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TEMPORAL DISTRIBUTION OF DESIGN STORM RAINFALL

by

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ABSTRACT

Basic characteristics of urban design storms are presented and discussed. One of these characteristics, the temporal distribution, is further examined by demonstrating its effect on simulated runoff peaks and by comparing various distributions found in the literature. General comparisons of temporal distributions indicate large differences among various distributions. Further comparative studies and exchange of experience are recommended.

SOMMAIRE

L'auteur présente et étudie dans sa communication les caractéristiques fondamentales que présentent les averses nominales des zones urbaines. Il examine de près l'une de ces caractéristiques, la répartition temporelle des pluies, en démontrant l'effet de cette répartition sur des ruissellements de pointe simulés et en comparant diverses répartitions signalées dans la documentation. Une comparaison générale des répartitions temporelles fait ressortir de grandes différences parmi diverses répartitions. L'auteur recommande d'autres études comparatives et d'autres échanges d'expériences.

MANAGEMENT PERSPECTIVE

The design of urban drainage structures such as sewers and gutters is mostly based on the use of design storms, i.e., a specified precipitation input. This paper outlines the various characteristics of design storms and examines one of these characteristics, the variation of precipitation intensity over the storm duration.

It is shown that various methods recommended in the literature for developing the precipitation distributions produce widely different results.

This paper is of interest to all who are concerned with drainage design and urban runoff modelling.

T. Milne Dick
Chief, Hydraulics Division

PERSPECTIVE-GESTION

La conception des installations de drainage urbain telles que les égouts et les caniveaux repose surtout sur l'observation des averses nominales, c'est-à-dire celle des quantités données de précipitation. La présente communication donne un apeçu des diverses caractéristiques des averses nominales et examine l'une de ces caractéristiques: la variation de l'intensité d'une précipitation au cours d'une averse.

La communication démontre que les diverses méthodes recommandées dans la documentation, qui servent à établir la répartition des précipitations, produisent des résultats sensibilement différents les uns par rapport aux autres.

La communication intéressera tous ceux qui s'occupent de conception d'installations de drainage et de construction de modèles de ruissellement urbain.

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TEMPORAL DISTRIBUTION OF DESIGN STORM RAINFALL

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ABSTRACT

Basic characteristics of urban design storms are presented and discussed. One of these characteristics, the temporal rainfall distribution , is further examined by demonstrating its effect on simulated runoff peaks and by comparing various distributions found in the literature. General comparisons of temporal distributions indicate large differences among various distributions. Further comparative studies and exchange of experience are recommended.

KEYWORDS

Design storms; temporal rainfall distribution; runoff computations; hydrological design; urban drainage.

INTRODUCTION

The arbitrary specification of a precipitation input for design calculations is well established as a criterion for the design of hydrotechnical structures. Examples of such inputs include the probable maximum precipitation in the U.S.A. and Hurricane Hazel or the Timmins rain in Ontario, Canada. The advent of stormwater modelling has brought about a profusion of so-called urban design storms, but to date none has received the same acceptance as the earlier mentioned examples.

The subject of urban design storms is somewhat controversial. Much of this controversy arises from the lack of accurate and complete definition of design storms and from confusion regarding the area of application. Consequently, the design storm concept is sometimes misused. Under such circumstances one may ask the question whether the design storm concept should be employed at all. The justifications for using design storms are as follows(Marsalek and Watt, 1983).

(i) Frequency analysis of actual flow records is not feasible in urban drainage projects because such records are generally not available. Also predevelopment flows are irrelevant to the postdevelopment conditions. Thus, it is not possible to select from the actual records design flows on the basis of either economic efficiency, specified risk and service period, or specified return period.

(ii) The cost of many installations does not justify extensive simulations of synthetic flow records and further frequency analysis as described earlier. Furthermore, the reliability of the synthetic records is often questioned because of the inherent need to calibrate the simulation model and the lack of calibration data for the design event return periods.

(iii) From the administrative point of view, the perception of an uniform level of protection seems to be an equitable objective preferable to economic efficiency with the concomitant variations in the level of protection. Thus for the sake of uniformity, an input and the method of computation are specified in urban drainage design.

(iv) The arbitrary specification of a design storm event in conjunction with the rational method is well accepted in practice and will continue to be used by practitioners.

(v) The specification of design storms requires minimal resources in terms of money and time.

(vi) The specification of a design storm minimizes misunderstandings on the parts of both the client and the approval agency.

In summary, a large portion of urban drainage structures has been and will continue to be designed on the basis of the design storm concept. For these purposes, the 'best' design storm should be selected. At the same time, however, there is a significant number of cases where the design storm concept is not appropriate at all and alternative approaches should be developed. A discussion of such approaches is beyond the scope of this paper.

DESIGN STORM APPLICATION CONSIDERATIONS

Design storms are most frequently applied to the calculation of runoff peak flows for the sizing of transport elements, such as sewers and gutters. This is perhaps the most appropriate application of design storms provided that the runoff generation in the catchment is primarily controlled by the impervious areas. Other applications of conventional design storms to such problems as storage design, or quality control and treatment are outside of the acceptable practice and should be avoided. Alternative approaches are required in such cases.

The application of conventional design storms is affected by catchment characteristics including the catchment size, imperviousness and suitability for runoff control by storage. Design storms are best applicable to small catchments of significant imperviousness and without runoff controls. Note that under such circumstances, the rainfall spatial distribution is not important and the runoff peak generation, which is controlled by impervious areas, is not significantly affected by antecedent conditions. Departures from the above catchment characteristics reduce the applicability of conventional design storms.

Finally, the selection of a design storm is obviously tied to the computational procedure and vice-versa. Where the rational method is applicable, the appropriate design storm is a uniform intensity rainfall of duration sufficient to reach equilibrium runoff conditions. Other cases will require the use of runoff simulation models which in turn require design storms of variable intensity derived from local data.

Following the preliminary considerations, the designer has established whether the design storm is applicable to the problem at hand and what general type of a design storm is required. The next step is to determine the storm characteristics.

DESIGN STORM CHARACTERISTICS

In a general case, a design storm is defined by its return period, total rainfall depth, temporal distribution of rainfall at a point, spatial characteristics (given by the storm movement and development or decay), and by some indication of the antecedent rainfall. The relative importance of each of these characteristics varies with a particular application of the storm and with the catchment characteristics. A brief description of individual storm characteristics follows.

Design Return Period

Ideally, a design return period should be selected on the basis of economic efficiency, i.e. to minimize total costs - investment plus damage, so as to yield an optimal design. Such considerations are included e.g. in the Wallingford Storm Sewer Design and Analysis Package (Price, 1981) and an algorithm for minimizing the sewer construction costs was earlier developed by Tang, Mays and Yen (1975). But as noted by Watt and Marsalek(1977), in urban drainage practice, the concept of optimal design is often replaced by a concept of a "level of protection', which generally applies to the exceedance probability of some rainfall event, rather than to the probability of failure of the hydraulic structure. In the Canadian practice, the return periods from 2 to 10 years are used in the minor drainage design and longer periods, up to 100 years, are used in the major drainage design.

Total Precipitation Depth

The total precipitation depth at a point is a function of the storm return period and duration. Generally, the estimates of the total precipitation depth can be obtained from the depth-duration-frequency maps which are prepared by meteorological agencies.

Storm Duration

The storm duration is an important factor which defines the storm rainfall depth for a given return period and affects the storm intensity and the resulting peak flows. The selection of the design storm duration depends on the catchment time constant which has been traditionally defined as the time of concentration. There is a tendency in the present practice to select the design storm duration as the time of concentration or longer.

Temporal Rainfall Distribution

The variation of precipitation intensity over the duration of the storm is an important factor in determining the timing and the magnitude of the runoff peak flow. A realistic estimate of temporal distribution can be obtained only from analysis of precipitation data from the recording gauge network. In Canada, the maximum daily intensity is reported for discrete intervals from five minutes to 24 hours. Therefore, five minutes appears to be a suitable minimum interval that can be used for the discrete representation of design storms unless the original records are reanalyzed. The peak intensity and the time of occurrence of this peak are then defined in terms of this interval.

Storm Spatial Characteristics

These characteristics are important only for larger drainage basins. The average basin precipitation may be significantly lower than the point value because of the limited extent of storm cells (James and Drake, 1980). In this case, data must be obtained from the precipitation gauge network to estimate the ratio of basin average to point values usually as an average for the storm. In addition, for these larger basins, storm movement and storm development and decay may also affect runoff both in terms of magnitude and timing. These considerations are particularly relevant to the case of operation and/or control of large systems.

In the earlier recommended applications of design storms to small urban catchments of significant imperviousness, the storm spatial characteristics may be neglected and the storm description is then reduced to the following characteristics: return period, duration, total rainfall depth, and temporal distribution. Among these characteristics, the temporal rainfall distribution is perhaps the most difficult one to establish. The significance of the temporal rainfall distribution is discussed in the following section.

EFFECTS OF TEMPORAL RAINFALL DISTRIBUTION ON SIMULATED RUNOFF PEAKS

The temporal rainfall distribution is usually expressed as the rainfall intensity distribution in time (a hyetograph) and, for a given rainfall depth and duration, it may be further characterized by the peak intensity and its timing. Both these parameters, which affect the simulated runoff peak, are defined with reference to the discretization interval of the event.

The sensitivity of the simulated runoff peak to the peak intensity is quite obvious and can be demonstrated by the equilibrium runoff model which states that the peak flow is directly proportional to the constant intensity of appropriately long duration. In the case of variable intensity, the runoff peak also increases with the increasing intensity, but the magnitude of such increases is affected by the catchment response. Note also that as the hyetograph is discretized, the choice of the discretization interval affects intensities in individual intervals. Thus the selection of the discretization interval may also affect the simulated runoff peaks.

Some data demonstrating the effects of the rainfall discretization interval on simulated runoff peaks are shown in Fig.1 below.

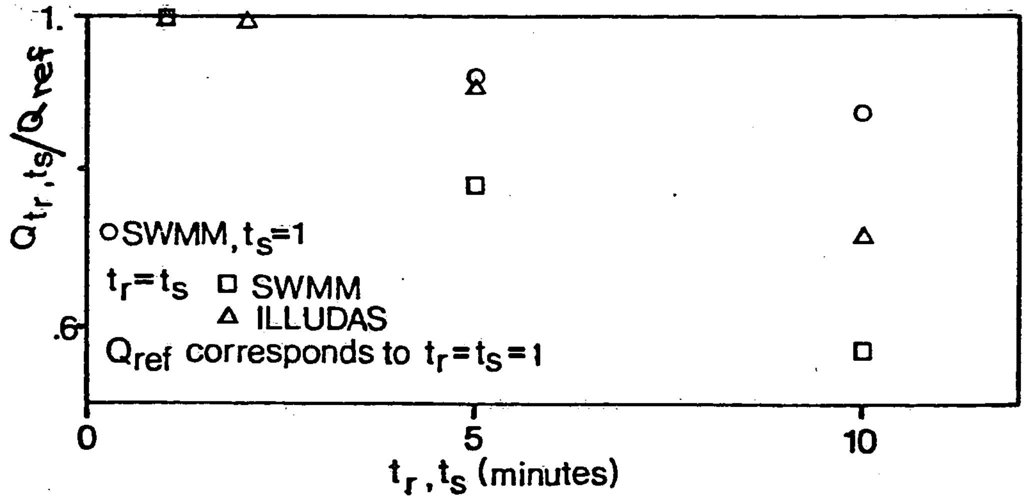

Fig.1. Effect of rainfall interval t_r and computational time step t_s on simulated runoff peaks Q_{t_r, t_s}

The data in Fig.1 were produced for a small catchment (area= 23 ha, 30% impervious) with a relatively fast response (the catchment time constant ≐ 15 min.) and a 5-year design storm derived from the intensity-duration-frequency (IDF) curves. It can be inferred from Fig.1 that the simulated peaks are not very sensitive to the rainfall discretization interval as long as a short computational time step is used. In some models, however, the time step has to be identical to the rainfall interval and the combined effect of increasing simultaneously both the rainfall interval and the time step is rather pronounced. As shown in Fig.1, the runoff peaks corresponding to the longer interval and time step of 10 minutes may amount to only 60% of those obtained for the short interval and time step of 1-minute.

Besides the peak intensity, the distribution of rainfall in relation to the temporal distribution of rainfall abstractions also affects the simulated runoff peaks. Advanced rainfall patterns with the peak intensity occurring in the early part of the storm generally produce lower peaks than delayed patterns of equal intensities. This has been demonstrated in some earlier studies (Marsalek, 1978; Patry and Raymond, 1979; Wenzel and Voorhees, 1981) and pertinent data are shown in Fig.2 .

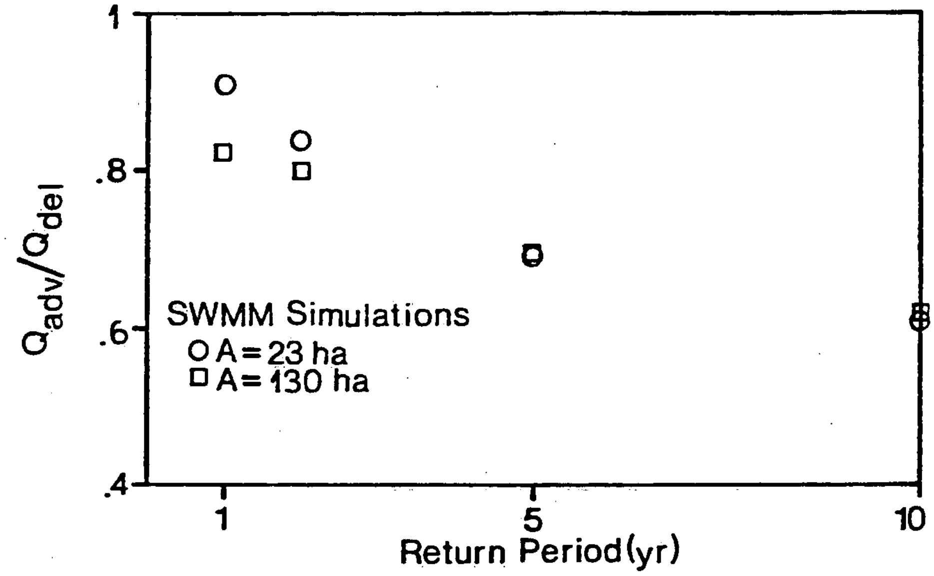

Fig.2. Relationship between runoff peaks simulated for advanced and delayed hyetographs (Q_{adv} and Q_{del}, respectively)

Thus in order to obtain good simulation results, the input hyetograph has to be derived from local data, has to contain intensities of the correct magnitude, these intensities have to be correctly distributed in time, and the hyetograph has to be discretized into fairly short time intervals and applied in conjunction with a properly selected computational time step. Various approaches to the development of design storm hyetographs are discussed in the next section.

SURVEY OF DESIGN STORM TEMPORAL DISTRIBUTIONS

A literature survey produced a number of methodologies for the development of design storm temporal distributions. The listing of such methodologies below is limited to those procedures which have been well publicized and used in comparative studies. For a more detailed listing, the reader is referred to the design storm bibliography which was prepared by Wenzel and colleagues (1983).

Hyetographs Derived From the IDF Curves

One of the first design hyetographs was proposed by Keifer and Chu (1957). This hyetograph, which is also referred to as the Chicago storm, is defined by the chronological location of the peak intensity (i.e. the parameter r, which is sometimes called the distribution skewness) and by the intensity distribution adopted from the IDF curves. Although this hyetograph can be easily derived, it represents one of the least satisfactory distributions because of large variations in the skewness of actual storm distributions and because of the incorrect assumption that all the points on the IDF curve may be assigned to a single storm.

A similar hyetograph, referred to as a composite method hyetograph, can be obtained by adopting incremental intensities from the IDF curves and arranging them in any order (Wenzel, 1982). Examples of two hyetographs derived from the IDF curves are shown in Fig.3 below.

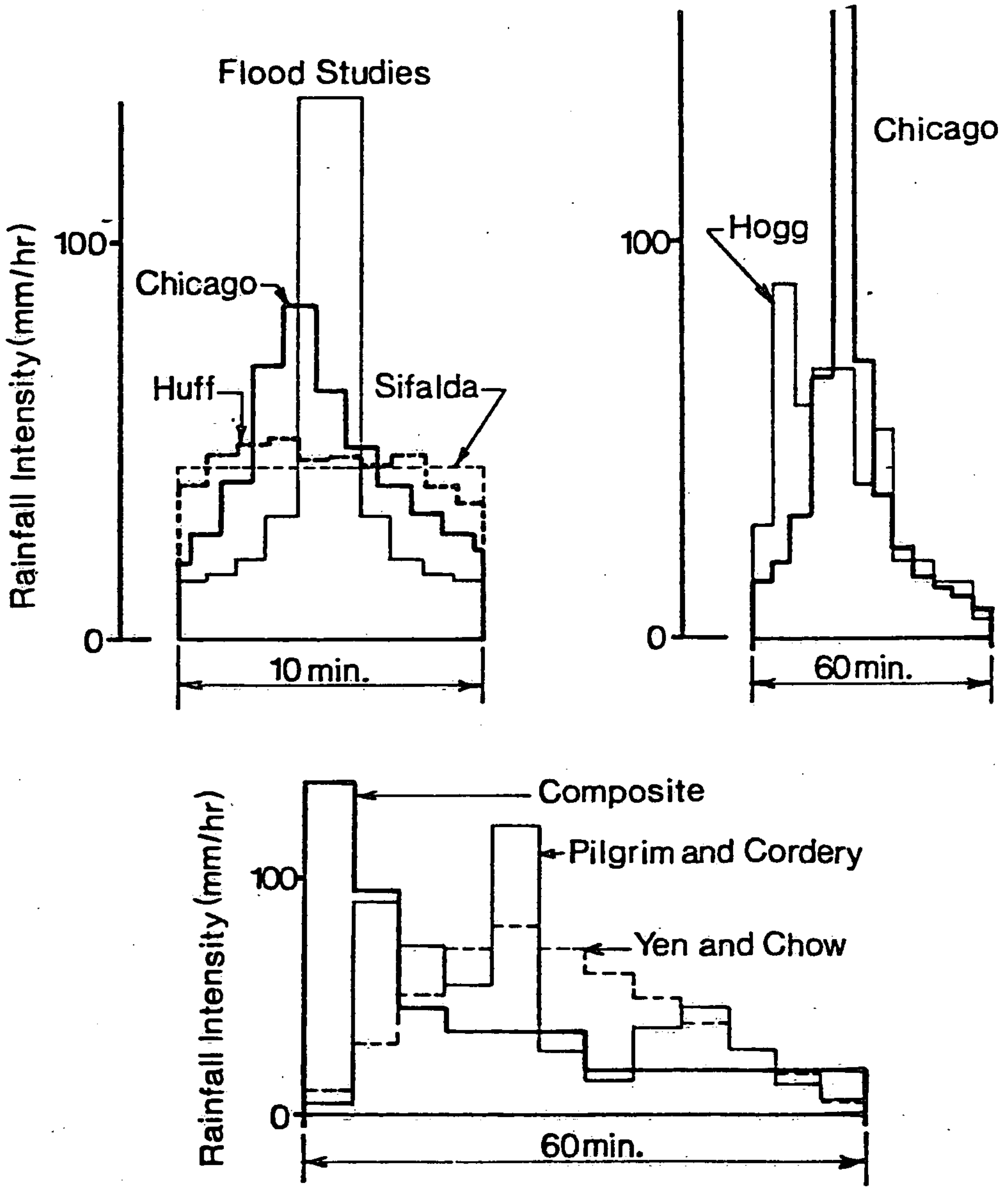

Fig.3. Design storm hyetographs (adopted from Arnell, 1982; Marsalek and Watt, 1983; Wenzel, 1982)

Flood Studies Report Hyetograph

The Flood Studies Report design storm (Natural Environment Research Council, 1975) is characterized by a series of symmetrical temporal patterns which are described by the probability of the hyetograph sharpness (referred to as the peakedness). For general applications, the distribution characterized by the 50 percentile of profile sharpness (i.e., 50% of all rainfall profiles are less sharp) is recommended. This profile is shown in Fig.3. Transferability to other climates may be infeasible because of the profile symmetry and the profile sharpness distributions derived from local data.

Normalized Rainfall Mass Curves

This approach was introduced by Huff (1967) who divided severe storms into four groups according to the timing of the peak intensity and, for each group, produced normalized curves of the rainfall mass vs. the lapsed time for various probabilities. For practical applications in Illinois, Terstriep and Stall (1974) recommended to use an advanced pattern and the 50 percentile curve.

Difficulties with the application of normalized curves arise when selecting the storm duration which affects the peak intensity. To alleviate such problems, Hogg (1980) produced normalized distributions for various Canadian climates for two fixed durations - 1 and 12 hours. Such durations were selected to provide samples of both convective shower events as well as synoptic scale cyclonic circulation events. The 30% probability curve was recommended for design. This means that only 30% of all data, plotted as the accumulated rainfall depth vs. the lapsed time, were above this curve. The need to recommend a more time-advanced (and usually also sharper) hyetograph than the 50 percentile curve may follow from the fact that Hogg (1980) did not separate storms into individual quartile groups as done by Huff (1967). Examples of Huff's and Hogg's distributions are given in Fig.3.

Pilgrim and Cordery Method

In this method (Pilgrim and Cordery, 1975), periods of each event are ranked according to the depth of rainfall. Using all events, average ranking is produced for each period. The percentage of the total rainfall for each event in each of the ranked periods for that event is determined. Average percentages are then calculated for individual ranks for all events. The hyetograph is then formed by arranging the periods in the most likely order by following the average period ranking and assigning to it the average percentages of rainfall. An example of the hyetograph derived by this method is shown in Fig.3.

Sifalda Method

Preprocessed actual hyetographs are characterized by mean values of the total rainfall depth, storm duration, the duration and magnitude of the peak intensity burst and the chronological location of this burst (Sifalda, 1973). The design storm hyetograph is then constructed from such characteristics. The hyetograph consists of a rectangular section with the peak intensity and of two trapezoidal sections, of different durations, preceding and following the peak intensity block (see Fig.3).

Yen and Chow Method

Actual hyetographs were described by the method of statistical moments. Using only the first moment, a general dimensionless triangular hyetograph was established. The original reference (Yen and Chow, 1980) referred to further possible extensions of this method.

All the temporal distributions discussed here are plotted in Fig.3 using the data presented by Arnell (1982), Marsalek and Wattt (1983), and Wenzel (1982). It is obvious from Fig.3 that various methods, when applied to a set of raw data, produce significantly different design storm hyetographs. Such differences can be noticed for the peak magnitude and timing as well as for the overall distribution. Some methods may even produce different rainfall depths for identical return periods and durations. As a consequence of differences in design hyetographs, the simulated runoff peaks will differ as well. The extent of variations in runoff peaks simulated for various design hyetographs will be affected by the catchment response and the selected rainfall interval and the computational time step.

CONCLUSIONS

The temporal distribution of rainfall intensity is one of the most important characteristics of the design storm. This distribution, and in particular its peak intensity and the peak timing, significantly affect the runoff peaks simulated for small urban catchments of significant imperviousness. Various methods for the development of design storm temporal distributions, recommended in the literature, produce widely varying temporal distributions. Such variations are then reflected to some extent in the simulated runoff peaks. Further comparative studies and evaluations of temporal distributions for design storms are needed.

ACKNOWLEDGEMENT

The descriptive parts in the first three sections of this discussion are partly based on an earlier paper written by Marsalek and Watt (1983).

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