.

The comparison of runoff peaks simulated for actual and synthetic storms yielded interesting results. For all return periods, both design storms produced flows larger than those produced by the actual storms of corresponding return periods. This overestimation was particularly large for the Chicago storm which produced peak flows from all the catchments about 80% larger than those produced by the corresponding actual storms. Some explanation of this overestimation was offered by Marsalek (1977) who pointed out the following shortcomings of the Chicago storm:

(1) All the maximum rainfall intensities which were observed for the specified durations during a number of actual storms, are attributed to a single design storm.

(2) The intensity-duration-frequency curves are extrapolated into extremely short intervals, thus yielding peak rainfall intensities exceeding the five-minute intensity by up to 60%.

(3) The description of the time of the peak intensity by a single t_r-value, which is an average of all the t_r-values observed for selected storms, is questionable in view of the probabilistic nature of this parameter. Large samples may be required to obtain a good estimate of t_r (Chen, 1975).

The ISWS storm produced better results than the Chicago storm. The peaks simulated for the ISWS storm were only slighltly (27%) larger than those simulated for the corresponding actual storms. There is, however, some degree of arbitrariness in the definition of this storm, particularly in the choice of the storm duration which affects the magnitude of rainfall intensities. The ISWS storm duration of one hour was recommended on the basis of some runoff simulations done with the ILLUDAS model for several urban catchments (Terstriep and Stall, 1974). The highest runoff peaks were obtained for the one-hour storm. Similar tests were done with the SWMM model for the rainfall data and catchments studied here. By reducing the ISWS storm duration from one to 0.5 hours, the runoff peaks increased by about one-third. For the five-hour storm duration, the simulated runoff peaks were much smaller than those produced by the one-hour storm. Note also that the actual storms analyzed here do not lend any support to the assumed duration of the ISWS design storm of one hour. Consequently, the relatively good performance of the ISWS storm reported here may be incidental, and the choice of durations of this storm should be further examined.

It is evident from the comparisons of runoff peaks simulated for various types of rainfall input that much more attention should be paid to the rainfall input than in the past. The synthetic design storms produced different results and these in turn differed from the results obtained for the actual storms. The uncertainty in simulated runoff peaks which was caused by the choice of a rainfall input appeared to be larger than the uncertainty inherent to the simulation process.

The actual storms used for runoff simulations were selected on the basis of peak intensities for duration of 5, 10, 15, 30 and 60 minutes. It is of interest to examine the efficiency of this selection process. For this purpose, the correlation between the ranks of peak intensities and run-off peaks was examined, for the individual durations, by means of the Spearman rank correlation coefficient. When considering all 27 storms, the values of the coefficient were larger than 0.545 which indicated a rank correlation significant at a 0.01 level of confidence (Siegel, 1956). The peak intensities appeared to provide a good selection criterion for the identification of important historical storms.

When attempting to directly correlate the simulated runoff peak flows and the peak intensities of actual storms, the highest values of the correlation coefficient varied from 0.629 to 0.734. This means that only 40 to 50% of the linear variation in the runoff peaks could be explained by the linear variation in the rainfall intensity. Evidently, not only the storm peak intensity but also other parameters of the rainfall distribution are important for the generation of runoff peak flows.

Though the results presented here are only valid for the conditions studied, the proposed methodology for the selection of actual storms and the establishment of frequency graphs of runoff flows may have a general applicability and will be tested for other areas. The graphs of runoff flow frequencies, analogous to those shown in Figures 2 and 3, could be used for quick estimates of runoff peaks from new urban developments or for checking design values.

Finally, the analysis presented did not consider the effects of storage reservoirs in the drainage system on runoff peaks. Such a less frequent case was analyzed previously and it was shown that storage effectively transposes the runoff flow frequency curve in the direction of smaller flow rates (Marsalek, 1977).

6.0 CONCLUSIONS

The comparison of runoff peaks simulated for two types of synthetic design storms and actual storms of identical nominal return periods produced widely varying results. The Chicago storm produced runoff flows 80% larger than those produced by the actual storms of corresponding return periods. Similarly, the use of the ISWS storm resulted in runoff flows about 27% larger than those simulated for the corresponding actual storms. The recommended duration of the ISWS storm of one hour, which affected the simulated peaks significantly, appears to be somewhat arbitrarily selected.

To establish the frequency of occurrence of runoff peak flows, continuous runoff simulation was approximated by a series of single-event simulations for 27 selected actual storms. The selection of these storms, which effectively replaced a 15-year rainfall record, was based on the ranking of storms according to their peak rainfall intensities for several durations.

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