United States
Environmental Protection
Agency

Office of Air
Land and Water Use
Washington DC 20460

EPA-600/9-78-035
October 1978

Research and Development

Urban Runoff Control Planning

Miscellaneous Reports Series

EPA-600/ 9-78-035
October 1978

URBAN RUNOFF CONTROL PLANNING

M.B. McPherson
Director, ASCE Urban Water Resources
Research Program
Marblehead, Massachusetts

Project Officer

Harry C. Torno
Office of Air, Land, and Water Use
Office of Research and Development
U.S. Environmental Protection Agency
Washington, D.C. 20460

OFFICE OF AIR, LAND, AND WATER USE
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

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FOREWORD

A major function of the Research and Development programs of the Environmental Protection Agency is to effectively and expeditiously transfer, to the user community, technology developed by those programs. A corollary function is to provide for the continuing exchange of information and ideas between EPA and users, and between the users themselves. In this latter spirit, the Office of Air, Land, and Water Use publishes work which, while not originally supported by EPA, is of sufficient interest and merit to be useful to engineers and planners working on EPA-supported programs.

This report, by the ASCE Urban Water Resources Research Council under the sponsorship of the National Science Foundation, provides insight into the urban runoff problem, and ashould prove invaluable to those engaged in the planning for or design of urban runoff pollution abatement or control projects.

Courtney Riordan
Acting eputy Assistant Administrator
Office of Air, Land, and Water Use

ABSTRACT

Section 208 of Public Law 92-500 (Federal Water Pollution Control Act of 1972) encourages areawide planning for water pollution abatement management, including urban runoff considerations where applicable. Areawide studies are under way or planned in just about every metropolitan area. Deadlines for initial areawide reports are not far off, and it is expected that many of the agencies preparing reports are presently resolving their projected activities beyond the current first planning phase. This report has been prepared to assist agencies and their agents that are participants in the preparation of areawide plans, from the standpoint of major urban runoff technical issues in long-range planning. Emphasized is the importance of conjunctive consideration of urban runoff quantity and quality and the need to development a factual basis that will support expected reliability of performance of proposed actions and programs. While not intended as a handbook for urban runoff control planning, this report delves into some important technical issues that are often slighted or poorly handled, such as the utilization of simulation. Recognizing that the ultimate test of any plan lies in its implementation, topics are viewed from the perspective and experience of the local government level where implementation takes place.

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INTRODUCTION AND SUMMARY

Synopsis

Section 208 of Public Law 92-500 (Federal Water Pollution Control Act of 1972) encourages areawide planning for water pollution abatement management, including urban runoff considerations where applicable. Areawide studies are under way or planned in just about every metropolitan area. Deadlines for initial areawide reports are not far off, and it is expected that many of the agencies preparing reports are presently resolving their projected activities beyond the current first planning phase. This report has been prepared to assist agencies and their agents that are participants in the preparation of areawide plans, from the standpoint of major urban runoff technical issues in long-range planning. Emphasized is the importance of conjunctive consideration of urban runoff quantity and quality and the need to develop a factual basis that will support expected reliability of performance of proposed actions and programs. While not intended as a handbook for urban runoff control planning, this report delves into some important technical issues that are often slighted or poorly handled, such as the utilization of simulation. Recognizing that the ultimate test of any plan lies in its implementation, topics are viewed from the perspective and experience of the local government level where implementation takes place. On-site elaboration of the principles presented in this report, and their adaptations to local conditions, will be made in seminars, workshops and conferences to be led by the author in a number of metropolitan areas over 1977-1979. During that period this report will be updated and supplemented in a series of subsequent releases.

Overview

Perhaps never before in our history has there been as rapid a change in the public's attitudes towards its surroundings as has occurred in the past few years. This has been fueled by unprecedented world-wide population growth, a resultant intensified international communication of pollutants as well as information, and a growing appreciation of the finite limitations of the resources of our planet. Manifestations are concerns over environmental protection, more rational use of land, better husbandry of energy and other resources and greater relief from natural disasters. Because three out of four Americans live in urban settlements that occupy less than one-fortieth of our land area, it is equitable to say that the winds of change are predominantly urban in character, or at least that urban dwellers will be the larger number affected.

The growth impact of projected areal enlargement of urban areas[1,2]* on present and planned urban water resource facilities is almost too stunning a reality to be comprehended fully at this time. Even if expected urban growth is checked by a renaissance of our central cities, the required reconstruction will still be monumental.

On the basis of past performance, it has been asserted that while the construction phase of water resource project implementation takes somewhere around 1 to 5 years for smaller projects and around 5 to 15 years for larger projects, because of protracted delays in conflict resolution the true lead-time necessary for accomplishing the objectives sought has been on the order of 20 to 25 years,

*: references are listed in Section 8.

particularly for larger projects, including certain large urban water supply developments.(3) Anticipated geometric increases in the impacts on urban water resources suggest that total project development time must be substantially compressed. Thus, means must be found for accelerating conflict resolution, the element most likely to cause protracted delays, despite the fact that new works will increase steadily in hydrologic complexity because of a growing interdependence among project components. That is, local governments are confronted with public demands for instant solutions while simultaneously there is more direct involvement by the public in decision-making as the problems and their solutions become increasingly more complex. On top of all this is the shifting of targets as national and State laws and regulations are constantly revised and augmented in an upward spiraling of stiffened requirements. There are numerous instances where elaborate plans have had to be abandoned or drastically altered because the "rules of the game" had suddenly become more demanding.

Problem

Considerable planning activity is being focused on the improved management of all types of water pollution, including that from urban runoff. Section 208 of Public Law 92-500 (Federal Water Pollution Control Act Amendments of 1972) encourages areawide management planning in areas which, as a result of urban-industrial concentrations or other factors, have substantial water quality control problems. The 208 studies are resulting in a many-fold increase in planning activities on urban runoff pollution management, with studies under way or planned for just about every metropolis in the nation. While an opportunity is presented through these studies "to plan and manage a comprehensive program based on integrated planning and control over such activities as municipal and industrial wastewater, storm and combined sewer runoff, nonpoint source pollutants, and land use as it relates to water quality,"(4) evidence is accumulating that, in order to meet initial reporting requirements, many of these plans may favor short-term solutions over the meeting of long-range goals.

Because knowledge on quantity and quality of stormflows is limited, long-range planning should be accompanied by programs for the acquisition of local field data, and its analysis by such means as simulation techniques. The 208 study schedule does not provide sufficient time to develop a suitable factual base where it did not already exist, and as a result in many metropolitan areas problem identification and definitive planning are expected to continue well beyond current initial reporting activities.(5)

Purpose

The major purpose of this report is to encourage long-range comprehensive planning for urban runoff quality and quantity management, as a supplement to applicable current Section 208 planning.

Another purpose is to stress the importance of planning not only for control of urban runoff quality but also quantity, a major economic and environmental consideration that might be inadvertently slighted in the new national emphasis on quality management.

Only a very few instances have been identified where local government planning that was started several years ago for urban runoff control has conjunctively addressed both quality and quantity management aspects. These and other but narrower leading-edge examples are cited whenever methods or procedures that have been used might be adapted to advantage elsewhere.

This report is _not_ a handbook for urban runoff control planning, and it is _not_ a definitive exposition of the planning process. It _does_ delve into some important technical issues that are often slighted or poorly handled, such as the utilization of simulation.

A synthesis of local government perspectives on planning and associated problems is presented with the intention of facilitating metropolitan intergovernmental cooperation and coordination. Because an intended audience is Section 208 planning agencies, a central purpose is to bring to their attention some potential technical impediments that might frustrate effectuation of their proposals. There are cautions also, directed at local governments, that are intended to be reminders of some technical pitfalls that should be avoided in their dealings with areawide plans. In short, we hope that the uninvited intervention of an outside referee will be of some assistance in improving the effectiveness of the participants in a very complex process.

Delivery

This report is being distributed to area-wide planning agencies and supportive entities in all metropolitan areas. On-site elaboration of the principles presented herein, and their adaptation to local conditions, will be provided in seminars, workshops and conferences conducted in a number of metropolitan areas over 1977-1979. As the delivery phase progresses, special topics not treated adequately or sufficiently elsewhere that are found to need synthesis and summarization will be reviewed in subsequent Addenda to this report. The two Addenda included with this report are examples of what is intended. Only a fraction of the metropolitan areas can be assisted via site visits made for the purpose of elaborating the report's contents, but through the distribution of post-report Addenda to all metropolitan planning agencies a wider audience will be reached. Also, the Addenda will serve as a vehicle for updating the content of the present report.

Intentions and Expectations

Just about all broad-scale, leading-edge, local government plans for urban runoff control we have encountered are called "master plans," and although this term appears to be abhorred by many metropolitan planning agencies we do not propose a wholesale semantic reform. This difference in viewpoint probably springs from the realistic need for areawide planning agencies to preserve the concept of a "living document" in principle and in fact, whereas local governments cannot afford this luxury because any planning dealing with capital improvements must lead to acts of implementation, and once construction starts the die has been cast.

The conventional local government master plan has generally dealt with only one aspect of water resources such as water supply, or wastewater treatment, or flood control, or pollution abatement. We favor and encourage the emerging type of comprehensive planning that provides integrated consideration of flooding protection, pollution abatement including erosion control, and water conservation. Even some of the advanced examples cited in this report have integrative deficiencies. There are simply not many local government plans under preparation that are truly comprehensive because the impact of changing policies and objectives has not been factored into many such plans. Moreover, because comprehensive local government urban water resource planning is a relatively new concept, very few of the broader plans have been carried to the threshold of implementation.

We have no illusions about the possibilities for wholesale change. Perhaps the most we can hope for in the near term, in a majority of metropolitan areas, is a conjunctive planning approach, at least with regard to the quantity and quality aspects of urban runoff. Comprehensive planning faces a number of obstacles, including fractionalized authority of administrative agencies and balkanized local governments in metropolitan areas,(6) specialist approaches taken individually on each water service component, and uneven advances in knowledge on the various components. The term "water management," widely used only in recent years, has not yet acquired a specific meaning. Although rational control of water is its objective, there is no common understanding of needed policies and institutional arrangements for achieving rational control. On the other hand, metropolitan area planning is being conducted formally almost nationwide, although the planning function is normally advisory, only; and in terms of comprehensive water resource development, the planning, implementation and operation of works is usually fragmented in both the central cities and in their metropolitan districts. Even water and wastewater services are commonly fragmented in metropolitan districts (e.g., our largest metropolis is served by some 400 separately managed water agencies). A major cause of administrative fragmentation is the existence of often gross dissimilarities in geographic areas: viz., differences in hydrologic, service and revenue areas, and in political jurisdictions. One of the consequences is that data collection and its analysis and related research and development is currently pursued independently by individual local government departments and metropolitan special districts, with only limited collaboration on a national or even metropolitan scale. Therefore, early coordination by local governments of data collection and its analysis is strongly advised.

"Although metropolitan-wide planning and coordination is necessary for successful urban water resource management, the key to the implementation of such planning generally lies at the local government level and at the State level in terms of what the State enables or requires local government to do."(7) (Emphasis in original).

While urban water resource implementation agencies may do some long-range or strategic planning, their basic functions as mandated by authorized responsibilities are management activities that emphasize specific local demands, a narrow range of alternatives, detailed rules for operation and maintenance, and the inclusion of measures such as service charges and service extension policies.(7)

"There have been some attempts to define an integration of regional planning with local guidance and management, but there have been few, if any, successful attempts to apply it."(7)

More comprehensive management approaches would integrate or give due consideration to all aspects of water in a metropolis, together with related environmental, energy and other relevant considerations. The need for a comprehensive overview becomes more crucial as the complexity of urban areas increases, caused by such things as population migration and growth, competition over available energy supplies, rising expectations of urban dwellers in the face of inflation of local government costs, and greater public awareness of environmental issues and enlarging demands for public participation. A means for assuring a comprehensive overview is the conduct of metropolitan water balances, or inventories, described in Addendum 1 of this report.

Summary, Section 2 - Selected Planning Considerations

Based on experiences with a leading-edge master plan for combined sewer overflow pollution abatement, a set of planning elements is presented that should serve as a useful check list.

The same master planning effort is reviewed in terms of project financing scenarios for meeting a variety of possible abatement goals. Illustrated by this example plan is the urgency of incorporating sufficient flexibility in a plan to satisfy future shifts in regulations as implementation proceeds.

Six discrete steps can be identified in the typical planning exercise: problem definition, establishment of goals and identification of constraints; data assembly and problem redefinition; formulation of alternatives; analysis of alternatives; identification of more promising alternatives; and evaluation of the trade-offs among selected alternatives. These fundamental steps are elaborated in a detailed block diagram of the urban runoff control master planning concept that incorporates all features of several less comprehensive depictions. Noted parenthetically is that a local agency can become so obsessed with long-range planning that more immediate, partially mitigative solutions can be overlooked or inordinately deferred.

Next, an example is given of a master plan for combined sewer overflow pollution abatement that was launched from the metropolitan level. The careful procedures that were followed in marshalling for the plan are detailed because they provide an example worth emulating.

Examples are then presented that include: a County master plan for flood control, erosion control and drainage; the master plan for the largest combined sewer overflow pollution and drainage control project in the nation; flood control master planning for a metropolitan area; and inferences of the Urban Studies Program of the Corps of Engineers.

The examples given in this Section deserve consideration by many readers because they are prototypes in many respects of desirable practices and they suggest, by what they do not include, possibilities for improvements elsewhere.

Lastly, the cyclic process of reviewing alternatives in a continuing planning process is briefly considered. Noted is the need for local government entities to be in close communication with regional planning agencies. That is, a firm, positive linkage should be forged between metropolitan planning and local government implementation. Noting that the detailed local government master plans alluded to earlier in this Section have required a minimum of three to five years for their development, it is important to accept that the local government planning time-frame must be measured in terms of a number of years.

Summary, Section 3 - Incentives for Comprehensive Planning

The thesis of this Section is that rational planning requires conjunctive consideration of the quantity and quality aspects of urban runoff within a comprehensive multiple-use framework. Some of the more obvious arguments are explored, in connection with public viewpoints on land-use controls, disparities in approaches to economic evaluation, potentials for water reuse, opportunities to reduce extraneous sewer flows, control of erosion and sedimentation, and disposal of solids from new joint or ad hoc treatment facilities.

Acting against the conjunctive approach is the tendency of citizen groups and Federal laws to dwell on land-use controls as a panacea while minimizing the interrelations and interconnections inherent in water resources. This point surfaces recognition of some of the challenges confronted by conjunctive planning and the great importance of public information, and even public education, in promoting the larger view.

Economic evaluation of water quality control projects at the local level appears to be restricted to some form of minimum cost analysis, as opposed to flood control economic justification via cost-benefit analysis at the river basin scale. Because of the trend towards the blending of river basin planning with metropolitan planning, local agencies will increasingly face the need to reconcile this dichotomy of project evaluation criteria. Further, these disparities in economic bases for project evaluation would seem to suggest rather strongly that planning for management of urban runoff quantity and quality should be conjunctive to avoid absolute confusion.

After briefly reviewing various national developments and trends it is concluded that the reuse of wastewater treatment plant effluents and reclamation of surface runoff is more likely to be via artificial recharge of groundwater supplies, in general. This eventuality can be provided for only through conjunctive planning.

A national Symposium concluded that, because the majority of publicly-owned wastewater treatment works will be committed by the time Section 208 plans are ready for review and implementation, the only questions that will still be open by that time will be nonpoint sources of pollution and growth management and who pays for growth. Thus, land-use control issues will continue to simmer for some time.

Erosion and sedimentation control is one of several water quality issues that cannot be divorced from their causative driving force, urban runoff.

With regard to treatment of stormwater and combined sewer flows, handling water volumes is only part of the problem, for once solids conveyed by water are removed they must be disposed of somewhere.

The point of resolution for inconsistencies, conflicts and duplications of legal instruments imposed by higher levels is at the local level of government. It is concluded that for this reason, among several, local governments are forced to define their own problems.

Summary, Section 4 - Some Flow Management Considerations

Because urban drainage and flood plains are almost always the responsibility of separate and different kinds of jurisdictions, their interconnected behavior is the responsibility of neither type of organization. A similar splintering of authority over water quality aspects plagues most metropolitan areas. Given this institutional patchwork, about the only avenue for conjunctive consideration of these otherwise disparate issues is via comprehensive planning and coordination at the metropolitan level.

Asserted in this Section is that the guiding principle should be to reduce the liabilities and increase the assets of urban runoff.

After reviewing a range of techniques applicable to urban runoff control the reader is warned that although numerous schemes have been postulated for controlling the quantity and quality of urban runoff, very few of these concepts have been tested in full-size system applications, and the research on which many of them are based is fragmented and the findings cannot be sufficiently generalized for universal applications. By default, local governments are obliged to assure themselves of the relevance and reliability of most such schemes before making a commitment to their wholesale use. This leads to an extension to the last conclusion above for Section 3: local governments are forced to define their own problems and to take the initiative in finding solutions to those problems.

Summary, Section 5 - Utilizing Simulation

While simulation is undoubtedly an effective means for defining problems and analyzing alternative urban runoff control strategies, its most important use can be in the assessment of expected system performance. Performance reliability is ultimately the fulcrum of local government political acceptability.

This Section commences with reasons for using simulation and a historical reckoning of practical reservations against incautious acceptance of runoff models. Emphasized is that simulation techniques adopted should not exceed the level of mastery of such tools by the user and that tools should be selected on the basis of their suitability for solving problems.

Performance reliability is defined in terms of four serially connected principal considerations, which must be completed in the following order to be fully effective: testing simulation models against local field data; using a long-term precipitation record as the basic reference; developing frequency rationales for levels of protection; and simulating a range of storms and system loadings. An emphasis is placed on the testing of simulation models against local field data because this issue is the crux of credibility as well as reliability. As things now stand, local governments are substantially on their own for the acquisition of field data.

The concept of spatial sampling for field instrumentation of catchments having representative land uses is outlined. Stressed is the importance of rapid application of data to models, to check data reliability but principally for expeditious calibration of models, the primary use of such data in planning.

Control of flooding and water pollution must be based on probabilities of occurrence because of the randomness of precipitation. Rationales are developed for using long-term rainfall records as the reference for defining urban runoff quality and quantity control objectives in probabilistic terms. Addendum 2 is an extension of this presentation.

Underscored in this Section is that the only realistic defense for planning in an atmosphere of ambiguous policy is to employ procedures the results of which have an inherent flexibility for conversion to meet alternative policy goals.

Reliability in the employment of calibrated tools for the exploration of alternatives planned for the future is a function of several things, but the most important are probably the shrewdness of design of the field data network and the extent to which its results approach an ideal set of representative samples.

Summary, Section 6 - Urban Runoff Models

Reiterated in this Section is the importance of calibrating any urban runoff model against local field data.

Models are characterized in this Section in terms of their applications. Specific models are identified, with an emphasis on sewered system planning applications, although associable analysis/design applications are duly noted. Receiving water modeling is handled separately because of the tendency towards customized adaptation of available models for that purpose. Examples are given of some regional receiving water simulation studies.

Summary, Section 7 - Some Modeling Considerations

Models can be used ineffectively if they are selected and employed without prior careful consideration of their data requirements and their place in overall information processing.

While flood plain mapping of 100-year stream stages is a minimum requirement, the mapping of more frequent occurrences is required for analysis of the economics of flood damage mitigation.

Permanent field data acquisition systems for operations, particularly for automatic control, can be readily justified, whereas only temporary installations can be justified for other purposes except for long-range monitoring.

The three themes described above are singled out in this Section because they are particular pitfalls that have been encountered in past planning efforts.

Also, the latest developments in the manipulation of storage via automatic control are described.

Summary, Section 8 - References

As a convenience to the reader, references are divided according to the Section in which they are cited. Considerable discrimination was employed in selecting these references from an initial listing with over three times as many entries. Rather than cite, say, six related references, the writer chose to use either the latest (which itself cites the ones omitted) or the one or two with the most useful information. Some references are cited to acknowledge credit for ideas or material while others are mentioned merely to document a point being made.

Knowing that every reader will have different interests and a unique familiarity with the body of literature involved, it would be folly to attempt to single out a short list as the "most important reading". The need to cite numerous references where each dwells on a narrow portion of the spectrum merely confirms a contention in this report that research and development on urban runoff control has travelled a disjointed, almost haphazard journey, with numerous efforts pursued independently.

Acknowledgment

This report has been prepared on behalf of and with the assistance of the ASCE Council on Urban Water Resources Research by its full-time operating arm, the ASCE Program bearing the same name. The activities and products of the Program are

well known.[8] Suffice it to say that the central objective of the Council is to help advance the state-of-the-art by identifying and promoting needed research and by facilitating the transfer of the findings from research to users. A Steering Committee designated by the ASCE Council gives general direction to its Program: Mr. S. W. Jens (Chairman); Dr. W. C. Ackermann; Dr. J. C. Geyer; Mr. C. F. Izzard; Mr. D. E. Jones, Jr.; and Mr. L. S. Tucker, Mr. M. B. McPherson is Program Director (23 Watson Street, Marblehead, Mass. 01945). Administrative support is provided by ASCE Headquarters in New York City.

Financial support for the preparation and duplication of this report is entirely from a grant to ASCE by the Research Applied to National Needs program of the National Science Foundation. Technical liaison representative for NSF/RANN is Dr. J. Eleonora Sabadell. Any opinions, findings, and conclusions or recommendations expressed herein are those of the author and do not necessarily reflect the views of the National Science Foundation. The ASCE Council is greatly indebted to NSF/RANN for supporting this nine man-month project.

Formal review of the two drafts of this report was generously provided by an ad hoc Advisory Panel assisting the ASCE Council:

Mr. Dennis Athayde, NPS Branch, Water Planning Division, EPA, Washington, D.C.
Mr. Harold C. Coffee, Jr., Department of Public Works, San Francisco
Dr. Charles N. Ehler, Office of Research and Development, EPA, Washington, D.C.
Dr. Neil S. Grigg, UNC-WRRI, North Carolina State University, Raleigh
Mr. Forrest C. Neil, Metropolitan Sanitary District of Greater Chicago
Mr. Harry C. Torno, Office of Research and Development, EPA, Washington, D.C.
Mr. L. Scott Tucker, Urban Drainage and Flood Control District, Denver
Dr. Stuart G. Walesh, Southeastern Wisconsin Regional Planning Commission, Waukesha
Dr. Donald H. Waller, Nova Scotia Technical College, Halifax

Information, assistance and review were also provided by scores of other persons from among the hundreds of ASCE Program cooperators. In particular, vital assistance was provided by officials responsible for some of the more innovative master planning projects. However, none of the persons named or alluded to in this subsection are to be held responsible for any of the contents of this report. They have nevertheless helped insure a better professional consensus on the issues involved and the ASCE Council and the writer are therefore greatly indebted to them for their invaluable assistance and generous cooperation.

A significant portion of this report draws upon relevant parts of a prior report that was prepared for the U.S. Geological Survey.[9]

Typing of this report and its drafts was by Mrs. Richard Symmes of Marblehead, Mass.

Principal Information Listings

Some readers may be interested in the sources of principal information listings in urban water resources research. Brief descriptions of on-going research on specific subjects are available from the Smithsonian Science Information Exchange. One service of the Exchange is the availability of "research information packages" that are updated every 90 days. Of particular interest here are the packages on urban water management, urban runoff, and effects of storm water runoff on water quality in receiving streams.[10]

For completed research, reference should be made to the selected water resources abstracts issued twice per month by the National Technical Information Service that are compiled by the Office of Water Research and Technology.[11] Nearly all of the reports cited in the abstracts that have been supported by public funds are available for a cost-recovery charge from NTIS, in which cases the NTIS identification numbers are included in the abstracts. Information searches on urban water publications can be greatly facilitated by referring to the annual cumulated indexes of the selected water resources abstracts.[12] In addition, NTIS has packages of bibliographies with abstracts on some specific subjects, such as urban surface runoff. Both NTIS and SSIE also make custom searches of completed and on-going work, respectively.

SECTION 2

SELECTED PLANNING CONSIDERATIONS

Definition

Comprehensive, areawide planning is conducted by metropolitan planning agencies. Implementation of such plans or their variants is by local units of government. Moreover, more detailed planning is done by local governments and their plans are carried through construction into operations. The term "comprehensive planning" thus means quite different things at the metropolitan and local levels. Also, local government plans can follow, be concurrent with, or lead areawide plans. It would be totally presumptuous and entirely inappropriate to attempt in this report to instruct regional planning agencies on the nuances of their responsibilities. Rather, a major purpose is to present a synthesis of local government perspectives on planning and associated problems which hopefully will facilitate metropolitan intergovernmental cooperation and coordination. Because an intended audience of this report is Section 208 metropolitan planning agencies, a central purpose is to bring to their attention some potential impediments that might frustrate effectuation of their proposals. There are cautions also, directed at local governments, that are intended to remind them of technical pitfalls that should be avoided in their dealings with areawide plans. In short, we hope that the uninvited intervention of an outside referee will be of some assistance in improving the understanding of the participants in a very complex process.

Regardless of the governmental level involved, when we use the term "comprehensive planning" we are referring to a multipurpose scope, such as for integrated management of flooding protection, pollution abatement including erosion control, and water conservation. That is, despite the label of comprehensiveness, we are really addressing only a subset of the overall metropolitan comprehensive planning matrix. Just about all broad-scale, leading-edge, local government plans we have encountered are called "master plans," although this term appears to be abhorred by many regional planning agencies. There are not many local government plans under preparation that are truly comprehensive because the impact of newer policies has not been factored into many such plans and, as a result, there are simply not many broad urban runoff control plans under way and those that are usually have been initiated to comply with a State directive against a particular local government. Further, comprehensive urban water resource planning is a relatively new concept to local governments and very few plans have been carried to the threshold of implementation, even for the narrower objective of urban runoff control.

An Example of Local Government Planning Elements

What is called for above is essentially a systems approach to facilities and management planning. For example, elements needed in water quality control combined sewer system planning are said to include the following:(13)

- An analysis of implementing governmental entities.
- A public information system.
- A set of goals, consistent with existing and proposed regulations for the foreseeable future.
- A realistic and attainable set of regulations.
- A definite understanding and description of the problems and their analysis.
- A complete description of the causes of the problems.

(Continued)

- A finite description of the existing physical system and its inadequacies.
- A finite description of the receiving-water environment for biota.
- A finite description of solids disposal locations.
- A detailed rainfall history perhaps spanning 30 to 50 years.
- A detailed history of water-demand and water-use factors.
- A detailed pollutant-constituent history.
- A complete water quality inventory showing constituents in the water supply, in wastewater prior to treatment, and in effluents.
- A means for evaluating treatment in terms of storage and overflow options.
- A series of viable alternatives based on real data.
- A recommended solution or set of solutions.
- A flexible facilities and operations staging program.
- A practical financing program.
- A source of funds.

Experience indicates that a steering committee comprised of outside experts can be very helpful in avoiding oversight or slighting of the above elements. While the above elements may be regarded as idealistic by some, the local government that ignores them does so at its peril. Because of unique combinations of local conditions, it should be evident that satisfaction of the needs listed above would necessarily vary in detail, methodology and procedure from one local jurisdiction to another, even in the same State. However, the institution of a public information program early in a local government planning endeavor is very important.

An Illustration of Local Government Planning

One of the most outstanding comprehensive, imaginative and thoroughly developed plans for municipal pollution control has been prepared by the Department of Public Works, City and County of San Francisco, California. San Francisco is served almost exclusively by a combined sewer system. The plan is founded on a data bank of unusual scale, ranging from runoff-quality field measurements initiated in 1965, through installation of a pilot wet-weather dissolved-air flotation treatment plant on a field catchment, to the use of a unique automatic rainfall-runoff monitoring system. A deliberate step-by-step approach was used in developing the plan, which was carefully phased with field information acquisition. The quality of the engineering work in the development of the plan was superb, and the plan was supported by several DPW studies and 25 ancillary reports prepared by consultants. Pioneering development of automatic control capability by the DPW is discussed in Section 7.

Water quality standards for both dry-weather and wet-weather flows have presented a moving target for pollution abatement compliance by local agencies, and more changes in regulations may be anticipated in the future. Recognized in the development of the "master plan" for wastewater and combined sewer overflow control in San Francisco was that for various abatement plans the timing imposed by Federal and State regulatory agencies and subsequently supported by their funding is the critical factor controlling the final selection of such a plan. The flexible plan adopted by the City is amenable to adjustment during its implementation to meet future shifts in regulations, but the feature to be discussed here is concerned instead with project financing scenarios for meeting a variety of possible abatement goals.

The preliminary master plan proposed in 1971 provided four alternates that would allow an overflow to occur from a maximum of eight times per year to a minimum of once in five years. Because the amount of Federal funds and the eligibility requirements for grants had changed almost continuously, and wet-weather projects had a lower priority than dry-weather projects (and were not scheduled for either California or Federal grants), the six variables affecting the scheduling of a bond issue were analyzed:

- Frequency of overflow occurrence (4 alternates).
- Effluent disposal site for a wastewater treatment plant (3 sites).
- Level of dry-weather treatment (3 levels).
- Number of years for completion of all facilities (10 to 60 years).
- Per cent of total costs obtained from grants (0% to 80%).
- Amount of the City's financial resources that might be allocated (four values).

Results of the analysis of the 1,944 combinations for each of the four alternates were presented in a single ingenious graph displaying all six variables.(14) Thus, for a given overflow control level, a given effluent disposal site, a given dry-weather treatment level, a given number of years for facilities completion, and a given per cent support by grants, the amount of the City's resources that would be required can be determined from the correlation graph. Through this graph the question of who would pay what share of the costs for meeting a particular water quality objective under a particular set of circumstances becomes immediately clear. This is an excellent illustration of the fulfillment of an engineer's responsibility in this instance: to present the range of options available to officials with the authority to make decisions and recommendations. In essence, the Department's engineers provided the City's elected officials with the means for negotiation with regulatory agencies having both enforcement powers and any funds for supportive grants.

While the above description is for abatement in terms of the average number of combined sewer overflows permitted per year, this could be readily converted into overflow volumes or pollutant loads or seasonal serial events instead, by reference to the City's performance simulation results described elsewhere in this report.

Parenthetically, although the basic features(15) of the preliminary master plan(16) have not been altered, some modifications were made in 1973(17) and again in 1976(18) when implementation of the first phase commenced.

Of the 28,000 acres in San Francisco, 24,000 are drained by the sewer system (plus 2,000 acres in adjoining San Mateo County). While the City is surrounded on three sides by receiving waters, the Pacific Ocean and the Bay, streamflow flooding does not exist in the City and therefore was not a consideration in development of the master plan. Also, the City is essentially completely developed and thus is not involved in the conversion of low-density land into suburban expansion. We now turn our attention to the more general situation where both sewer systems and streams drain the land and where both existing and projected drainage systems are involved.

A Broader Perspective

A review of a number of local government planning projects has revealed a commonality of six discrete steps, which might be regarded as the framework of the standard planning process:(19)

- Define problems, establish goals and identify constraints.
- Assemble data and refine definitions of problems.
- Formulate alternatives.
- Analyze alternatives.
- Identify more promising alternatives.
- Evaluate the trade-offs among selected alternatives.

Embracing the above six parts of the basic planning process, Figure 1 is a block diagram depicting the urban runoff control planning concept, which attempts to accommodate situations with sewer systems and streams and land already developed and projected for development. The term "master planning" appears in the title to insure a local government application connotation. The sequence of steps shown is not intended to specify the scheduling of the steps or the degree of their completion needed, but the order in which decisions should be reached in order to assure an efficient planning progression. Considerable feedback would be involved from any given major step to various preceding steps, but no attempt is made to show the morass of possibilities on the diagram because it is complicated enough as it is. Simulation and related hydrological and water quality field data acquisition aspects are discussed in detail in Section 5.

In the preparation of Figure 1, all the significant features of three selected representations were also taken into account, for water flow control[20] and water quality control[21,22] local government planning.

For conjunctive flow control and quality control planning, water quality considerations tend to dominate. This is primarily because flow control is concerned solely with quantity while quality control is concerned with the complex relationships between runoff and its pollutant burdens. A compelling reason for engaging in conjunctive planning, however, is that the overall flow processes for quantity and quality planning are shared in common, making it difficult to justify two independent planning efforts which draw upon the same field information resources and often use the same or similar or related tools of analysis. Other arguments favoring conjunctive planning are presented throughout the remainder of this report.

Before leaving Figure 1, it is important to note that a local agency can become so obsessed with long-range planning that more immediate, partially mitigative solutions can be overlooked or inordinately deferred. Zoning ordinances, flood insurance and incorporation of detention storage in new land development are examples that come quickly to mind. Preparation of most local government plans completed to date has required a minimum of three to five years. There are interim or initial actions that can be taken early in this period that do not require significant local government funding and hence are not likely to be as highly controversial politically.

Marshalling for a Plan

Mentioned earlier was that the first phase of the San Francisco master plan is being implemented, reflecting the effectiveness of its careful preparation. We now turn to a planning effort recently completed for metropolitan Milwaukee. To be discussed here are the deliberate, careful steps that should be taken to organize and marshall forces for the conduct of plan preparation. While the San Francisco effort originated with and was pursued by an operating department of a municipality, that for the Milwaukee area was launched from a metropolitan level. The description which follows is a digest of coverage in an earlier ASCE report.[9]

A preliminary engineering study leading to a plan for the abatement of pollution from combined sewer overflows in Milwaukee County was initiated by the Metropolitan Sewerage District of the County, a special-purpose metropolitan agency. The combined sewers in the County are owned and operated by the City of Milwaukee and one of its adjacent suburbs, the Village of Shorewood, both of which are entirely within the County and the Sewerage District. Three rivers pass through the area served by combined sewers: the Kinnickinnic, the Monomonee and the Milwaukee Rivers,

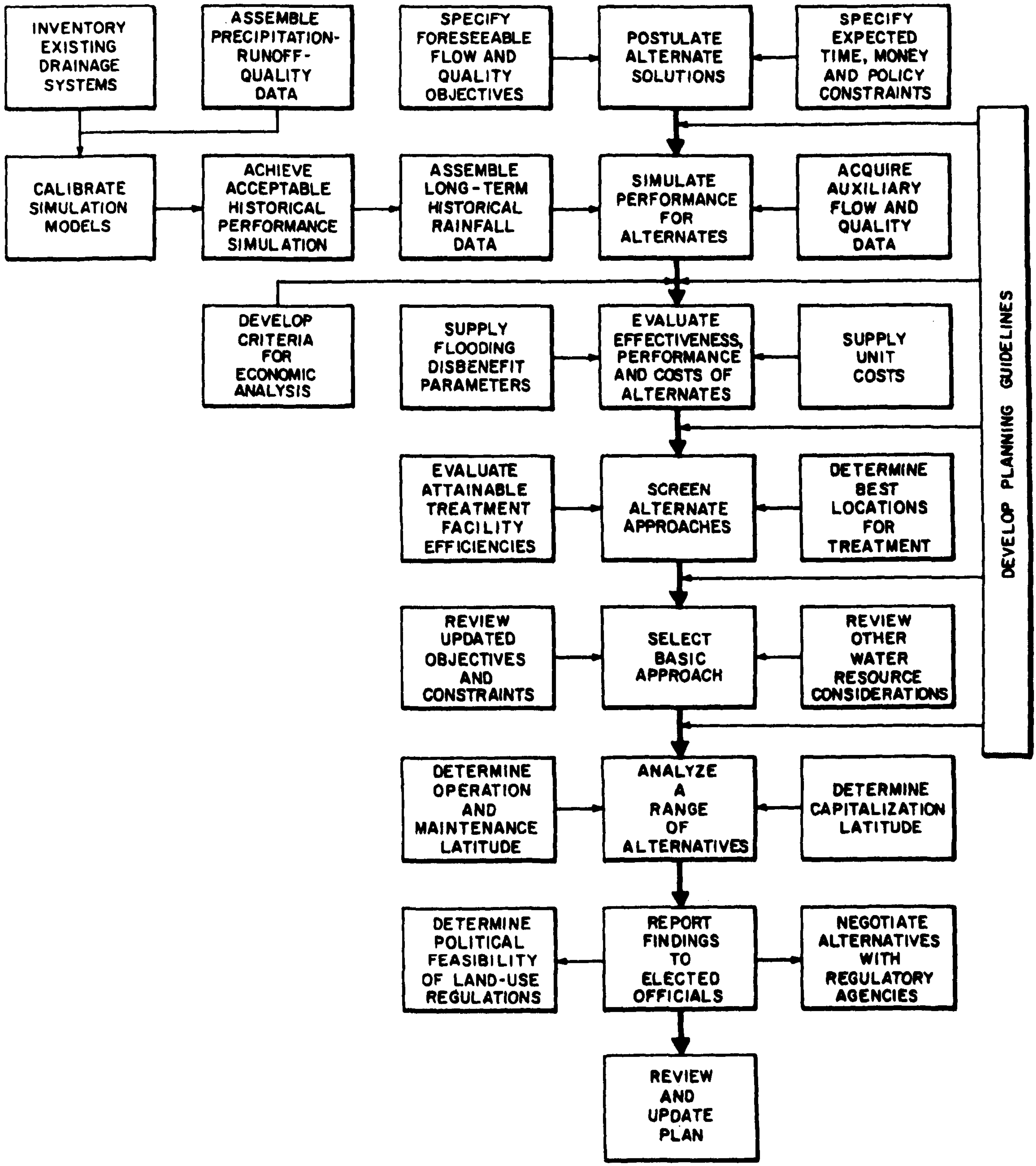


FIGURE 1 - BLOCK DIAGRAM OF URBAN RUNOFF CONTROL MASTER PLANNING CONCEPT

all in the Lake Michigan Basin. The three rivers merge in the downtown area of the City of Milwaukee and enter Lake Michigan through a common channel. The mouth, the confluence and reaches part way up each river are navigable and used for various types of commercial shipping.

Alternative abatement methods for the master plan were investigated as a part of a comprehensive study of water-related problems of the Milwaukee River Watershed conducted by the Southeastern Wisconsin Regional Planning Commission (SEWRPC) at the request of the Common Council of the City of Milwaukee and the Board of Supervisors of Milwaukee County. After three years of intensive work by SEWRPC, retained consultants and a technical advisory committee comprised of Federal, State and local citizens, public officials and technical experts, the report(23,24) was published and later adopted as a regional guide for development and for solution to existing problems. That part of the report that dealt with the combined sewer problem investigated many different methods of abating pollution from combined sewer overflows and recommended, based on the pre-feasibility investigation, the use of a combination of methods entitled "Deep Tunnel/Mined Storage/Flow-Through Treatment". The report also recommended that an existing sub-regional agency, the Metropolitan Sewerage District of the County of Milwaukee, agree to conduct a more in-depth engineering study leading to a master plan.

In April, 1973, the Metropolitan Sewerage District, through the agencies of its constituent operating bodies, the Sewerage Commission of the City of Milwaukee and the Metropolitan Sewerage Commission of the County of Milwaukee, requested SEWRPC to prepare a Prospectus for a preliminary engineering study for combined sewer overflow abatement for use in selecting a consultant to conduct the study. The Prospectus was prepared by SEWRPC with the assistance of a Technical Advisory Committee of outstanding Federal, State and local specialists in the water pollution field and completed in July, 1973.(25) The Prospectus outlined a proposed engineering study based on the work contained in the Milwaukee River Watershed report and directed a more in-depth study of possible alternatives as well as further advances in the field that had occurred since the publication of the report. The Prospectus was used as an invitation for proposals and was sent to many of the most prominent consulting engineering firms in the nation. Twenty-one proposals were received, as evidence of the interest in such a project, with six consultants invited for oral presentations.

The selection of the six consultants, as well as the recommendation to the District of the consultant to be retained, was made by the same Technical Advisory Committee; and the Committee was later transferred to the jurisdiction of the District. The District then entered into a contract with a consultant for the preliminary engineering study. The consultant worked under the direct supervision of the staff of the Metropolitan Sewerage District assisted by the Technical Advisory Committee. The primary jurisdiction of the Sewerage District is the planning, construction and operation of the sanitary sewerage facilities for approximately 420 square miles, comprising the Metropolitan Milwaukee area. The District operates two regional wastewater treatment plants, 240 miles of interception sewers and 20 miles of improved stream channels.

Three basic approaches for abatement and control were considered in the Milwaukee River watershed study: storage of overflows with their subsequent slow release for conventional treatment at existing wastewater treatment facilities; flow-through and inflow treatment of overflows at special treatment facilities during peak flow periods; and complete separation of combined sewer systems. Ten different storage schemes and three different methods of separation were explored in the total of fifteen different alternatives considered.(24) On the basis of careful economic, engineering and legal analysis of the fifteen alternatives, it was recommended that a combination of deep-tunnel conveyance, mined storage and flow-through treatment be utilized for combined sewer overflow abatement for the entire County combined sewer service area of 27-square miles.(25) The purpose of the preliminary engineering study was to verify the feasibility of the recommended plan or the best alternative thereto, and to determine the needed system configuration with particular emphasis on the balance to be struck between capacities for conveyance, storage and flow-through treatment.

The study area not only encompassed the combined sewer service area of the three rivers and the lakefront but was extended to include all areas that contribute to the hydraulic flows in the combined and intercepting sewers, and to permit proper analysis of related wastewater flows and stormwater discharges and related drainage and flooding problems. Provided was all information necessary for the respective political entities involved to make the public policy determinations required to proceed with the pollution abatement program.

Also considered in the study was the potential application of various combinations of collection, conveyance, storage and treatment facilities for the additional handling of stormwater runoff from throughout the Milwaukee urbanized area, should stormwater runoff treatment become necessary in the future to protect surface water quality, particularly in Lake Michigan and the interconnected Milwaukee Harbor estuary.

Tunnel systems have been considered in Boston, Chicago, Detroit, Milwaukee and Washington, D.C.(26) A brief description of the Chicago plan is included as a case study in an ASCE Program Technical Memorandum.(16) The Chicago studies were far more advanced than those for Milwaukee in 1973(26) and, as noted later in this Section, a major phase of that master plan is under construction. Because little has been published on the Milwaukee master plan(26,27) more description is given here than would otherwise be justified.

Local government plans do not evolve overnight and carefully weighed plans are usually founded on several years of deliberate planning and development, as epitomized by the initiation of the Milwaukee master plan.

Work on the Milwaukee Combined Sewer Overflow Pollution Abatement Project commenced late in 1974 and the first of three phases was completed less than a year later.(28) The organizational and scheduling plan for conducting the project, by tasks,(29) would be of interest to planning entities embarking on the development of such plans. The last phase was completed in mid-1977, and the final report provided the basic information required to start design for construction.

"Reliable local municipal stormwater drainage facilities cannot be properly planned, designed, or constructed except as integral parts of an areawide system of floodwater conveyance and storage facilities centered on major drainageways and perennial waterways designed so that the hydraulic capacity of each waterway opening and channel reach abets the common aim of providing for the storage, as well as the movement, of floodwaters. Not only does the land pattern of the tributary drainage area affect the required drainage capacity, but the effectiveness of the floodwater conveyance and storage facilities affects the uses to which land within the tributary watershed, and particularly within the riverine areas of the watershed, may properly be put."(24) The preceding is given as the guiding principle for water control facility development for the Milwaukee River watershed, and hence for the subject master plan. Flood mitigation aspects are discussed later in this report.

It is important to note that in the 1970 inventory,[23] specific model information needs were not an issue. In fact, most of the hydrologic models deployed in the master planning were only partially developed, at most, in 1970. This illustrates the recent phenomenon where information requirements shift in only a very few years as modeling capabilities are improved.

An Emphasis on Flooding and Drainage

An outstanding example of a comprehensive local government plan for flood control, erosion control and drainage is the program in Fairfax County, Virginia. The County lies to the west and southwest of Washington, D.C. The County's population leaped from 100,000 in 1950 to 560,000 residents in 1974. With this population spread over an area of 405-square miles the County can be said to be "suburbanizing" because it is being transformed from a predominantly rural area to a residential area for many people employed in the District of Columbia. Such explosive growth has strained various public facilities and services.

Problems associated with stormwater runoff were parried in November, 1971, when the voters of the County passed an $11-million storm drainage bond referendum. The objectives of the bond program are to solve existing flooding and drainage problems, to make plans for avoidance of such problems in the future, and to improve the storm drainage system operation and maintenance capability of the County staff. In the spring of 1972, a consulting firm was awarded a contract on the planning program, which will be completed late in 1977. Although the program has been described elsewhere,[9,30,31] some of its salient features deserve mention here.

Briefly stated, the specific goals of the program are to: develop immediate solutions for existing flooding and channel erosion problems; improve methods and procedures for developing budget projections for storm drainage capital facilities; establish a standard baseline from which to measure environmental changes; rewrite the drainage code and policy by incorporating a balance of on-site detention and off-site pro rata costs; and make full utilization of stormwater runoff by regarding it as a resource out of place.[32]

The thrust of the planning program is on flooding and erosion. Because it commenced prior to the enactment of PL 92-500, water quality was not considered to be a major issue at that time. However, included among the planning activities were "environmental baseline studies"[33] and environmentally-oriented design solutions, in all of which water quality was a consideration. Detailed study of water quality management in the County commenced with the Occoquan basin.

The "environmental baselines"[33] established for each of the 29 watersheds provided the environmental framework for developing basin plans and the necessary data for analysis of projected environmental conditions. Rather than use a benefit-cost approach for evaluating alternative plans, a "problem identification matrix" was chosen to permit better inclusion of benefits and costs for such usually elusive elements as visual impact, aesthetics, and creation or destruction of wildlife habitat and aquatic systems. The matrix approach, which has also been used elsewhere, was a technique for systematically identifying the general extent and importance of stormwater-related problems in the County and evaluating the adequacy of proposed solutions to those problems. Its use required three different elements: the objectives of the drainage planning program; the relative importance of each objective; and the degree to which each objective would be met under a given set of circumstances. Although program objectives can and may be changed or regrouped as experience by the County in the application of the method is gained, initial objectives, not necessarily in order of importance, were: freedom of residences from flooding damage; freedom of commercial and industrial facilities from flooding damage; freedom of public and institutional facilities and equipment from flooding damage; control of bank and channel erosion; protection of aquatic ecosystems; protection of wildlife habitats; freedom of parks, recreation and aesthetic areas from flooding damage; and prevention of traffic interruptions. Although protection of health and safety is not specifically included in this listing, it is an overall consideration and would take precedence over other objectives as part of any flood control plan.

By the end of 1977 the consulting firm will make available a summary report on the program, encapsulating procedures and findings embodied in the dozens of program reports for the County.

Some Metropolitan Examples

Chicago Area. There are more than 5,000-miles of combined sewers within a 375-sq. mi. portion of the territory served by the Metropolitan Sanitary District of Greater Chicago (MSDGC), which includes the 220-sq. mi. of the City of Chicago. The District serves more than 5,500,000 people in an 860-sq. mi. area, exercising control over 75-miles of open waterways with respect to drainage, pollution control and navigation, much of which is monitored for water quality.(34) Lake Michigan is the water supply source for the area, which is drained by three rivers away from the lake.

An intergovernmental committee resolved a "Tunnel and Reservoir Plan" for combined sewer overflow pollution abatement(35) that incorporates flood control features. In 1972 the cost of implementing the plan was estimated at 3.3-billion dollars.(36) Currently, the first major phase is under construction. "A unique feature of the plan is its inherent flexibility to take advantage of discoveries made during its ten year construction period."(36) Principal features of the plan have been outlined.(16)

The primary function of the MSDGC is the collection and treatment of the water-borne wastes from the City of Chicago and its suburbs. Because flooding is related to such collection and treatment, the District has initiated long-range planning and local flood plain control. The District has used its authority over the granting of sewer permits to require local authorities to enact flood plain ordinances that safeguard flood plain developments against high water and requires on-site detention of excess runoff from new developments.(37) A streamflow model developed by the District(38) is used in the preparation of watershed master plans. One of the completed plans, for a 52-square mile watershed, calls for measures consisting of: five floodwater retarding structures; a multiple-purpose structure (flood protection and recreation); 261-acres of flood plain preserve; about 1.8-miles of channel modification; and a land treatment program for more than half of the watershed.(37) Participants in this planning, in addition to the MSDGC, were the local Soil and Water Conservation District, the Cook County Forest Preserve District, the State of Illinois, and four villages and four park districts,(37) illustrating the multiple jurisdictions commonly involved in flood control planning. (The MSDGC spearheaded the combined sewer overflow pollution abatement master plan mentioned above, in cooperation with the State of Illinois, Cook County and the City of Chicago(35)).

Denver Area. A local government special district, the Urban Drainage and Flood Control District was established in 1969 to solve growing drainage problems in metropolitan Denver. The District covers about 1,200-square miles of all or part of six counties. A joint project by the Denver Regional Council of Governments and the District, partly supported by the HUD Urban Systems Engineering Demonstration Program, developed procedural methodology for urban drainage and flood control master planning in the region.(39)

Major drainageways were targeted in the methodology development, defined as those streams, creeks and gulches that have definable flood plains. Three types of principal activities to be pursued over two decades are called for under the Denver regional program: (1), preventive master planning for areas where flood plain regulation, land use controls and other preventive actions can be utilized; (2), design master planning for areas where problems already exist and facility construction is known to be required; and (3), construction for developed areas where preventive measures are not feasible and where channels, culverts, sewers and other structures are needed to provide protection.(40) Activities included are: delineation of flood plains on major drainage channels; regulation of all unoccupied and occupied 100-year flood plains; 100-year protection on occupied flood plains; National Flood Insurance Program coverage on occupied flood plains where protection is not economical; the provision for limitation of runoff from new real-estate development by ordinance and State law; flood storage capacity and spillway protection for dams in the region; integration of major drainage measures with the regional water resource management scheme; and a flooding early warning system.(40) The first master plan for a major drainage area using the methodology developed was completed in 1971.(41)

Motivation for preventive master planning was strong. The cost of protection for the quarter of the total District flood plain area already occupied, in 1972 dollars, was estimated at $110-million.(39) Use of a preventive approach for the remaining three-fourths is proving to be comparatively inexpensive.

Urban Studies Program, Corps of Engineers. Congress and the Office of Management and Budget in 1970 expressed an interest in having the Corps of Engineers conduct pilot studies of wastewater management for several major metropolitan areas, to assist metropolitan agencies in their planning efforts. "The initial objective for each study area was to produce a feasibility report that would identify alternative means of reaching very high standards of wastewater quality on a regional basis. A further requirement was that the Corps of Engineers consider land treatment as an alternative to advanced plant treatment systems."(42) Initial feasibility studies were completed in 1971 for five metropolitan areas: Merrimack Basin in New England, Cleveland-Akron, Chicago, Detroit and San Francisco Bay. Detailed studies for these five pilot areas were completed in 1973. By the end of 1974 a total of 37 such planning studies had been authorized, and the number has since increased. Several studies have been completed.

Provided will be a range of urban water resource plans that are compatible with comprehensive urban development goals of the region under study, and these plans will provide "an integrated approach to water resources management".(43)

Features have been summarized of the Chicago(42) and San Francisco(44) area studies, and the growth of interest in the land disposal of wastewater concept has been briefly defined.(16) A significant feature of a study of possibilities for land disposal in the San Francisco Bay region was the exclusion of places where urban development might be anticipated by the year 2000.(45) The important point here is that not only are the effects of local pollution often felt well outside the metropolis of origin, but with land disposal the abatement of pollution via this form of treatment could also take place beyond the metropolis.

Corps responsibilities under the Urban Studies Program can include: urban flood control and flood plain management; municipal and industrial water supply; wastewater management; bank and channel stabilization; lake, ocean and estuarine restoration and protection; recreation management and development at Civil Works projects located in close proximity to urban areas; and regional harbor and waterway development. Urban runoff quantity and quality is addressed in several of the Urban

Studies. Where associated master plans have been adopted by local governments they are treated mostly as givens in the Urban Studies.

Two-Direction Communication and Time Horizons

A 1970 analysis of regional land use and transportation planning drew inferences from case studies of thirteen metropolitan areas. At that time the conventional planning concept was the selection of a metropolitan plan from several complete and integrated development proposals. Proposed as a more viable substitute was a style in common use today, a cyclic use of alternatives. In the cyclic style, "alternatives become a means of exploring and understanding the effects and implications of diverse objectives, assumptions, plans and policies, often in response to a specific problem. ..... flexible and partial alternatives should be prepared and evaluated for several types of plans at appropriate geographic scales, time horizons and levels of detail."(46)

Table 1(46) characterizes the cyclic employment of alternatives in a continuing planning process. The reasons for introducing Table 1 here are to critique the role of local government master plans therein and to remind the reader of the long time horizons required for such plans.

Seemingly overlooked in Table 1 under "Sub-metropolitan Studies" is, in addition to the impact of metropolitan plans, the identification of potential problems at the local level, the input of such signals to the metropolitan phases, and the acknowledgment of these problems in all metropolitan planning. Consequently, the allocation of only ten per cent of the total effort in Table 1 to Sub-metropolitan Studies appears to be too small. On the other hand, the more detailed planning for local development generally fails to address the broader considerations involved in metropolitan planning, with the consequence that there is often an appreciable planning void between the two levels, with no organized or assured effort made to accommodate these considerations. These comments illustrate the common occurrence of a discontinuity between metropolitan and local planning efforts. Needed is not only the interdisciplinary interaction that is becoming more widely practiced, but also a firmer interinstitutional approach. The Consulting Panel on Water Resources Planning of the National Water Commission indicated in 1973 that there was a long way to go, claiming that: "Water-resource planning has rarely been integrated or coordinated with overall urban planning".(47) Section 208 planning has a great potential for closing this gap.

As noted in Section 1, and as illustrated in Table 1, considerable time is required for the resolution of plans. The detailed local government master plans cited in this report have required a minimum of three to five years for their development. Most will require at least ten additional years for their full implementation. Those being implemented have faced some degree of delay for resolution of conflicts, right up to the letting of construction contracts.

TABLE 1

SUMMARY GUIDELINES FOR PLAN COMPONENTS OF CONTINUING PLANNING PROCESS[46]

Plan Type	Purpose	Time Horizon	Level of Detail	Proportion of Effort
Metropolitan Development Framework	general framework for urban development and basis for metropolitan public facility and service system plans and review of local plans and programs	20 years	pattern of urbanization, location and density of activities and layout of facility systems	25%
Facility-Service System Plans	metropolitan system plans and basis for designing the location and characteristics of services and facilities and programming their implementation (transportation, water-sewer, open space, sub-regional centers, education, health, housing)	20 years	sufficiently detailed for preliminary design studies	30%
Capital Improvements Program	coordination of the allocation of fiscal resources at all levels of government and the timing of implementation of projects, both within and among facility systems	10 years	large or composite projects as basis for programming and budgeting by operating agencies	15%
Metropolitan Studies and Alternatives	identification and reconciliation of development objectives, policies and combinations of systems and examination of long-run impact of technological, social and economic change on urban development, as basis for the metropolitan development framework and facility-service system plans	20-50 years	highly generalized to very specific as appropriate to particular purpose	20%
Sub-Metropolitan Studies	impact of plans on the development process and quality of urban environment at the community to project scale	various	detailed to generalized	10%

SECTION 3

INCENTIVES FOR COMPREHENSIVE PLANNING

Synopsis

Emphasized in a 1975 national symposium(48) co-sponsored by the ASCE Urban Water Resources Research Council was that urban water quality control plans should be comprehensive, making provision for and fully taking into account future water needs and withdrawals and recognizing and making provision for appropriate future usage of waterways and adjacent areas as wildlife habitats and for recreation. Recognized was that water supply, flood control, drainage and pollution control are interrelated in urban planning and that independent development of drainage and flood control plans might result in higher costs for achieving water quality objectives.

The thesis of this Section is that rational planning requires conjunctive consideration of the quantity and quality aspects of urban runoff within a comprehensive, multiple-use framework. Some of the more obvious arguments are explored, in connection with viewpoints on land-use controls, disparities in approaches to economic evaluation, potentials for water reuse, opportunities to reduce extraneous sewer flows, control of erosion and sedimentation, and disposal of solids from new joint or ad hoc treatment facilities. Admittedly, difficulties and problems are emphasized, but for the sole purpose of indicating that the only rational way in which a reasonable degree of order can replace possible chaos is probably through highly skilled comprehensive planning and policy coordination at the metropolitan level.

Land-Use Controls

Citizen participation is urged or required in various national laws dealing with urban runoff management. While the public is perceived by various observers as demanding more holistic or overall solutions to problems from its elected officials, there is a countervailing tendency for responsible public groups to single out specific parts of the puzzle for detailed scrutiny and simultaneously to oversimplify the interconnections among closely related issues.

Exemplifying this inconsistent view, the national Citizens' Advisory Committee on Environmental Quality has defined planning as "the conscious selection of policy choices in land use,"(49) whereas a more generally acceptable definition would be that planning is the process of deciding what resources to allocate, over time and space, to achieve a set of specified objectives. Noted by the Committee is that many communities begin the planning process with a comprehensive study that takes into consideration interactions between all present and potential man-made activities and various natural characteristics, resulting in a document called a master plan or comprehensive plan for use in guiding community development. Emphasized by the Committee is the importance of a land-use inventory as "a useful data collection device with which to obtain a fresh view of an area's relevant features. It is a device which will focus attention on the important problems and proposals that are evolving around us — often gradually, without shock to our daily lives — but with a tremendous long-range impact on our future."(49) Water aspects seem to be regarded by the Committee as mostly locational issues in an inventory, as for other natural resources such as flora and fauna and geologic features. By stressing the location of water aspects, the transitory nature of water resources and the intermingling of waters in multiple uses is thereby completely ignored, as opposed to the water orientation to supplement land-use inventories advocated in Addendum 1 of this report. Independent of planning issues, it has been alleged that management of urban water resources has been viewed for the most part independently from land-use policy and management rather than as an integral element.[50] These points are raised in an attempt to indicate some of the challenges confronted by comprehensive planning and the great importance of public information, and even public education, in promoting the larger view.

Attempts by Congress to force a marriage between land and water management are exemplified by 40 CFR Part 131.11 supporting Section 208, P.L. 92-500. Although intended by Congress to exploit the police power of local governments over land use, it is nevertheless expected that in a number of urban areas it may be politically impossible to utilize land-use controls for pollution abatement.[51] For this and other reasons, there are strong local biases towards capital-intensive solutions. One of the other reasons is that nonstructural solutions, which seem to be favored by EPA for nonpoint source control,[52,53] tend to require additional investment in operation and maintenance of affected urban services, and thus would inflate already burgeoning local agency operating budgets. Also, the efficacy of most nonstructural solutions has not been adequately demonstrated with the support of field data. The intents of Congress in Section 208 are said to be the establishment of a land-use regulatory mechanism to achieve water-quality objectives over the longer term, and to provide an incentive and a mechanism for States and local governments to come to grips with nonpoint source pollution.[54] Local governments face difficulties in acceptance of land-use regulatory mechanisms in the absence of reliable demonstration of their effectiveness.

EPA has released in three stages a two-volume manual covering procedures available for water quality management, with particular emphasis on urban stormwater, to assist the conduct of State and areawide planning under Section 208 of P.L. 92-500.[55] The manual notes that the establishment of an overall wastewater management plan calls for examination by State and areawide planning agencies of the wide variety of pollutant sources and corresponding receiving water impacts in terms of the necessity and feasibility of their control.

Existing and future surface water quantity and quality problems are inextricably tied to existing and future land use patterns. For example, because land use significantly influences the volume and rate of runoff from the land surface to stream systems, land use is a primary determinant of the location and severity of stormwater inundation problems and of riverine area flooding problems. The general locations of discharges of wastewater treatment facility effluents to surface water systems are likely to be influenced by overall land-use patterns, and the nature and density of land use will affect the quantity and quality of pollutants received by and passed through such facilities. Similarly, the type and quantity of pollution from diffuse sources and the points of entry of these pollutants into stream systems are also largely determined by existing and future land-use patterns. Engineering and planning studies for amelioration of existing surface water quality and quantity problems and for the avoidance of future problems should be undertaken within the framework of an agreed-upon areawide land-use plan. To do otherwise is to ignore the interdependence of land and water resources.

Because the land-use control issue is so central to the implementation of any urban runoff control scheme, this writer can find no realistic alternative to comprehensive planning and policy coordination at the metropolitan level. Because we have only a handful of areawide, general-purpose metropolitan agencies with operations authority, for nearly all metropolitan areas the baton must go by default to their regional planning agencies.

Disparities in Economic Evaluations

Implementation of plans ultimately hinges on political acceptability, which in turn depends on economic justification. Explored in this subsection are disparities in measures of economic efficiency employed, and again the regional planning agency appears to be the most likely agent for reconciliation.

A review[56] of the report[57] from a symposium sponsored by EPA concluded that, because of the emphasis in the current Federal water pollution control program on waste control at the source, it appears that environmental decisions will be based on tests of cost-effectiveness and noted that EPA is inclined in that direction. Symptomatic is the agency's proposed mandatory requirement for the use of value-engineering in certain wastewater projects.[58]

From a conservationist point of view, there are evaluation hazards in attempting to assign monetary values to what are usually regarded as non-resources, such as endangered animals and plants,[59] which is one of the reasons why a cost-benefit approach finds little popular support as a basis for environmental protection justification.

Such terms as "public welfare," "social well-being" and "quality of life" encountered energetic discussion at a recent international urban water workshop where agreement on terminology appeared to be better for the concept of "the good life" and fairly definitive on basic human and social needs requisite to survival.[60]

Because aesthetically-oriented water uses can in many ways be incompatible with utilitarian uses, conflicts have arisen in multiple-objective water resource management. There has been a definite trend in recent years towards greater legal recognition of aesthetic values associated with natural processes, especially with regard to water, in essentially all areas of the law, including water quality protection, allocation and planning, and authorization of projects.[61] An outstanding example is the environmental impact analysis requirement of the National Environmental Policy Act of 1969.

Because both facts and value judgments are involved, environmental decision analysis is both economic and political in character, conditions which are said to lead to the conclusion that models for environmental planning can be no more definitive than the political processes to which they provide information on the consequences of alternative decisions.[62] Thus, cost-benefit analysis is truly helpful only in handling the monetizable assets and liabilities, the economic effects but not the environmental and social effects, although some of the latter can be assigned subjective market values.

Federal flood control planning proceeds essentially on a river basin basis and uses cost-benefit analysis for project justification. Thus, we find in urban runoff management a tendency towards cost-benefit evaluation for water quantity and a tendency towards least-cost evaluation for water quality. The traditional approach for water project evaluation by local governments has been minimum costs, which should facilitate the water quality side. However, <u>because of the trend towards the blending of river basin planning with metropolitan planning, local agencies will increasingly face the need to reconcile the dichotomy of project evaluation criteria</u>. A problem that local governments have always had with cost-benefit analysis is that, being a test of economic efficiency, identification of neither beneficiaries nor bearers of costs is included, whereas the issues of "who benefits" and "who pays" are central feasibility questions in local government projects.

Disparities in economic bases for project evaluation would seem to suggest rather strongly that planning for management of urban runoff quantity and quality should be conjunctive to avoid absolute confusion. Federal policy with regard to both flood control and runoff pollution abatement in urban areas is biased towards non-structural remedial measures, underlining the need for conjunctive planning. In sum, and to repeat the allegation at the beginning of this subsection, the metropolitan planning agency appears to be the most likely agent for the reconciliation of disparities in approaches to economic evaluation mandated by a tangle of Federal and State laws.

Water Reuse

Water conservation has often been emphasized as one of the advantages of wastewater reuse in urban areas.[63,64] Whereas reuse of reclaimed wastewater for domestic purposes is still controversial, recycling for industrial uses is not. Furthermore, the requirements of P.L. 92-500 for wastewater treatment in the 1980's are expected to result in such stringent controls on the disposal of effluents into receiving waters that recycling via groundwater recharge and augmentation of industrial supplies may become more cost-effective than one-time use.

As local shortages and conflicts over the enlargement of facilities continue to grow, more serious attention to reuse in more places can be expected; but this attention is now only beginning to emerge.

There is considerable internal recycling of process water by industry. Irrigation and cooling water for industry are essentially the exclusive direct reuses of treated municipal wastewater, a practice that is not widespread.[65] Because of its location in a semi-arid region where the bulk of its water must be imported, the City of Los Angeles has been increasingly leaning towards large-scale reclamation for uses such as aquifer injection as a seawater barrier, surface spreading for water supply augmentation, industrial cooling, and park irrigation. In 1973 it appeared that the only instance of a carefully engineered system for water reuse in a major U.S. city was in the wastewater portion of the City's legally adopted master plan. Figure 2[66] shows the general types of reuse that have been considered for Los Angeles.

Los Angeles practices surface spreading of storm and imported water in an area to the north, to replenish water pumped as part of the supply from aquifers normally affected by river waters.[66] The hundreds of recharge basins on Long Island, New York, are very effective in offsetting the effects of urbanization on the groundwater supply, according to water budget studies made of the catchments for three basins.[67] Sooner or later, stormwater will more commonly be reclaimed as a source of water supply, probably mostly through recharge of groundwater aquifers. However, in prior-appropriation doctrine States, legal questions may arise when urban runoff is captured and put to a beneficial use.[68] There are other formidable management obstacles to water reuse and conservation,[69] and groundwater sources are incompletely developed in a number of metropolitan areas.[70]

We can conclude from the observations above that reuse of wastewater treatment plant effluents and reclamation of surface runoff is more likely to be via artificial recharge of groundwater supplies, in general. Where this occurs, the water supply-through-wastewater treatment cycle will interconnect with the stormwater disposal cycle (an interconnection shown in Figure 1 of Addendum 1), particularly for combined sewer systems, through the long-term storage replenishment of groundwater. This eventuality can be provided for only through conjunctive planning.

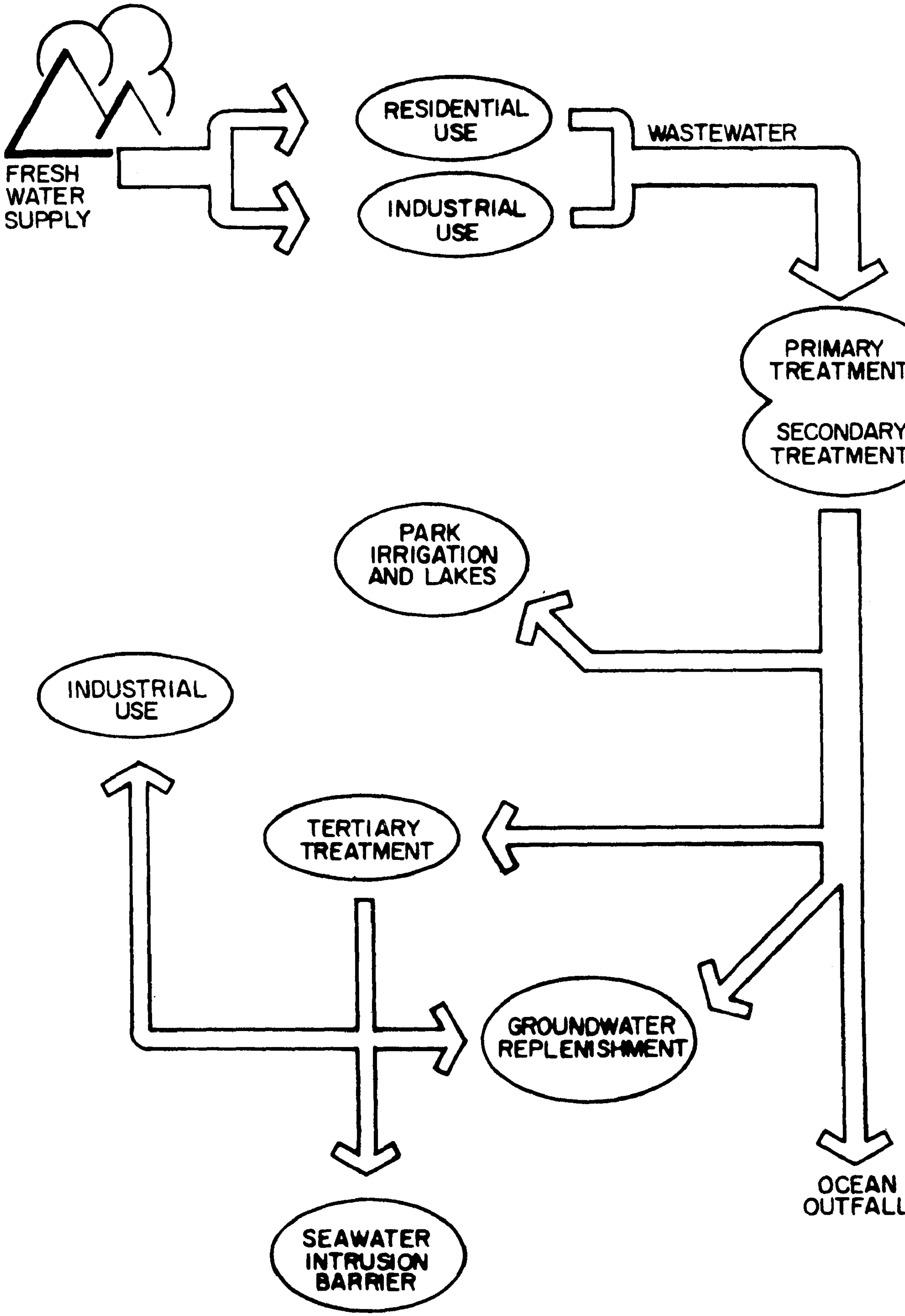


FIGURE 2 - USES FOR RECLAIMED WATER(66)

Considerable capabilities exist for analysis of aquifer performance[71] and for monitoring groundwater pollution.[72] Also, knowledge on groundwater pollution has greatly expanded in recent years,[73] partly because of national concern for control of water pollution and protection of public water supplies.

Extraneous Sewer Flows

Whereas the preceding subsection dealt with beneficial aspects of groundwater manipulation, the entrance of extraneous flows that include leakage of groundwater into sewers is a liability.

Flows of stormwater and infiltrated groundwater into wastewater and combined sewer systems may considerably exceed domestic flows where poor sewer construction and illicit connections are prevalent.[74] Results from a survey of communities in the U.S. and Canada[75] indicated that such infiltration and inflow problems are widespread. More stringent Federal pollution abatement requirements (P.L. 92-500) and related increased investment in wastewater treatment facilities have necessitated consideration of ways to diminish these extraneous sewer flows in order to reduce the capacities of new or enlarged treatment plants. Specifically, applicants for treatment works grants must demonstrate that each sewer system discharging into the treatment works is not subject to excessive infiltration and inflow.[76] It has been calculated that almost $5.3 billion will have to be spent to correct inflow and infiltration problems nationally.[77] Diminishment of extraneous flows would also increase the effective carrying capacity of wastewater sewers, which in some cases would offset future growth in wastewater loads. Under certain circumstances, auxiliary capacity could be added by installing flow-smoothing basins at key locations in a system, as an alternative to the provision of relief sewers.[78]

Considerations for the conduct of infiltration/inflow evaluations have been outlined.[79,80]

The connection between stormwater and wastewater is obvious for combined sewer systems, in terms of burdens of infiltrated groundwater that must pass through wastewater treatment plants. Perhaps less obvious is the fact that the promotion of consolidation or regionalization of treatment facilities to achieve economies of scale may prove to have been a poor policy when such facilities are later called upon to handle attenuated stormwater flows from combined and separate sewer systems from dispersed storages. Thus, schemes for optimizing the regionalization of wastewater treatment plants[81,82] should be modified to accommodate wet-weather flow considerations where their joint treatment at central facilities is projected. Further, while regionalization of wastewater collection and treatment has been encouraged by various Federal and State regulations, diseconomies of scale are encountered with increase in collection system size, and minimum-cost system size is highly sensitive to the density of the population in the area being served.[83]

At a national Symposium on Regional Solutions to Regional Problems held in Portland, Oregon, in 1976, concluded was that because the majority of publicly-owned wastewater treatment works will be committed by the time Section 208 plans are ready for review and implementation, only two questions will still be open by that time, nonpoint sources of pollution and growth management and who pays for growth.[84] Growth management complexities are compounded by the proliferation of restrictive zoning, moratoria on sewer or water connections, building bans and dedication ordinances, adopted as measures to slow growth rates and control the location of development.[85] Although a variety of motivations have spurred these actions, local growth control is viewed as a unifying theme.[86] Some of the antigrowth mood can be laid to the higher costs of unlimited urban sprawl. A study for the U.S. Council on Environmental Quality explored the cost of urban sprawl.[87] On the basis of quantified results from a supporting analysis of energy consumption related to alternative metropolitan futures for the Washington, D.C., area, it was concluded "that the present concern for land management or growth management can be widened to total resource management, including consumption of energy".[88] We can expect increasing pressure for the integration of energy costs into any metropolitan study of urban water resources versus land use.

Erosion and Sedimentation

Clearance of land for urban construction exposes it to rapid erosion.[89] In 1965, Montgomery County, Maryland, adopted the first sediment control program in the nation.[90] A Statewide Sediment Control Law was adopted by the Maryland Legislature in 1970. Every project involving grading requires a permit from either the local government or the State Department of Natural Resources.[91] Each County and municipality serves as the primary unit of government for administration, inspection and enforcement for sediment control, the statewide program utilizes the U.S. Soil Conservation District in each County as technical advisor for erosion and sediment control, and the program is fully integrated with the State's pollution control activities.[92] The statewide sediment control program is gradually being expanded to include storm water management. Preliminary evaluations of "individual site" stormwater detention storage, such as mandated for erosion and sediment control in Maryland, suggest that such dispersed storage might aggravate downstream flooding and that a regional approach to stormwater management might be superior.[93]

<u>Of particular significance here is the fact that erosion and sedimentation control is one of several water quality issues that cannot be divorced from their causative driving force, urban runoff</u>. This is one more reason why conjunctive quantity and quality planning for urban runoff control is needed.

A manual has been developed for use in connection with the master plan for control of flooding and drainage in Fairfax County, Virginia.[94] Some of the features in the manual have been summarized.[95,96]

For the person concerned with erosion and sediment control there are a number of important references available.[2,97-102]

A Metropolitan Issue

Reference is made to Figure 2 in Addendum 1, which shows the annual quantities of water passing through a hypothetical metropolitan area of one-million inhabitants. Total wastewater solids for the million residents would be on the order of 100-tons/day, <u>dry weight</u>, most of which would be removed in conventional wastewater treatment. However, sludge is often transported to disposal sites as a slurry. Costs of transportation and handling can be reduced by slurry volume-reduction through concentration or "thickening" of sludge solids. Based upon the maximum concentration that could be hydraulically transported for the hypothetical metropolis producing 100-tons/day, dry weight, the minimum slurry load would be about 400-tons/day. However, normal practice, which reflects reliability of operation as well as cost-attractiveness, would result in about a 1,000-tons/day slurry for the hypothetical metropolis, or in the neighborhood of half the weight of solid wastes handled per day. As opposed to solid waste disposal, the disposal of wastewater sludge encounters two monumental liabilities: an extremely large water content and a very high organic composition. As a result, the feasible avenues of disposal generally differ from, and often are more restricted than, those for solid wastes. In sum, handling water volumes is only part of the problem, for once solids conveyed by water are removed they must be disposed of somewhere.(69)

A number of major cities, such as New York and Philadelphia, transport wastewater sludge for disposal outside their metropolitan areas. With many of them on a desist notice to comply with new environmental regulations, they are very hard pressed to find acceptable solutions for alternative disposal. Wherever urban runoff is captured for treatment in combined sewer and storm sewer systems in the future, the sludge disposal problem will be magnified. Even if the added annual amounts captured were only about one-tenth to one-quarter more, their episodic and unpredictable accumulation (because of the random nature of storm occurrences) would tax the scheduling and operation of sludge disposal systems that otherwise handle fairly uniform loadings throughout the year.

Conclusions

Explored have been some of the more cogent arguments for comprehensive urban runoff control planning. Additional arguments will surface in succeeding Sections, particularly in the next Section on conjunctive planning for flow management. There appear to be very few local government plans extant that have integrated water quantity management with water quality management. Perhaps the most we can expect, given the institutional constraints on planning in metropolitan areas, is an integrated, or comprehensive, or systems approach; but such an approach is subtle in a documentary sense, and its existence would probably be difficult to detect from the outside. Optimistically, much more rational conjunctive planning may be going on than external indications seem to imply.

As an experienced observer of the local water resources operating agency scene, the writer is impelled to note that any inconsistencies, conflicts and duplications between and among Federal and State laws, policies, guidelines and regulations must be and are, reconciled, arbitrated and resolved mostly at and by the local level of government. For this reason, among several, local governments are forced to define their own problems. Numerous examples abound of dire consequences where this responsibility has been abrogated.

SECTION 4

SOME FLOW MANAGEMENT CONSIDERATIONS

Synopsis

This Section is essentially a continuation of the previous Section on "Incentives for Comprehensive Planning," and has been given separate billing because the issues involved require a comparatively greater exploration.

Integrated consideration of convenience drainage systems and urban flood plains has long been recognized as a logical and efficient basis for their management. More recently, the logic of integrated or conjunctive consideration of runoff quantity and quality has become widely apparent. Impeding these ideals is a fractionalized authority in nearly all metropolitan areas, with responsibilities for drainage, flood control and their associated water quality aspects splintered among numerous units of local government. Additionally, in many cases there are State and Federal agencies exercising specialized authority. Given this institutional patchwork, about the only avenue for conjunctive consideration of these otherwise disparate issues is via comprehensive planning at the metropolitan level. While institutional reform has had strident advocates for decades there have been only rare indications of wholesale change.

Asserted in this Section is that the guiding principle should be to reduce the liabilities and to increase the assets of urban runoff. Some of the more promising ideas for accomplishing this are cited. However, the reader is cautioned that very few of these concepts have been tested in full-scale system applications, the research on which many of them are based is fragmented and the findings cannot be sufficiently generalized for universal application. Here, as in later Sections of this report, local governments are advised that they are mostly on their own in the conduct of the research needed to rationalize fiscal commitments for improved resource management.

Drainage Versus Flood Control

Geohydrologic processes have formed natural drainage channels that convey storm waters to the seas. Because of an early dependence by commerce on water transport, most large metropolitan areas originated as urban centers on or near streams, lakes, estuaries or seacoasts. Intrusion of urban development on natural flood plains has resulted in damages to occupying structures and sometimes loss of life. Subterranean systems of conduits facilitate human occupancy by draining sheet-flow runoff from the land surface. That is, fluvial drainage areas contributing to urban flood-plain inundation are often of gigantic size compared with individual underground conduit catchment areas which rarely exceed 10-square miles in extent. Thus, underground drainage systems are more of a convenience or amenity than a preserver of public safety. Structural means for mitigating flood-plain inundation are designed to provide a much higher level of protection than that for storm drainage systems because of the much greater threat to human life and more apparent community-wide economic implications.

The function of underground drainage conduits is to remove storm water from urban surfaces (except combined sewers, which in addition convey wastewater on a perennial basis). The smallest catchment area (on the order of an acre in size) is that tributary to a street inlet. Flow in storm and combined sewer systems is principally by gravity. Like natural drainage basins, smaller sewer branches unite with larger branches, and so on, until a main sewer is reached. Thus, a main sewer not only transmits upper reach flow to a receiving watercourse but serves as a collector of surface runoff all along its route.

Whereas there is a continuum between the subsystems of water supply, water use and wastewater reclamation (Figure 1, Addendum 1), stormwater has been historically regarded as purely a negative good or nuisance and its subsystem (Figure 4, Addendum 1) has seldom been deliberately connected to the other urban water subsystems.

Urban drainage facilities are generally owned, operated and maintained by local governments, and designed and constructed by local governments and private land developers. Human life is seldom threatened by the flooding of these facilities. Because the principal local detrimental effects of flooding are damage to the below-ground sections of buildings and hindrance of traffic, the consequences of flooding range from clearly assessable property destruction to annoying inconvenience. It follows that provision of complete protection from flooding can only rarely be justified. Instead, facilities are designed which will be overtaxed infrequently. However, because of the marginal level of protection afforded, storm drainage flooding damages are also of considerable magnitude, probably equalling those in urban flood plains. In addition, indirect damages from local drainage flooding are much more extensive than for stream flooding and generally recur more often, and direct damages are usually much more widely dispersed throughout a community.

Because urban drainage and flood plains usually are the responsibility of separate and different jurisdictions, their interconnected behavior is the responsibility of neither type of organization. Thus we find storm sewers designed for common storm events without an investigation of their expected performance during rare events, and flood plain improvements selected on the basis of rare events but with little investigation of flood damages during more common events. This dichotomy is epitomized by a reliance on local detention storage for common storm events and the unexplored discounting of the possible effects of such storage on receiving waters during rare events.

The safety of people and properties occupying flood-prone areas is the concern of every level of government, and Federal, State, special districts and other local government units are involved in the development of an ever-greater arsenal of remedial or mitigative measures and policies. While storm drainage facilities and land use regulation are provided by local governments, development of most major stream flood-management works is undertaken by national agencies.

Flood control, drainage and the quality of receiving waters are all closely related. (Refer to Section 3, "Incentives for Comprehensive Planning," for a review of current trends). Frequently overlooked is the fact that precipitation cleanses the land surface. However, because pollutants together with aesthetically objectionable materials are washed off the land and transported to receiving waters in runoff, the result is merely a transfer of land surface pollution to water pollution despite the benefits accruing to the land. Considering that urbanization increases the rates and volumes of runoff delivered locally to receiving waters, it is evident that the conveniences of surface cleansing and land drainage are obtained at the expense of higher stages and greater pollutant burdens in receiving waters.

Flood Aspects

One out of six acres of urbanized areas is in the 100-year natural flood plain. There are about 20,000 flood-prone communities and some 16,500 square miles of urban flood plains in the nation, an area equivalent to the States of Maryland and Hawaii combined or Massachusetts and New Jersey combined. More than one-half of the nation's flood plains in urbanized areas have been developed, an area of 8,800 square miles or more than the entire State of Massachusetts. These estimates, based on a 1973 evaluation,[103] appear in the proceedings of a 1976 seminar on nonstructural flood plain management measures.[104]

A supporting study[105] indicated that slightly more than half of all federal investments in structural flood control works for flood damage prevention can be credited towards flood protection or reduction of flood damage potential in urban areas. While average annual urban area stream flooding damages are not known, the U.S. Water Resources Council estimated the rural-urban total at more than $1.7-billion for 1966, of which the urban portion probably would not be much more than $1-billion. In contrast, the American Public Works Association estimated 1966 average annual flooding damages in sewered systems as being in excess of $1-billion.[106] The major portions of urban areas are served by underground systems of storm drainage. Thus, somewhere around half of all urban annual flooding damages occur on the large segment of urban land outside of 100-year flood plains, a matter of considerable importance to the National Flood Insurance Program.

Conjunctive Planning

There was a time when stream flood plains and sewered drainage systems could be considered separately, and when urban runoff could be regarded as merely an adjunct consideration in land-use planning. New public demands and policies require widespread use of true comprehensive planning, which includes integrated land and water management. The guiding principle should be to reduce the liabilities and increase the assets of urban runoff.[107] The National Flood Insurance Program has been supported by investigations and analyses concerned mostly with stream flood-plains. It is to be expected that the scope will greatly enlarge as more communities become eligible for insurance against storm sewer flood damages. There will be severe difficulties, alone, in defining what is "improper" drainage or "inadequate" storm sewers. Determination of criteria for defining "construction and land use practices that will reduce flooding" will be quite difficult for several reasons: (1), techniques for hydrologic analysis of rivers and urban streams are at a stage of verification far beyond that for local drainage, mostly because of a much broader data base for the former; (2), designs by municipalities, developers and consulting firms, and tools of analysis developed by universities and other research organizations, have predominantly emerged as an assortment of unconnected, independent products; (3), the potential transferability of innovative plans and designs has been drastically impaired because a range of alternatives was very rarely explored in any given case and a bias of specialized site conditions was generally suspected; and (4), new requirements for protecting the quality of water supplies and for abating the pollution of water bodies add a complex dimension rarely considered in past development projects.

All the basic engineering methods for in-stream flood regulation are applicable to flood mitigation works within local urban catchments, but generally on a smaller scale: acceleration of flood flows by canalization through threatened reaches with consequent reduction in stages; isolation of contiguous land in threatened reaches from flood flows by means of embankments, such as levees and flood walls; and the attenuation of flood peaks by means of storage, located either upstream from threatened reaches or at lateral points fed by diverted flows. Similarly, possibilities also exist for social-economic-administrative-legal controls to preclude some degree of damages, such as flood-plain zoning, flood-proofing of structures, flood insurance and related schemes.

Increased volumes of direct runoff from underground drainage conduits clearly can aggravate flooding of urban flood plains. On the other hand, increased receiving-stream stages can cause or induce flooding of underground drainage systems, because of the intimate hydraulic linkage between them. Extensive programs of integrated flood plain management are particularly crucial for metropolitan areas that have little topographic relief, such as greater Chicago.(108) There are large cities that early dedicated most of their flood plains over to parkways, such as Philadelphia, or developed extensive systems of major drainage channels in step with urban development, such as Los Angeles.(109,110) Regardless, new suburbs have more than occasionally aggravated flooding with subsequent induced damages in many major cities located on large streams or bodies of water because their low topographical elevation makes them hydrologically subservient.

It has been suggested that for a growing metropolitan area the thrust of drainage and flood control solutions should be in two basic directions: preventive activities in the form of flood plain management together with good planning; and remedial actions where flood plains have been improperly occupied and developed and where local drainage problems have not been adequately considered and handled.(111) Preventive activities and remedial actions can involve both structural and non-structural considerations. Structural components include storm sewers, inlets, curbs and gutters, culverts, and channelization and detention facilities; and non-structural activities include flood plain management, flood plain warning and flood insurance.(111)

Environmental corridors in southeastern Wisconsin encompass about 18 per cent of the total area served by the regional planning commission, 486 out of 2,700 square miles.(112) Asserted is that riverine areas should be viewed as environmental corridors managed in such a way as to minimize flood problems and simultaneously satisfy a variety of noneconomic human needs.(113) In concept, an environmental corridor is essentially a continuous linear pattern in the landscape consisting of a composite of natural resource and natural resource-related elements that are important for maintenance of overall environmental quality. This concept is partly founded on the demonstrated unsuitability of riverine areas for urban development, and favors, instead, multiple-purpose floodland management in terms of flood damage mitigation and environmental corridor protection. From this perspective, it is impossible to conduct planning separately for streamflow quantity and quality.

Considerations in water-oriented amenities in the urban environment have been evaluated and tested in an analysis that included case studies of: Harrisburg, Pennsylvania; Waterloo, Iowa; Houston and San Antonio, Texas; Sacramento, California; Minneapolis-St. Paul, Minnesota; and the two new towns of Flower Mound, Texas, and Columbia, Maryland.(114)

Detention Storage

Discharges from conventional storm drainage sewer facilities and flood-plain intrusion by structures both tend to aggravate flooding, and thereby jointly tend to raise the potential for stream flooding damages. Revising storm sewering criteria, such as by including much more in-system storage, can be an effective adjunct in flood plain management. While there is universal agreement that the planning and development of drainage systems and flood-plain management programs should be coordinated and integrated, and there is evidence that such efforts are increasing, prospects for accommodation may tend to diminish rather than improve because of an increasing concern over water quality considerations in sewered systems and a contemporary neutrality or indifference on water quality matters by some agencies dealing predominantly with flood plains. Often overlooked is that much of the flood-plain flooding problem as well as the land runoff water quality problem could possibly be more effectively countered on the land feeding urban watercourses.

While the advantages of local detention storage in lieu of the traditional rapid removal of storm flows have long been recognized,[115] such storage has been only occasionally employed as part of overall flood mitigation, as in Denver[116] and Fairfax County (Section 2). However, specific projects attempting to derive general storage-requirement parameters have heretofore been necessarily site-specific, insensitive to storm actualities, limited to one or very few storage options, and usually have ignored water quality considerations. For example, while an excellent study in its own right, the research by North American Rockwell[117] typifies these limitations.

Findings from a general study of stormwater detention usage have been reported.[118-120] Documented in another study were management techniques in the use of detention storage.[121] While the principle of the use of local detention storage is often championed in lieu of main channel improvements for metropolitan areas, sharp differences of opinion have been known to arise, partly or perhaps principally because implementation of the two methods commonly falls within different jurisdictions of authority.

There is evidence which indicates that the effects of urbanization on the magnitudes of streamflows may decrease as rarer floods are approached,[122] to the extent of a difference that may be undetectable or at least relatively insignificant at the 100-year level. That is, the effects of land use changes due to urban development are more pronounced the more common the occurrence. Underground conduit systems for urban drainage and adjunct detention storage are deliberately designed to be overloaded rather often (such as once in 5 years, on the average), whereas waterways flooding protection is usually much greater. Because storm sewers drain to waterways, they are obviously connected and their performance is related. More attention should be accorded the interrelations between land runoff and streamflow, over a range of occurrence frequencies, in a conjunctive planning sense. For example, plans for upstream local detention storage for flow attenuation and/or pollution control might be found to aggravate unduly streamflow rates and/or pollutant burdens in a given instance. A study in the Atlanta, Georgia, area indicated that small detention storage basins are effective in holding runoff from newly developed areas to their former peaks under natural conditions but become progressively less effective with increasing watershed size.[123]

A computer-based procedure for application in comprehensive flood plain information studies[124] is in advanced stages of development. Accommodated are land use changes and interrelations between conduit systems, detention storages and streamflows from the standpoint of both quantity and quality.

Through use of simulation, it is feasible to analyze the influence of alternative urban development on stream hydraulics and to assess expected flood damages. Elucidation of incremental increases in average annual flood damages are an effective means for demonstrating flood damage effects of arbitrary or indiscriminant urban development.[125] Also, there are ways to analyze via simulation the influence of alternative urban development on storm water quality.[55]

Whereas cost information on local detention storage has not been comprehensively assembled, a manual is available for estimating construction and O & M costs for combined sewer overflow storage and treatment.[126] Also, some useful data has been assembled on costs of traditional drainage[127] and on storage capacities.[128]

Urban lakes have been a subject of concern to the U.S. Geological Survey[129,130] and the American Geophysical Union.[131]

Other Measures

New national urban planning priorities include: enhancement of urban environments; conservation of water resources; and reduction in water pollution. These new priorities require use of stormwater management practices that are much more comprehensive and complex than generally used at present. Research on desirable practices has been fragmented and diffused, a severe handicap in making deliberate efforts to encourage abandonment of historical practice, where urban settlements have been drained by underground systems of sewers that were intentionally designed to remove stormwater as rapidly as possible from occupied areas, with little regard for resultant receiving-stream flooding and ignorance or oversight of lost environmental enhancement opportunities.

There are at least four basic methods for reducing sewered system flood peaks and controlling runoff pollution: (1), retardation of flows by induced friction; (2), control of flow paths and gradients by grading; (3), induced infiltration of stormwater into the ground; and (4), provision of detention or retention storage. All of these are more effective the nearer they are to the sources (individual properties) under the conditions for which they are designed. However, as noted in subsections above, during rarer runoff events the benefits of such means may decline substantially by the time flows reach downstream waterways and other receiving bodies of water.

Alternatives have been conceptualized that would cause a reduction in quantity or improvement in quality of urban runoff, yet could be implemented by investment in other than the construction of major new facilities. Among the possibilities are the following:[132]

Source Control Alternatives
- Roof storage
- Ponding
- Porous pavements
- Erosion control
- Street cleaning
- Deicing methods
- Utilization of natural drainage features

Collection System Control Alternatives
- Sewer flushing
- Inflow/infiltration
- Sewer cleaning
- Polymer injection
- In-line storage
- Remote monitoring and control.

These and other alternatives are discussed in an EPA report.[133] A "desktop" procedure is available for the comparison of selected alternative control technologies for stormwater management.[134]

"Urban hydrology is one field which has failed to attract major Federal support in spite of strenuous efforts of professional groups such as the Urban Water Resources Research Council, ASCE. The development of acceptable ways to measure water quality in urban streams and storm sewers has been even more neglected. The Federal data-gathering program has concentrated on the larger streams, presumably because of their traditional use in formulating basin plans. There has been a tendency to leave urban data-gathering to the municipalities themselves. Even so, Federal research funds have been available only to a very limited extent."(135)

Thus, the reader is warned that although numerous schemes have been postulated for controlling the quantity and quality of urban runoff, very few of these concepts have been tested in full-scale system applications, the research on which many of them are based is fragmented and the findings cannot be sufficiently generalized for universal application. By default, local governments are obliged to assure themselves of the relevance and reliability of most such schemes before making a commitment to their wholesale use. One way to investigate expected performance is the employment of hydrological simulation methods founded on local field data, the subjects of the remainder of this report.

Miscellaneous

A very useful compilation of 800 references has been prepared that emphasizes nonstructural measures for flood damage abatement, flood insurance and floodproofing.(136) Citations span 1928-1976.

A study of a 29-acre urban catchment in West Lafayette, Indiana, investigated the economic and environmental impact of alternative drainage systems for varying land use and population density.(137) Drainage alternatives included different levels and kinds of runoff treatment plants and either open channel or closed conduit drainage, but consideration of detention storage was not included. However, the techniques for cost evaluation of single-family versus multiple-dwelling versus commercial development are potentially useful for applications elsewhere.

Several drainage ordinances, including those in Fairfax County and metropolitan Chicago, have been examined in terms of the effectiveness of their specific provisions and the problems encountered in administering them, and a model drainage ordinance based on the survey was promulgated.(138)

SECTION 5

UTILIZING SIMULATION

Synopsis

While simulation is undoubtedly an effective means for analyzing alternative urban runoff control strategies, its most important use can be in the assessment of expected system performance. Local government officials must be able to answer pointed voter questions on the exactness of cost estimates versus the degree of confidence that may be placed on proposed facilities and management programs to meet specified abatement goals. Performance reliability is ultimately the fulcrum of political acceptability.

This Section commences with reasons for using simulation and a historical reckoning of practical reservations against incautious acceptance of runoff models. Emphasized is that simulation techniques adopted should not exceed the level of mastery of such tools by the user and that tools should be selected on the basis of their suitability for solving defined problems.

Performance reliability is defined in terms of four serially connected principal considerations, which must be completed in the following order to be fully effective: testing simulation models against local field data; using a long-term precipitation record as a reference; developing frequency rationales for levels of protection; and simulating a range of storms and system loadings. The first two of these four steps are more or less inconveniences that must be accepted to get to the threshold of reliability required for undertaking the latter two steps where the true payoff resides.

An emphasis is placed on the testing of simulation models against local field data because this issue is the crux of credibility as well as reliability. As things now stand, local governments are substantially on their own for the acquisition of field data. The concept of spatial sampling of catchments for field instrumentation having representative land uses is outlined. Stressed is the importance of rapid application of data to models, to check data reliability but principally for expeditious calibration of models, the primary use of such data in planning.

Control of flooding and water pollution must be based on probabilities of occurrence because of the randomness of precipitation. Rationales are developed for using long-term rainfall records as the reference for defining urban runoff quality and quantity control objectives in probabilistic terms. (Reasons for using actual records rather than synthetic storms are reviewed at length in Addendum 2).

Reference is made to indications in other Sections of a history of moving targets for flood management and pollution abatement over the past decade and the likelihood that policy targets will continue to shift over the next several years. Underscored in this Section is that the only realistic defense for planning in an atmosphere of ambiguous policy is to employ procedures the results of which have an inherent flexibility for conversion to meet alternative policy goals.

This Section closes with a discussion of what was termed "simulate performance for alternates" in Figure 1, Section 2. In runoff control planning, simulation is mostly for future or extrapolated conditions. Reliability in the employment of calibrated tools for such extrapolations is a function of several things, but the most important are probably the shrewdness of design of the field data network and the extent to which its results approach an ideal set of representative samples.

Most of the written record on simulation has been authored by modelers and data collectors and reflects their research orientation and interest. Details over which these specialists fret and expostulate are of only passing interest to persons responsible for developing comprehensive plans within limited budgets and rigid schedules. Therefore, an attempt has been made in this Section to dwell only on fundamental simulation issues in planning and to sidestep niceties that are the grist of the researcher's mill. Thus, some readers concerned with applications may not necessarily appreciate the full reasons for some of the trenchant assertiveness of this Section.

Why Simulation?

The name of the game is system performance. Readers who are disinterested in the reliability of performance of plans for projected drainage can skip the remainder of this report.

All but a small fraction of storm sewers here and elsewhere in the world have been sized by means of wholly empirical methods. Given a lack of evidence of superior methods, these overly simplistic procedures proved adequate when the primary purpose of storm sewers was to drain the land and express the accelerated convergence of surface runoff to receiving waters. Out of sight, out of mind. Once restrainment or containment of flows and their pollutant burdens become added primary objectives, traditional procedures of analysis are no longer adequate because of added system complexities for which conventional tools are unsuited.

Why not use observed discharge variations as a guide? There are several compelling reasons precluding this possibility: (1), very few urban catchments, particularly sewered ones, have been gaged; (2), a statistical approach requires a period of record spanning at least ten years, substantial physical changes commonly take place on most urban catchments over this long a time, and the mixed statistical series that results is not interpretable; (3), while such a statistical series would characterize the existing situation, there would be substantial uncertainty over its extension to differing future situations; and (4), the clinching reason, in the usual case where no field measurements have been made, is that it would be necessary to postpone planning and analysis until new long-term field records were accumulated, an unacceptable option under contemporary imperatives. An even less acceptable alternative would be to rely solely on empirical tools and determine prototype system performance after system changes had been instituted, a procedure that would indicate the overall errors implicit in the tools used, but would be very expensive experimentation.

Thus, in order to anticipate future system performance under changed conditions, because these changes can very rarely be simulated by manipulating prototype systems, recourse must be made to performance simulation by calculation or analogy. Once having accepted simulation as the means for analyzing future performance, the central issue remaining is the degree of reliability of system performance that is desirable, under what conditions and at what costs, the central theme of this Section.

Background

Hydrology may be defined as the science that is concerned with the waters of the earth, their occurrence, circulation and distribution, chemical and physical properties, and their reaction with the environment, including their relationship to living things. Thus, hydrology embraces the full history of water on the earth.[139]

Because the complex interactions of human activity in concentrated settlements with air, water and land must be taken into account, urban hydrology is a distinctive branch of the broad field of hydrology. As opposed to conventional hydrology, because urban development everywhere has been in continuous states of expansion and flux, urban hydrology contends with the dimension of dynamic change. Also, urban water resources management utilizes the social and biological sciences as well as the physical sciences. Reflecting the lag in the recognition of the fact that America became an urban nation over half a century ago, the term urban hydrology gained currency less than two decades ago. Since then, the term has been tacitly expanded to include all urban water resource matters in, or interfacing with, the hydrologic cycle, including water quality considerations.

Whereas generic hydrology is termed a geoscience, engineering hydrology is strictly an applied science. There are no truly fundamental principles exclusive to engineering hydrology, and this is particularly true of urban hydrology. Fundamental laws of soil-water movement are borrowed from soil physics and fundamental equations of water motion are adapted from hydrodynamics. For example, the principle of conservation of mass is applicable to all kinds of systems, not merely the water cycle. It is important to recognize that urban hydrology is a complex art, but an art all the same, because otherwise a more faithful replication of system performance would be expected from simulation than warranted by the attainable perfection of the art.

Without question, the advent and rapid evolution of electronic computation has accelerated development of tools of analysis in urban hydrology as in every other field. Advances are clearly evident in a recent review.[140] Perhaps more obvious advances have been made in water supply and receiving water hydrology simply because these are essentially suprametropolitan. Another factor is undoubtedly the fact that they have benefited from advances in traditional hydrology that came about because of national and State imperatives. Federal legislation on water pollution, flood insurance and environmental protection, has resulted in a great intensification of urban water model development over the last few years, with sewered catchment models perhaps very recently eclipsing urban receiving-water model development.[141]

Relatively few runoff-quality field gagings in sewered catchments have been made, and these have been mostly at outfalls. Source quality has been investigated principally as a function of street surface pollutants accumulated between rainfalls. In order to accommodate cause-effect relationships required for modeling, it is current practice to estimate potential street loadings, separately for individual parameters, on the basis of the few documented solids-accumulation histories. Arbitrary allowances are then added to account for off-street contaminant accumulations, expressed as multiples of the potential street loadings. Thus, no direct verification of the hypothesized buildup of pollutants and their transport to receiving waters is presently available. It is reasoned that when "pollutographs" generated by models reasonably approximate field observations for a catchment, that the overall accumulation and transport hypothesis is validated. As a result, it might be concluded that model development has already greatly outstripped the data base for model validation, in the sense of bracketing probable reliability. However, if field research and model testing continue at anywhere near the level of activity of the past decade, substantial advances in reliability appear to be an inevitable result.

Or, as summarized in a recent national conference on urban runoff: "When we consider the length of time that studies have been made of treating sanitary wastes and the principal industrial wastes, or of hydrology in general, it is apparent that water quality analysis of urban runoff is a relative newcomer and a neglected field".[142] Much more also has to be learned on the quantitative aspects of urban runoff.

A recent appraisal of future modeling needs[140] revealed that, from the standpoint of runoff quality simulation, progress is stymied principally by a lack of sufficient field data to evaluate thoroughly the quality segments of various models, let alone to evolve improvements.

A plan for a national program for acquisition and analysis of field data from urban sewered catchments was designed by ASCE in 1969.[143] Despite the best of intentions,[144] only a token portion of the proposed, critical, field gaging portion has been implementable. <u>As things now stand, local governments are substantially on their own for the acquisition of field data and no integrated or national program exists or is in sight</u>.

The fact that three-fourths of our people live in metropolitan America continually escapes the attention of framers and implementers of national policy. In the meanwhile, fractionalized, largely independent, fretful but perhaps impressive progress is being made in urban hydrology research, and accelerating planning activities nationwide imply even greater attention in the immediate future.

Acknowledging that significant advances have been made in many aspects of urban water resources, research on drainage has long been neglected, particularly with regard to field data acquisition, and information needs are growing faster than new knowledge is accumulating. This neglect can be traced to the continuing ambivalence of the Federal government on its role in metropolitan areas, an inheritance from our colonial past. One view holds that a national program of drainage research would require the least public expenditure, because the primary objective would be transferability of findings. However, the prevailing view is that each of the more than 10,000 local units of government in metropolitan areas involved with stormwater management should "do their own thing" in such research. Seemingly overlooked is the fact that national investment in urban facilities has already constituted seven-tenths of total investments in all water resource facilities, with its share still growing. While it is of small comfort, the U.S. is far from unique in this regard because the most urbanized countries still exhibit the rural origins of their institutions, and urban water resources research around the world commonly has suffered from inadequate attention and support and from discontinuous and erratic efforts.

Against this historical perspective is a viewpoint that deserves quoting: "There does not seem to be a 'perfect' model for analysis of stormwater. The models are either too complicated, do not allow for distributed inputs and parameters, do not simulate continuous streamflow, or have not been tested extensively on hydrologic data. ..... There remains much uncertainty in stormwater modeling. There appear to be enough parametric models available which have been shown to be feasible conceptualizations of the stormwater runoff process. What is needed now is a continued and accelerated verification of the existing models and a follow-up regionalization of the parameters."[22] All this will take some time. What can be done in individual metropolitan areas in the meanwhile is to be discussed in the remainder of this Section.

Performance Reliability

Users of the newer simulation models are confronted with a seeming dilemma. While the risk of inadequate representation of prototype systems decreases as more flexible models are used, the complexity of the models necessarily increases at the same time, as does the difficulty in obtaining an acceptable simulation of expected performance. A recommended rule is "the premise that the simplest model that will do the job is usually the best".[132] Further, the more complicated the model the greater the demands on the user for a mastery of complex hydrodynamic and biochemical relationships. Indeed, the technological sophistication of some urban runoff models is as great as in any water resources application. This may be seen by reviewing the process details of modern models.[22] There is evidence of simulation failures that have been attributed to model inadequacies where the blame more properly belonged to improper handling by the user because of insufficient comprehension of the complex processes involved. Thus, the first cardinal rule is that simulation techniques adopted should not exceed the level of mastery of such tools by the user. Putting it in a more pertinent way, the range of simulation possibilities can be constrained by the level of skills of the user. The finesse and mastery of the technical personnel involved directly in the simulations can therefore be a major contributing influence on the reliability of system performance that can be attained.

A second cardinal rule is that tools should be selected on the basis of their suitability for solving defined problems. This is to say that what is wanted from simulation should be defined first, and the selection of techniques should follow, not lead, this decision. Models may be useful as tools in solving problems but they have no inherent capability for defining them. Further, there being no single or universal model, a stable of tools will be involved over the course of comprehensive planning, starting with preliminary screening models through post-implementation operations models. It should be evident that model needs should be projected through the total planning epoch, to assure compatibility (at least for data required) and to avoid extravagant costs and almost irreversible commitments to particular data processing schemes. Also, the package of tools used should permit the incorporation of expected improved versions because such changes have accumulated in the past in the midst of comprehensive plan evolvements.

We now turn to other principal considerations affecting the reliability of projected prototype system performance. Four of these principal elements are itemized in Figure 3:

- Testing simulation models against local field data. Without some adjustment of model factors to reflect local peculiarities, whatever they may be, any model is necessarily under a cloud of suspicion. This item is included in Figure 1, Section 2.
- Using long-term precipitation record as reference. Water quality and quantity control objectives must be defined in probabilistic terms, for which this is usually the only way open. This item is included in Figure 1, Section 2.
- Developing frequency rationales for levels of protection. Flood levels and receiving water quality goals must be expressed in several ways, such as frequency of events or degrees of protection, as implied in Figure 1, Section 2.

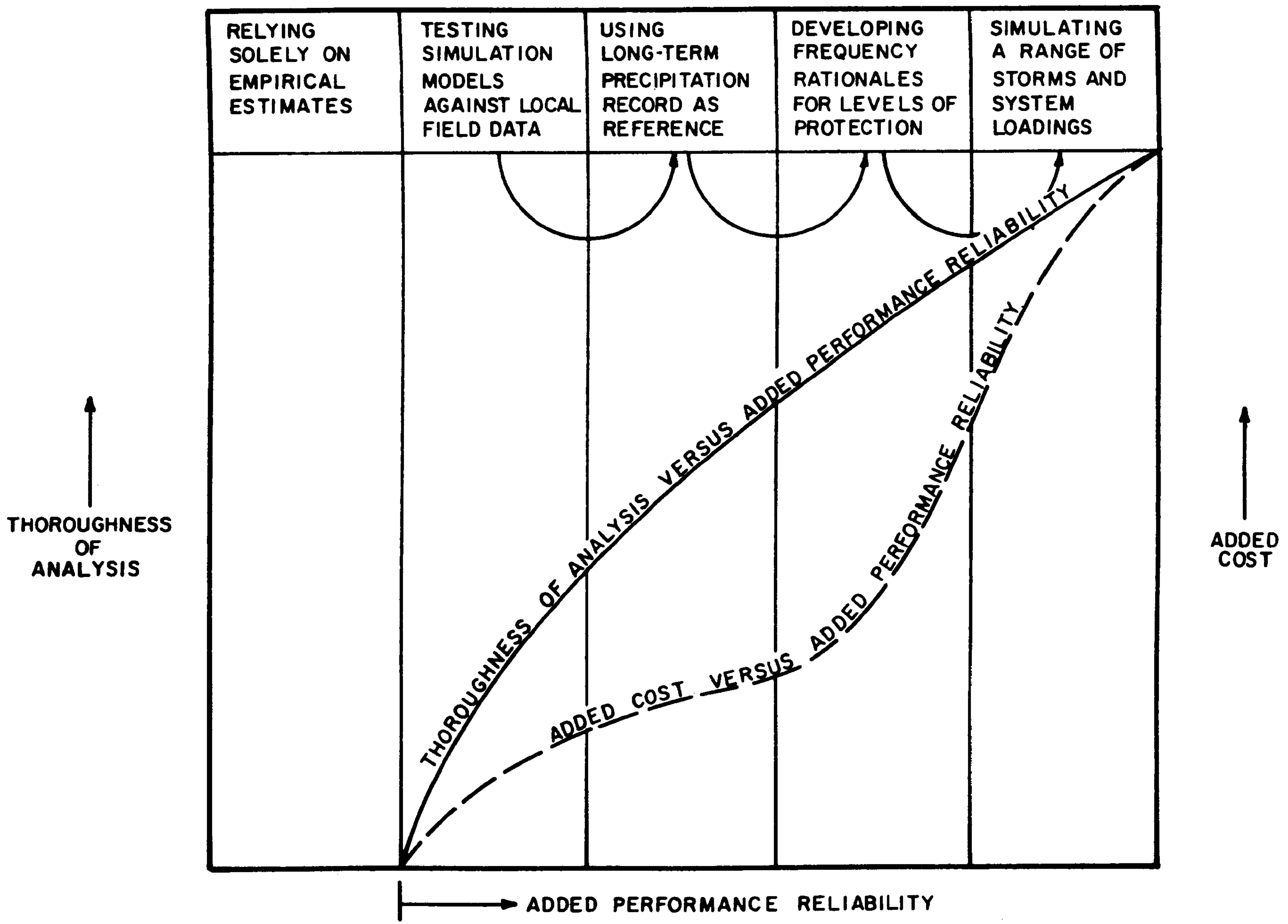


FIGURE 3- SUBJECTIVE ESTIMATE OF INCREASED RELIABILITY AND ASSOCIATED COSTS FOR MORE THOROUGH ANALYSIS

. Simulating a range of storms and system loadings. This, too, is implied in Figure 1. Unless a system is operated (albeit via surrogate simulation) over a reasonable range of expected conditions, how can we be assured it will perform as we expect and under what circumstances may it be expected to be "overloaded"?

These elements are discussed at greater length below. However, before leaving Figure 3 there are important points that should be made. First, it should be noted that the four components are serially connected. That is, a "range of loadings" depends on the "frequency rationales" developed, which depends on reference to a "long-term precipitation record," all of which hinges for collective reliability on prior "testing of the simulation models" that are used against local field data. Second, as graphed in Figure 3, the first two elements probably provide a greater payoff relative to their cost than the other two, although the latter are the really important ones from the standpoint of comprehensive planning. That is, the first two are more or less inconveniences that must be accepted to get to the threshold of reliability required for undertaking the latter two steps. Figure 3 is a purely subjective estimate by the writer, made in an attempt to hypothesize a nationally applicable representation. It will be some time before supporting evidence accumulates. There does not appear to be a suitable way to accommodate in Figure 3 the question of the use of tools with complexities compatible with master plan objectives, discussed at the beginning of this subsection.

Testing Simulation Models

In the analysis of water distribution systems, the first major objective, prior to the investigation of requirements for the future, is the attainment of satisfactory simulation of existing system performance. System simulation is considered validated during preliminary analysis for design when calculated pressures are satisfactorily close to observed field-gage readings for given field water source send-out and storage conditions. "The simulation capability for the existing system having been verified, analysis of the system or district is pursued for conditions expected within the next 5, 10, 25 and possibly 50 years."(145) Water distribution system simulation is singled out to illustrate the fundamental planning principle of regarding present conditions as prologues or points of reference for the future. Good information on current actualities is needed to proceed confidently with extrapolations, where the degree of reliability of projections diminishes as estimates move into more remote time-frames.

Returning to urban catchments, the "systems" involved become considerably more complicated. Even if there are parallel similarities between drainage conduits and water distribution piping, stormwater catchments are characterized by their large number and variegated land use characteristics, and to this is added the complex performance interaction of underground conduits with receiving channels, streams and other water bodies. Thus the calibration of simulation model variables is much more involved. It is important that the field data used be for catchments "representative" of prevailing land uses. In principle, identification of candidate catchments would be on the basis of an inventory of existing catchments in the metropolitan area, classifying them by predominant land use, size of area, extent of storm sewer drainage, and location of outfall relative to receiving waters. Detailed guidelines for the selection, instrumentation and processing of data of catchments have been presented recently.(146)

Table 2(147) lists the principal physical characteristics of the nine catchments presently being gaged in the City of Philadelphia to obtain regional model calibration data. Note that uniformity of land use is deliberately high for

TABLE 2

PHYSICAL CHARACTERISTICS OF REPRESENTATIVE CATCHMENTS BEING GAGED IN PHILADELPHIA[147]

PREDOMINANT LAND USE	CATCHMENT NAME	UNIFORMITY OF LAND USE	SEWERAGE	SIZE (Acres)	IMPERVIOUSNESS (Per Cent)	DWELLING UNITS (No./Acre)
<u>Residential</u>						
Single Houses	Linden	86	Separate	145	43	1.4
	Overbrook	88	Combined	32	31	3.4
Semi-Detached	Tustin	100	Separate	82	63	10.9
	Whitaker	81	Combined	120	68	8.4
Row Houses	Large	98	Combined	74	84	16.0
	Mifflin	90	Combined	80	98	24.4
<u>Commercial</u>						
Shopping Center	Leo Mall	100	Separate	9	100	-
<u>Industrial</u>						
Light Industry	Industrial Park	100	Separate	43	75	-
Heavy Industry	Erie	100	Combined	100	76	-

the catchments selected, to ensure a representativeness of land use types for later application to the Philadelphia metropolitan area of the models to be calibrated. There are about 250 separate storm and combined sewer catchments within the City of Philadelphia alone.

For smaller metropolitan areas it is more feasible to make comparatively more extensive gagings simply because of the few catchments involved. An example is the portion of metropolitan Rochester, New York, served by combined sewer systems. There, all thirteen combined sewer outfalls involved have been instrumented for automatic flow-sensing and water quality sampling.(148) The land-use features of the catchments are listed elsewhere.(132) The catchments range in size between 230 and 2,600 acres, residential occupancy ranges between 37% and 94% and imperviousness ranges between 35% and 80%. Ten of the thirteen combined sewer outfalls discharge overflows to the Genesee River, which is also monitored for water quality. Considerable modeling work has been undertaken.(132,148)

A number of considerations are involved in the deployment of gaging stations,(146) such as in the Philadelphia case above, which may be termed a spatial network design. The purpose of these stations is to serve planning and design functions. (The use of such networks for real-time operation is discussed in Section 7). Specifically, short-term records are necessary to calibrate models so that long-term performance simulations can be made. That is, rainfall variations are sampled over a season or two on catchments that are samples of predominant land uses. (The issue of storm variations over time is discussed in the next subsection).

The goal of spatial network design is to attempt an optimization of the best set of "representative catchments samples" that can be realistically funded for gaging; and the maximum number of station points where measurements of flow-quality can be made, for a given total budget, is then affected by how fast a point can be abandoned and measurements at some other point can be initiated with the same equipment. Speed of abandonment is a function of how rapidly the data are used to calibrate a model or models, and the deliberate rush to calibrate may cause an extra non-data cost for analysis which has to be taken into account in weighing budget schedules. Because such field work can often be funded only on a year-by-year basis, the staging of investment is therefore affected by the staging strategy employed, in addition to considerations of total project and yearly project budgets. That is, the staging over the project should maximize results from year to year, with an early payoff and sufficient payoff thereafter to discourage premature abandonment by officials with fiscal veto power. All this is to say that considerable judgment, wisdom and foresight are required. It is obvious that spatial network design is presently more art than science. On top of all this is the restriction that some catchments cannot be gaged at all (e.g., very complex backwater flooding of a site at a wide flow range, such as by tides), and some can be gaged only at inordinate expense (e.g., because of difficulties of access). ASCE had proposed a 3-year national gaging program in 1969,(143) and it is therefore expected that any metropolis could get nine-tenths of its work done within 3 years.

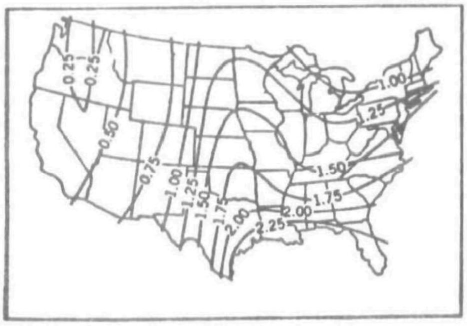
Long-Term Precipitation Record as a Reference

Control of flooding and water pollution must be based on probabilities of occurrence because of the randomness of precipitation. Reasons for using actual records rather than synthetic storms are reviewed at length in Addendum 2.

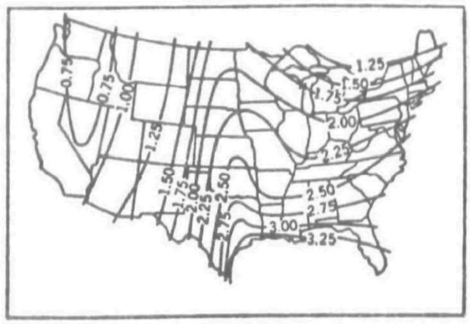
The primary long-term rainfall data base for sewered catchments in any metropolis is the (usually) single U.S. Weather Service first-order station five-minute interval data, commonly spanning fifty years or more. It is necessary to assume that short-term rainfall samples from field-gaging programs lasting a season or two comprise a representative sample of the long-term record. To repeat, the single rainfall station record is the primary (and sometimes sole) source of long-term data available. So far, there has been no successful simulation of 5-minute single-point data via stochastic methods, so there is no other way. On top of the assumption that our precipitation field gaging is for a representative sample, we must further assume that the long-term single-point record would be more or less replicated over a similar period of time at any other point in the same metropolis. The latter is a generous assumption because we have proof of areal persistence only for pieces of storms: i.e., maximum intensities of a specified duration. Figure 4 shows examples of the areal persistence of one-hour maximum average rainfalls of particular frequencies, after Yarnell[149] (only six of the total of 56 charts he prepared). The same persistence is implicit in later mappings of the U.S.W.S., including Technical Report No. 40[150] and in recent updates. In addition, the Corps of Engineers studied the Yarnell maps and found that the depth for any duration was an approximate function of the same frequency one-hour duration depth for all national station data, regardless of recurrence interval, Figure 5.[151] It should be carefully noted that the "isohyetals," such as in Figure 4, connect equal values of precipitation for a given duration and frequency; and that these values are for different storms at different years from one location to another. (Details on such data are given elsewhere[152]). Thus, there is an implication of long-term persistency from one point to another over long distances, and hence over relatively short distances, tenuously based on the documented strong persistence of separate pieces of storms regarded independently.

Theory of catchment raingage network design is relevant only for initial placement; and optimum or critical placement can be assured only by modeling data as it accumulates to see if reliability of the objective function (runoff-quality) is adversely affected by the initial placement. The underlying assumption in using a single point (Weather Service gage) long-term record for all points in a metropolis is that variations are completely random and therefore over a sufficiently long period (whether 10-years, 50-years, or more, nobody knows), essentially the same objective function results would be obtained at any point in the metropolis as at the single reference point. However, we have no adequate theory, based on objective function response and not input response, on the statistical relation between more than one point of input data and metropolitan-wide points. Based on our fundamental assumption, it follows that if more than one raingage is required to characterize a catchment we will not be able to extend the record as above-described realistically. This leads us to the inescapable conclusion that all catchments with a flow-sampler installation should need only a single raingage. At first, this would seem to restrict consideration to catchments no larger than perhaps 500 acres, but it does not. Rather, what it tells us is that subcatchments should be no larger than some magic limit (say, 300 acres), that each subcatchment of about that size should have its own raingage, and that each subcatchment should have its own runoff-quality station at its outlet, so that when long-term records are simulated the simulation will take account of the separate subcatchments' calibration peculiarities. This is still stretching our basic assumption, in the application of long-term single-point rain data, because we will be assuming perfect joint correlation of objective function results for the multi-point catchment (i.e., between each subcatchment) as we have for between the reference Weather Service record and some other single point. This, however, is the best we can possibly do!

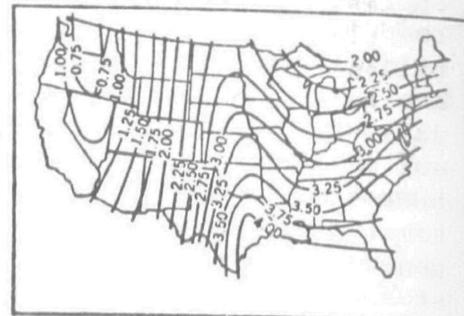
An important conclusion that results from the above reasoning is that, except where a catchment does not change in character over a long period of time



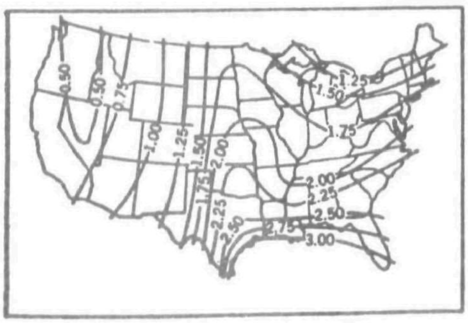
One-Hour Rainfall, in Inches, to Be Expected Once in 2 Years



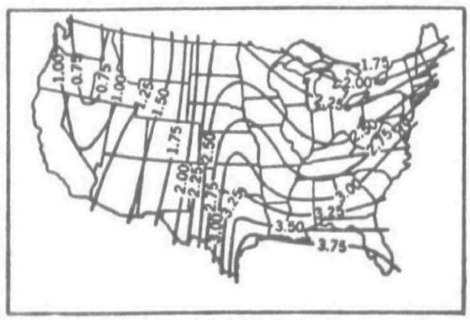
One-Hour Rainfall, in Inches, to Be Expected Once in 10 Years



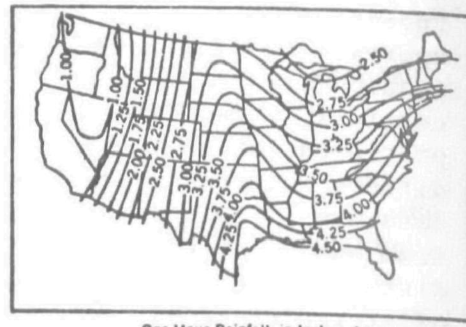
One-Hour Rainfall, in Inches, to Be Expected Once in 50 Years



One-Hour Rainfall, in Inches, to Be Expected Once in 5 Years



One-Hour Rainfall, in Inches, to Be Expected Once in 25 Years



One-Hour Rainfall, in Inches, to Be Expected Once in 100 Years

FIGURE 4 - SAMPLE OF YARNELL CHARTS [149]

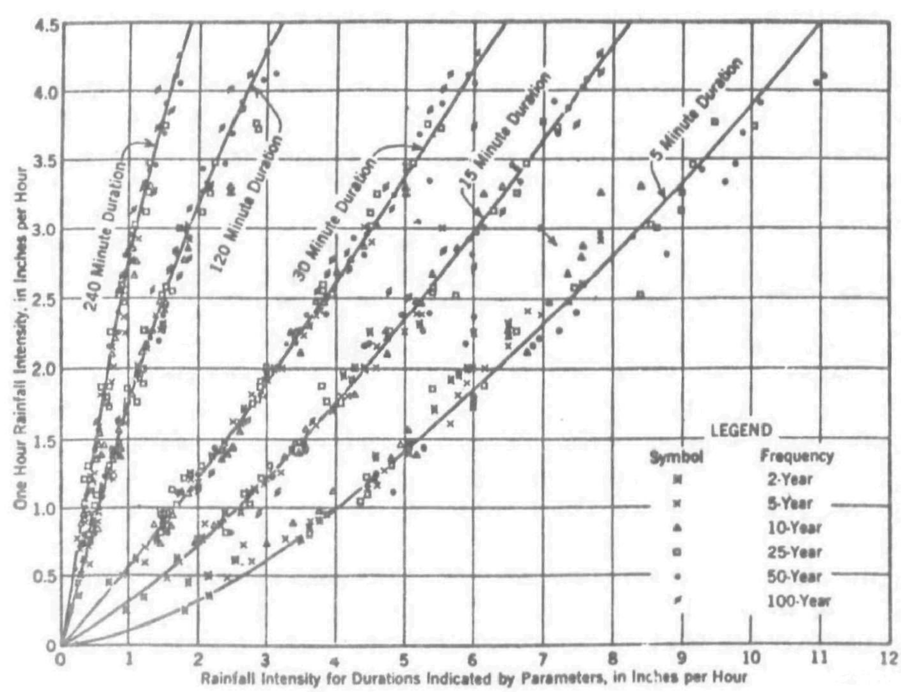


FIGURE 5 - RELATION FOR VARIOUS DURATIONS [151]

(unchanged land use and hydrology over 10 years or more, but how can this be realistically anticipated in advance?) the justifiable duration of data collection for planning purposes is limited to that amount necessary to calibrate the catchment response with whatever model or models will be employed to extend the record of that catchment. Dallying could be justified on the basis that the "right" model or models is either not known with assurance at the outset, or that improvements in all models will be coming, sometime, but such temporizing must be weighed against the cost of acquiring data only as a "holding action" and the more important fact that the equipment tied down by equivocation cannot be moved to calibrate another catchment, and the extent of "catchment sampling" (see the preceding subsection) will accordingly be shorted. Of course, there is merit in the perpetuation of a few stations as a means for sharpening reliability as planning proceeds. But this is virtually identical to an extra-cost option on an insurance policy and should be so regarded. The only major exceptions are for long-term monitoring and where automatic control in planned operations is contemplated, discussed in Section 7.

Developing Frequency Rationales for Levels of Protection

Emphasized throughout this Section, and in Addendum 2, is the importance of putting results of simulation on a probabilistic footing as a means for maximizing the reliability of performance of facilities and programs planned for the future. Noted several times in earlier Sections was the history of moving targets for flood management and pollution abatement over the past decade, and the likelihood that policy targets will continue to shift over the next several years. The only realistic defense for planning in an atmosphere of ambiguous policy is to employ procedures the results of which have an inherent flexibility for conversion to meet alternative policy goals.

In Section 2 we noted that the policy goal of the San Francisco master plan for combined sewer overflow pollution abatement was to limit the average number of overflows per year. We also noted that because the City had founded its estimates of attainable system performance on a long-term precipitation record (62-years), it would be relatively simple to convert the original simulation results summaries into their equivalent average annual or seasonal or serial expected occurrences, such as in terms of overflow volumes or pollutant loads or seasonal number of overflows or whatever. Such a conversion would be facilitated by the existence of field data that has accumulated over the past six years in San Francisco in support of a total system automatic control development program (Section 7). Although the first phase of the master plan is now committed in terms of physical deployment and capacities of a about half of the projected facilities, the ultimate incorporation of automatic control and interbasin dispatching features will provide sufficient flexibility in operating the total system so that reasonable shifts in policy targets could be accommodated. Had the above elements of flexibility not been included, the City would be rather helpless if the policy target was shifted. The cost of add-on revisions to new facilities could quickly escalate abatement costs to completely unrealistic levels.

While flexibility of analytical tools is the issue in this Section, it is important to note that total project flexibility is much more important.

Simulating a Range of Storms and System Loadings

The preceding three steps are prologues to this step, called "simulate performance for alternates" in Figure 1. To reiterate, it is necessary to acquire short-term field data records over one or two seasons to calibrate models for the subject simulation. That is, rainfall variations need to be sampled over a season or two on catchments that are samples of predominant land uses. Because urban catchments, whether fully storm sewered or only partially storm sewered, often change in land use and drainage system characteristics from year to year, the river basin analysis approach of deriving hydrologic indicators from a long series of field observations (such as 20 years or more) is simply not feasible. If reliability of the results of analysis is a genuine issue, the only recourse is a time-series synthesis of runoff-quality events via simulation using hydrologic models.

Now for the moment of truth. What are we going to do with the resulting catchment calibrations? We are going to extrapolate them. We are going to assume that we have a fix on the characteristics of a "representative sample" of catchments and that the model hydrologic constants, exponents and other factors that were found to work on a given class-size-land use catchment will apply to all catchments of that given character and (sending up a prayer) that we can extrapolate (and perhaps interpolate) these findings for other class-size-land-use combinations where we did not have enough money to sample for their characterization. Obviously, the level of reliability of the tools we have calibrated is not merely a function of the goodness of fit of hydrographs and pollutographs in the calibrations or in inherent instrument or recording errors, but also on the "representativeness" of the gaging network; and this latter, very important point must be brought home to the so-called decision-makers. That is, reliability of the employment of calibrated tools is a function of several things, but the most important are probably the shrewdness of design of the data network and the extent to which its results approach an ideal set of representative samples. Let the modelers lament observed-calculated differences and let the field engineers sermonize on instrument error, but always remember that the degree of data network adequacy is a much more important measure of the reliability of planning or design conclusions based on simulations founded on data from the network. At this point we must qualify the preceding assessment of overall reliability of results, downward, but by a necessarily qualitative amount, to allow for the universal use of a single-point rainfall data record within the metropolis, described earlier in this Section, as the representative historical rainfall for the entire metropolitan area. There are a few cities and metropolitan areas that have raingage networks that have been in operation for five years or longer. A few of these have been able to derive empirical relationships between rainfalls in various sectors of their jurisdictions on the basis of selected storms. While such indicators can certainly be useful in operations, and perhaps also in design, planning applications are necessarily referenced to the long-term record of single-point rainfall data.

SECTION 6

URBAN RUNOFF MODELS

Contents

Reiterated in this Section is the importance of calibrating any urban runoff model against local field data. The national dearth of field data (particularly for sewered catchments) available for validating models is cited as one of the principal liabilities in the use of such models. However, there are a number of potential advantages in the use of models that can be exploited when certain liabilities, such as a national scarcity of validating field data, can be overcome.

Models are characterized in this Section in terms of their applications. Specific models are identified, with an emphasis on sewered system planning applications, although associable analysis/design applications are duly noted. Of the several attempts to compare the performance of various models, the most useful are those defining extents of application flexibility. Receiving water modeling is handled separately because of the tendency towards customized adaptation of available models for that purpose. Examples are given of some regional receiving water simulation studies.

Role of Simulation in Planning

A special session at an annual meeting of the American Geophysical Union(153) attempted to define appropriate rationales and incentives for the more extensive use of urban runoff mathematical models, for planning, analysis/design and operations. Among the advantages cited for the use of such models for planning were that: tests can be made of alternative future levels of development and their impact on facilities needed in the future; several models well-suited to master planning are in the public domain and are regularly upgraded and made readily available by the Federal agencies that supported their development; when detailed models are used in advanced stages of planning the user is able to understand better the physical performance of a system; the interrelation between land-use projections and planned mitigative programs and their costs can be made more apparent; revisiting plan assumptions to update projects can be done with consistency and relative ease; joint consideration of quantity and quality of runoff in sewered catchments and in streams can be accommodated; hydrologic-hydraulic effects of future urbanization can be explored; and deficiencies in existing facilities and prevailing management programs can be identified.

Of the liabilities, outstanding is the dearth of field data on rainfall-runoff-quality, particularly for sewered catchments, for development of more acceptable measures of reliability of all types of models. A recent workshop conducted by the ASCE Urban Water Resources Research Council resolved guidelines for the acquisition of such data by local governments.(146) The spectrum of investigative stages utilizing field data for the sewered areas and receiving waters of a metropolitan area include the following:

- Identification and evaluation of quantity and quality problems.
- Exploration of alternatives for pollution and flooding abatement.
- Analysis of the most attractive alternatives.
- Preliminary design of adopted alternatives.
- Detailed design of adopted alternatives and their implementation.
- Post-implementation operation via a range of possibilities extending from simple monitoring to automatic control.

Consistent with the comprehensive planning scope outlined in Figure 1 (Section 2), this Section will concentrate on the use of models up to preliminary design. However, because models used in the implementation and operation stages that evolve from a comprehensive plan rely on extensions of the same data base, some mention must be made of their characteristics.

Before embarking on a discussion of model capabilities, it is necessary to establish certain caveats on the state of the art.

Some Reservations

Because complex processes, such as in the hydrological response of a sewered catchment to a precipitation occurrence, can never be fully replicated in a computation due to incomplete technical understanding of the processes and the infeasibility of detailing the literally myriad pieces involved, resort is made to simulation of response of a conceptually equivalent system. The simulation package is commonly called a "model". Reality dictates that a model should be selected on the bases of the type of application involved, how it is to be used, how much can be invested in its use, how often it would be used, what levels of precision are required or desired, what kinds of outputs are wanted, how much time can be spent to get the model to work, and how much can be committed to verify and calibrate the model. Calibration is the process of varying model parameters to minimize the difference between observed and simulated records.

We have been reminded that until each internal module of an overall catchment model can be independently verified, the model remains strictly a hypothesis with respect to its internal locations and transformations.(154) Because of the very limited amount and kind of field data available, just about all sewer applications model validation has been for total catchment response, at outfalls. That is, under contemporary conditions a distributed system model deteriorates into a lumped system model for all practical purposes. It should therefore be evident that validation using transferred data by the model's developer is not nearly enough. Credibility requires at least token calibration using some local rainfall-runoff-quality data. Unfortunately, the acquisition of such data is commonly regarded as the exclusive problem of local governments, and too many planning and analysis exercises have proceeded without benefit of local field data, using one model or another.

Calibration and validation is further confused by the fact that much more field data are available for partially sewered catchments, where flow is measured in receiving watercourses, than for totally sewered catchments. (That water quality samples have been taken for only a fraction of these gaging sites does not help). Adding streamflow hydraulics to sewer hydraulics hardly simplifies the lumped system dilemma alluded to above, yet much of the data used to verify various models has been from such mixed catchments. This should add additional incentive for calibration with local data. "General EPA guidance to date (1975) has tended to minimize basic sampling and analysis of urban runoff. Collection of new stormwater data is severely needed to define the magnitude of its impact on water quality."(155)

Concluded in a comprehensive Canadian study was that sufficient information is not available on relationships between street surface contaminants, their pollutional characteristics, and the manner in which they are transported during storm runoff periods. Also concluded was that basically only one type of model exists for analysis of urban runoff quality, and that the accuracy of the water quality computations using models extant has not been sufficiently established to be used with confidence for prediction purposes, in particular the formulation relating water quality with land use.(156)

A fundamental objective in water pollution abatement is public health protection, yet little information is available on the level of pathogenic microorganisms in stormwater runoff in urban areas.(157)

Water resources are not an exclusive major consideration in environmental protection, and interest is growing in comprehensive or integrated planning, management and control of all types of pollution sources. For example, a model for metropolitan application has been developed which considers environmental pollution as a set of interrelated problems, using submodels for land use, residuals, and dispersion-disposal of residuals.(158) While analogous completely comprehensive modeling of urban water resources processes has been found to be technically feasible,(159) conclusions reached in a study of future needs in urban water models included the opinion that emphasis in urban water modeling in the immediate future should remain on the simulation of discrete processes (individual subsystems such as storm runoff) before attempting simulation or optimization of connected subsystems.(140) This viewpoint was predicated on the likelihood that the functional responsibilities of local government water agencies will remain as splintered as they typically are now. "It is likely that only where wastewater reclamation, including municipal reuse, becomes a viable alternative will a single agency require comprehensive urban water modeling approaches."(140)

Categories of Model Applications

Mathematical models used for the simulation of urban rainfall-runoff or rainfall-runoff-quality can be divided into three different application categories: planning, analysis/design and operations. Some particular models have been employed in both planning and analysis/design, and a few models have been applied in analysis/design and operations applications, making it difficult to allocate them to a single category. Additionally, the reader is cautioned that on no account should most of the models to be mentioned be regarded as typical tools. Rather, common practice still favors rudimentary techniques and only a modest number of offices currently employ the more advanced tools routinely.

Planning applications are at a macro-scale, such as in metropolitan or city-wide master plans. Model requirements for planning are less rigorous and require and permit less detail than for analysis/design because investigation of a range of broad alternatives is at issue. What are sought for planning tools are general parameters or indicators for large-scale evaluation of various alternative schemes. Hence, the degree of model detail required in jurisdictional planning is much less than in analysis/design. However, a certain amount of conjunctive intensive detailed modeling nevertheless is needed to establish parameters and indicators and to provide an underlying understanding of the governing hydrological processes, so that simplified expedients are not inadvertently misused.

Total lengths of underground drainage conduits dwarf those of open watercourses in major cities. For example, total lengths in the 97-square miles of the City of Milwaukee as of the beginning of 1970 were as follows:(160)

Lakefront length	-	8-miles
River lengths	-	37-miles
Combined sewers	-	550-miles
Storm sewers	-	820-miles.

(In addition, there were 685-miles of wastewater sewers). These drainage conduits are distributed over 465 drainage catchments having a maximum size of 1,820-acres and a median size of 25-acres.[161] When dealing with so many components the model used must be as simple and as flexible as possible. That is, data processing for planning applications becomes a much more important practical consideration than the level of sophistication of hydrological process modeling.

Models used for analysis/design applications are more sophisticated and thus are more detailed tools. They are used for analyzing individual catchments and subcatchments where the simulation of detailed performance of discrete elements within a subcatchment must be achieved. Whereas hourly rainfall data is an appropriate input for planning models and for simulating flows in larger urban streams, 5-minute interval rainfall data (the shortest duration reported by the U.S. Weather Service) is the appropriate input for simulating flows in sewers and small urban streams for design applications. That is, the level of sophistication of hydrological process modeling for design becomes a much more important practical consideration than data processing, just the opposite of the emphasis imposed by planning requirements.

Models used for operations applications are likely to be more use-specific because of wide diversities in management practices, operating problems and individual service-system configurations. However, the most potentially transferable technology will be for automatic operational control of total community runoff, a capability that is currently receiving intensive development (Section 7). The mathematical models required feature control algorithms that have to be painstakingly derived from numerous indicator applications of both detailed analysis/design models (for generalization of the performance of individual process components by simulation) and planning models (for generalization of community-wide system performance by simulation). Here also, analysis/design models are used as tactical tools and planning models are used as tools of strategy.

Models for Planning Applications

STORM (Storage, Treatment and Overflow Model). Designed specifically for urban runoff and quality evaluation for total jurisdiction master planning, this computer model is eminently suited for that purpose and currently enjoys, in one version or another, the most extensive use of any urban drainage planning model. Not only is it non-proprietary but the computer program, model documentation[162] and a users' manual[163] are all readily available. The original version of the model was employed in part of the development of the Department of Public Works, City and County of San Francisco, master plan for combined sewer overflow abatement, described in Section 2.

A simplified logic diagram for STORM is presented in Figure 6.[28] Note that this model focuses on structural means for flow and pollutant containment (storage and treatment). It is designed for use with many years of continuous hourly precipitation records (but can be used for individual storm events). For example, a 62-year U.S. Weather Service hourly precipitation record was used for the San Francisco master plan. Essentially, the model employs an accounting scheme that, for each storm event, allocates runoff volumes to storage and treatment, noting volumes exceeding storage or treatment capacities (overflows, in the case of combined sewer systems) as these capacities are exercised from one event to the next. Water quality is handled as a function of hourly runoff rates, with generated quantities of constituents allocated to storage, treatment and non-capture as for runoff volumes. Statistics are generated for each event and collectively for all events processed, including average annual values.

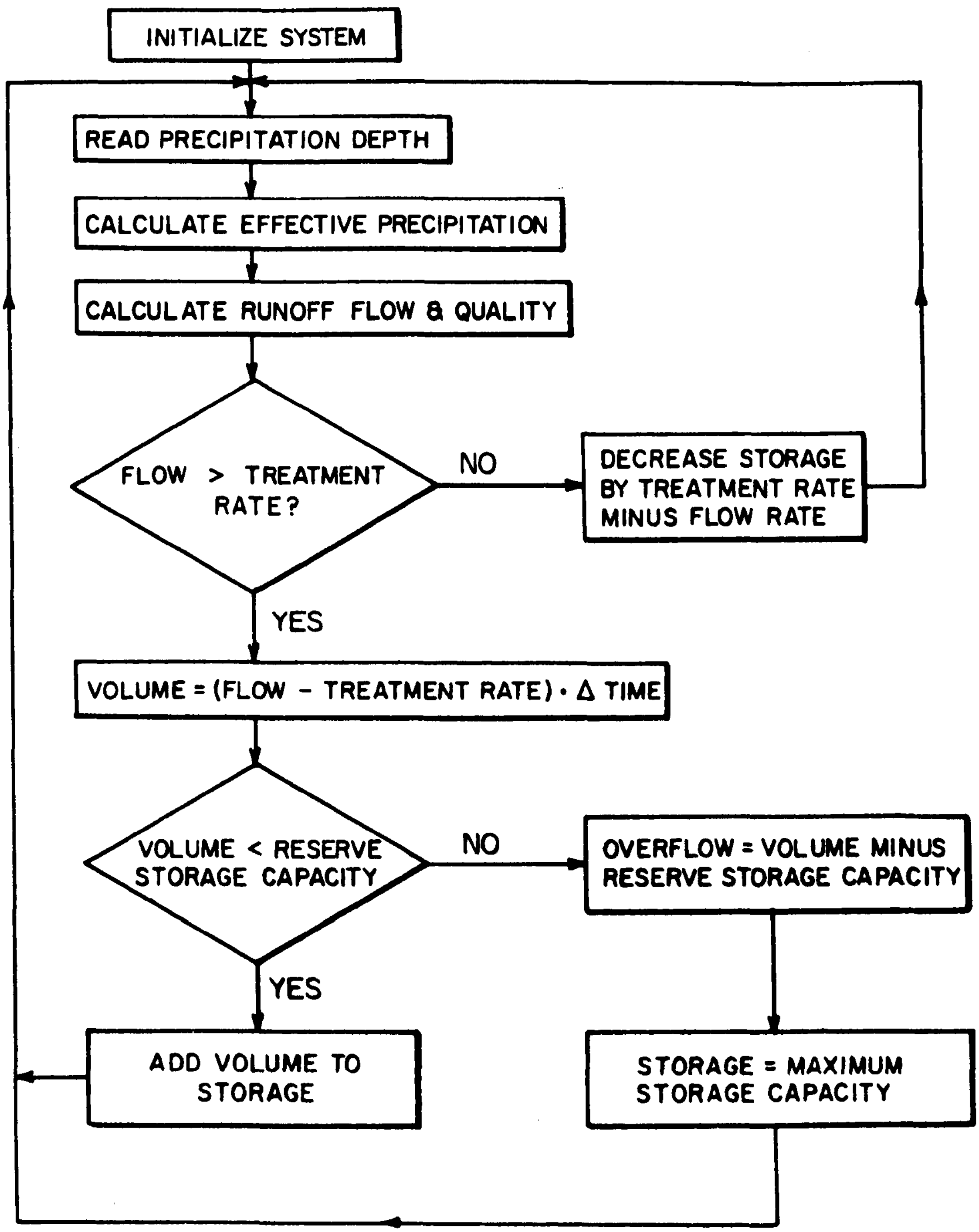


FIGURE 6- "STORM" SIMPLIFIED LOGIC DIAGRAM[28]

STORM accommodates non-urban catchments and snowpack accumulation and snowmelt, and land surface erosion for urban and non-urban areas can be computed in addition to basic water quality parameters.

Until recently, hydraulic simulation or flow routing was not incorporated in STORM. The latest Corps of Engineers' version includes a capability for routing to the outlet of each sub-basin through the use of triangular unit hydrographs based on the Soil Conservation Service procedure.(163) Planned for future versions are such added capabilities as an expanded procedure for sizing detention reservoirs. Hence, we find a tendency towards enlargement of a model originally designed for broad scale planning to include preliminary analysis/design capabilities. Simultaneously, detailed models for analysis/design are being simplified to make them more amenable for planning applications, and both types of models are being interfaced to facilitate feedback of information from one to the other. All this is making it more hazardous than ever to categorize a model in terms of its major applicability.

A modification of the STORM model has been linked with the receiving water module of the SWMM analysis/design model (noted below) for continuous simulation of receiving water quality.(164,165)

As part of a nationwide assessment of stormwater pollution control costs a "desktop" procedure was developed, by streamlining the STORM model, that can be used to estimate the quantity and quality of urban runoff in combined sewer and storm sewer areas and unsewered portions of a jurisdiction. Combinations of storage and treatment for pollution abatement and their costs can be estimated taking advantage of generalized results from the nationwide assessment.(166)

A methodology has been suggested for screening urban development alternatives and preliminary design of storage and treatment capacities for runoff control on the basis of water quality impacts utilizing STORM.(167)

<u>Other Planning Models</u>. A simplified stormwater management planning model has been developed that is an inexpensive, flexible tool for planning and preliminary sizing of stormwater facilities.(132,168) Time and probability considerations are incorporated in the model. Joint usage of a complex model and a simplified planning model, such as this one, is said to be not only compatible but also complementary.

A very simple methodology has been advanced for preliminary screening of stormwater pollution abatement alternatives.(169,170) While the method was conceived for combined sewer system applications, it could as easily be applied to stormwater systems. Developed for use at the national or State decision-making level for early identification of poor candidates for abatement project funding, it might as readily be applied for rough, early assessment of the maximum pollutional impacts of storms at the metropolitan level.

Reported verifications of the simple process planning models, including STORM, have been limited, although hearsay indicates that the number of verifications is growing. Because suggested magnitudes of model coefficients are based on the sparse amount of field data available nationally, it is very important that local rainfall-runoff-quality field data be used to calibrate such models for the sake of enhanced reliability of results. Too many planning exercises have proceeded without benefit of local field data, using one model or another.

The original MIT Catchment Model was predominantly an analysis/design type. However, a modified version has been employed for estimating sewered catchment and stream runoff in the Fairfax County master plan development,(30,31) Section 2.

Synthetic unit hydrographs, based on regional parameter generalizations,(171) were employed in planning the drainage for the large "Woodlands" development(172) near Houston. However, unit hydrograph applications have been mostly in analysis/design. For example, a unit hydrograph is in general use for designing drainage and flood protection works in metropolitan Denver.(173,174)

Models for Analysis/Design Applications

There are a number of this type, including the two noted immediately above.

Although it is essentially an analysis/design type when applied to smaller-scale catchments, the Hydrocomp Model can and has been used for broad-scale receiving-water planning applications, described later. The Hydrocomp Model has also been employed in several metropolitan sector studies.(175-178)

There have been some applications of versions of the British Road Research Laboratory Model.(179-181) Available to the public is the computer program for a very simple streamflow and drainage system model used in the Chicago area(182) for preliminary project planning and design.

The USGS has modified the MIT and ILLUDAS models for continuous simulation, detention-storage accommodation and water quality simulation.(183)

As far as drainage system simulation is concerned, the EPA SWMM(184) (Stormwater Management Model) and its variants have enjoyed more applications and been subjected to more verification than any other model. For example, a version of SWMM has been used to supplement and compare results obtained by the Chicago Department of Public Works from its own models(185,186) as part of the master plan for combined sewer overflow abatement in metropolitan Chicago, Section 2. Another version, used in connection with the San Francisco master plan,(187,188) has the most versatile transport module of those extant. This version is now included as an option in the computer program for SWMM. The latter is continually updated.(189)

For the Milwaukee master planning project, described in Section 2, three models were used: STORM, SWMM and Harper's Multiparametric Water Quality Model.(28)

Model Comparisons

There have been a rash of projects comparing the merits of various models on the basis of a variety of criteria.(156,190-197) Instances where tests had been published of the performance of various types of models against field data were reported in 1975.(141) Advances in modeling capability occur almost too rapidly to keep track, and in 1975 it could be said that mathematical model development for sewered system applications had already seemingly greatly outpaced the data base for model validation.(141)

Results of the various tests are mixed, mostly because there is no acceptable basis for multiple-objective comparison. Peak flow is the major consideration in sizing conduits, volume and hydrograph shape are critical for sizing storage, and concentrations and loadings of pollutant emissions are essential for evaluation of receiving water impact and helpful in sizing treatment facilities. Each model has its strengths, weaknesses and outright faults for a given application. Over and above the correlation definition problem is the inherent difficulty with any runoff model in the necessarily subjective separation of abstractions (infiltration, depression storage, etc.) from total rainfall to resolve rainfall excess (amount and pattern), which is the input from which an equal volume of direct runoff is generated by models of one kind or another. After analyzing the performance of a variety of models, it was concluded that the weakest link is the proper estimation of rainfall excess.[198] All this is to reiterate that urban hydrology is still, in the absence of an adequate body of field data, more of an art than a science, and that under this circumstance the choice of a model for a given application is largely a matter of taste.

Receiving Water Modeling

Receiving waters are the common repository of effluents from just about every community and self-supplied industry in a metropolis, constituting perhaps the most shared aspect of urban water resources. Impressive advances have been made in receiving water modeling. Initial attention was on hydrology and hydraulics in support of flood control objectives. Water quality modeling capability has evolved more recently, with a tendency to use tailor-made models for discharge-quality simulation in planning applications, and earlier development was focused on estuaries. The choice of a model or models to be used in any given planning effort therefore requires careful and discriminating study. Consequently, it is appropriate to cite recent capability summaries and describe a few examples of applications. Reference has been made earlier in this Section to SWMM capabilities.

A compendium,[193] two companion reports,[199,200] a North American summary,[201] and a text,[202] survey features of large-scale water quality models; and an annotated bibliography of models for tidal rivers, estuaries and coastal waters is available.[203] Tidal water models have been comprehensively classified,[204,205] and capabilities for modeling estuary and streamflow water quality have been assessed.[206]

Aquatic ecosystem submodels have been delineated for process analysis.[207] Aquatic ecosystem models were surveyed in 1974 for the National Commission on Water Quality via a questionnaire,[208] and while several models reported are more generally applicable, nearly all have been developed or tested on a specific water body and only a fraction of the applications have urban implications. Computer graphics techniques might be useful for displaying oceanographic data for coastal metropolises.[209]

Water quality modeling for systems containing rivers and reservoirs has been advanced through the issuance of a description of a combination of models.[210] The Hydrologic Engineering Center is having dynamic flow routing routines added to the model and plans to upgrade the documentation as new developments occur.[211] Dynamic or unsteady-flow water quality modeling is particularly important in the case of significant pulse loadings from urban runoff or when man-made controls such as dams are involved.

Effects of risk and uncertainty in the application of operations research techniques, including hydrologic modeling, have been critically reviewed at length.[212]

Although receiving waters represent only a part of the total urban water resource, they commonly traverse entire metropolitan areas and are affected by the actions of a multitude of local jurisdictions. Recent emphasis on regionalized wastewater treatment and disposal has resulted in some receiving water simulation studies on a grand scale, a few of which are noted here as examples.

<u>San Francisco Bay</u>. The study area extends upstream well beyond the San Francisco metropolitan area. About two-fifths of all the surface runoff in California passes beneath the Golden Gate bridge via the Bay estuary. About 800-mgd of treated

wastewater effluents from industrial and municipal sources enter the Bay. Funded by the California State Water Resources Control Board, the estuary water quality project is modifying and calibrating an existing set of interrelated hydraulic, water quality and ecologic models of the Bay.[213] The hydraulic models were designed to permit rapid evaluation of alternative wastewater management plans at a reasonable cost and to serve as a screening tool to reduce alternative plans to a reasonable number. Once the screening of plans has been resolved, the ecologic models are to be used in the simulation of the ecologic response of the Bay for the more attractive plans. Descriptions are available of some of the water quality[214] and flow[215] characteristics of the group of models used.

Denver Area. Partly supported by an EPA planning grant, a metropolitan water quality management plan[216] is being resolved by the Denver Regional Council of Governments that encompasses: a plan for an areawide system of wastewater treatment facilities; financial arrangements and legislation and institutional guidelines for implementation of the plan; a water quality surveillance system for regional water quality management; and an environmental impact assessment of the proposed facilities. Virtures of alternative treatment plant locations, sizes and groupings were analyzed using the Hydrocomp model. Flow and water quality simulations were performed for the mainstem and principal tributaries of a 750-square mile sector of the South Platte River basin that essentially includes the metropolitan Denver area. Numerous flows enter the study sector from upstream. Stream sections within the sector were divided into 33 reaches for the simulations and hourly water quality levels and streamflows were computed for each reach. Model calibration was complicated by a necessity to account for upstream reservoir releases and irrigation diversions and shortcomings in long-term water quality data. Simulations were for anticipated future land use. An earlier DRCOG study[39] (Section 2) had developed some contemporary runoff-related land use characteristics for 398 sub-basins, most of which are in the study area, and this information was exploited in characterizing streamflow inputs from urban land runoff and its associated pollution burden.

Seattle Area. In response to a State of Washington directive to develop water pollution control and abatement plans for two basins, the Municipality of Metropolitan Seattle and surrounding King County formed the River Basin Coordinating Committee (RIBCO), which adopted a coordinated approach to insure development of not only an integrated plan but an integrated planning process as well.[217] Study elements included water resource management, water quality management, urban runoff and basin drainage, solid waste management, and land use allocation. Because of an emphasis on a continuing planning process, a systems approach was specified. This led to extensive use of computer models and provisions for their future updating and use.

The water resource management study[218] included use of the Hydrocomp model[219] to develop a regional water supply plan for municipal and industrial usage, navigation lockage, stream flushing and fish flows. Both streamflow quantity and quality characteristics were included, together with firm yields and upper estuary considerations. Seventeen chemical and biological parameters were simulated. Model input/output is responsive for segments 5-square miles or larger in size.

A variant of the EPA SWMM model was used in connection with the urban runoff evaluation portion of the RIBCO Study.[220]

Hydrocomp has since extended the usefulness of its model for non-point pollution simulation.[221]

Milwaukee Area. Responsible for finding solutions to areawide developmental and environmental problems in a rapidly urbanizing seven county area, the Southeastern

Wisconsin Regional Planning Commission was created in 1960. The Commission has completed comprehensive studies for the Root River, Fox River, Milwaukee River and the Menomonee River watersheds, and is conducting a similar study of the Kinnickinnic River watershed. The Milwaukee master plan, outlined in Section 2, is an extension of part of the Milwaukee River watershed comprehensive study.

Among the outputs of the comprehensive watershed studies are flood hazard maps for computed 100-year recurrence interval flood stages and estimated annual flood damages for floodprone reaches. Extensive simulations are made using a Corps of Engineers backwater computer program(222) and a Soil Conservation Service flood-routing computer program.(223) SEWRPC also uses the Hydrocomp model. There are several other computer programs suitable for practical application that are readily available.(224) The watershed studies have included an examination of both structural and non-structural alternative plan elements for the resolution of existing flood problems and for determination of the effect of changing land use in an evaluation of the 1990 regional land use plan.(225) Also included have been the development of guides for designation of floodland regulatory zones, analysis of floodland encroachments, provision of bridge design data and criteria, provision for subsequent updating and refinement of flood stage profiles and floodland mapping, preparation of Federal Flood Insurance reports and dissemination of flood hazard information.(225) Other modeling has been undertaken in connection with Section 201 and Section 208 planning.

More recently, the Hydrocomp water quality submodel has been incorporated by its developers into flow simulation capabilities in a Hydrologic, Hydraulic, Water Quality and Flood Economics Model.(153)

San Diego Area. Under an Urban Systems Engineering Demonstration Program comprehensive planning grant from the Department of Housing and Urban Development, the Comprehensive Planning Organization of the San Diego Region commissioned a joint venture to formulate water resource planning methodologies and technical approaches for estimating costs of alternative land use plans. Five interrelated computer programs were developed: Data Management System; Water Supply Model; Wastewater Model; Flood Control Model; and Economic Analysis Program.(226)

SECTION 7

SOME MODELING CONSIDERATIONS

Pitfalls

Models can be misused or be used ineffectively if they are selected and employed without prior careful consideration of their data requirements and their place in overall information processing. Because the response formulation of current models is keyed to land-use parameters, the extent and detail of land-use characterization that can or should be assessed becomes a function of the intended uses of model outputs.

While flood plain mapping of 100-year stream stages is a minimum requirement, the mapping of more frequent occurrences is required for analysis of the economics of flood damage mitigation.

Permanent field data acquisition systems for operations, particularly for automatic control, can be readily justified. In Section 5, only temporary installations could be justified for planning and analysis/design purposes. On the other hand, a skeletal instrumentation network should be maintained throughout the planning through operations cycle, but such long-term foresight has not been well received in the past.

The themes described above are discussed in this Section. They have been singled out because they are particular pitfalls that have been encountered in past planning efforts.

A special feature of this Section is a brief review of the latest developments in the manipulation of storage via automatic control.

Land-Use Data

Land-use information needs for planning and analysis/design models are similar, differing mostly in the amount and detail required, the latter model category being more sophisticated and demanding by far. For current models, the key land-use variable for calculating runoff is the imperviousness for each land-use category being considered. Runoff rates and volumes rise as the degree of imperviousness increases. Because the basic hypothesis for source quality for current models hinges on street-surface pollutants accumulated between rainfalls, the key land-use variable for calculating pollutant accumulations is the extent of street gutters, expressed in terms of length of street curbs. Indeed, these two parameters may be the most sensitive of the parameters affecting the estimation of runoff and pollutant loading amounts, respectively, according to the results from limited sensitivity tests.

Intuitively, land-use type, degree of imperviousness and extent of curbs should be correlatable, using residential population density as the dependent variable. Such correlations have been found for the State of New Jersey,[227] sectors of metropolitan Washington, D.C.[228,229] and for a few other samples in the U.S. and Canada.[166] Similar relationships can be developed for any metropolitan area or major jurisdiction for use in planning, by characterizing representative samples of various land-use population densities.

However, because more detailed representation may be justified for later preliminary design, recourse to total jurisdiction or metropolitan area assessment of current (and perhaps prior) land use might be more consistent. While aerial photographs can certainly be used for land-use surveys, new technology for computer-aided interpretation of satellite data might be exploited for the explication of imperviousness.[230] Compared with costs for interpretation of aerial maps, the new method appears to be considerably faster and less expensive.[231] Interpretive resolution is of about the same precision as for manual reading of aerial photographs, but only for watersheds of a minimum of about one square mile in size.[231] This tentative conclusion is based on a case in which comparative model results were obtained for both types of mapping.

At this juncture it is again important to remind the reader that planning is a dynamic process concerned with alternative futures. Present conditions are prologues or points of reference for the future and projections are best made from the springboard of a reasonably reliable assessment of current conditions. Good information on current actualities is needed to proceed confidently with extrapolations, where the degree of reliability of projections diminishes as estimates move into more remote time-frames. That is, one of the tests of contemporary information usefulness relates to its amenability for the projection of expected ranges of future conditions or requirements, a consideration over and above the question of model response reliability for a given land-development configuration (Section 5). This is true for all levels of planning but certainly most visible at the implementation level. Land-use maps show current or recent land use, but planning for specific facilities requires projection of land use through the full period of implementation, which would be around two decades for most comprehensive plans.

Because anticipated future conditions cannot be simulated by manipulating actual land use, recourse is made to simulation by some form of calculation or analogy. Again, as in the analysis/design of urban water resource facilities, the first major requirement, prior to investigating requirements for the future, should be attainment of a satisfactory simulation of existing conditions. This might be called the tool calibration phase. Immediately the question emerges: to what detail should basic land-use information for existing conditions be acquired? Because cost of information acquisition rises with the degree of detail sought, it is prudent first to derive collaterally an indication of simulation precision through sensitivity testing. In the case of hydrological modeling, the subject of the preceding Section, there is usually too little field data for conducting a realistic sensitivity analysis. On the other hand, even a year of rainfall-runoff-quality field data for one to a few catchments would lead to the use of much more reliable model coefficients than the use of values transferred from elsewhere by default.[232] All this is to say that if local field data is not available for some degree of hydrological model calibration, what would be the logic in determining land use, imperviousness and curb lengths at a scale of, say, 10-acres, when a scale of perhaps 100-acres or 200-acres would be more consistent with the probable errors in assumed or transferred hydrological model coefficients? Similarly, why determine model information input for every bit of land involved, when a sampling of perhaps a tenth of the planning area would be more consistent? Conversely, when land-use projections of only specious quality are available, why insist on process elegance in the hydrological tools employed?

Perhaps a suitable answer to the information scale and detail question would be to suggest that: when information on a small scale and with good detail is available it should simply be used to the maximum extent possible consistent with available budgets for simulation; and when only coarser raw information is available the scale and detail used should be weighed against the presence or absence of field rainfall-runoff-quality data for hydrological model calibration in tempering the level of model sophistication adopted.

There are a number of computer models for land-use projection and land allocation. An example of a projection model is one advocated for wastewater system planning.[233] A land-allocation model was employed in the Fairfax County master planning (Section 2) to derive future runoff characteristics for the hydrological models employed.[9,234] For both of these examples there are computer-graphics subroutines for display of variables. Of course, land-use mapping is only one component of composite mapping and associated environmental impact analysis.[235]

Prior to conducting inventories of land-use data and establishing procedures or designing systems for the storage, retrieval, analysis, and display of such data, it is important to consider the potential need, either in the modeling process or in other aspects of master planning, for other types of areal data. These other types of areal data, that is, natural resource and man-made features data having an areal characteristic as opposed to point location, might include soil type, vegetal cover, population density, land market values, and flood-prone status. All such data types are similar to the land-use data discussed above in the sense that they have an areal dimension and, therefore, may be readily inventoried and managed in a manner that parallels that used for land-use data. The potential utility of a systematic approach to the inventory, storage, retrieval, manipulation, and display of land-use and other areal data has been demonstrated.[236]

Guidelines for acquisition of information on natural characteristics of land for land-use planning are available.[237]

Flood Plain Mapping

It has been posited that economic criteria can and should be used in planning nonstructural flood control measures generally, just as they have long been used in planning structural measures.[238] But little data is available at the local level for making benefit analyses.[111,239] Fundamentally, what is needed from a hydrological standpoint is stage-frequency information, which would have to be developed by simulation.

As noted earlier, there is considerable evidence[122] which suggests that the effects of urbanization on streamflows decreases as rarer floods are approached, to the extent of a difference that may be undetectable or at least insignificant at the 100-year level. That is, the effects of land use changes are more pronounced the more common the occurrence, adding further substantiation to the need for some sort of regional stage-frequency information over and above the mapping of the 100-year event. Without a reasonable indication of stage-frequency relations for existing conditions, how can one make flood plain planning projections with reasonable confidence?

An attempt has been made to generalize the 2 1/3-year, 5-year, 10-year, 20-year and 50-year frequency of peak flows for urban waterways on the basis of available data.[240] Input variables of interest here are drainage area slope and degree of imperviousness, akin to the needs for conduit flow simulation models.

Over half of natural 100-year flood plains in urban areas are already occupied.(103) Damages to buildings are a function of the height of flooding level at individual buildings in a flood plain. That is, two identical buildings located at different elevations of a waterway flood plain would suffer differing damages for a given flood, assuming that both were not totally destroyed. Looking at it another way, in any given instance some buildings are exposed to more frequent flooding hazard than others. There is so much attention accorded the 100-year flood level(241) that more frequent occurrences can be inadvertently overlooked. There is a convincing indication that potential flooding losses might be significantly mitigated in numerous situations if flood protection was provided up to some relatively frequent recurrence interval, meaning that conventional protection against a rarer flood level could be much more costly.(242)

In essence, needed are stage-frequency relations for as many streamflow locations as is feasible if benefits are to be assessed realistically. Clearly, delineation of 100-year flood plains is not enough. The National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973 call for measures implying the necessity of using stage-frequency relations. Whether, say, 10-year, 25-year and 50-year flood plains should or could be mapped along with the 100-year levels is a moot point.

Developers of the Fairfax County master plan (Section 2) were fortunate that the USGS had mapped not only the 100-year County flood plains but also the 25-year and 50-year flood plains. The Southeastern Wisconsin Regional Planning Commission (Section 2) not only mapped the 100-year flood plains of the Milwaukee River watershed but also the 10-year highwater surface profiles using a simulation model.(23)

Storage Manipulation Via Automatic Control

While we may not go that far as a nation, the ultimate solution for the problem of abating pollution from urban storm sewer discharges and combined sewer overflows is the treatment of such flows prior to their release into receiving waters. Outflow surface-water hydrographs of sewered catchments exhibit very large peaks, on the order of two or more times those of equivalent non-sewered areas. Capturing, transporting and treating all discharges/overflows, unattenuated, would require gigantic collection sewers, pumping stations and treatment facilities, all of which would be used less than the equivalent of about an hour a day, on the average, over a typical year. Furthermore, there will be periods running into several weeks or even months where the discharge/overflow control works will not operate at all, because of little or no precipitation. Therefore, schemes for system-wide discharge/overflow collection and treatment incorporate some form of auxiliary storage for the purpose of attenuating sudden inflows to collection and treatment facilities, with the objective of scaling down the size of such facilities to reasonable and manageable proportions. The complexity added by converting a facility with simple gravity flow into a multiple-component interacting scheme, where practically instantaneous response to flow incidences must be made, requires incorporation of some degree of automatic operational control. In addition, automatic control permits manipulation of dispersed storage capacities for the purpose of maximizing the effective use of their collectively available volumes, and the diversion of uncontainable releases to lower impact receiving water reaches, by taking advantage of the areal non-uniformity of rainfall over large jurisdictions.

There are three distinct levels of complexity in automatic control for the operation of urban water resource systems. In order of complexity, they are automated monitoring, remote supervisory control, and complete, or "hands off," automatic control.(243) So far, the use of automatic control for the manipulation of storage has been considered seriously only for existing combined sewer systems, although the same technology could be applied in the future to existing or new separate stormwater sewer systems.

Two basic schemes have been considered: the exploitation of ambient storage capacity in existing trunk and interceptor sewers in combined systems by adding dams and diversion structures that can be remotely actuated; and the much more expensive provision of altogether new auxiliary in-line or off-line storage facilities that can be remotely controlled. The level of automatic control adopted has been scaled to the investment in storage capacity added. Remote supervisory control is the highest level of automation to which schemes for exploitation of ambient storage capacity have aspired. This is exemplified by the metropolitan Seattle remote supervisory control system.(244,245) At least two other cases have not progressed beyond automated monitoring.

Originally planned features have been outlined of the San Francisco master plan for combined sewer overflow pollution abatement.(246) Implementation of the master plan (Section 2) represents the exclusive instance where complete automatic control capability is being sought. There are fundamentally two distinctive modes of automatic control, reactive and predictive. The reactive mode refers to the manipulation of facilities in response to the actual occurrence of a rainstorm, as the storm progresses over the affected jurisdiction. The predictive mode refers to the anticipation of how facilities should be handled just prior to the beginning of a rainstorm, and blends with the reactive mode once rainfall on the affected jurisdiction commences, but the predictive feature is retained until the cessation of incipient rainfall. Reactive strategies are apt to be site-specific whereas predictive strategies are likely to be adaptable to any system.

San Francisco is currently engaged in automatic control reactive capability development but plans to advance to a predictive capability soon.(15) In cooperative projects with the City, Colorado State University has advanced the automatic control capability state of the art with respect to individual catchments and the total City system.(247,248) Mathematical models employed for developing control algorithms for both control levels have had somewhat more specific information requirements than for STORM but much less detailed requirements than for SWMM. That is, simpler models than SWMM were used for developing control algorithms for individual catchments. However, use of analysis/design models such as SWMM are needed to monitor and provide a physical understanding of results from wholesale applications of far simpler models. That is, a certain amount of intensive detailed modeling is needed to establish parameters and indicators and to provide an underlying understanding of the governing hydrological processes and system responses so that simplifying expedients are not inadvertently misused.

With respect to predictive control capability development, a procedural scenario can be hypothesized with regard to storm characterization. The data source would be at least two years of rainfall data for a network of raingages surrounding the jurisdiction, with their ideal density heavier towards the side from which storms passing over the metropolis tended to prevail. Because of the importance of time of occurrence, synchronization of raingage records would be mandatory. By means of computer-mapping, the depth of rainfall over any point within the network at a given time interval could be interpolated and the prevailing storm direction at a given instant would be indicated by the slope of the maximum tilt of the fitted surface. Storm path and traversing speed would be indicated by the progression of prevailing storm directions across the network from one time interval to the next. The statistical means and a measure of variance would be derived for prevailing direction, path and speed of storms and for total storm depths outside and inside the metropolitan area. As an adjunct of total storm depth characterization, similar statistical characteristics of the time-pattern of accumulation of total depths would be obtained. Next, a theory of storm temporal and spatial variations would be postulated, the statistical indicators would be applied to the resultant theoretical storm characterization model, and the validity of the model would be verified by testing it against at least a season of data that had not been used in the prior analysis. It might take some adjustment in approach or theory to reach an acceptable predictive capability. Of some help would be the fact that advances continue to be made on the statistical characterization of rainfall records for a single point for intervals of an hour or more.[249]

Consider now the following capability, expected to become available in 1977: at each node of a metropolitan spatial grid (such as a 64-point by 64-point grid with nodes 3-miles apart) the digital read-out of rainfall intensity every few minutes (such as at a 3-minute interval), the cumulative depth of rainfall at each time of reporting (such as at a 3-minute interval), and rates and depths for each node predicted as far in advance as 6 hours. This capability is emerging from the use of special radar equipment in conjunction with the deployment of 200 raingages in a recently concluded study in metropolitan St. Louis and a subsequently implemented study in metropolitan Chicago involving a network of 300 raingages. Storm characterization will be possible by analysis of the data from these extensive networks; however, in system control applications later it will be necessary to have only a radar facility and only a few telemetry-connected raingages, with the emerging special software converting radar readings into rainfall intensities and depths, occurring and predicted, for a grid of points that could be as large as almost 200-miles by 200-miles in size. These exciting developments are part of the "Chicago Hydrometeorological Area Project," partially supported by an NSF/RANN grant to the Illinois State Water Survey.[250]

Criticism of the use of synthetic storm patterns for planning and analysis/design applications is elaborated in Addendum 2, with particular reference to cases where substantial new storage facilities would be involved. The criticism centers about the point that for these applications the issue is system output characterization, not rainfall characterization. This substantial reservation does not apply for real-time applications, as in computer-assisted supervisory control or in complete automatic control, where storm prediction capability would be directly interfaced with facilities operation simulation capability. The remaining element is then the development of predictive mode tactical algorithms for operation of facilities, such as rule curves or decision trees or the like, as is being done for the San Francisco system.

Raingage Networks

In 1968, fifteen of the largest metropolitan areas had a network of recording raingages of between about 5 and 192 instruments, with records spanning periods of 2 to about 50 years.[251] There has since been a growth in the number and size of networks, but the most advanced automatic network was installed in San Francisco in 1971, where each 0.01-inch of rainfall from 30 raingages are transmitted at the specific second of occurrence to a central data acquisition and recording station.[252] A very similar, but smaller, telemetry and automatic data logging facility has since been installed in Portland, Oregon.[253] (Both facilities also automatically log signals from a number of field flow-indicating instruments).

A computer-mapping program (SYMAP[9]) operated off-line has been employed by the San Francisco DPW to process a number of storm records obtained from their network. For a given storm, the program interpolates between raingage network readings at any specified clock time and calculates rainfall depths for an imaginary fine grid covering the entire City. On the basis of the grid values, the program prints a contour map showing the variation of depths over the City for the time interval in question. Visual inspection of a series of such maps for an entire storm, plotted for some specified time interval such as five minutes, provides dramatic evidence of rainfall variability over time and space and the movement of the most intense rainfall sectors across the City.

Study of storm mappings showed that, for more intensive rainfalls, only portions of San Francisco are subjected to the higher rainfall rates at any given time. Storms of low total depth are more uniformly distributed, but these would not tax facilities as severely. The spatial and temporal differences observed in the occurrence of rainfall has confirmed the presumption of the 1971 master plan that a system of interconnected components would result in more efficient utilization of facilities. That is, use of real-time computer-actuated control, based on sensing the direction and likely volumes of rainfall, accompanied by a constant concurrent updating of system status, would permit fully efficient use of all storage and flow capacity throughout the system. The result would be a wider latitude in staging the master plan implementation, potential improved system performance reliability, and greater flexibility in meeting water quality standards as they are refined or revised. In essence, system-use allocations will be balanced between storage vacancies available and unencumbered treatment capacity, to meet sensed or predicted rainfall loadings. Further, when overflow occurrences are unavoidable, the sites of their release can be selected on the basis of minimal environmental impact.

Use of a raingage network for developing and using some form of remote supervisory or complete automatic control can be rather readily justified. There is considerable merit in installing networks for surveillance of much less exotic systems, such as those in several other metropolitan areas. However, older networks have had raingages with their own on-site recording charts. Synchronization of charts was always a problem, with time correlations within 15-minutes about the best that could be expected, but the much larger liability was the onerous and time-consuming task of reading the inked traces and reducing the results to a digital tabulation. The result was that usually only the data for selected storms of unusual interest were reduced. Unless field data is reduced and analyzed, it is difficult to justify indefinitely the maintenance of a raingage network.

The above considerations lead us to the inescapable conclusion that the principal value in the use of raingage networks is in the operation of systems, but that full exploitation of such networks requires automatic data logging and reporting, an expensive commitment. However, when raingage networks are deployed as part of an automatic control scheme their cost compared with their importance is almost trivial. (These qualifications apply equally to flow monitoring stations, which are an integral and indispensible part of any automatic control system).

Installation of raingage networks for planning and analysis/design is more difficult to justify. There are some obvious advantages when employed simultaneously with the deployment of temporary runoff and water quality field stations for the calibration of simulation models. Also, they could prove invaluable in the checking of post-improvement performance of systems, and there is much to be said for the maintenance of monitoring stations on selected catchments and receiving water sites from the beginning of planning through operation, but this type of reliability insurance has always been one of the most difficult to sell. Not to be overlooked are possibilities for extending features of temporary data acquisition systems into permanent urban flood warning systems, such as in metropolitan Melbourne.[254] It is not entirely inconceivable that the day when real-time bacteriological and chemical warning systems might be added may not be too distant.

Parenthetically, mention should be made of recommendations that have been offered for raingage densities in the operation of in-line storage or selective discharge systems.[255] Also, there are new useful references on the effects of storage in underground systems[256] and in surface systems,[257-259] but most of these are marred by a reliance on hypothetical cases and a dependence on synthetic storms. The latter is a practice deplored in Addendum 2 when used for important projects.

EPA Overview

An important reference is the recent review of EPA's research and development program for urban runoff pollution control.[133] Outstanding features are: estimated national costs for stormwater pollution control; a summary of references on simulation models; more numerous and more detailed urban stormwater quality management alternatives than could be justified for inclusion in Section 4 of the present report; and identification of 174 EPA reports and 74 on-going EPA projects, plus 15 additional references.

EPA personnel have provided annual literature reviews on urban runoff and combined sewer overflow papers and reports.[260]

Difficulties with planning called for under P.L. 92-500, particularly in Section 208, have been discussed by experts from outside of EPA.[261,262]

Lastly, a methodology has been developed for assessing secondary impacts of wastewater treatment facilities on urban ecosystems in a project by The Institute of Ecology for EPA.[263]

SECTION 8

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ADDENDUM 1

METROPOLITAN INVENTORIES

by M. B. McPherson

Hydrologic Complexity

Figure 1 is a simplified schematic summary of the interconnections of water occurrences, uses and processes in the urban hydrologic system. Not shown are water quality aspects, which would be still more complicated. Considerable detail is hidden from Figure 1. For example, most of our major cities have dozens and even hundreds of stormwater drainage catchments with cumulative conduit lengths often exceeding 1,000 miles, all of which are subsumed under "storm drainage" in Figure 1.

Accounting for Water Quantities

Figure 2[1] is a graphical representation of water quantities in a hypothetical urban area of one-million inhabitants. National averages have been used to calculate the magnitude of the various components. These magnitudes are thus individually typical, but their combination in Figure 2 is not. Nonetheless, two important facts are illustrated: the bulk of the supplied water originates outside of urban areas; and well over half of all the water handled one way or another is in private hands. This means that there is typically only a partial overlap in local government water supply source planning and metropolitan general planning, and that the usual approach of focusing on public water supply can overlook a much larger user group. The situation for water pollution control is similar, as over half of the volumes of discharges to receiving waters are from private corporation lands and the region impacted by water pollution often extends well beyond the metropolis of origin.

Cooling water withdrawals of thermoelectric power plants are essentially all self-supplied. While such withdrawals in urban areas have not been segregated from national totals,[2] they are quite possibly on the order of three times the non-thermoelectric industrial withdrawals of Figure 2.

Figure 3[3] depicts a breakdown of that portion of Figure 2 termed "public water supply withdrawals". Figure 3 is drawn to scale and is again based upon a composite of national annual averages that resulted from a reconciliation of estimates for individual components made by various experts. Thus, the distribution of component amounts for any given community will vary from those shown. For example, there are cases where industrial use is minuscule and cases where it predominates. Because of difficulties in quantity accounting generally, there is considerable uncertainty about the extent of "unaccounted-for" water and "infiltration and inflow". This is the result of the necessity to arrive at amounts for these two elements by determining the residuals or leftovers remaining after the magnitudes of all other elements have been taken into account. Incomplete system quantity measurements and errors in registration of measuring devices also contribute to the accounting uncertainties. Finally, difficulties in calculation arise from the fact that water and wastewater jurisdictions often differ in size.

Figure 4 depicts a breakdown of that portion of Figure 2 termed "urban runoff". Whereas Figure 3 is a composite of water and wastewater national annual

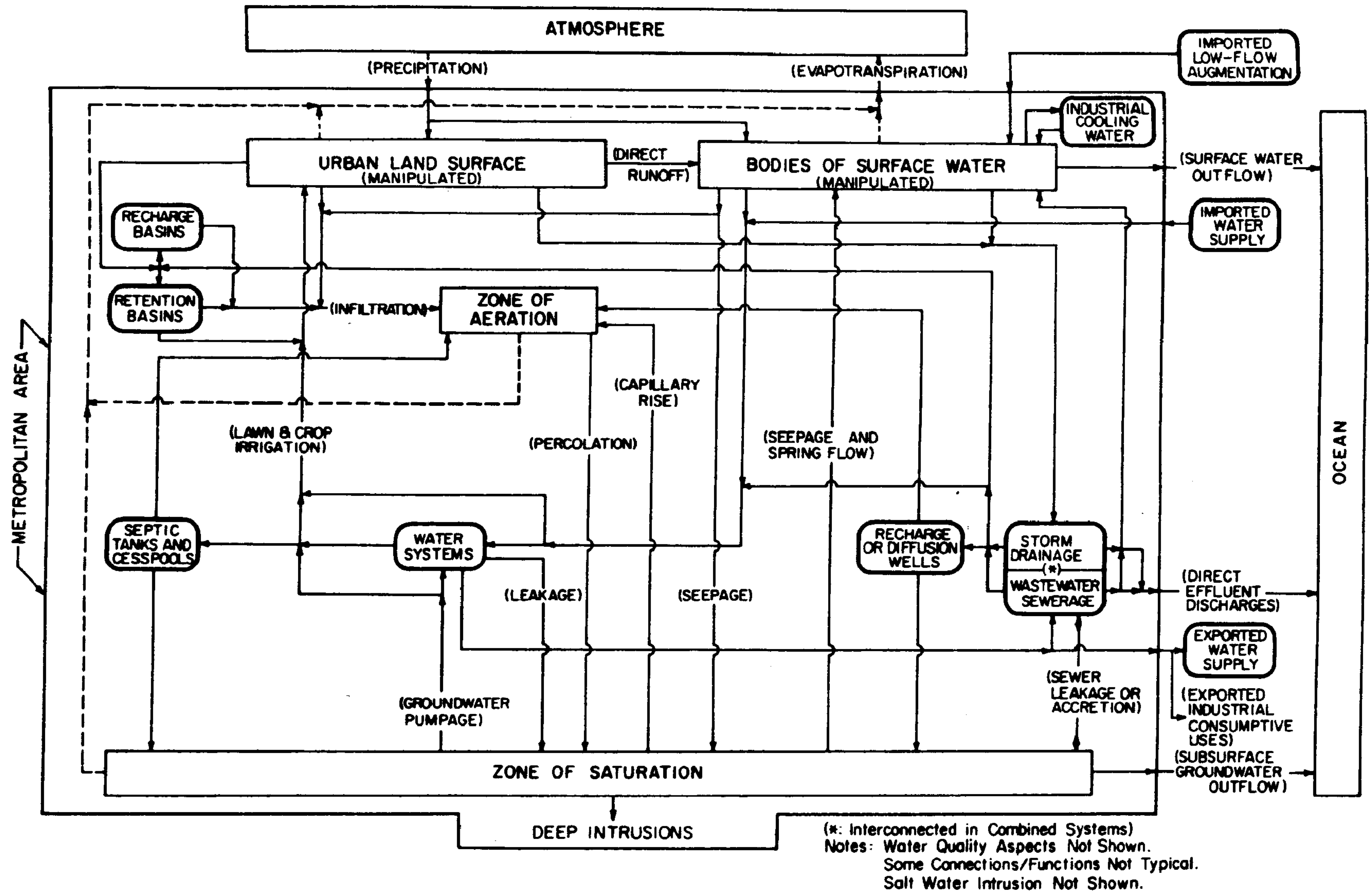

FIGURE 1-URBAN HYDROLOGIC SYSTEM
(Adapted From: "Summary of the Hydrological Situation in Long Island, N.Y. as a Guide to Water Management Alternatives", by O.L. Franke and N.E. McClymonds, U.S. Geological Survey Professional Paper 627-F, 1972).

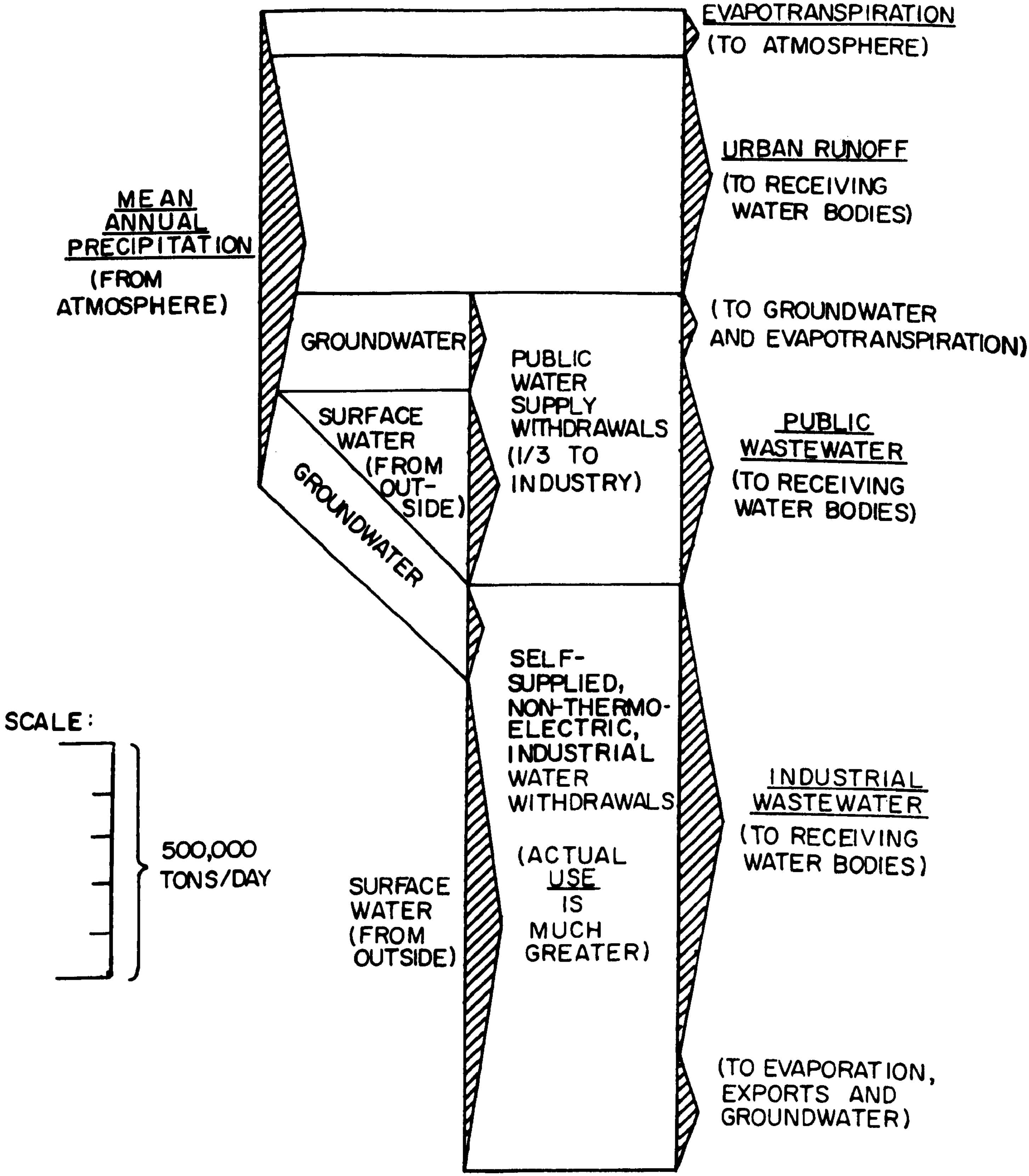

FIGURE 2 - ANNUAL AVERAGE WATER QUANTITY BALANCE FOR AN HYPOTHETICAL URBAN AREA OF ONE-MILLION INHABITANTS, IN TONS PER DAY[1]

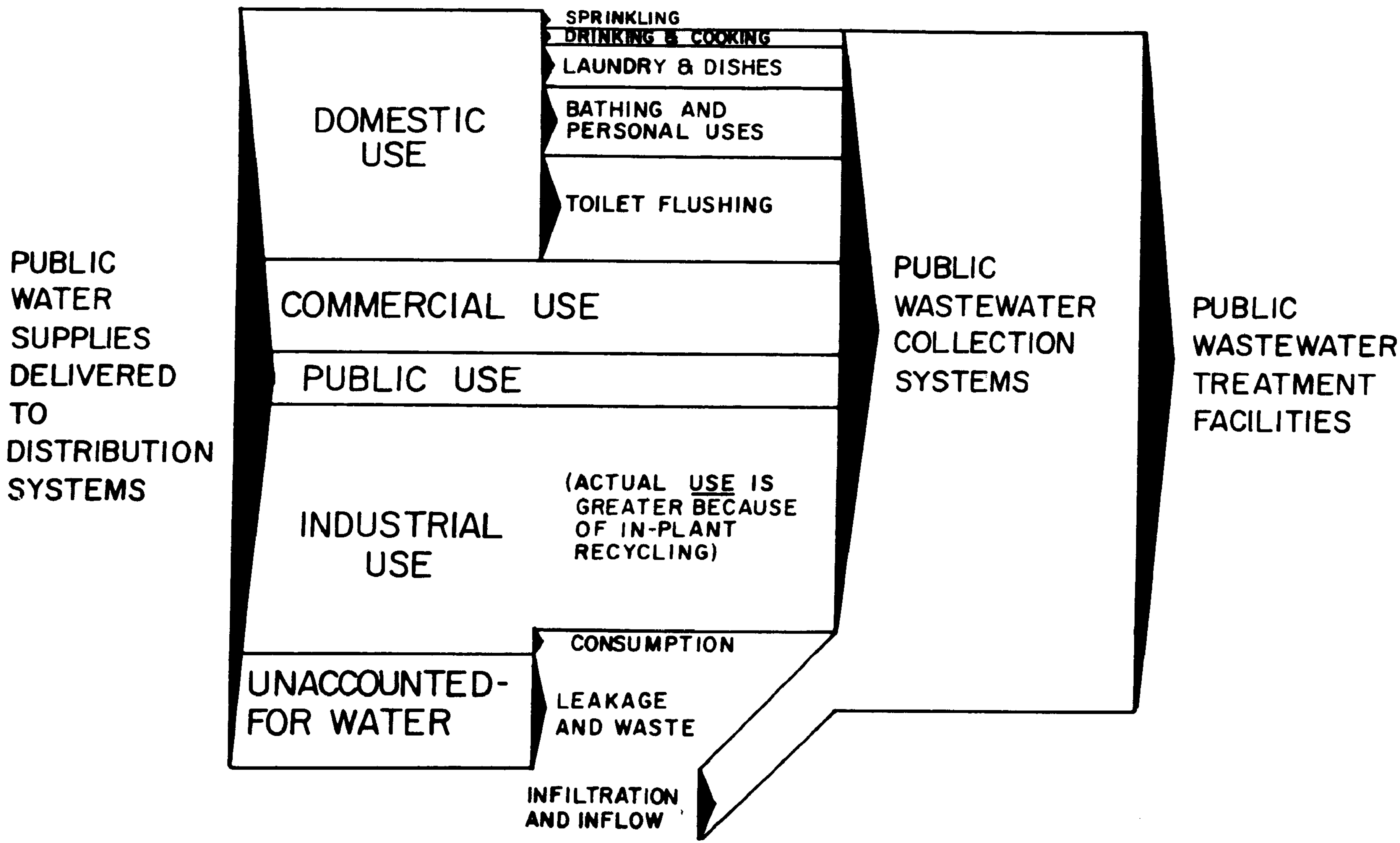

FIGURE 3- COMPOSITE PUBLIC WATER/WASTEWATER INPUT-OUTPUT BALANCE[3]

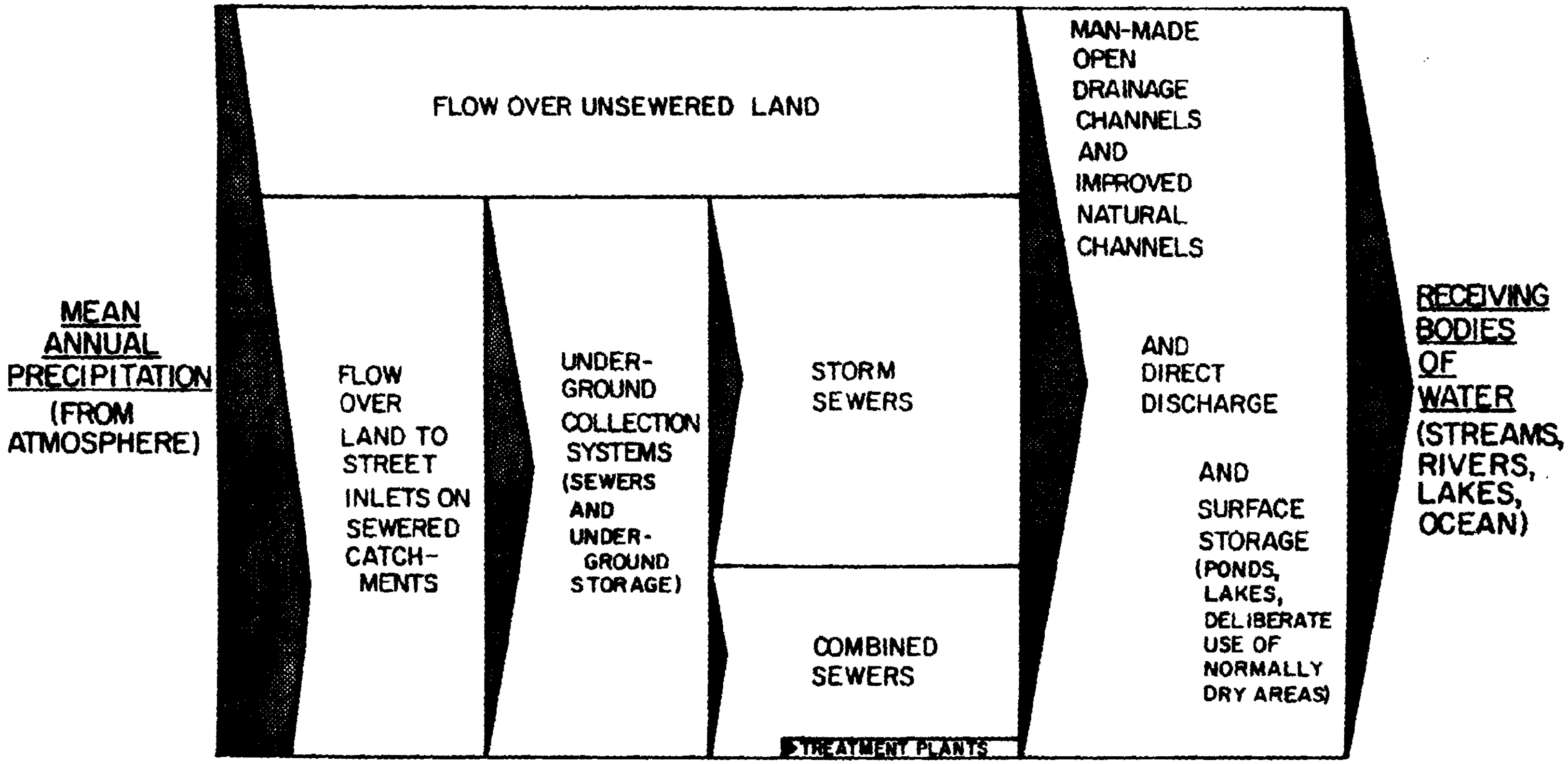

FIGURE 4- COMPOSITE URBAN SURFACE WATER RUNOFF INPUT- OUTPUT BALANCE

averages drawn to scale, and there are reservations about how precise some of these estimates are, we confront much greater uncertainties with urban runoff. The principal reason is that there is a substantial body of data on metropolitan water and wastewater volumes and practically none on storm and combined sewers. The indication in Figure 4 that national annual average flows from storm sewers are about twice as large as from combined sewers is based on an indirect indication[4] of the population served by combined sewers versus wastewater sewers. The indication in Figure 4 that about one-fourth of urban runoff flows over unsewered land is purely conjectural but within reason. After leaving sewer systems, runoff may be stored in surface reservoirs and may pass through artificial and improved channels on its way to receiving waters, and some of it will enter receiving waters directly. There is no basis for estimating the division of national annual averages among these three modes. Further, there are complex mixtures of open drainage channels and surface storage in many metropolitan areas and it might be next to impossible to assign a typical simplified chain of occurrence (vis-a-vis Figure 3) to the flow through these elements, even for a particular metropolitan area. The writer's guess is that nationally less than one-fourth of urban runoff passes through open artificial or improved channels and that one-tenth or less is subjected to surface storage on its way to receiving waters. How much that is subjected to surface storage also passes before or afterwards through open artificial or improved channels is anyone's guess.

Obviously, an input-output balance of urban surface water runoff pollutant burdens would be even more complicated, and much less is known about urban runoff quality than on urban runoff quantity.

No attempt has been made to show the interrelations between groundwater and surface water in Figure 4. While these are taken into account in Figure 1, exactly how to depict them quantitatively in Figure 4 presents quite an enigma because of the complexities involved and the lack of data.

Despite the absence of national indications, as described above, responsible management would appear to call for definitive breakdowns of urban runoff components, in terms of quantity and quality, in each metropolis. How can we solve a presumed problem when we have not assessed its magnitude? Such breakdowns would form a part of a metropolitan water balance inventory, the subject of the remainder of this Addendum.

Metropolitan Water Balances

Comprehensive planning faces a number of obstacles, including fractionalized authority of administrative agencies and balkanized local governments in metropolitan areas,[5] specialist approaches taken individually on each water service component, and uneven advances in knowledge on various water components. More comprehensive management approaches would integrate or give due consideration to all aspects of water in a metropolis, together with related environmental, energy and other relevant considerations. The need for a comprehensive overview becomes more crucial as the complexity of urban areas increases, caused by such things as population migration and growth, competition over available energy supplies, rising expectations of urban dwellers in the face of inflation of local government costs, and greater public awareness of environmental issues and enlarging demands for public participation.

Water balance inventories, describing the quantity and quality aspects of the fate of water from its appearance as precipitation through its departure from a metropolis as runoff and evapotranspiration, have been advocated as a referencing tool for planning in metropolitan areas.[6] Such inventories provide a basis for better recognition of the interrelation, interdependence, and interconnection of the elements of the water resources of a metropolis. "Attempts to study any inventory system apart from the total system does not enable the investigator to see the total impact of alternatives."(7)

In Sweden, urban water inventories have been approached from two directions: the "outer system," embodying natural processes governed by seasonal and random variations; and the "inner system," involved with the conveyance and distribution of water for uses within urban areas. The collective average annual water budget for urban areas in Sweden is shown in Figure 5(8) for the outer system and in Figure 6(8) for the inner system. Numerical values indicated are volumes in millions of cubic meters per year. The total urban water budget can be described as the sum of the outer and inner systems, Figure 5 plus Figure 6, where the sole connections would be "via combined sewers" (340×10^6 - m^3/year) and "receiving waters".

Inventory methods used in San Antonio for the quantification of soil water, surface water and groundwater have been described.(9) Long Island, New York, an instance of a nearly "closed" hydrologic system covering about 1,400 square miles, has been the subject of extensive water-budget studies.(10)

Figure 7(11) is a water balance, on an annual average basis, for metropolitan Chicago (3,714-square miles). All but the smallest of the seven drainage basins involved receive flows of water from outside the metropolitan area. Variations within individual years were also inventoried. "Water in the metropolitan area is constantly on the move. The three principal components of the water resource (atmospheric moisture, surface water and groundwater) are interconnected, and water moves in a continuous cycle between them. Any comprehensive study of water resources, therefore, must first recognize the existence of this cycle and then define the total water system, even while realizing that only a limited part of the water moving through the system is actually available for use. The situation is further complexed in that the metropolitan area is part of several larger systems which embrace much of the United States."(11)

Getting the Act Together

In an effort to facilitate the preparation of water balance inventories, an attempt has been made in Figure 1 to describe, in flow-chart form, the urban hydrologic system,(12) of which stormwater is a part or a subsystem. Figure 8(13) describes the urban stormwater runoff subsystem, interpreted from a drainage standpoint, and deals with water quantities. In contrast, Figure 9(14) is a schematic diagram of the relationship between urban activities and water quality, where urban runoff is a major element. Not many urban runoff control master plans have been developed, fewer still have been implemented, and rare are the instances where master plans have integrated quality control with quantity control. All this despite the increasing importance of their conjunctive planning.

Strategies for improving water quality have been delineated from the perspective of the urban planner concerned with stormwater control.(15)

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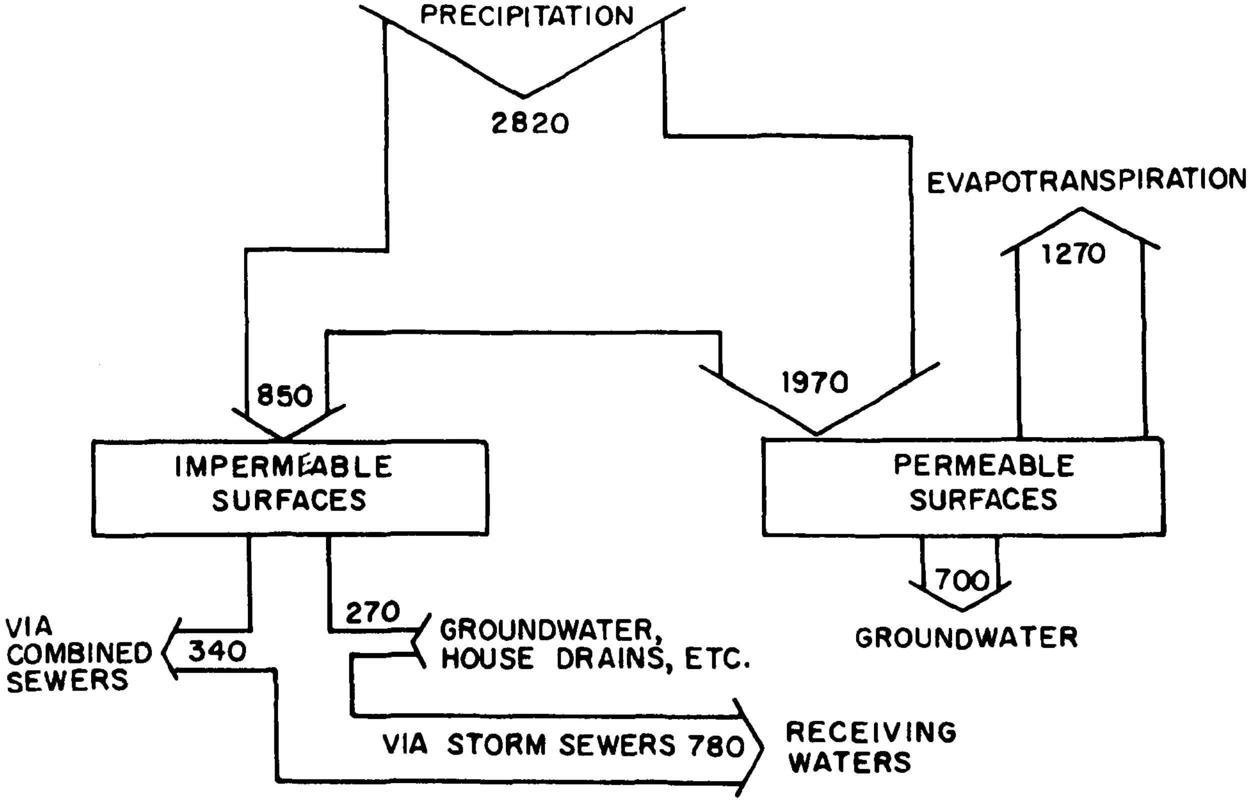

FIGURE 5 - GENERAL URBAN AREA WATER BUDGET FOR SWEDEN, OUTER SYSTEM.(8)

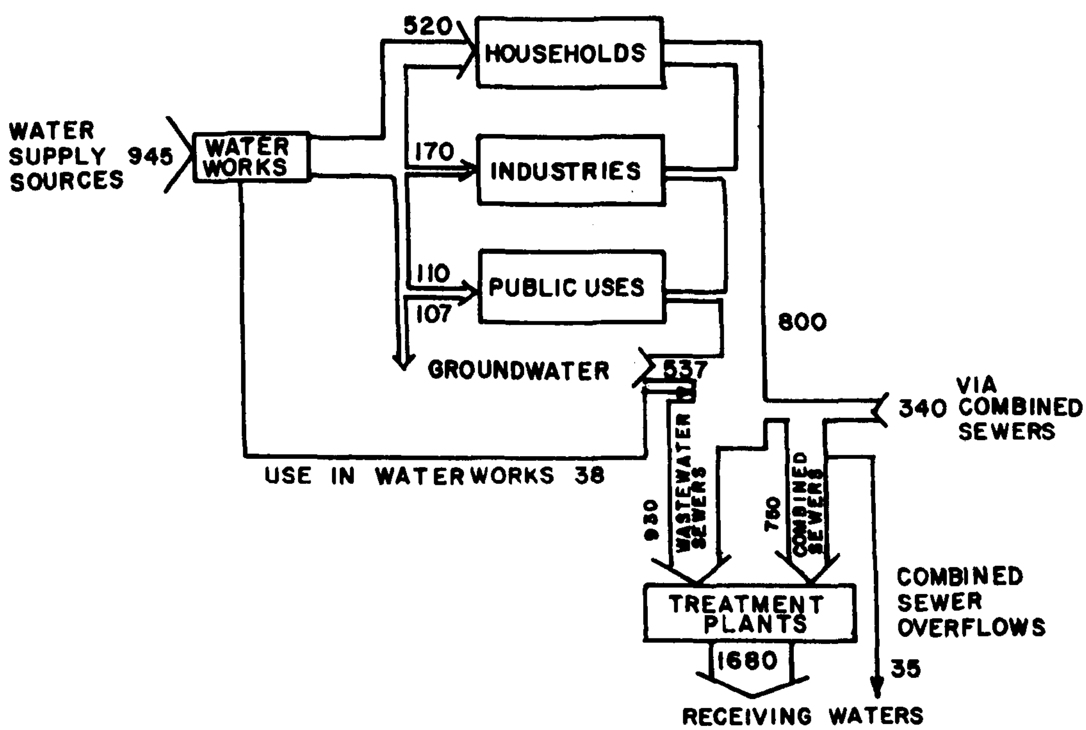

FIGURE 6- GENERAL URBAN AREA WATER BUDGET FOR SWEDEN, INNER SYSTEM.(8)

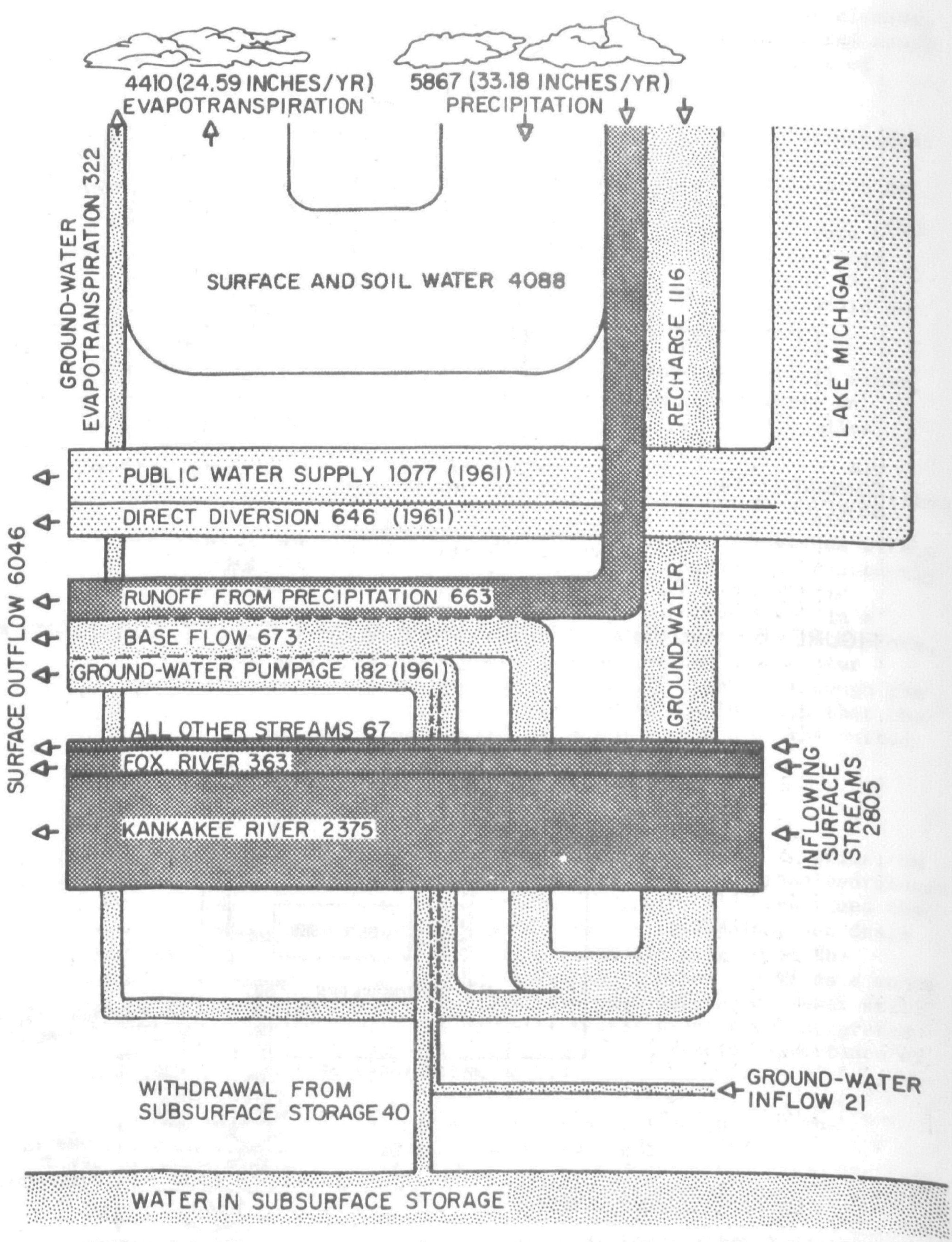

FIGURE 7 - AVERAGE ANNUAL WATER BALANCE, METROPOLITAN CHICAGO (MILLION GALLONS PER DAY, 1961)[11]

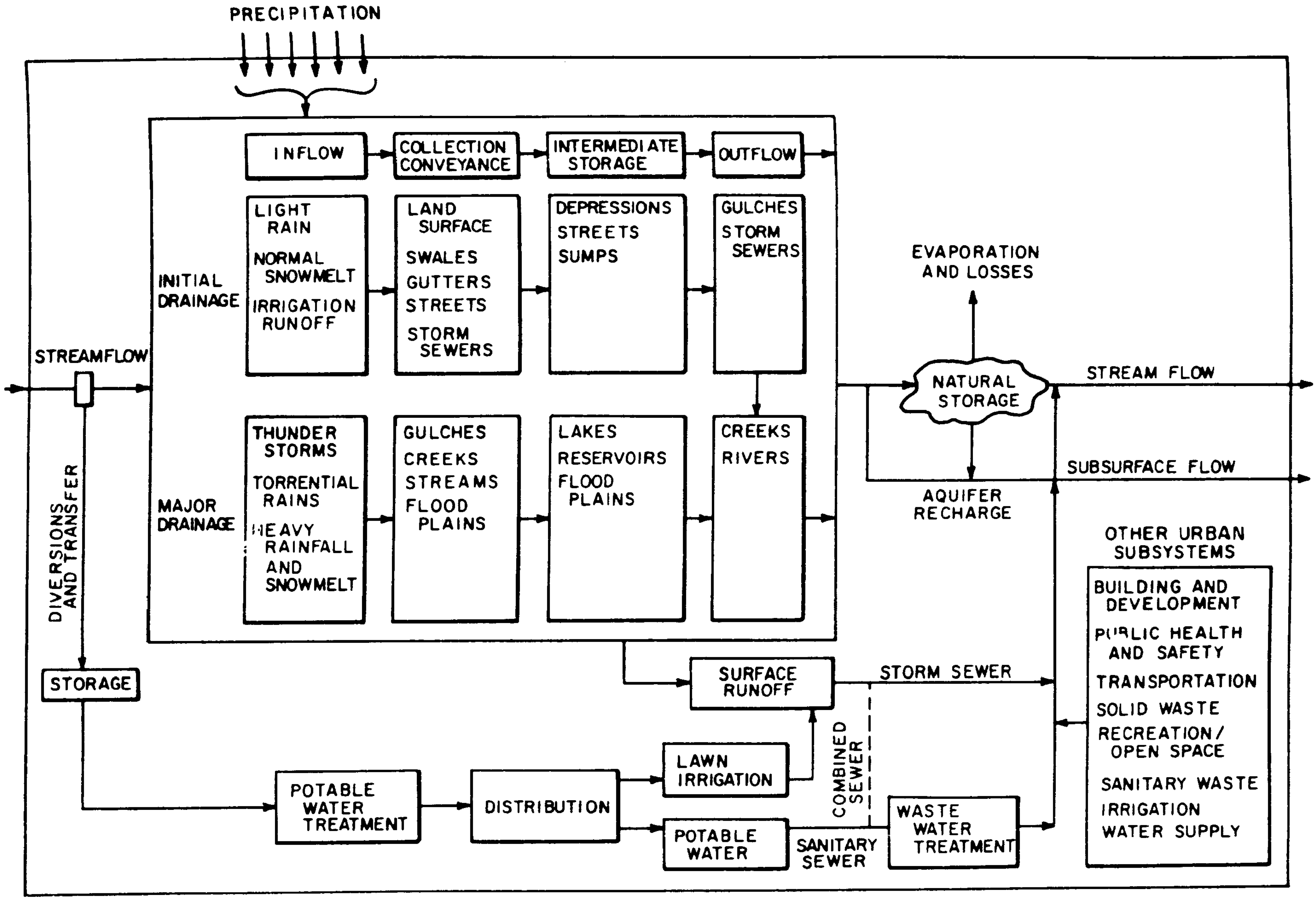

FIGURE 8 - THE URBAN STORM DRAINAGE SUBSYSTEM[13]

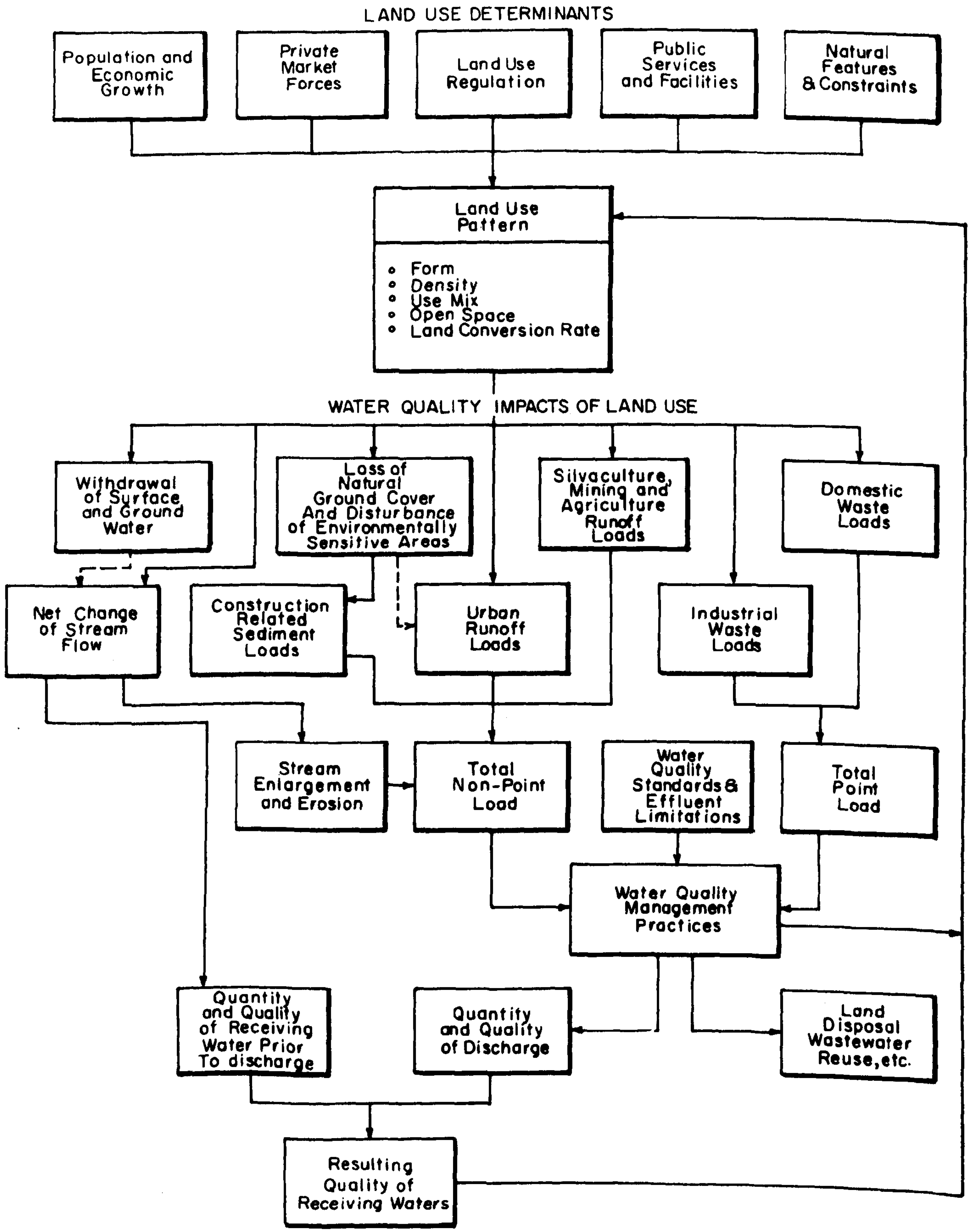

FIGURE 9- SCHEMATIC DIAGRAM OF THE LAND-USE/WATER-QUALITY RELATIONSHIP[14]

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ADDENDUM 2

THE DESIGN STORM CONCEPT

by M. B. McPherson

Introduction

Historically, urban areas have been drained by underground systems of sewers that were intentionally designed to remove stormwater as rapidly as possible from occupied areas. Discharges from conventional storm drainage sewer facilities and flood-plain intrusion by structures both tend to aggravate flooding, and thereby jointly tend to raise the potential for stream flooding damages. The advantages of local detention storage in lieu of the traditional rapid removal of storm flows has long been recognized(1) and there is evidence that the usage of such storage is on the rise.(2) However, local detention storage has been only occasionally employed as part of overall flood mitigation, as in Denver(3) and Fairfax County, Virginia.(4)

Detention storage is recognized as one of the principal means for abatement of pollution from stormwater discharges and combined sewage overflows.(5) There are opportunities in new land development to incorporate detention storage at the ground surface.(6) However, for existing drainage systems, there may be few opportunities to add detention storage except underground, for both combined sewer and separate storm sewer systems. Because abatement of pollution from the dispersed sources served by drainage systems began with a focus on combined systems, our knowledge on storage requirements for them is greater. Recognized very early was the need for some form of automatic control,(7) because of system complexity and the need to manipulate flows in order to insure their containment as a means for reducing overflows.

System Performance

When rapid removal of stormwater was the primary objective, hydrological considerations were mostly restricted to the sizing of conduits. Retardation of flows and attenuation of their peaks is another matter, as is containment of flows and their diversion for treatment to abate pollution. Whereas simple concepts sufficed for conduit sizing, their perpetuation in more complex applications inevitably leads to lowered reliability.

An attempt is made herein to indicate how precipitation data can best be employed in planning and design of stormwater facilities under emerging imperatives. Responsibilities of local government operating agencies were formerly restricted to the management of underground conduit systems that had been deliberately designed to be overloaded upstream of their outlets rather often, such as once in five years on the average, with little regard for the impact of system flows on receiving waters. Thus, the sole criterion of performance was how effectively the land had been drained. Once containment of flows and their pollutant burdens become additional objectives, performance criteria enlarge to include the meeting of regulations and statutes. As a result, public officials will no longer be able to depend only on the relatively obvious incidence of system overloading evidenced by flooded basements and streets. They will also have to show concrete evidence of compliance acceptable to regulatory agencies. This means that they will have to know when, how much and how often allowable limits have been exceeded. Regulatory limits necessarily have to be founded on some level of frequency, or probability, such as containment on the average of all but one combined sewer overflow every five years, or of all but a certain volume of stormwater over a season, or that would permit no more than a certain maximum pollutant discharge per storm.

The only way an event frequency can be determined is by referring to a reasonably large series of prior events. A long history of overflow or storm discharge amounts is almost never available and, even if it were, it would not be suited for systems revised with new storage and controls. Thus, recourse is made to simulation of system performance via some sort of calculation, such as the use of hydrological models. That is, a historical record of rainfall is transformed by calculation into a simulated historical record of stormwater flow and possibly quality. Simulation of an existing system thereby yields a synthesized historical record of pre-modification conditions. System revisions are then targeted to contain all but some relatively rare events from the synthesized performance record, as prescribed by regulations. But how will compliance be judged after a system has been modified to meet a regulation? Of course, monitoring all overflow/discharge points by means of rainfall-runoff-quality instrumentation would indicate when violations had occurred, but would not indicate why. Further, reality dictates that, from an economical standpoint, only a sample of overflow/discharge points could probably be monitored, and simulation of post-modification performance can be employed to interpret sparsely monitored systems and explain whether or not violations have occurred and why.

At least a decade ago there was widespread acceptance of the need for complete storm hydrograph simulation if major system modifications were contemplated, particularly if new storage was involved. However, as we enter an age of regulation another need arises, for simulation of the performance of systems modified to meet regulations. Unfortunately, concepts that were adequate for sizing conduits have been carried into the sizing of detention storage and now threaten to be perpetuated in the future testing of compliance performance in flooding relief and pollution abatement facilities planned for construction. For this reason, it is necessary to review concepts that have been used for decades, to show why they are not suited for investigating performance, either expected or experienced. These now outmoded concepts originated in connection with the sizing of conduits.

Simplistic Traditions

The overwhelmingly used technique for sizing combined and storm sewers is known as the "rational method".[8] Its limitations have been discussed at length elsewhere,[9] and present purposes are served by concentrating on its fundamental premise.

Intuition, theory and laboratory tests indicate that when a rain of a constant intensity falls on a catchment, the outflow from the catchment will ultimately rise to a maximum rate, which would be sustained as long as the constant rainfall intensity continued thereafter. The time required to reach the maximum outflow rate (to reach a state of equilibrium) is commonly called the "time of concentration". The equilibrium discharge is merely a fixed fraction of the constant rainfall intensity in the rational method, with the ratio of equilibrium discharge to rainfall intensity defined by a constant that is usually interpreted in terms of type of land use. Because for a given catchment, equilibrium discharge is a fixed multiple of input constant rainfall intensity, it follows without reservation that the frequency* of an equilibrium discharge is identical with the frequency of its associated constant rainfall intensity.

Storm and combined sewers are usually designed to flow full without surcharging at some selected frequency, such as on an average of once in five years or once in ten years. Thus, the only type of rainfall information needed for use of the rational method is an array of intensities for each of several durations from which various frequencies can be interpolated. The result is called "intensity-duration-frequency curves". Such curves have been developed in a number of municipalities and advantage has been taken of the spatial persistence of curve values in the preparation of national maps that contain lines of equal rainfall for given durations and frequencies, such as by Yarnell,(10) Hathaway(11) and Hershfield.(12) Certain characteristics of such information must be reviewed next in order to demonstrate some rather severe limitations.

Intensity-Duration-Frequency Relations

The almost universal data source is the U.S. Weather Service, from its first-order stations in principal cities.

In principle, one would search the recorded rainfall for each storm at a given station for the largest catch over a particular duration. Because engineers have been more interested in the rarer events, and to reduce the data processing and reporting burden, the USWS processes and reports data for only those storms where at least some rainfall depths exceed a specified threshold level. The level is sufficiently low to include 1-year frequency and somewhat lower values. Data from storms having depths above the threshold are termed "excessive precipitation". Because for a given duration the approximately largest depth of rainfall is reported, it is therefore the maximum amount for that duration for that storm at that gage. Because there is considerable variability in rainfall over time, the selected amount, when expressed as an intensity, is necessarily the average intensity for the given duration. Hence, we will hereinafter use the term maximum average when describing excessive precipitation.

In deriving intensity-duration-frequency relations, rainfall values for each duration are regarded independently from other durations, the first step being a separate ranking of values for each duration in descending order of size. A mathematical fit is made to the array of depths or intensities for each duration and the line of best fit values are computed. In the last step, fitted values for each duration for a specified mean recurrence interval (or frequency) are plotted with intensity as ordinate and duration as abscissa; and smooth fitted lines are drawn through points of equal frequency.

Starting in 1936 the USWS published maximum average excessive precipitation for various durations, presumably to facilitate rational method applications, but which makes it necessary to refer to the National Climatic Center, NOAA Environmental Data Service, Asheville, N.C., for the true sequence of occurrences. From 1895 through 1935 the USWS published maximum average excessive precipitation as it had accumulated over time, mostly for a 5-minute interval. The report of a study of frequency analysis methods includes a listing of such data for 1913-1935 for Chicago,(13) used in the succeeding discussion.

*: (The terms "frequency," "return period" and "recurrence interval" are used interchangeably in drainage practice, as they will be here. Frequency is the reciprocal of probability).

Listed in Table 1 are the top 14 values of maximum average excessive precipitation for Chicago, 1913-1935. Casual inspection of Table 1 will reveal the fact that maximum average depths for a given average return period are not necessarily from the same storm, e.g. see Rank No. 5. Inspection of actual records of accumulation would reveal that 5-minute maximum average depths did not necessarily occur in the first five minutes, and so on for other durations. Also, the record includes storms that lasted as little as five minutes, with a majority of the 130 storms reported lasting less than 35-minutes, and with very few lasting more than an hour.

Further, in order to insure a complete data matrix for regression analysis to develop frequency curves, a dummy value is inserted for all durations beyond the cessation of excessive precipitation for a given storm. For example, for the storm of 7/16/14, to be examined later, 1.61-inches had accumulated at the end of the storm, which lasted 35-minutes, and for frequency analysis purposes a fictitious value of 1.61-inches would be inserted for durations from 40 to at least 120 minutes for that storm prior to the ranking of maximum averages. The insertion of dummy values (called "extended duration" values) is consistent with the underlying concept of the rational method, but further distorts intensity-duration-frequency curves from the actual history of occurrences. This distortion is illustrated in Figure 1, where maximum average values of rainfall for a T_E (the average return period) of 4.6-years and 5.8-years from Table 1 have been converted into intensities. Compared is a "5-year" curve for Chicago attributed to Eltinge and Towne.(14) The falloff of plotted points beyond a duration of 40-minutes, compared with the uninterrupted smooth curve, reflects the exclusive use of actual data for the former and the traditional use of extended duration values for the latter. Note that the plotted points in Figure 1 cease at 80-minutes because only one storm from the record used had lasted any longer but was a rarer value.

In referring to a frequency curve in the context of the rational method, some engineers erroneously use the term "storm frequency". For example, values taken from a 5-year curve or the resulting computed flow rate are often stated to be for "a five year storm". Considering that a given frequency curve can represent values from different storms, in a time sequence different than actually occurred, and might include non-existent dummy values, it is obvious that any reference to the term "storm" in rational method applications is at least misleading and is certainly technically imprecise.

Further, it should be noted that the time of concentration varies from point to point in a catchment area. Thus, using a given frequency curve, various portions of a drainage area may be designed on the basis of pieces of different storms that might have occurred several years apart. Hence, the presentation of water surface profiles or hydraulic gradients from upstream extremities to the outlet for "design conditions" can be greatly misleading.

In summary, the term "storm" should never be used in connection with the rational method, whereas the term "rainfall" is adequately vague and innocuous. In fact, an outstanding limitation of the rational method stems from its complete independence from storm pattern. To reiterate, the maximums of the several durations from a given storm record as used in the rational method are not necessarily in their original sequential order; and the resulting tabulation of maximums ordered by size of duration may bear little resemblance to the original storm pattern.

TABLE 1 - RANKING BY MAXIMUM DEPTH FOR VARIOUS DURATIONS, CHICAGO, 1913-1935

RANK	AVERAGE RETURN PERIOD, YEARS	MAXIMUM DEPTH IN INCHES FOR STATED DURATIONS; AND DATE OF RAINFALL													
		5-min.	10-min.	15-min.	20-min.	25-min.	30-min.	35-min.	40-min.	45-min.	50-min.	60-min.	80-min.	100-min.	120-min.
1	23.0	0.55 (6/26/32)	0.96 (8/11/31)	1.22 (8/11/31)	1.39 (8/11/23)	1.58 (6/20/28)	1.70 (6/20/28)	1.86 (6/20/28)	1.92 (6/20/28)	2.04 (6/20/28)	2.06 (6/20/28)	2.30 (6/20/28)	2.30 (8/11/23)	1.36 (6/23/31)	1.53 (6/23/31)
2	11.5	0.53 (6/29/20)	0.94 (6/20/32)	1.16 (6/20/28)	1.39 (8/11/31)	1.49 (8/11/23)	1.61 (8/11/23)	1.77 (6/13/26)	1.85 (6/13/26)	2.03 (6/29/33)	1.85 (8/11/23)	2.02 (8/11/23)	1.55 (9/10/22)	-	-
3	7.7	0.51 (8/11/31)	0.88 (6/20/28)	1.16 (6/26/32)	1.38 (6/20/28)	1.45 (7/7/21)	1.55 (6/13/26)	1.71 (8/11/23)	1.81 (6/29/33)	1.80 (8/11/23)	1.40 (7/19/31)	1.53 (9/10/22)	1.54 (8/5/24)		
4	5.8	0.50 (6/20/28)	0.80 (8/11/23)	1.12 (8/11/23)	1.35 (7/7/21)	1.34 (6/13/26)	1.49 (7/7/21)	1.61 (7/16/14)	1.77 (8/11/23)	1.31 (7/19/31)	1.23 (9/10/22)	1.27 (8/10-11/31)	1.45 (8/19/21)		
5	4.6	0.48 (8/8/27)	0.80 (7/2/33)	1.08 (7/7/21)	1.21 (8/27/31)	1.29 (10/5/19)	1.48 (7/16/14)	1.59 (7/20/24)	1.75 (7/20/24)	1.25 (6/24/25)	1.17 (8/10-11/31)	1.22 (8/5/24)	1.23 (3/31/29)		
6	3.8	0.45 (8/11/23)	0.76 (7/7/21)	1.00 (7/2/33)	1.16 (7/2/33)	1.27 (8/27/31)	1.44 (10/5/19)	1.53 (10/5/19)	1.24 (7/19/31)	1.14 (9/10/22)	1.15 (8/5/24)	1.21 (7/11/22)	1.22 (6/23/31)		
7	3.3	0.42 (8/27/31)	0.74 (10/5/19)	0.96 (8/27/31)	1.13 (6/13/26)	1.25 (7/20/24)	1.43 (7/20/24)	1.53 (6/29/33)	1.23 (8/15/34)	1.12 (8/5/24)	1.12 (7/11/22)	1.13 (6/23/31)	0.96 (7/5/30)		
8	2.9	0.41 (10/5/19)	0.74 (8/27/31)	0.93 (10/5/19)	1.10 (10/5/19)	1.21 (7/16/14)	1.38 (6/29/33)	1.38 (8/13/25)	1.22 (9/5/20)	1.06 (7/11/22)	1.02 (6/23/31)	1.12 (8/19/21)	-		
9	2.6	0.41 (8/13/25)	0.71 (8/2/35)	0.87 (6/13/26)	1.05 (7/20/24)	1.21 (8/13/25)	1.31 (8/13/25)	1.30 (8/29/28)	1.13 (6/24/25)	1.05 (8/10-11/31)	0.87 (6/15/25)	0.99 (6/24/28)			
10	2.3	0.41 (8/2/33)	0.68 (8/13/25)	0.87 (8/2/35)	1.05 (8/13/25)	1.18 (6/29/33)	1.20 (8/29/28)	1.21 (8/15/34)	1.10 (9/10/22)	1.00 (6/23/31)	0.80 (9/17/27)	0.97 (9/17/27)			
11	2.1	0.39 (9/5/20)	0.68 (5/18/26)	0.86 (7/20/24)	0.97 (7/16/14)	1.07 (8/15/34)	1.19 (8/15/34)	1.17 (7/19/31)	1.08 (7/19/16)	0.98 (6/12/15)	0.79 (3/31/29)	0.89 (3/31/29)			
12	1.9	0.39 (7/7/21)	0.68 (6/29/33)	0.86 (6/18/35)	0.96 (6/29/33)	1.05 (9/5/20)	1.12 (9/5/20)	1.16 (9/5/20)	1.08 (8/5/24)	0.82 (6/15/25)	0.78 (4/10/22)	0.82 (7/5/30)			
13	1.8	0.39 (7/20/24)	0.67 (6/18/35)	0.84 (8/13/25)	0.96 (8/2/35)	1.05 (7/1/27)	1.11 (7/1/27)	1.09 (6/25/26)	1.06 (8/2/35)	0.74 (9/17/27)	0.78 (6/24/28)	0.81 (9/23/26)			
14	1.6	0.38 (5/18/26)	0.66 (7/16/14)	0.83 (8/9/14)	0.95 (5/18/26)	1.04 (5/18/26)	1.05 (8/2/35)	1.05 (7/19/16)	1.03 (6/18/35)	0.73 (4/10/22)	0.74 (8/19/21)	-			

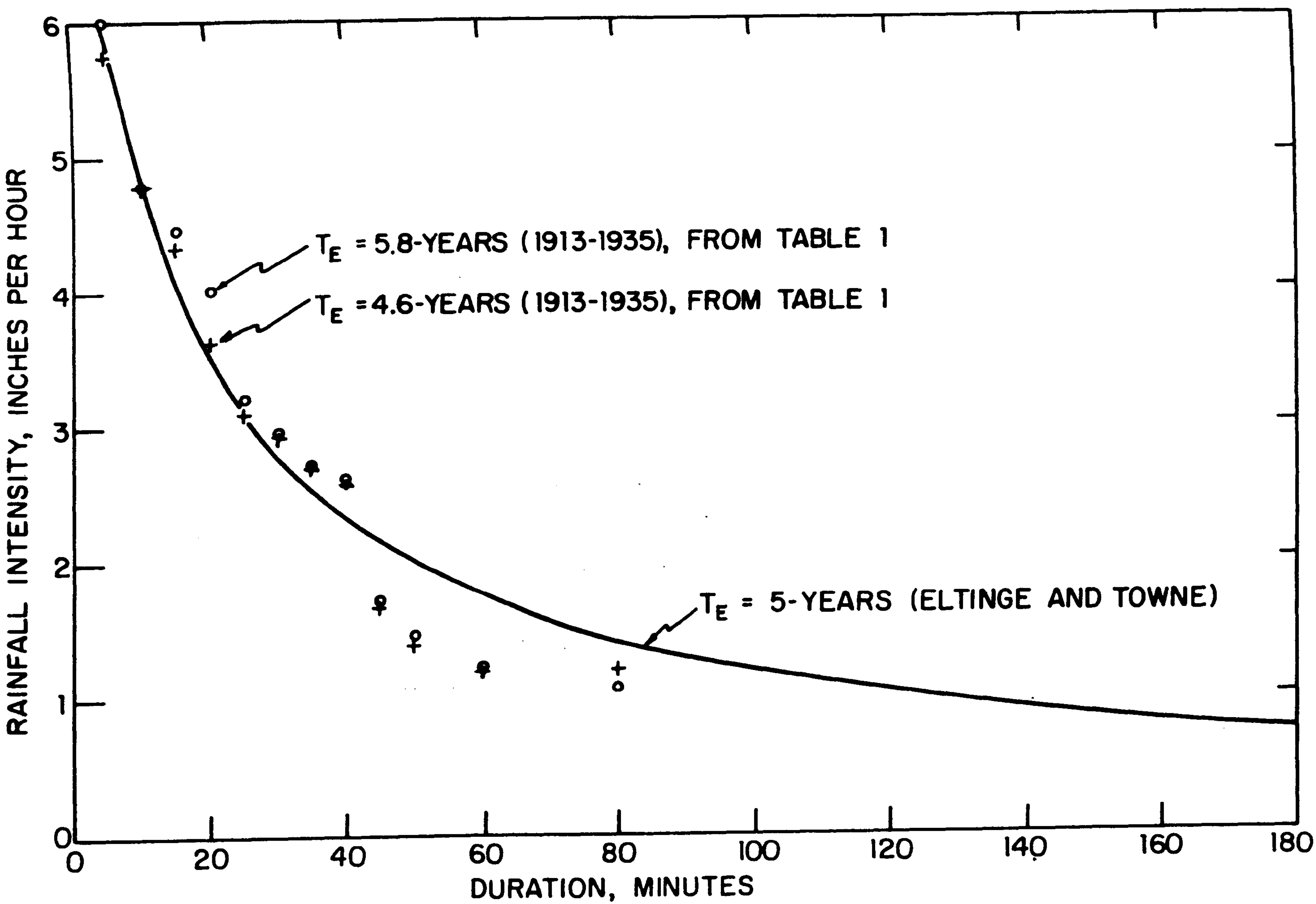

FIGURE 1 - INTENSITY-DURATION-FREQUENCY COMPARISONS, CHICAGO DATA

Volumetric Considerations

Because the sizing, deployment and operation of detention storage facilities is determined by the time patterns and amounts of rainfalls selected, the use of actual precipitation records is greatly preferred, for reasons already given. Listed in Table 2 are the top 14 storms in terms of total rainfall depth for Chicago, 1913-1935, for the same record from which Table 1 was derived. Shown are the recorded depths for successive intervals of time. It may be noted that the larger amounts of rain occur early, late, and in the middle of storms, and some have double peaks, such as Rank No. 5.

In order to illustrate the mixture of individual frequencies of separate parts of storms, the actual depths for two storms given in Table 2 have been traced through Table 1 (and its extension, not shown), and the result is given in Table 3. Note the wide disparity in individual storm component frequencies. Also to be considered is the distortion in timing of the maximum average intensities:

		Occurrence During Storm of Maximum Average Depth for Stated Duration in Minutes							
Rank	Storm	5	10	15	20	25	30	35	40
4	6/13/26	30-35	30-40	20-35	20-40	15-40	10-40	5-40	0-40
5	7/16/14	20-25 and 25-30	20-30	20-35	10-30	5-30	0-30	0-35	-

which may be checked by inspection of Table 2.

Hopefully, some indication has been presented of the folly of attempting to define a design storm by using concepts applicable solely to the rational method.

Another Example

As a companion to Table 2, listed in Table 4 are the top 14 storms in terms of total rainfall depth for Philadelphia, 1913-1935. What has been said about the Chicago data is generally applicable for the Philadelphia data. In addition, because the writer has been supplied a copy of the complete USWS record by the Philadelphia Water Department, certain features from that record can be cited —

- For the storm of August 7, 1921, 0.65-in. fell between 3:40 p.m. and 5:05 p.m., 0.81-in. between 6:10 p.m. and 7:00 p.m., and the 1.91-in. indicated in Table 4 followed between 10:15 p.m. and 11:40 p.m.

- For the storm of May 27, 1918, 1.28-in. fell between 7:38 p.m. and 8:45 p.m., followed by a 0.25-in., and then the 1.62-in. indicated in Table 4 followed between 10:18 p.m. and 11:33 p.m.

- For the storm of June 27, 1914, the 1.60-in. indicated in Table 4 fell between 3:21 p.m. and 4:15 p.m. and was followed by 0.56-in. between 5:40 p.m. and 8:10 p.m.

- Note that in Table 4 the storm of July 10, 1931 with a total depth of 1.57-in. was preceded only three days earlier by a storm with a total depth of 1.43-inches. Also, 0.91-in. fell in a little over half an hour on July 14, 1931, followed by 1.03-in. on July 18, 1931, which fell mostly in a 26-minute period.

TABLE 2 - RANKING BY DEPTH OF STORM, CHICAGO, 1913-1935

RANKING BY MAGNITUDE OF TOTAL STORM DEPTH			RECORDED DEPTH IN INCHES FOR STATED TIME INTERVAL IN MINUTES													
RANK	AVERAGE RETURN PERIOD, YEARS	TOTAL STORM DEPTH, INCHES	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50	50-60	60-80	80-100	100-120
1	23.0	2.30 (6/20/28)	0.22	0.50	0.38	0.28	0.20	0.12	0.16	0.06	0.12	0.02	0.24			
2	11.5	2.30 (8/11/23)	0.10	0.27	0.35	0.45	0.32	0.09	0.13	0.06	0.03	0.05	0.17	0.28		
3	7.7	2.03 (6/29/33)	0.31	0.37	0.12	0.05	0.25	0.28	0.15	0.28	0.22					
4	5.8	1.85 (6/13/26)	0.08	0.22	0.21	0.21	0.31	0.23	0.33	0.26						
5	4.6	1.61 (7/16/14)	0.27	0.24	0.19	0.12	0.33	0.33	0.13							
6	3.8	1.55 (9/10/22)	0.08	0.23	0.08	0.04	0.16	0.15	0.13	0.01	0	0.21	0.44	0.02		
7	3.3	1.54 (8/5/24)	0.12	0.12	0.23	0.10	0.12	0.11	0.08	0.20	0.04	0.03	0.07	0.32		
8	2.9	1.53 (10/5/19)	0.09	0.19	0.33	0.41	0.17	0.19	0.15							
9	2.6	1.53 (6/23/31)	0.18	0.23	0.26	0.09	0.12	0.08	0.03	0.01	0	0.02	0.11	0.09	0.14	0.17
10	2.3	1.49 (7/7/21)	0.32	0.39	0.37	0.27	0.10	0.04								
11	2.1	1.45 (8/19/21)	0.20	0.16	0.22	0	0	0	0	0	0.07	0.09	0.38	0.33		
12	1.9	1.40 (7/19/31)	0.15	0.21	0.32	0.23	0.07	0.06	0.13	0.07	0.07	0.09				
13	1.8	1.39 (8/11/31)	0.26	0.45	0.51	0.17										
14	1.6	1.38 (8/13/25)	0.10	0.21	0.16	0.27	0.41	0.16	0.07							

TABLE 3 - RETURN PERIODS FOR MAXIMUM DEPTHS
OF TWO STORMS, CHICAGO, 1913-1935

RANK	AVERAGE RETURN PERIOD, YEARS	FROM RANKING BY DEPTH OF STORM	OCCURRENCE OF MAXIMUM DEPTHS FOR GIVEN STORMS							
			5-min.	10-min.	15-min.	20-min.	25-min.	30-min.	35-min.	40-min.
1	23.0									
2	11.5								6/13/26	6/13/26
3	7.7							6/13/26		
4	5.8	6/13/26					6/13/26		7/16/14	
5	4.6	7/16/14						7/16/14		
6	3.8									
7	3.3					6/13/26				
8	2.9						7/16/14			
9	2.6				6/13/26					
10	2.3									
11	2.1					7/16/14				
12	1.9									
13	1.8									
14	1.6			7/16/14						
15	1.5									
16	1.4									
17	1.3									
18	1.3				7/16/14					
19	1.2									
20	1.2			6/13/26						
21	1.1									
22	1.0									
23	1.0									
24	1.0									
25	0.9									
26	0.9		6/13/26							
27	0.8		7/16/14							

TABLE 4 - RANKING BY DEPTH OF STORM, PHILADELPHIA, 1913-1935

RANKING BY MAGNITUDE OF TOTAL STORM DEPTH			RECORDED DEPTH IN INCHES FOR STATED TIME INTERVAL IN MINUTES													
RANK	AVERAGE RETURN PERIOD, YEARS	TOTAL STORM DEPTH, INCHES	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50	50-60	60-80	80-100	100-120
1	23.0	2.70 (6/26/30)	0.38	0.32	0.32	0.29	0.08	0.02	0.04	0.05	0.10	0.08	0.33	0.69		
2	11.5	2.09 (8/16/17)	0.41	0.30	0.37	0.23	0.49	0.27	0.02							
3	7.7	1.91 (8/7/21)	0.12	0.02	0.08	0.24	0.24	0.17	0.18	0.04	0.02	0.02	0.06	0.62	0.10	
4	5.8	1.82 (7/15/26)	0.06	0.04	0.05	0.05	0.17	0.14	0.13	0.26	0.24	0.13	0.15	0.40		
5	4.6	1.82 (4/21/27)	0.07	0.08	0.13	0.09	0.08	0.11	0.07	0.01	0.20	0.24	0.31	0.20	0.23	
6	3.8	1.76 (7/3-4/26)	0.12	0.11	0.18	0.15	0.15	0.09	0.08	0.16	0.15	0.24	0.28	0.05		
7	3.3	1.74 (5/24/33)	0.38	0.49	0.29	0.23	0.12	0.03	0.02	0.01	0	0.02	0.05	0.06	0.02	0.02
8	2.9	1.63 (7/13/19)	0.06	0.05	0.14	0.28	0.04	0.02	0.20	0.36	0.23	0.18	0.07			
9	2.6	1.62 (5/27/18)	0.09	0.08	0.03	0.04	0.11	0.23	0.20	0.11	0.15	0.08	0.05	0.45		
10	2.3	1.60 (6/27/14)	0.15	0.22	0.21	0.21	0.14	0.09	0.01	0.02	0.22	0.20	0.13			
11	2.1	1.57 (7/10/31)	0.25	0.18	0.20	0.16	0.16	0.03	0.06	0.02	0.10	0.21	0.20			
12	1.9	1.48 (7/7/31)	0.32	0.40	0.46	0.30										
13	1.8	1.43 (6/20/13)	0.34	0.19	0.20	0.22	0.38	0.10								
14	1.6	1.39 (8/28-29/23)	0.08	0.10	0.11	0.13	0.12	0.11	0.08	0.12	0.15	0.15	0.24			

On August 3, 1898, 5.43-inches were recorded for the period between 10:50 a.m. and 12:35 p.m., only 105 minutes; and on September 14 and 15, 1904, an equal amount was recorded, 3.65-in. in 149 minutes preceded by almost 3-in. which had ended only about seven hours earlier and most of which occurred in a period of only one hour.

These examples should give a clear indication of why synthetic storms can be a liability in detention storage evaluations. Only by referring to actual records as a basis for simulating performance can multiple events be realistically taken into account. Additionally, a synthetic storm approach neglects surficial and subsurface effects of prior storms on the magnitude of runoff and pollutants from a given storm (commonly called antecedent conditions).

Synthetic Storms

Graphed in Figure 2 is a "5-year" synthetic storm developed in Chicago.[14] Also graphed thereon are the two storms from Table 2 with average return periods of 4.6-years and 5.8-years on the basis of total storm depth. It is difficult to decide just how the two actual storms should be placed with respect to time for comparison, and the positions shown are quite arbitrary. The extension of the synthetic storm to 180-minutes and its overstated total depth (2.28-inches versus 1.61-inches and 1.85-inches for the actual storms) reflect its origin as an intensity-duration-frequency curve containing extended duration values described earlier.

The method of storm synthesis for Chicago reported in 1957 was criticized on the grounds that it retained too many of the fallacies and empiricisms inherent in the rational method to recommend its principle for general use by others.[15] The authors[14] were asked[15] why could not actual storms be used instead, in the later development of general design criteria.[16] To be fair, all this occurred before high-speed, general-purpose computers were readily available and the processing of a number of storms rather than only one was simply not practicable at that time. However, when it came time to prepare a master plan for combined sewer overflow abatement in metropolitan Chicago that incorporated extensive underground storage,[17,18] hourly precipitation records for 1949 through 1969 for twenty raingages in the metropolitan area were used in conjunction with over a score of computer models as a means for determining system component sizes, and necessary waterway improvements and waterway quality for each plan alternative.[19]

Perhaps because the Chicago synthetic storm was featured in a widely used handbook[8] that had been first issued in 1960 and last in 1969, there have been a number of reports and published papers that either adopt the method or allude to its use. This paper has attempted to indicate why synthetic storms are a poor choice for simulating system performance, particularly where detention storage of any significance is involved.

Availability of Chicago data makes possible the illustration of another synthetic storm procedure. A computer model has been developed for storm sewer design called ILLUDAS (Illinois Urban Drainage Area Simulator),[20] that had been described earlier in an ASCE journal.[21] A design storm procedure for Illinois sites is recommended in the users' manual,[20] Graphed in Figure 3 are the two storms from Table 2 with average return periods of 4.6-years and 5.8-years on the basis of total storm depth. Also graphed are "5-year" synthetic storms for ILLUDAS criteria, setting the total duration in each case equal to the duration of the actual storm compared.

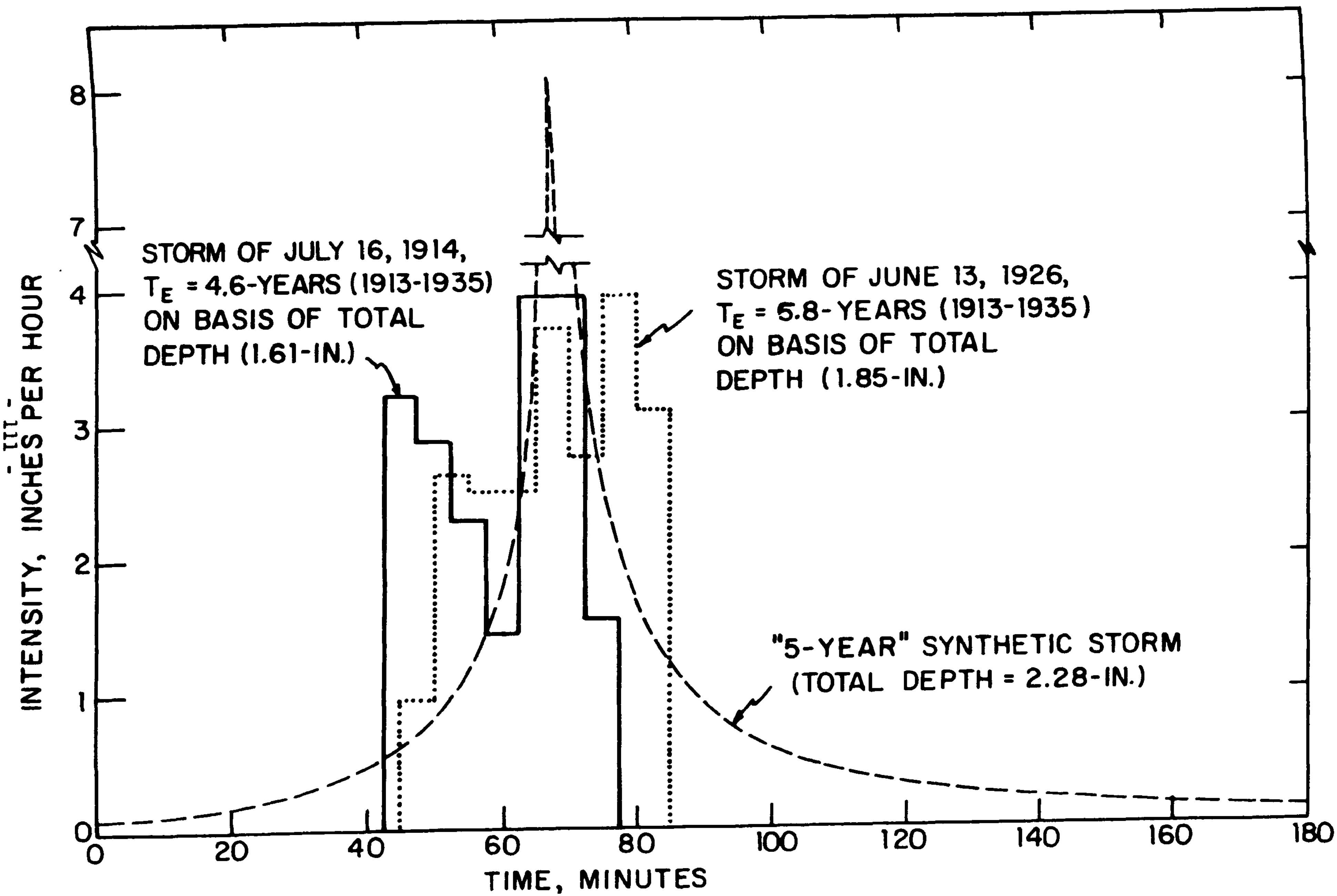

FIGURE 2-ACTUAL VERSUS SYNTHETIC STORM PATTERNS, CHICAGO

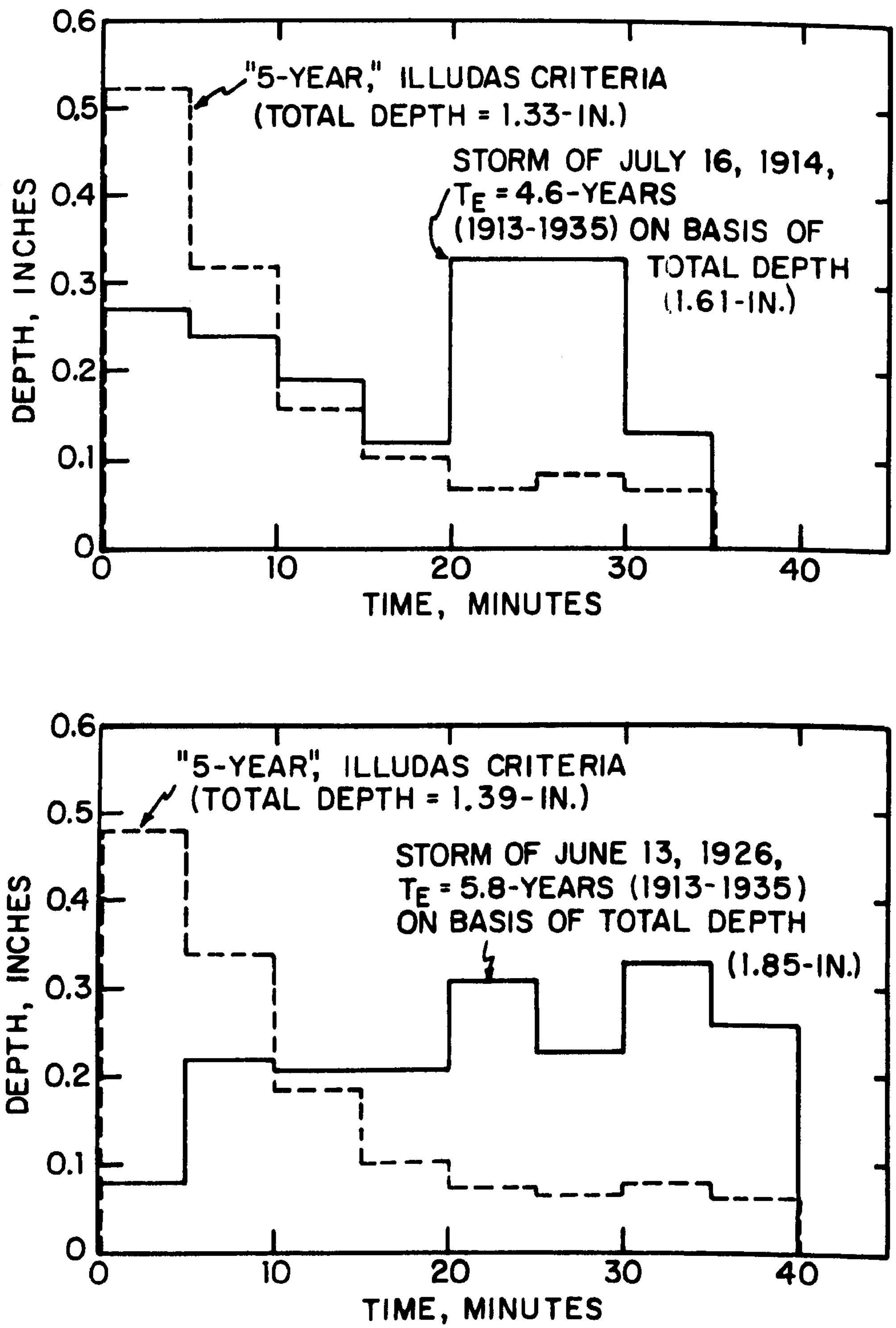

FIGURE 3- COMPARISON WITH ILLUDAS, CHICAGO

The synthetic storms yield lesser volumes. They have an advanced type of pattern, with decreasing depth over time, the characteristics of distributions from intensity-duration-frequency curves such as the one illustrated in Figure 4. It seems evident that the ILLUDAS synthetic storm criteria also evolved from the rational method concept.

Because there are inherent non-linearities in most methods for processing inputs (subtracting assumed abstractions from total rainfall to derive the net rainfall associated with runoff) for linear models, and dynamic models are non-linear by definition, the statistics of the input array may differ appreciably from those of some or all of the arrays for runoff and quality characteristics. Attempting to assign a mean frequency of probable occurrence to a "design storm" is meaningless because of statistical nonhomogeneity of rainfall, runoff and quality.

In an attempt to test the design storm transformation hypothesis, a graduate student at Purdue University calibrated the ILLUDAS model for a 13-acre Chicago catchment for which synchronous rainfall-runoff field data had been collected, and input both a 35-year rainfall record for Chicago(13) and ILLUDAS synthetic storms of various frequencies.(22) Concluded was that the ILLUDAS synthetic storms of given return periods produced peak runoffs of essentially the same return period as those generated from the long-term rainfall record, at least for return periods of 25-years or less. Unfortunately, while the student reproduced the table of accumulated depths for the 35-year record in an appendix of his report, he had mistakenly used, instead, another table listing maximum values for each duration, skewing all storm data into advanced patterns. Therefore, the only conclusion that can be reached is that advanced rainfall patterns give very similar peak discharge arrays. What happens with actual storm pattern data thus remains to be demonstrated, because the well-intended Purdue experiment appears to have been the first reported.

However, an inkling of what might be found is suggested in a test of a synthetic storm used in a preliminary study of stormwater pollution control for a major metropolitan area.(23) Because a planning model(24,25) was used for the study that normally operates on hourly precipitation data, one-hour interval rainfall data was used for the test. The model was calibrated for a catchment draining into the Charles River, and then run with a long-term precipitation record for Boston and with the synthetic storm. From the test it was found that storage sized to contain either 90% or 95% of the total runoff volume resulting from the synthetic storm would be capable instead of containing only about 60% or 80%, respectively, of the total runoff volume.

So far, we have restricted discussion to the shorter return periods associated with storm drainage system sizing. A synthetic storm expected to yield the peak 100-year urban streamflow was developed in a study of the Four Mile Run in northern Virginia(26) following the scheme developed in Chicago.(14) However, the 100-year peak flow obtained from an extrapolation of a series of peak flows in simulations with a long period of actual storm data was used to check the validity of the synthetic storm. The 100-year peak flow simulated using the synthetic storm was only 11% lower, and the synthetic storm was enlarged accordingly for its application in subsequent simulations using a much more elaborate model. Demonstrated in this study was that a simple continuous simulation model can be used to develop a synthetic storm for later use in a complex single-event simulation model for more reliable estimation of a rare event such as a 100-year peak flow. Use of this approach for more frequent events would be even more reliable, provided the simple simulation model was first calibrated with local rainfall and runoff data as in the case above.

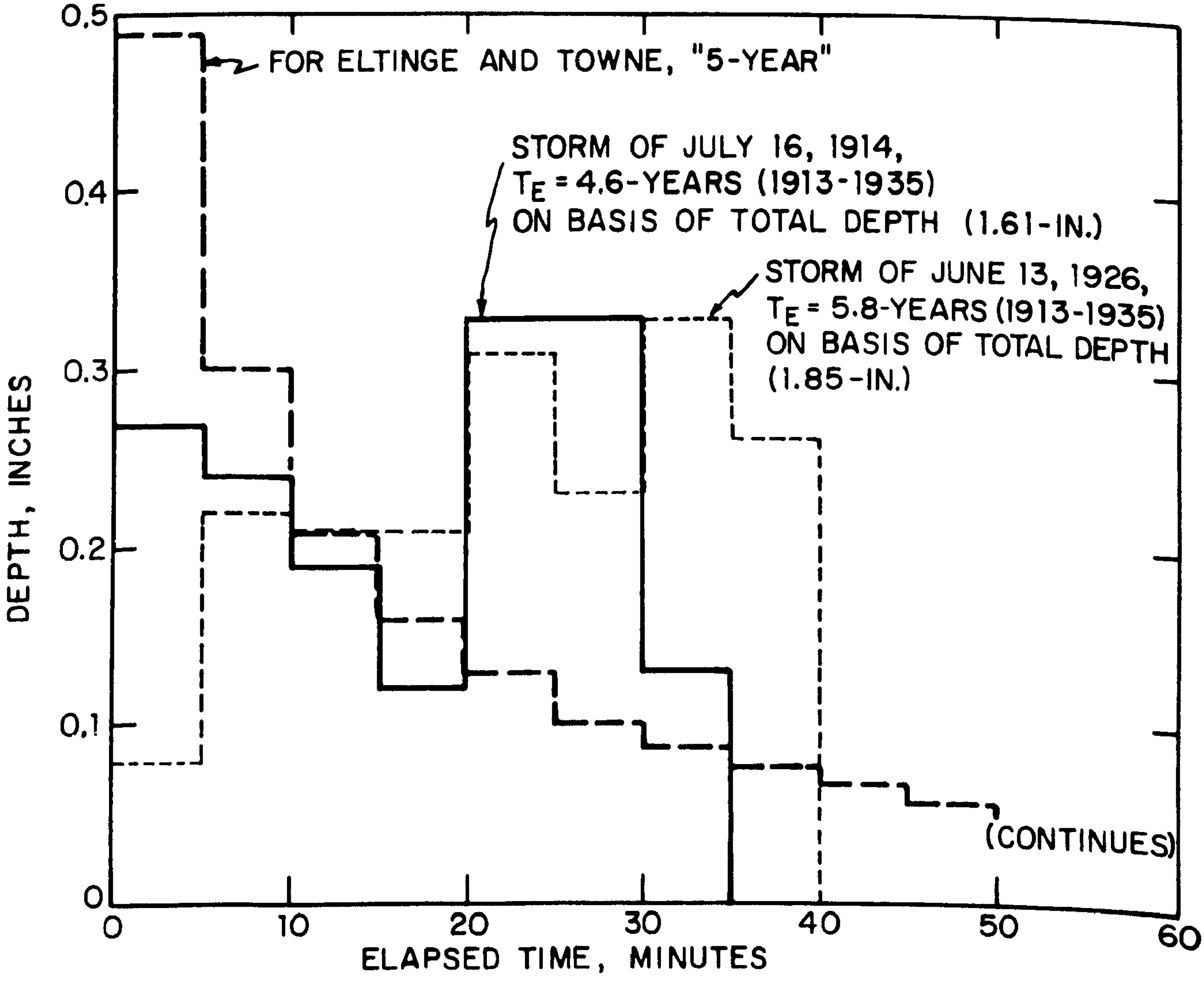

FIGURE 4- STORM PATTERN COMPARISONS, CHICAGO

Some Preliminary Planning Implications

Preferred inputs for modern planning models are reasonable lengths of actual rainfall records, perhaps spanning at least 20 years. It seems more reasonable to route rainfall data of local record through a model to arrive at output parameter frequencies than to synthesize a storm of some assumed probability. Use of meteorological expedients can conceivably be unnecessarily hazardous and the results obtained thereby can be extremely misleading.

In the development of the San Francisco Master Plan for abatement of pollution from combined sewers, 62-years (1907-1968) of hourly rainfall from the local U.S. Weather Service gage were used(27,28) in conjunction with an early version of an uncomplicated planning model that has since been improved(24,25) and extensively used elsewhere.

As mentioned earlier, for the development of the Chicagoland Master Plan for abatement of overflow pollution and reduction of flooding, 21-years (1949-1969) of rainfall data from the local network of twenty raingages was used.(19) Some 104 individual drainage basins in the 375-sq. mi. study area were involved, and thirty-two computer simulation models were developed as part of the planning effort.

Features of both of the above-mentioned master plans are outlined in an ASCE Program report.(29)

Another example is the use of a 60-year rainfall record with a rainfall-runoff model. The model was calibrated using 4-year to 10-year periods of concurrent rainfall and runoff observations for each of 26 partly sewered catchments ranging in size from 1/2 to 88 square miles, and an imperviousness of 2 to 35 per cent.(30) Primary interest was on peak discharges.

Some Design Implications

Because there is insufficient evidence to justify reform, it is difficult to fault the use of synthetic rainfall distributions and simplistic methods in routine design of storm sewers and detention reservoirs of small size. Major facilities that cost substantially more are another matter entirely.

Because the cost of running the more elegant design/analysis models per storm event is high, many defenders of these models champion acceptance of a synthetic storm as an expedient to save time and money, but at the expense of credibility of results. This is not to suggest that all catchments of a jurisdiction where large capital investments are contemplated should be analyzed using 20 years or more of continuous rainfall records on a complex model. Rather, as some of the leading practitioners do, such a long record should be applied to a calibrated catchment near the reference weather station to segregate those storms of design importance. Because only the unusual occurrences are of design interest, there may be perhaps only two dozen or so actual storms of concern. To be consistent, any project sufficiently important to call for the use of a complex model should also be important enough to apply a few storms rather than a single synthetic storm. Thus, the handful of storms selected on the basis of simulated catchment response become a family of design storms for use in connection with other catchments in the jurisdiction. Officials in charge of urban drainage facilities are hard-put to explain an artificial synthetic storm's frequency to irate citizens who have been flooded or to a State official regulating overflows. Defense against storms of record is rather direct, and in the opinion of the writer the only realistic option open to a local government official. The temptation to use artificial confections as input data should be resisted. In addition, the only way to check the expected performance of a facility is to simulate not only the design loading but greater possible loadings as well.

Even if very simple procedures are used, e.g. to determine only a peak flow rate, the collateral, occasional monitoring of computations by means of one of the more complete models can serve as an auxiliary guide to sharper judgment.

In the United Kingdom, a national study has been undertaken that demonstrates that economic analysis-planning-design of storm sewer systems is technically feasible, on a limited scale.(31) However, needed in particular would be development of damage cost-curves and risk cost-curves. Underpinning damage cost curves would be performance evaluation, which would have as its objective to simulate conditions of expected system performance "well enough to be able to measure how well the thing works and thence, if possible, to get an estimate of value before the design is committed. ..... the degree of stress that a system can cope with is a measure of its performance."(31) This would be a considerable departure from conventional evaluation where, for example, pipe sizes are selected on the basis of negligible hydraulic stress (no surcharge, flowing full: no flooding). Concerning performance, it is important to realize that the adoption of a particular design frequency imputes acceptance of a particular risk of system capacity or water quality level being exceeded. Thus, employment of a "design storm" freezes the level of risk, and very little information is available to assess the reality of presumed levels of risk in a cost-benefit context. Unfortunately, little data is available even in the U.S. for making suitable benefit analyses.(32)

Conclusion

Current storm drainage analysis procedures too often emphasize the use of a synthetic storm, which is a device for facilitating analysis but at the expense of reliability. For small projects such an approach is useful and can be appropriate in helping to define marginal costs among alternatives, but may be acceptable only when gross differences in levels of protection from flooding or pollution are sought. However, for more important projects, local officials should make reference to actual rainfall histories for the planning, design and operation of new facilities. This recommendation is particularly appropriate where detention storage is involved, and even more so when detention storage is interconnected as in schemes for reduction of combined sewer overflows. For the latter, and especially where outflow from storage would be governed by treatment plant capacities, dewatering may take a number of hours or days.

When flooding or pollution occur from system overloading, local officials should have a firm rainfall data base on which to defend themselves against criticism and liability, and the use of synthetic storms would leave them all too vulnerable. Similarly, they should be encouraged to make some field rainfall-runoff-quality measurements to sharpen the validity of whatever analytical methods and models are used. Lastly, crudities used in preliminary planning should not be carried into advanced stages of design or into the operation of complex interacting systems.

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ADDENDUM 3

NOMOGRAPHS FOR TEN-MINUTE UNIT HYDROGRAPHS FOR SMALL URBAN WATERSHEDS

ASCE Urban Water Resources Research Program

Technical Memorandum No. 32

by

William H. Espey, Jr., and Duke G. Altman
Espey, Huston & Associates, Inc.

and

Charles B. Graves, Jr.
City of Austin

Austin, Texas

December, 1977

American Society of Civil Engineers
345 East 47th Street
New York, N.Y. 10017

PREFACE

by M. B. McPherson

Background

The following technical memorandum is Addendum 3 of a recent ASCE Program report on "Urban Runoff Control Planning".[1] Addendum 1, "Metropolitan Inventories," and Addendum 2, "The Design Storm Concept," were appended to the latter report. The present technical memorandum is the first of several that will contain additional, individual Addenda over the period 1977-1979.

The principal intended audience of the recent ASCE Program report was the agencies and their agents that are participating in the preparation of areawide plans for water pollution abatement management pursuant to Section 208 of the Federal Water Pollution Control Act Amendments of 1972 (P.L. 92-500). While the presentation which follows is also directed to areawide agencies and their agents, it is expected that it will be of interest and use to many others, particularly local governments.

ASCE Program

The American Society of Civil Engineers' Urban Water Resources Research Program was initiated and developed by the ASCE Urban Water Resources Research Council (formerly the Urban Hydrology Research Council). The basic purpose of the Program is to help establish coordinated long-range research in urban water resources on a national scale.

Abstracts of the twenty-eight reports and technical memoranda of the Program for the 1967-1974 period are included in a readily available paper.[2] The two reports and the six technical memoranda of the regular series completed since are identified in a recent publication.[3] Also included in the latter is a listing of all but one of the twelve national reports in the special technical memorandum series for the International Hydrological Programme; and the last national report[4] and an international summary[5] have been released since.

A Steering Committee designated by the ASCE Council gives general direction to the Program: S. W. Jens (Chairman); W. C. Ackermann; J. C. Geyer; C. F. Izzard; D. E. Jones, Jr.; and L. S. Tucker. M. B. McPherson is Program Director (23 Watson Street, Marblehead, Mass. 01945). Administrative support is provided by ASCE Headquarters in New York City.

Unit Hydrographs

Unit hydrographs have been developed from rainfall-runoff field data for completely sewered drainage catchments in Louisville, Kentucky,[6] and Atlanta, Georgia.[7] However, there is considerable doubt over the transferability or universality of the findings from a single jurisdiction. Moreover, most of the unit hydrographs developed across the nation have been for partially sewered catchments where the streamflows measured had included a significant contribution from non-sewered sectors. An example is the testing of unit hydrographs using urban streamflow data from large catchments nearby by the Georgia Institute of Technology.[8]

Dr. William H. Espey, Jr., the senior author of the following Technical Memorandum, pioneered in the regionalization of unit hydrographs (the regional synthesis of unit hydrograph characteristics) for urban streams. His initial work dealt mainly with streamflow data for urban watersheds located in Texas, Kentucky, Illinois, Indiana and Ohio.[9] This initial unit hydrograph analysis was subsequently expanded utilizing the USGS data for Houston, Texas.[10] His first syntheses of characteristics were for thirty-minute duration unit hydrographs, suitable only for direct application to larger-sized urban watersheds. In contrast, the characteristics reported herein are for ten-minute duration unit hydrographs. The synthesized equations and associated nomographs are derived from data for forty-one urban watersheds of which eighteen are in Texas. Flow gaging in nearly all instances was in streams, a limitation discussed below.

Unit hydrographs have been employed in investigations of the effects of urbanization or in the improvement of prediction techniques.[11-16] Field data from urban catchments in Indiana and elsewhere have been used in tests of various linear process methods, including "instantaneous" unit hydrographs.[17] A series of papers has been devoted to the development of unit hydrographs for gaged urban catchments.[18]

Local Validation

Users of the equations herein, or of the nomographs representing them, are very strongly urged to check their local validity by deriving as many unit hydrographs as possible using rainfall and runoff data from local catchments for comparison. This is particularly important where applications will be for wholly sewered catchments, a condition not truly represented by the formulations presented. One's first reaction might be that if local unit hydrographs can be derived, why do we need the nomographs? What the nomographs provide is a generality of characteristics so that local unit hydrographs for particular catchment physical characteristics can more reliably be extrapolated as part of a family of characteristics for application in catchments where the physical characteristics differ. Also, the fact that the nomographs are founded on data from around the country should reinforce the credibility of any indications obtained using local data.

An excellent manual on unit hydrograph analysis is readily available[19] together with a computer program user's manual[20] for development of unit hydrographs from field data and for routing flows from one point to another. The computer program has been used extensively in urban projects of the Corps of Engineers. Alternative methods for deriving unit hydrographs (linear programming versus least squares) have been compared.[21]

As is the case for some of the newer runoff planning tools, access to a digital computer is not required for the use of the information in this report. The user can calculate synthetic unit hydrograph parameters directly from the equations with a pocket calculator of modest capability or simply use the nomographs. Moreover, complete program listings have been published for use of eleven different small programmable calculators for: deriving a unit hydrograph from an observed hydrograph; converting a unit hydrograph of a given duration to its equivalent for another duration; and synthesizing a hydrograph using a given rainfall pattern and a unit hydrograph.[22] Also included among the 40 complete program listings are three different methods for routing hydrographs from one point to another. The cost of the calculators for which programs are listed span essentially the full price range. A sample of the programs available, but for water distribution system analysis, is provided in a technical magazine.[23]

We are not advocating the use of unit hydrographs as a replacement for more versatile or comprehensive tools of analysis but as a supplementary element among the range of tools needed in the development of urban runoff control plans. For example, use of locally validated synthetic unit hydrographs based on indicators from the following report would be an improvement over employment of hypothetical triangular unit hydrographs as descriptors of subcatchment inputs in some of the more comprehensive runoff models.

Validation Example

The Colorado Urban Hydrograph Procedure (CUHP) has been used extensively in the design of flood abatement facilities throughout the Denver metropolitan area. When the CUHP was introduced in 1969 it was founded on a very small amount of field data. On the basis of much more subsequent data, the Procedure was modified in 1975[24] and again in 1977.[25] The Executive Director of the Urban Drainage and Flood Control District of the Denver metropolitan area, Mr. L. Scott Tucker, was asked to check the equations in the following technical memorandum against local data. (Mr. Tucker is also a member of the ASCE Program's Steering Committee and Chairman of the ASCE Council). Mr. Ben Urbonas of his staff reported their findings,[26] summarized below:

	Stream Gaging Station			
	S. Platte R. Tributary at Englewood	S. Platte R. Tributary at Denver		Sand Creek Tributary at Denver
Area, A, mi^2	1.03	0.75	0.51	0.270
L, mi	1.93	2.02	1.56	0.80
S, ft/ft	0.0076	0.007	0.006	0.0069
I, %	46	42	60	43
ϕ	0.8	1.0	0.8	0.6
Hydrographs checked	(1) 8/19/71 (2) 8/24/72	(3) 8/7/73	(4) 7/21/76	(5) 7/24/73 (6) 8/7/73 (7) 7/30/74

Hydrograph	Peak Discharge, cfs, Generated and Actual: Via This Tech. Memo.	Via CUHP	Recorded
(1)	185	164	122
(2)	204	180	122
(3)	99	93	59
(4)	57	53	36
(5)	100	105	104
(6)	218	223	252
(7)	170	176	250

For the South Platte River Tributary at Denver, rather high values of ϕ were selected to account for induced surface storage expected in that particular case whenever storm sewer capacity is exceeded. (Considerable care should be exercised in the selection of ϕ-values because of their profound influence on the magnitude of unit hydrograph peak discharge and the time to the peak discharge).

No special attempt was made to modify the originally selected values of ϕ to force a better agreement between observed and generated peak discharge for any of the above cases. The tests reported above were for different streams than the two in the Denver area from which data was used in the development of the nomographs herein.

For the Sand Creek Tributary at Denver, the "Hydrograph (6)" storm of 8/7/73 had a maximum average 15-minute intensity close to the once in 100 years on the average historical value. The 100-year design peak flow would be about 335-cfs for this location, using the CUHP, only about one-third higher than the peak flow recorded for storm "Hydrograph (6)".

Reiterating the point made earlier, users of this Technical Memorandum are emphatically urged to check the local validity of the relationships presented by deriving as many unit hydrographs as possible using rainfall and runoff data from local catchments for comparison. The exercise of such care will not necessarily be easy, because there is an inherent difficulty with any runoff model in the necessarily subjective separation of abstractions (infiltration, depression storage, etc.) from total rainfall to resolve rainfall excess (amount and pattern), which is the input from which an equal volume of direct runoff is generated by models of one kind or another. After analyzing the performance of a variety of models, it was concluded that the weakest link is the proper estimation of rainfall excess.[27]

Acknowledgments

The empirical hydrograph equations presented in this Technical Memorandum were developed as part of the City of Austin Master Drainage Study under the direction of Charles B. Graves, Jr., Director of Engineering. Pursuant to this study, a drainage criteria manual was written which contains design discharge curves to be used in hydrologic studies in and near the Austin, Texas, area. These curves were developed utilizing synthetic unit hydrographs determined from the empirical equations along with local rainfall data for application in the Austin area.

The ASCE Urban Water Resources Research Council is indebted to the authors for their generous contribution of this report as a public service.

Processing, duplication and distribution of this Technical Memorandum was supported by grants to ASCE from the Research Applied to National Needs program of the National Science Foundation. Technical liaison representative for NSF/RANN is Dr. J. Eleonora Sabadell. Any opinions, findings, and conclusions or recommendations expressed herein are those of the contributing authors or the writer and do not necessarily reflect the views of the National Science Foundation.

New References

To the references of Section 4 of the parent report[1] should be added a useful document containing brief summaries of 18 selected publications dealing with nonstructural measures for reduction of flood losses.[28]

In the report[1] to which this Technical Memorandum is Addendum 3, reference 55 on page 72 states that the EPA "Areawide Assessment Procedures Manual" is in two volumes, whereas a third volume on "Best Management Practices" has since been released. Also, reference 99 on page 75 is available from NTIS as PB 239 333, and reference 221 on page 84 is available from NTIS as PB 257 089 (July, 1976).

Reference 236 on page 85 of the parent report[1] has been published in the Journal of the Water Resources Planning and Management Division, ASCE Proceedings, Vol. 103, No. WR2, pp. 177-192, November, 1977. Other papers in the same issue that are related to the subject of urban runoff control planning are on: "Urban Flood Management: Problems and Research Needs," by D. H. Howells, pp. 193-212; "Innovative Management Concept for 208 Planning," by J. W. Bulkley and T. A. Gross, pp. 227-240; and "Flood Management for Small Urban Streams," by W. Whipple, Jr., pp. 315-324. Overlooked in a previous issue was a reference on control of sedimentation via detention basins.[29]

The report of a nationwide evaluation of combined sewer overflows and urban stormwater discharges (including the cost of control or abatement of receiving water pollution from such sources) has been released in three volumes. The first volume is an executive summary.[30] Assessments were for all 248 "Urban Areas" in the U S., of which about 46 per cent is undeveloped. Land uses for the 54 per cent that is developed are approximately as follows: residential, 58%; industrial, 15%; commercial, 9%; and other, 18%. Also, about 14% of the developed urban land is served by combined sewers and 38% by separate storm sewers, with the balance containing unsewered storm drainage. "Average annual dry-weather flow is significantly greater than average wet-weather flow only in the arid areas. However, in most parts of the country, dry-weather flows represent 30 to 50 per cent of the total (wet plus dry) runoff from urban areas."[30]

Mr. Carl F. Izzard, member of the ASCE Program Steering Committee, has re-analyzed the original experimental data for full-size street inlets and developed a graphical solution for the hydraulic design of curb-opening inlets.[31]

An overview and assessment of current catchbasin technology has been reported recently.[32] Included are evaluations of hydraulic and pollutant removal efficiencies.

To the transport module of the version of the Storm Water Management Model embodying the fundamental hydrodynamic equations of motion has been added a capability for analyzing alternatives for the abatement of deposition and scour in storm and combined sewers. The project report[33] includes listings of the new computer program subroutines that have been added for solids transport characterization. A related report[34] describes procedures for estimating dry-weather pollutant deposition in combined sewers.

Four papers presented at a recent national meeting, under the auspices of the ASCE Urban Water Resources Research Council, have been preprinted in a single document.[35]

A critique of traditional land-use control mechanisms is included in a report on economic incentives for such control.[36] An extensive bibliography occupies almost a third of the content.

National state-of-the-art indications on urban stormwater runoff control possibilities have been blended with local information in a comprehensive report for the metropolitan Milwaukee region.[37]

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NOMOGRAPHS FOR TEN-MINUTE UNIT HYDROGRAPHS FOR SMALL URBAN WATERSHEDS

(Addendum 3 of the ASCE Program report "Urban Runoff Control Planning," June, 1977)

by
William H. Espey, Jr., President, and
Duke G. Altman, Staff Engineer,
Espey, Huston & Associates, Inc.,
3010 S. Lamar
Austin, Texas 78704
and
Charles B. Graves, Jr.,
Director of Engineering,
City of Austin,
P.O. Box 1088,
Austin, Texas 78704

December, 1977

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NOMOGRAPHS FOR TEN-MINUTE UNIT HYDROGRAPHS FOR SMALL URBAN WATERSHEDS

Introduction

Due to rapid runoff response and the resulting flood potential characteristic of many relatively small urban and surrounding area watersheds, a need is evident for adequate and efficient description of the dynamic process involved. In some instances, procedures such as the "rational method" may be sufficient in describing a hydrologic process in a small watershed but at other times a more definitive method is required. A short-duration unit hydrograph has considerable potential for describing the dynamic runoff process of small watersheds.

The basic theory of the unit hydrograph appears to have been suggested first by Folse (1929). The Boston Society of Civil Engineers (1930) stated that "the base of the flood hydrograph appears to be approximately constant for different floods, and peak flow tends to vary directly with the total volume of runoff". Three years later, in 1932, Sherman introduced the basic concept of the unit hydrograph. The unit hydrograph has found wide acceptance as an outstanding contribution to engineering hydrology. The general concept of the unit hydrograph has been summarized as follows (Morgan and Johnson, 1962):

> "for a given drainage area, the time-base of surface-runoff hydrographs resulting from similar storms of equal duration are the same regardless of the intensity of rainfall;
>
> "for a given drainage area, the ordinates of the surface-runoff hydrographs from similar storms of equal duration are proportional to the volume of surface runoff; and
>
> "for a given drainage area, the time distribution of surface runoff from a particular storm period is independent of that produced by any other storm period."

Because the unit hydrograph is such a valuable hydrologic tool, many investigations have been undertaken to develop synthetic unit hydrograph relationships based on watershed features.

Because it is generally accepted that the best unit period should be somewhat smaller than the watershed lag time (time from the center of mass of rainfall to the peak of the hydrograph), the need for short-duration unit hydrographs to describe the runoff process for small, quick-responding watersheds becomes obvious. Empirical equations given subsequently were developed to aid users in describing or synthesizing the shape of a ten-minute unit hydrograph for any small watershed for which adequate rainfall-runoff relationships had not been previously developed through use of site-specific data.

The following hydrologic parameters were chosen to describe the shape of the ten-minute unit hydrograph:

T_R	-	the time of rise, in minutes;
Q	-	the peak discharge, in cfs;
T_B	-	the time base, in minutes;
W_{50}	-	the time, in minutes, between the two points on the unit hydrograph at which the discharge is half of the peak discharge; and
W_{75}	-	the time, in minutes, between the two points on the unit hydrograph at which the discharge is three-fourths of the peak discharge.

For clarification, these parameters are illustrated in Figure 1. Once the five parameters have been determined by use of the empirical equations, and an auxiliary means for distributing a runoff volume of one inch has been employed, the ten-minute unit hydrograph can be easily constructed for a watershed. Dimensionless unit hydrographs, such as those found in the Soil Conservation Service hydrology manual (USDA-SCS, 1971), can be used as an aid in determining the basic shape of the ascending and descending portions of the unit hydrograph. Generally, the lower portions of the ascending and descending unit hydrograph limbs can then be adjusted to obtain the one inch runoff volume.

From the empirical equations, parallel, scaled nomographs have been developed to provide a straightforward, simple procedure enabling graphical determination by the user of the five hydrologic parameters. The nomographs are featured in this document.

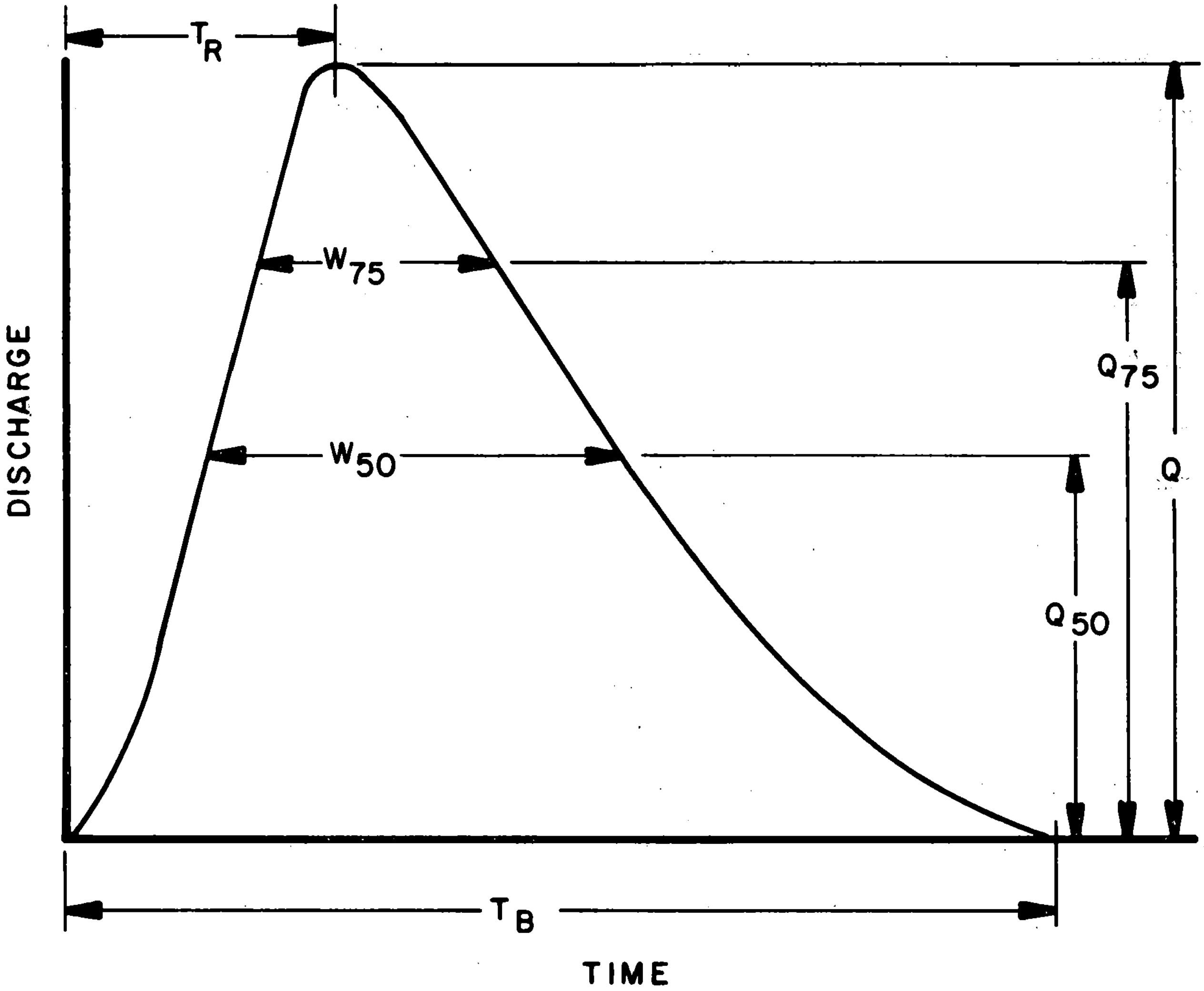

FIGURE 1 - DEFINITION OF UNIT HYDROGRAPH PARAMETERS

Watershed Data

The five hydrologic parameters that describe the fundamental shape of the ten-minute unit hydrograph were developed from rainfall-runoff data collected for forty-one small watersheds throughout the United States. Of the forty-one watersheds, eighteen are located in Texas. Listed in Table 1 are the name, location, and physiographic characteristics of each of the forty-one watersheds, which have a range in size of drainage area between 0.014 and 15.0 square miles. As indicated in Table 1, size of drainage area (A), main channel length (L), main channel slope (S), extent of impervious cover (I), and a dimensionless watershed conveyancy factor (ϕ) were selected as the most important indices describing the physiographic characteristics of each watershed. These physiographic characteristics are defined as follows:

(A) is the watershed drainage area (in square miles).

(L) is the total distance (in feet) along the main channel from the point being considered to the upstream watershed boundary.

(S) is the main channel slope (in feet per foot) as defined by H/(0.8L), where L is the main channel length as described above and H is the difference in elevation between two points, A and B. A is a point on the channel bottom at a distance of 0.2L downstream from the upstream watershed boundary. B is a point on the channel bottom at the downstream point being considered.

(I) is the extent of impervious area within the watershed (in per cent).

(ϕ) is the description of the conveyance efficiency of the watershed's drainage system. The basic drainage system conditions that effect the conveyance efficiency or hydraulic response of an area can be evaluated in terms of this factor. These basic drainage system conditions include: degree of main channel improvement; upstream main channel roughness coefficient (Manning 'n'); storm sewer density; area or length of streets; and amount of impervious area. The hydraulic response of a watershed is a function of the drainage efficiencies of both the main channel and contributing subareas, respectively described by a weighted Manning 'n' value and per cent impervious cover in Figure 2.

TABLE 1

LOCATION AND PHYSIOGRAPHIC DESCRIPTION OF WATERSHEDS

Watershed Name	Watershed Location	Drainage Area, A (Square miles)	Main Channel Length, L (Feet)	Main Channel Slope, S (Ft. per Foot)	Impervious Cover, I (%)	Dimensionless Conveyance Factor, ϕ
Bachman Branch	Dallas, Texas	10.0	28,512	0.006	30.0	0.8
Hunting Bayou at Cavalcade Street	Houston, Texas	1.03	5,800	0.002	27.0	1.25
Stoney Brook Street Ditch	"	0.50	3,700	0.0006	33.0	0.60*
Sims Bayou at Carlsbad Street	"	4.99	12,400	0.0008	4.0	1.30
Cole Creek at Guhn Rd.	"	7.05	18,500	0.001	4.0	1.30
Bering Ditch at Woodway Drive	"	2.59	11,400	0.0007	17.0	0.85
Berry Bayou at Forest Oaks Street	"	11.10	18,300	0.0016	13.0	0.65*
Berry Bayou at Gilpin Street	"	3.26	4,500	0.0015	9.0	1.00
Berry Bayou at Globe St.	"	1.58	7,900	0.0006	15.0	1.00
Brickhouse Gully at Clarblak Street	"	2.01	12,700	0.0011	2.5	1.05
Halls Bayou at Deertail Street	"	5.27	20,600	0.0011	2.0	1.30
Keegans Bayou at Keegan Road	"	5.77	31,200	0.0005	2.0	1.30
Willow Waterhole at Landsdown St.	"	1.15	1,700	0.0006	33.0	0.95

*Modified ϕ selection process

(Continued)

TABLE 1 (Continued)

Watershed Name	Watershed Location	Drainage Area, A (Square miles)	Main Channel Length, L (Feet)	Main Channel Slope, S (Ft. per Foot)	Impervious Cover, I (%)	Dimensionless Conveyance Factor, ϕ
Waller Creek at 38th Street	Austin, Texas	2.31	23,080	0.009	27.0	0.80
Waller Creek at 23rd Street	"	4.13	27,560	0.009	37.0	0.80
Helotes Creek at State Hwy. 16	San Antonio, Texas	15.00	35,600	0.01	6.0	1.00
Brown Creek at State Fairgrounds	Nashville, Tenn.	11.80	28,512	0.0072	15.7	0.80
Three Mile Creek	Jackson, Miss.	1.10	9,504	0.0079	19.7	0.75
Crane Creek	"	0.45	4,224	0.0067	27.5	0.80
Walton Run	Phila., Penn.	2.17	14,784	0.0068	24.7	1.00
Little Sugar Creek at Brookcrest Dr.	Charlotte, N.C.	0.84	9,029	0.0175	21.0	0.80
Little Sugar Creek at Burnley Road	"	0.44	5,544	0.0157	14.0	0.80
Paw Creek at Allenbrook Drive	"	0.62	7,022	0.0143	18.0	0.80
Briar Creek at Sudbury Road	"	0.56	5,808	0.0112	16.0	0.85
Briar Creek at Shamrock Drive	"	0.52	5,650	0.0148	20.0	0.85
Irwin Creek Tributary	"	0.27	5,914	0.0193	19.0	0.90

(Continued)

TABLE 1 (Continued)

Watershed Name	Watershed Location	Drainage Area, A (Square Miles)	Main Channel Length, L (Feet)	Main Channel Slope, S (Ft. per Foot)	Impervious Cover, I (%)	Dimensionless Conveyance Factor, ø
Silas Creek at Pine Valley Road	Winston-Salem, N.C.	0.89	8,554	0.0193	19.0	0.90
Tar Branch at Walnut Street	"	0.59	6,706	0.0295	28.0	0.80
Brushy Creek at U.S. Hwy. 311	"	0.55	5,808	0.0271	37.0	0.90
Sanderson Gulch Tributary	Lakewood, Colorado	0.47	4,752	0.0139	51.0	0.70
Tuck Drain	Northglenn, Colorado	0.067	1,584	0.0163	33.0	0.80
Western Outfall Sewer	Louisville, Kentucky	2.77	22,000	0.0009	70.0	0.60
17th Street Sewer	"	0.22	4,900	0.0038	83.0	0.60
Northwest Trunk	"	1.90	16,000	0.0012	50.0	0.60
Southern Outfall	"	6.43	34,000	0.0014	48.0	0.60
Southwestern Outfall	"	7.52	34,200	0.0015	33.0	0.60*
Freeman Field A	Indiana	0.015	900	0.009	21.6	1.00
Freeman Field B(1) + Apron	"	0.0128	555	0.0058	100.0	1.00
Freeman Field B + Taxi	"	0.014	1,200	0.004	100.0	1.00
St. Anne Auxiliary Field	Indiana	0.114	2,600	0.008	5.1	1.00
Godman Field No. 1	Kentucky	0.0205	1,300	0.015	22.1	1.00

*Modified ø selection process

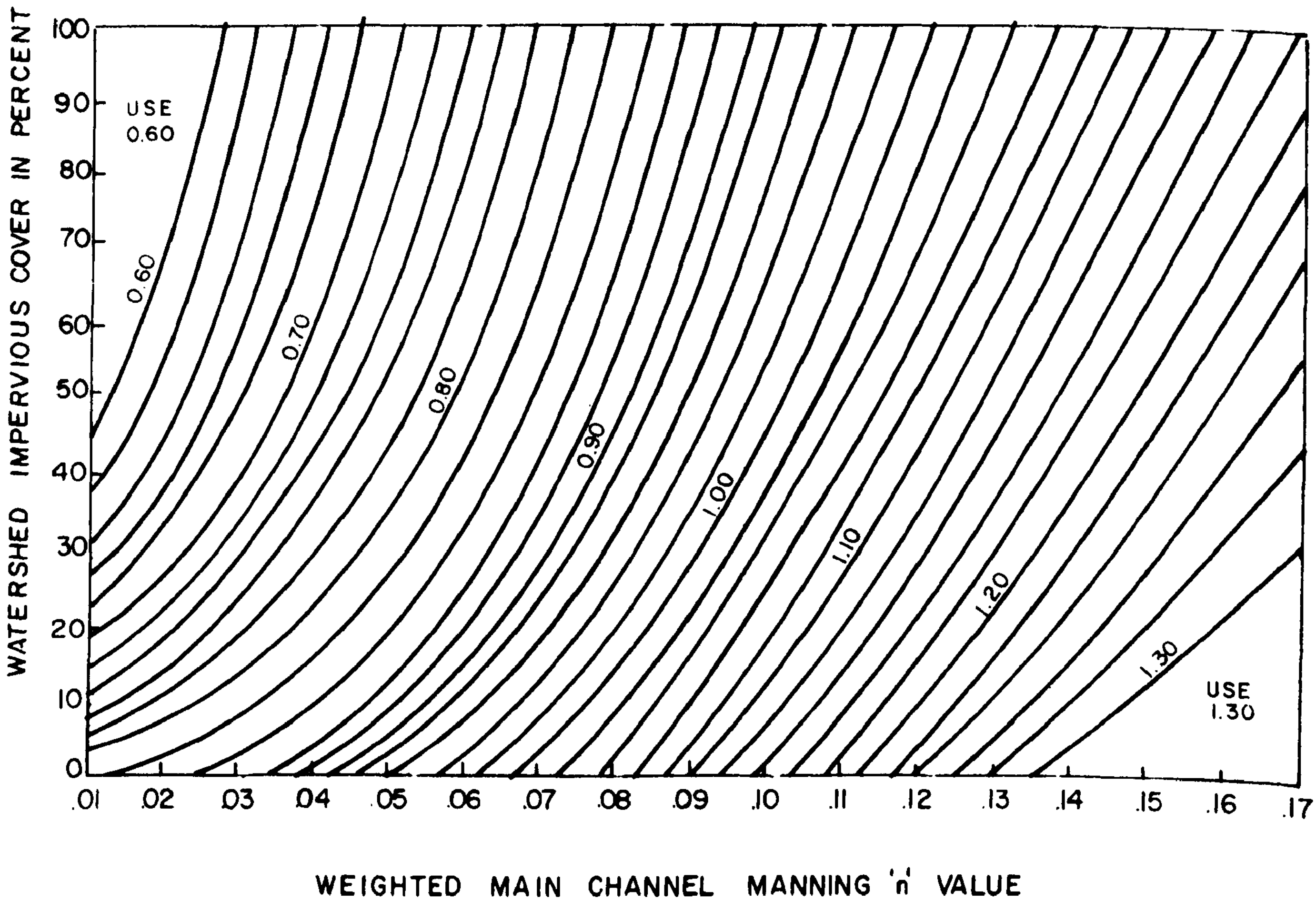

FIGURE 2 - WATERSHED CONVEYANCE FACTOR, Φ, AS A FUNCTION OF PER CENT WATERSHED IMPERVIOUS COVER, I, AND WEIGHTED MAIN CHANNEL MANNING 'n' VALUE. (Austin/Drainage Manual, 1977)

The effects of urbanization on the unit hydrograph can generally be accounted for in the percentage of impervious cover (I) and the watershed conveyance factor (ϕ). Man-made improvements such as buildings, paved streets, and parking lots effectively reduce infiltration into the ground and often increase the volumes and peak rates of runoff. The I and ϕ values are related to these improvements and are indicators of a watershed's runoff response characteristics.

A brief history concerning the evolution of the ϕ factor is appropriate at this point. Espey <u>et al</u>. (1965) found that the introduction of watershed impervious cover as an index of urbanization was not sufficient for describing the runoff characteristics of some urban watersheds. It was noted in this early study that in most cases when the channel had been improved, and/or a storm sewer system existed, the predicted values of the time of rise (T_R) were high compared to measured values. Therefore, a new urban factor (ϕ) was introduced to account for the reduction in the time of rise due to channel improvements or storm sewers. Further studies by Espey <u>et al</u>. (1968) refined the determination of the ϕ factor to include channel vegetation conditions. In the latter study, for several watersheds, predominately in the Houston area, a significantly shorter response time was indicated than for watersheds in other areas. The only apparent variable that could account for the shorter response times was channel vegetation. To explain the effect of channel vegetation on the time of rise, the watershed conveyance factor (ϕ) was redefined as the sum of: the channel characteristics, ϕ_1, previously defined by Espey <u>et al</u>. (1965), which accounts for the amount of channel improvement and the type of secondary drainage; and ϕ_2, which accounts for flow-retarding vegetation within the channel. Values of these two drainage system characteristics are listed in Table 2.

The most recent work dealing with the ϕ factor and its hydrologic significance was performed in conjunction with the City of Austin Master Drainage Study. In

Φ_1	CLASSIFICATION
0.6	EXTENSIVE CHANNEL IMPROVEMENT AND STORM SEWER SYSTEM, CLOSED CONDUIT CHANNEL SYSTEM.
0.8	SOME CHANNEL IMPROVEMENT AND STORM SEWERS; MAINLY CLEANING AND ENLARGEMENT OF EXISTING CHANNEL.
1.0	NATURAL CHANNEL CONDITIONS.

Φ_2	CLASSIFICATION
0.0	NO CHANNEL VEGETATION.
0.1	LIGHT CHANNEL VEGETATION.
0.2	MODERATE CHANNEL VEGETATION.
0.3	HEAVY CHANNEL VEGETATION.

$$\Phi = \Phi_1 + \Phi_2$$

TABLE 2 - CLASSIFICATION OF WATERSHED DRAINAGE SYSTEMS (Espey *et al.*, 1968)

that study the ϕ factor was determined for each watershed shown in Table 1 and used as one of the watershed physiographic characteristics in a statistical analysis to determine the ten-minute unit hydrograph parameter equations. Figure 2 was then developed to aid users in the determination of the ϕ factor for urban watersheds.

It must be pointed out that the normal process for selecting a watershed's ϕ value (Figure 2) may not be applicable for watersheds with atypical drainage characteristics. For that reason, judgment must be exercised in the use of Figure 2 because the data base used in the development of the figure was limited (Table 2). Future study and collection of additional data may indicate a need for modification or refinement of the ϕ selection process. If flow-modifying conditions exist in a watershed, such as excessive surface detention storage, combined sewer systems, or overtaxed storm sewer systems combined with a lack of surface escape routes, additional hydrologic procedures may be required for the description of the rainfall-runoff process.

Rainfall-runoff data for numerous watersheds were reviewed for applicability in describing the composition of the ten-minute unit hydrograph equations. The major share of this data was obtained from a TRACOR report (Espey, et al., 1968) and a report to the Texas Water Commission (Espey, et al., 1965). Reported in these two documents were reduced runoff data from fifty watersheds providing average thirty-minute unit hydrographs for the respective watersheds. Data compilations reported by the U.S. Geological Survey were also reviewed, from which a few more watersheds were found with adequate hydrologic data that could be incorporated into the data base. The previously derived thirty-minute unit hydrographs were transformed into ten-minute unit hydrographs by the S-curve procedure, which is described in most comprehensive hydrology texts. The results of the S-curve reductions combined with new unit hydrographs were then studied and the resulting ten-minute unit hydrographs having the best shape were chosen for further analysis. From this process the final forty-one watersheds listed in Table 1 were selected to determine the relationships between physical watershed characteristics and the shape characteristics of the ten-minute unit hydrograph to be generalized. Definitions of the five hydrograph parameters discussed earlier were derived from among the variables affecting the ten-minute unit hydrographs for the forty-one selected watersheds.

Development of Empirical Ten-Minute Unit Hydrograph Equations

The objective of developing empirical equations to describe the runoff response characteristics of watersheds (i.e., a unit hydrograph) is to provide a means for taking known or relatively easily obtained information, such as rainfall and watershed physical characteristics, and to describe synthetically the runoff hydrograph for a particular rainfall event. Of course, there are other considerations that must be accounted for along with the synthetic unit hydrograph in describing runoff patterns. These considerations include such items as infiltration rates, rainfall surface-storage, base flow, and other flow-modifying conditions.

The problem of statistically synthesizing a unit hydrograph is by definition one of approximating the hydrograph shape. Empirical equations allow the description of a hydrograph in terms of typical hydrograph dimensions (parameters), avoiding the difficulties that would have to be overcome in attempting to derive a mathematical function for the hydrograph. Having described the hydrograph for each of the forty-one watersheds in terms of five parameters, each parameter was statistically described as a function of the physiographic features and/or other unit hydrograph parameters of the watershed.

To describe the five parameters mathematically, a multiple non-linear regression formula of the following form was used:

$$Y = KX_1^{A_1} X_2^{A_2} \ldots X_N^{A_N},$$

where Y is one of the five hydrograph parameters,

X_i (for $i = 1, \ldots, N$) are physiographic characteristics or hydrograph parameters for the watershed, and

K and A_i (for $i = 1, \ldots, N$) are regression coefficients.

The above equation was expressed in the following logarithmic form:

$$\log Y = \log K + A_1 \log X_1 + \ldots A_N \log X_N,$$

and the method of least squares was then applied to evaluate the regression coefficients. This method, instead of minimizing the sum of the squares of the deviations of the function itself, minimizes the sum of the squares of the deviations of the logarithm of the function.

Table 3 presents the results of the regression analysis in the form of the ten-minute unit hydrograph equations. For easy reference, brief definitions are included in Table 3 of the watershed physiographic characteristics and the five hydrograph parameters. Table 3 also presents the statistical accuracy of the empirical ten-minute unit hydrograph equations obtained using the data from the forty-one watersheds, by giving the total explained variation of the logarithmic predictions for each of the five equations, where the value 1.000 would be for a perfect fit.

Nomographs and Examples

To simplify the process of synthesizing ten-minute unit hydrographs, a set of parallel, scaled nomographs were constructed using the empirical equations, to facilitate user determination of values for all five of the unit hydrograph parameters. A ten-minute unit hydrograph can then be constructed resembling the one shown in Figure 1. These nomographs are presented in Figures 3 through 7. A description of the respective nomographs shown in each of these five figures will now be given along with an example of their use.

Determination of the unit hydrograph parameters using the developed nomographs

TABLE 3

TEN-MINUTE UNIT HYDROGRAPH EQUATIONS

Equations	Total Explained Variation
$T_R = 3.1\ L^{0.23} S^{-0.25} I^{-0.18} \phi^{1.57}$	0.802
$Q = 31.62x10^3\ A^{0.96} T_R^{-1.07}$	0.936
$T_B = 125.89x10^3\ A\ Q^{-0.95}$	0.844
$W_{50} = 16.22x10^3\ A^{0.93}\ Q^{-0.92}$	0.943
$W_{75} = 3.24x10^3\ A^{0.79}\ Q^{-0.78}$	0.834

L is the total distance (in feet) along the main channel from the point being considered to the upstream watershed boundary.

S is the main channel slope (in feet per foot) as defined by H/(0.8L), where L is the main channel length as described above and H is the difference in elevation between two points, A and B. A is a point on the channel bottom at a distance of 0.2L downstream from the upstream watershed boundary. B is a point on the channel bottom at the downstream point being considered.

I is the impervious area within the watershed (in per cent).

ϕ is the dimensionless watershed conveyance factor as described previously in the text.

A is the watershed drainage area (in square miles).

T_R is the time of rise of the unit hydrograph (in minutes).

Q is the peak flow of the unit hydrograph (in cfs).

T_B is the time base of the unit hydrograph (in minutes).

W_{50} is the width of the hydrograph at 50% of the Q (in minutes).

W_{75} is the width of the unit hydrograph at 75% of Q (in minutes).

$$T_R = 3.1\,(L^{.23})\,(S^{-.25})\,(I^{-.18})\,(\Phi^{1.57})$$

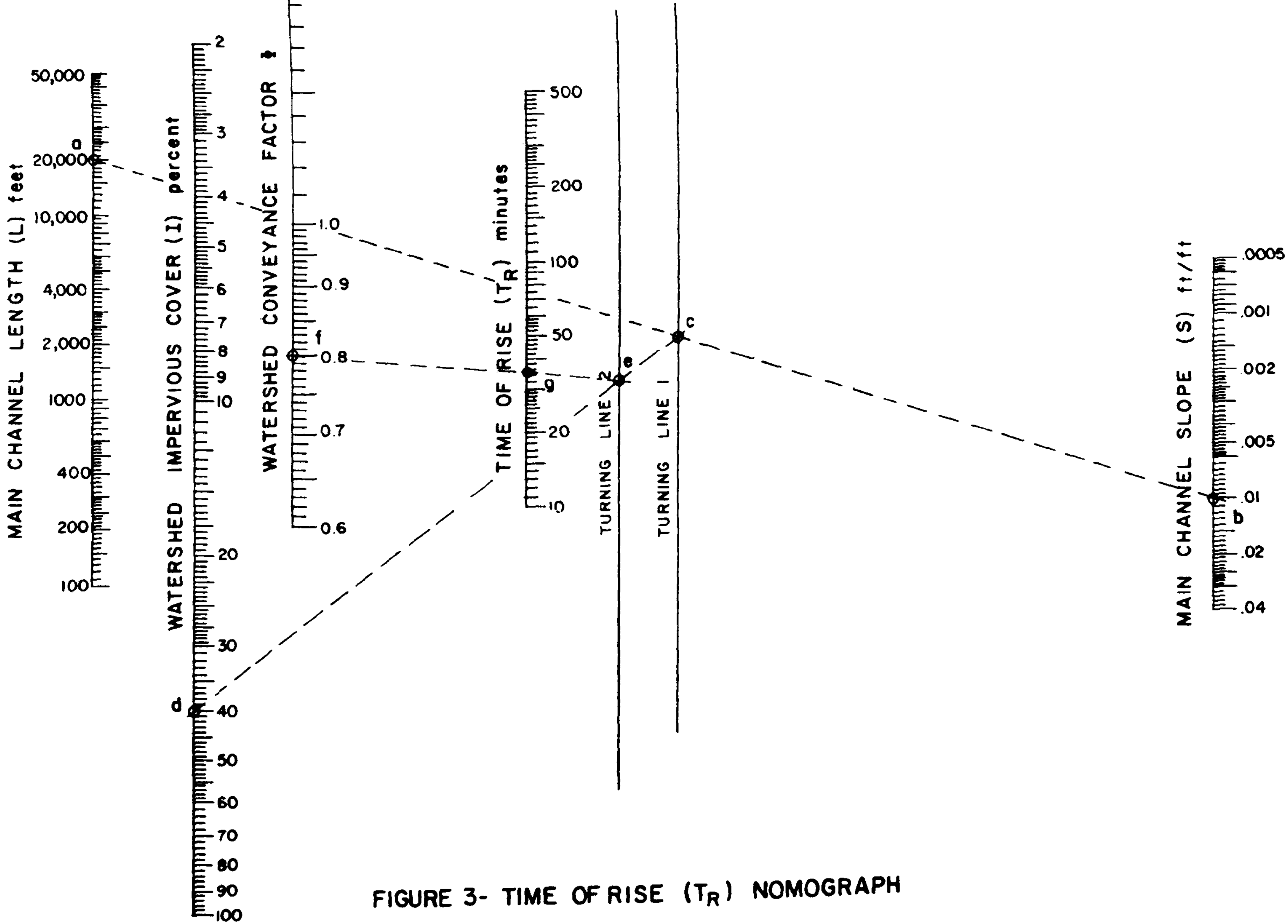

FIGURE 3- TIME OF RISE (T_R) NOMOGRAPH

$$Q = 31.62 \times 10^3 (A^{.96})(T_R^{-1.07})$$

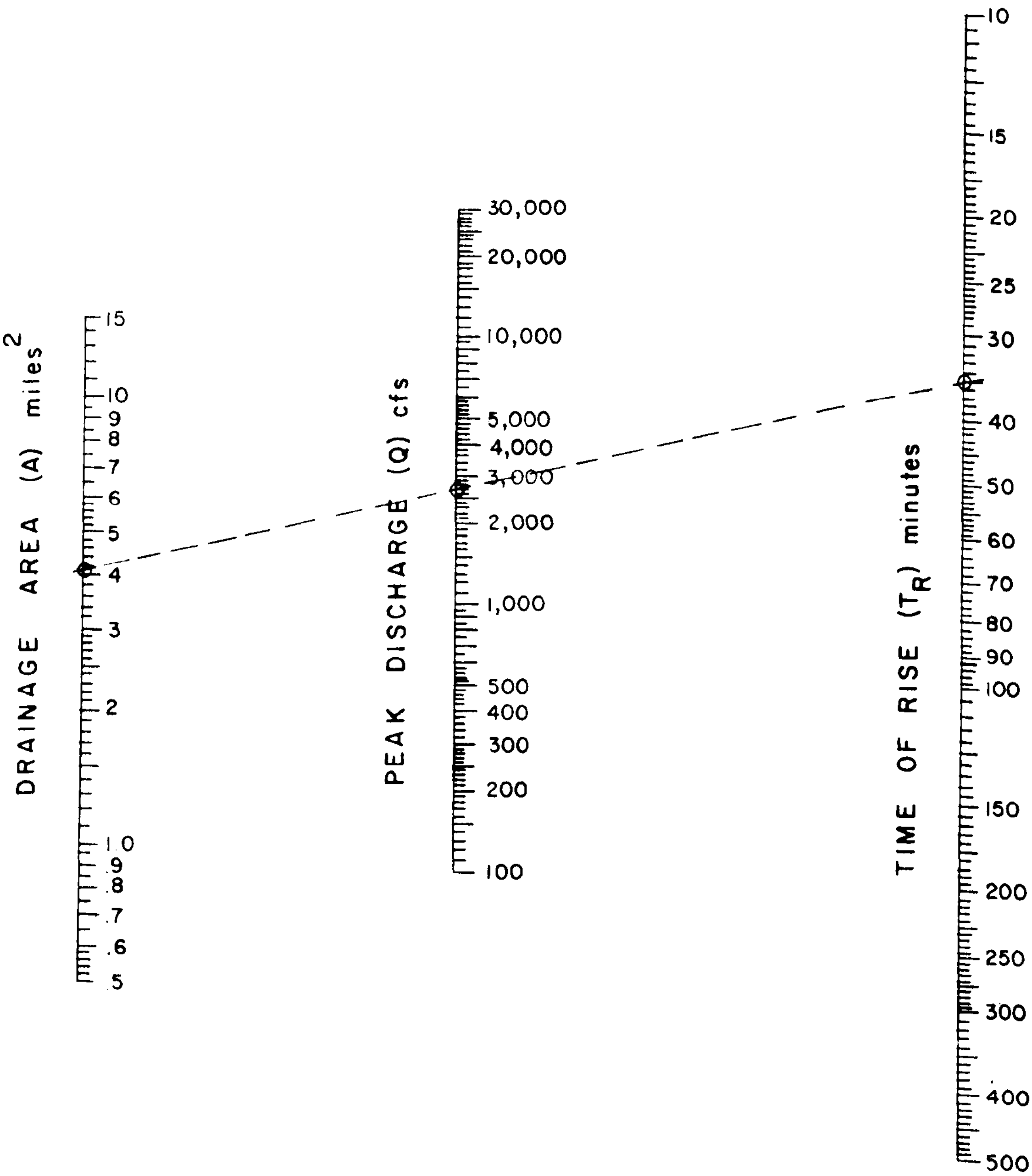

FIGURE 4- PEAK DISCHARGE (Q) NOMOGRAPH

$$T_B = 125.89 \times 10^3 (A)(Q^{-.95})$$

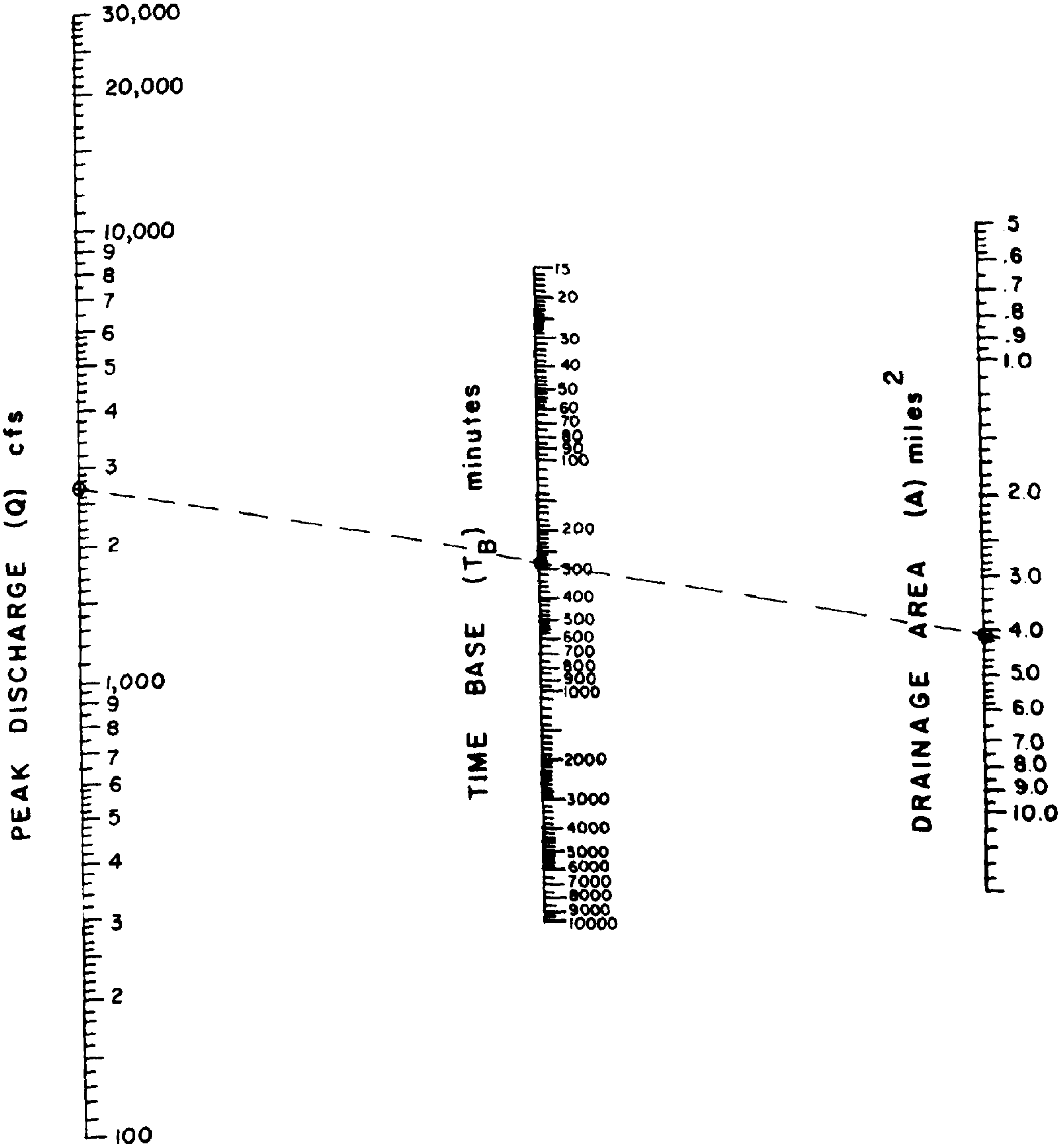

FIGURE 5- TIME BASE (T_B) NOMOGRAPH

$$W_{50} = 16.22 \times 10^3 (A^{.93})(Q^{-.92})$$

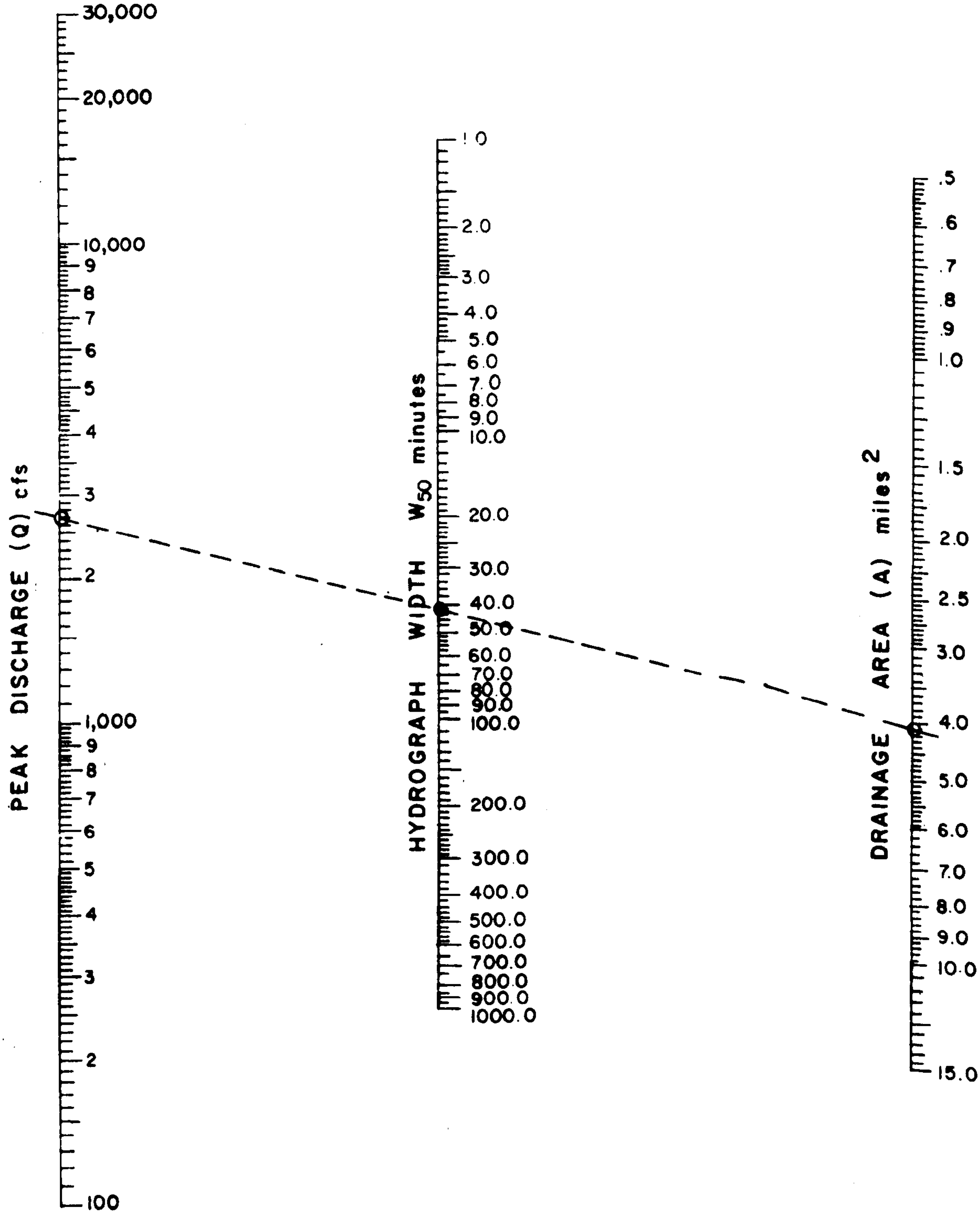

FIGURE 6- HYDROGRAPH WIDTH (W_{50}) NOMOGRAPH

$$W_{75} = 3.24 \times 10^{3}(A^{.79})(Q^{-.78})$$

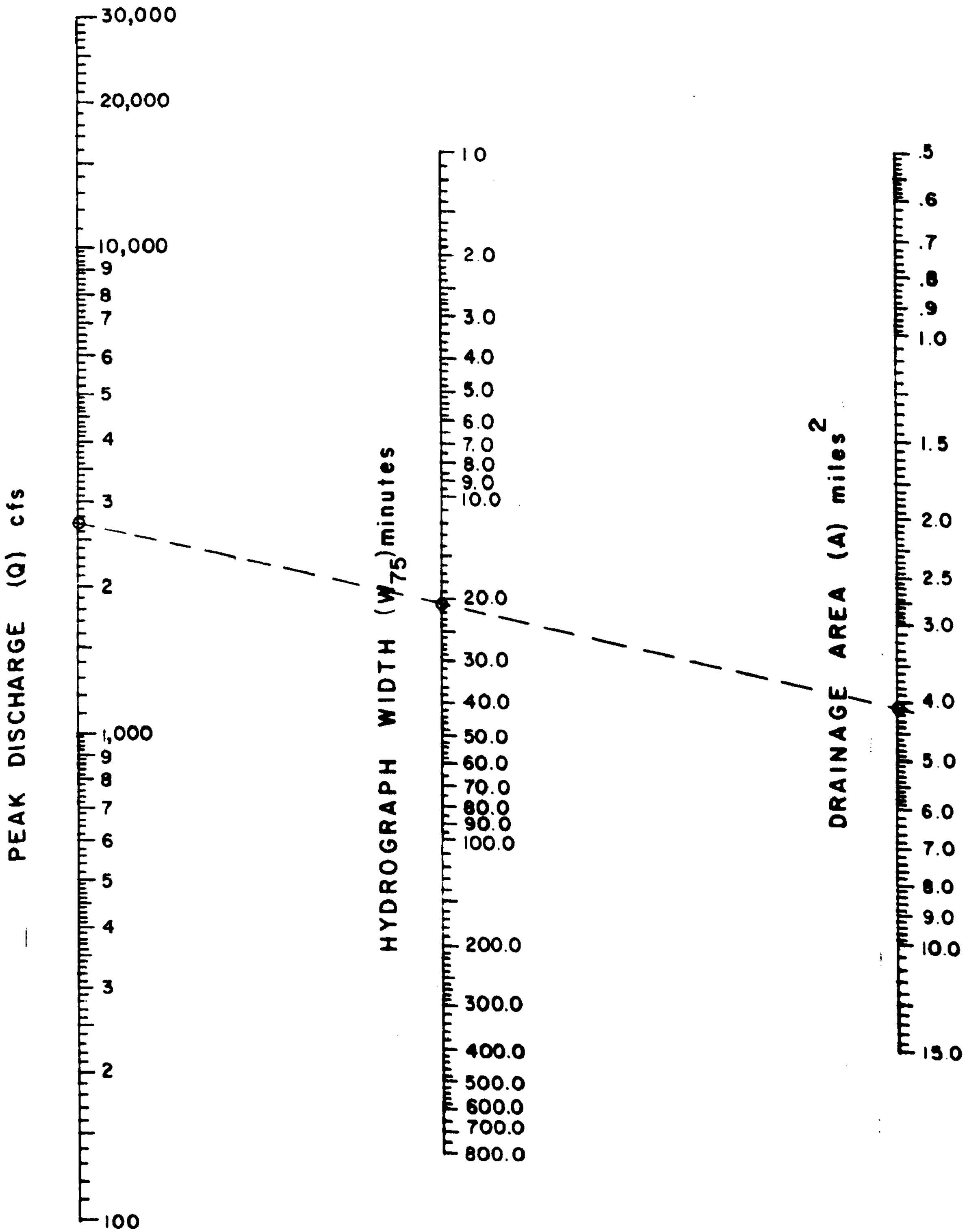

FIGURE 7- HYDROGRAPH WIDTH (W_{75}) NOMOGRAPH

is shown below for a watershed having the following physiographic characteristics:

A = 4.1 square miles

L = 21,000 feet

S = 0.01 ft/ft

I = 40 per cent and

ϕ = 0.8

The unit hydrograph time of rise (T_R) determination is a multiple-step process that can be followed in Figure 3. Align the scale values for L (21,000 at point a) and S (0.01 at point b) with a straightedge and mark point c on turning line 1. Align point c and the scale value for I (40 at point d) along a straight line and mark point e on the intersection with turning line 2. Align point e and the scale value for ϕ (0.8 at point f) with a straightedge and read the time of rise (T_R) value at the intersection with its scale (35 at point g).

The peak discharge (Q) of the unit hydrograph can be obtained from Figure 4 by simply aligning the drainage area scale value (4.1) and the time of rise scale value (35) with a straightedge and reading the Q value from the peak discharge scale in the middle (2700).

Figure 5 provides the unit hydrograph time base (T_B) by aligning the peak discharge scale value (2700) and the drainage area scale value (4.1) with a straightedge and obtaining the value from the time base scale in the middle (285).

Hydrograph widths W_{50} and W_{75} can be respectively found by aligning the drainage area (4.1) and peak discharge scale values (2700) with a straightedge in Figures 6 and 7 and reading the appropriate values from the hydrograph width scales in the middle of each nomograph (W_{50} = 42 and W_{75} = 21).

The above determinations are summarized below for the five ten-minute unit hydrograph parameters:

TR = 35 minutes

Q = 2700 cfs

T_B = 285 minutes

W_{50} = 42 minutes

W_{75} = 21 minutes

Figure 8 shows a constructed ten-minute unit hydrograph satisfying these five hydrograph parameter values. The dimensionless unit hydrograph shape presented in the USDA-SCS (1971) hydrology manual was used to approximate initially the shape of the ascending and descending portions of the constructed hydrograph in Figure 8. The unit hydrograph's ascending and descending portions below 50 per cent of peak discharge were then adjusted until the represented runoff volume equalled one inch.

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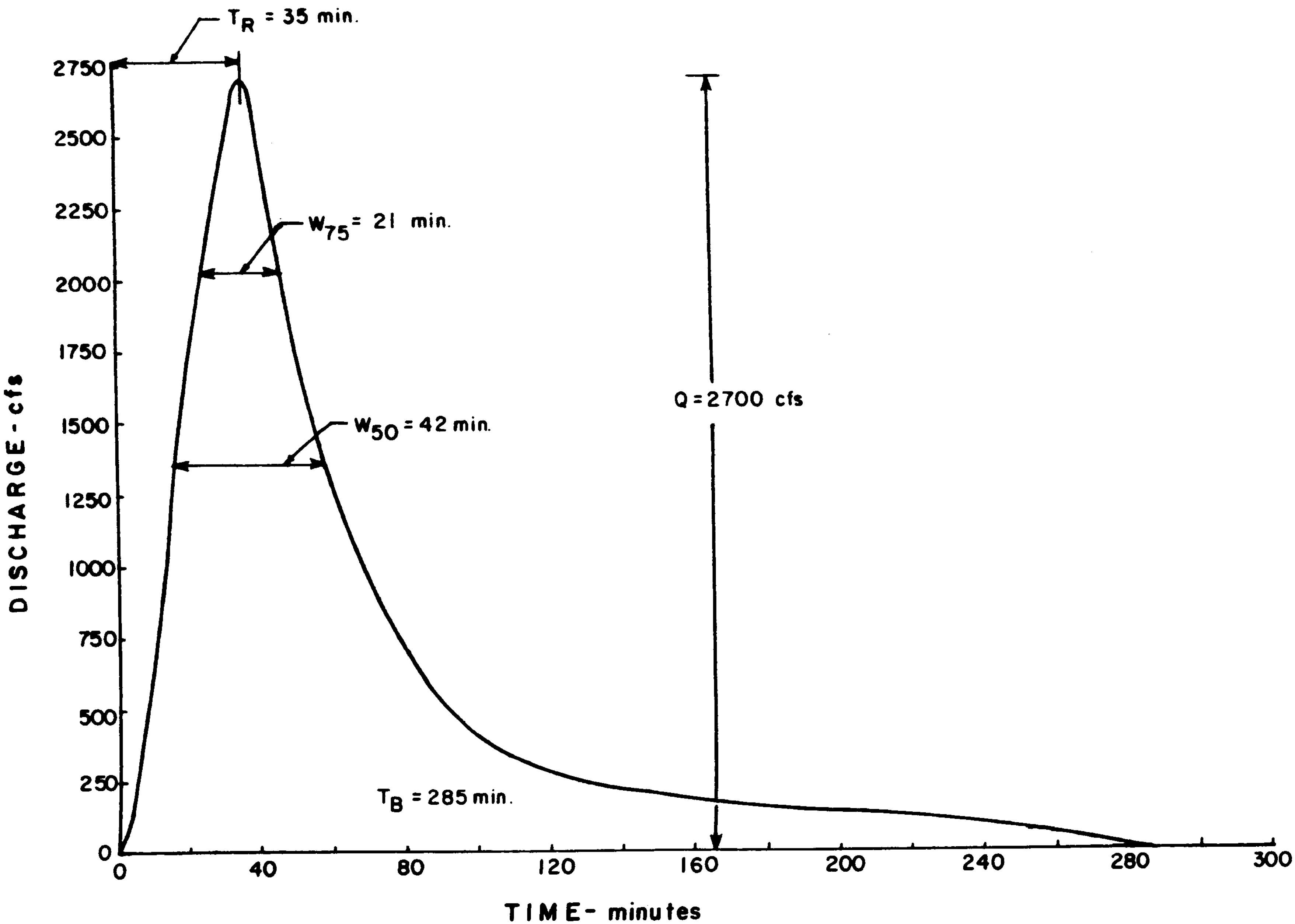

FIGURE 8- EXAMPLE OF TEN-MINUTE UNIT HYDROGRAPH CONSTRUCTION

RESEARCH ON THE DESIGN STORM CONCEPT

ASCE Urban Water Resources Research Program

Technical Memorandum No. 33

by

Jiri Marsalek
National Water Research Institute
Burlington, Ontario,
Canada

September, 1978

American Society of Civil Engineers
345 East 47th Street
New York, N.Y. 10017

PREFACE

by M. B. McPherson

Background

The following Technical Memorandum is Addendum 4 of a 1977 ASCE Program report on "Urban Runoff Control Planning".[1] Addendum 1, "Metropolitan Inventories," and Addendum 2, "The Design Storm Concept," were appended to the latter report. Addendum 3[2] was the first of several additional, individual Addenda to be released over the period 1977-1979.

The principal intended audience of the ASCE Program's June, 1977, report was the agencies and their agents that are participating in the preparation of areawide plans for water pollution abatement management pursuant to Section 208 of the Federal Water Pollution Control Act Amendments of 1972 (P.L. 92-500). While the presentation which follows is also directed to areawide agencies and their agents, it is expected that it will be of interest and use to many others, particularly local governments.

ASCE Program

The American Society of Civil Engineers' Urban Water Resources Research Program was initiated and developed by the ASCE Urban Water Resources Research Council. The basic purpose of the Program is to promote needed research and facilitate the transfer of findings from research to users on a national scale.

Abstracts of the twenty-eight reports and technical memoranda of the Program for the 1967-1974 period are included in a readily available paper.[3] The two reports and the six technical memoranda of the regular series completed since are identified in a companion publication.[4] Also included in the latter is a listing of all but one of the twelve national reports in the special technical memorandum series for the International Hydrological Programme; and the last national report[5] and an international summary[6] have been released since.

A Steering Committee designated by the ASCE Council gives general direction to the Program: S. W. Jens (Chairman); W. C. Ackermann; J. C. Geyer; C. F. Izzard; D. E. Jones, Jr.; and L. S. Tucker. M. B. McPherson is Program Director (23 Watson Street, Marblehead, Mass. 01945). Administrative support is provided by ASCE Headquarters in New York City.

Design Storms

In our Addendum 2, "The Design Storm Concept," in our June, 1977, report[1] we attempted to indicate the hazards that might be encountered in interpreting results of analyses based on that concept. However, we were forced to dwell almost exclusively on the characteristics of rainfall because demonstrations of the effects of transforming synthetic rainfalls into runoff were lacking. The report that follows provides the first such demonstration. While the findings are site-specific and are certainly not universal, they effectively show the potential liabilities in the adoption of synthetic storms for the planning and design of important works.

We have long advocated reference to a long period of historical rainfall for simulation of important sewered systems. It was not proposed that the entire record would be used in every simulation exercise. Rather, we suggested that the about two-dozen actual storms of analysis interest be identified from total record simulations of token catchments, and that the resultant set of "design storms" be applied thereafter in analyses for other catchments in the jurisdiction involved. The report herein describes an alternative method for accomplishing essentially the same objective. A somewhat similar procedure was reported in 1972[7] for segregating storms of primary interest to facilitate simulation of urban catchment flows for a long period of record. More recently, a storm screening procedure applied to the rainfall data for several Canadian cities has been described.[8]

An alternative to the above approaches has been sought in an attempt to preserve the statistically attractive features of continuous hydrological modeling while reducing the extensive computer costs of such simulations. To this end, a "fixed recurrence interval transfer technique" has been developed by the Southeastern Wisconsin Regional Planning Commission.[9] Using this technique, once a full continuous simulation has been completed for a given catchment condition, it is possible to explore revised conditions via simulation of a few selected meteorological intervals of data. In this way, alternative plans and projections can be compared. The technique was developed "for preliminary screening and assessment of the impact of alternative land use conditions and structural water control measures".[9] While the technique is illustrated in the reference with flood flow simulation examples, it is noted that the concept has been used already in water quality simulations.[10]

Testing of the design storm concept is also under way overseas, for example in Sweden.[11]

Results have been reported from a preliminary test of design storms versus actual rainfall data in Denver.[12] The 190-acre catchment is drained by separate storm sewers and concrete lined channels. A unit hydrograph derived from field data for the catchment was used in the tests. Concluded was that design storms can produce results that can vary significantly from the probability distribution of runoff simulated using actual rainstorm data. Planned are similar analyses for about fifteen other catchments for which rainfall and runoff data are available.

Initial indications from an investigation at the University of Illinois at Urbana of the design storm concept will be reported in December 1978.[13] This writer was advised in private correspondence with the University investigator that preliminary findings, for a large urban watershed for which the model used was calibrated against field data, were analogous to those in Denver.

Both the Denver and University of Illinois correspondents have warned that the magnitudes of assumed rainfall abstractions used in simulations can have as large an effect on the results as the use of design storms.

We may state with little fear of contradiction that exploration of the design storm question has only barely begun. This is not too surprising when we consider that most of the more comprehensive simulation tools emerged in the 1970's.

Acknowledgments

This report draws on and is an extension of findings from papers presented at four international conferences. Mr. Marsalek has assembled in this

report the salient results of his research on implications in the use of the design storm concept in urban sewered system planning and design. The Hydraulics Research Division of the National Water Research Institute at the Canada Centre for Inland Waters has spearheaded a substantial research effort on urban hydrology, resulting in products of considerable international value, mostly under the Canada-Ontario Agreement on Great Lakes Water Quality. In 1976, Mr. Marsalek assembled a national state-of-the-art report on urban hydrological modeling and catchment research in Canada[14] for the ASCE Program, which was reprinted in Canada[15] and included in a Unesco publication.[16]

The ASCE Urban Water Resources Research Council is indebted to Mr. Marsalek and the Hydraulics Research Division of the National Water Research Institute for their generous contribution of this report as a public service.

Processing, duplication and distribution of this Technical Memorandum was supported by Grant No. ENV77-15668 awarded to ASCE by the U.S. National Science Foundation. However, any opinions, findings, and conclusions or recommendations expressed herein are those of Mr. Marsalek or this writer and do not necessarily reflect the views of the National Science Foundation.

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Footnote

(Areawide planning agencies should have the following report in their libraries: Setting the Course for Clean Water, "A Citizen's Guide to the Section 208 Water Quality Management Program," National Wildlife Association, Education Division, 1412 16th St. N.W., Washington, D.C. 20036, 64 pp., March, 1978).

RESEARCH ON THE DESIGN STORM CONCEPT

(Addendum 4 of the ASCE Program report "Urban Runoff Control Planning," June, 1977)

by
Jiri Marsalek
Hydraulics Research Division
National Water Research Institute
867 Lakeshore Road
Burlington, Ontario, Canada L7R 4A6

September, 1978

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SECTION 1

SUMMARY AND CONCLUSIONS

This report draws on and extends the findings presented in four recent papers,(1-4) assembling in one document the salient results of research on implications in the use of the design storm concept in urban drainage design and planning. Analysis was restricted to sewered catchments in a southern Ontario municipality.

Urbanization substantially increases the volumes and peak flows of surface runoff. The cumulative effect of increased runoff volumes and localized peaks then contributes to flooding of downstream areas. Such adverse effects of urbanization are particularly pronounced in the case of catchments in which the downstream part is developed and the upstream part is undergoing development. This type of catchment is fairly common in southern Ontario.

To control the increase in stormwater runoff due to urban development, many government regulatory agencies have introduced criteria for urban drainage design. Such criteria require various degrees of control of runoff from areas undergoing development.

In order to meet requirements stipulated by runoff control policies, the frequencies of runoff flows of various magnitudes need to be determined. In streamflow flood frequency studies, the frequency of occurrence of various floods is often determined directly from a flow record. Since adequate flow records are virtually nonexistent in the case of urban catchments, runoff models are employed to produce surrogate flow records from which the frequencies of occurrence can be determined. Such an approach was taken here and the frequencies were determined in two ways. Firstly, runoff flows were simulated for design storms of assigned frequencies of occurrence and it was assumed that the frequencies of the runoff peak flows were identical to those ascribed to the design storms. Secondly, runoff flows were simulated for a large number of historical storms and the frequencies of occurrence of various runoff flows were determined directly from the simulated runoff flow record. Since most sewer systems are typically designed for return periods of from one to 10 years, such a range of return periods is of primary interest here.

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 all of the storms according to their peak rainfall intensities for several durations.

In Section 3, runoff peak flows simulated for synthetic as well as actual rainfall events are compared for an actual catchment and for several hypothetical catchments patterned after typical urban developments in southern Ontario. Although the results obtained are valid only for the conditions studied, the comparisons give a general indication of the relationship between synthetic and actual storms and demonstrate some shortcomings of an approach based on a synthetic rainfall hyetograph. The analysis is restricted to runoff peak flows on small and intermediate catchments (a maximum of 130-ha).

Comparison of runoff peaks simulated for two types of synthetic design storms and for actual storms of nominally equal return periods produced widely varying results. One type of synthetic storm produced runoff peak flows much larger than those simulated for actual storms of corresponding return periods. The use of another type of synthetic storm resulted in runoff peak flows that were also larger than those simulated for the actual storms of corresponding return periods.

The use of design storms simplifies preparation of rainfall data for runoff calculations and drainage design. It appears plausible that these subjectively defined storms could be replaced by historical storms which produced runoff flows of certain frequencies of occurrence on similar catchments in a given locality. These historical storms would then be used in future drainage design. To identify such events, one needs to carry out either a continuous simulation of runoff for a reasonable time period (10 to 20 years), or to carry out single event simulations for a number of selected events. In the latter case, the initial catchment conditions may be adjusted as necessary to account for antecedent precipitation.

Selection of historical storms to be used for design is affected to some extent by the characteristics of the urban catchment on which such a storm would be applied. It is therefore recommended that a wide range of catchment parameters be covered in runoff simulations serving for the selection of historical design storms. For the design of storage, runoff volumes also have to be considered in such an analysis. On the basis of simulated performance for a catchment, described in Section 4, synthetic storms were found to be incapable of representing the true volume, timing and multiple-peak nature of actual hyetographs. For the analysis of storage, it is preferable to employ historical storms producing runoff events of known frequencies of occurrence rather than to use a design storm. Implications of the concept requiring that urban runoff be held to pre-development levels were explored through a series of runoff simulations for different degrees of detention storage.

The last Section of this report is devoted to some water quality simulation considerations. Under the conditions explored, it is concluded that the design storm concept and single-event runoff simulation are of limited use for water-quality oriented design, because of statistical nonhomogeneity of runoff quantity and quality events.

SECTION 2

SYNTHETIC DESIGN STORMS

Background

Sizing of storm drainage conduits is universally based on the concept of a design storm. In application, it is assumed that the frequency of occurrence of a runoff peak flow is identical to that assigned to the associated rainfall. Design storms have been traditionally represented as a block rainfall, and more recently by synthetic hyetographs. Such hyetographs are typically derived by synthesizing generalizations from a large number of actual events.

The concept of a design storm and its use in urban drainage design and other applications has been a subject of considerable criticism. Particularly criticized have been attempts to assign mean frequencies of probable occurrence to storms of various intensities and durations, and the presumption of identical frequencies of occurrence of coupled rainfall and runoff events has been questioned because of the inherent statistical nonhomogeneity of rainfall and runoff processes.(5) Although such criticism seems to be generally justified, the shortcomings of the design storm concept have been only tenuously demonstrated with actual rainfall data, and only indirectly in conjunction with runoff calculations. Included herein are findings from a study of the design storm concept. Two synthetic storm configurations were analyzed, termed the "Chicago"(6) and "Illinois State Water Survey (ISWS)"(7) design storms. The Chicago design storm was singled out for attention because several Canadian municipalities have adopted this type of storm representation as part of their design criteria for urban drainage. The ISWS design storm was included because of its association with a well-known computer simulation model developed for the sizing of storm drains.

Chicago Design Storms

Formulation of the Chicago synthetic hyetograph was presented over twenty years ago in August, 1957.(6) Adaptation of the method to local rainfall data elsewhere has been reported, for example, in India,(8) the U.S.(9) and Canada.(10) The Chicago method has been rather widely incorporated in North American practice because it can be readily derived from available rainfall intensity-duration-frequency relationships and partly because of limited alternative approaches. A contributing factor was undoubtedly its inclusion in a widely used handbook.(11) When the method was presented it was criticized on the grounds that it retained too many of the fallacies and empiricisms inherent in the Rational Method to recommend its principle for general use by others.(12) The fact remains that several Canadian municipalities have included this type of design storm in their design criteria for urban drainage.

In an attempt to preserve correspondence with actual rainfall events, the Chicago method takes into account the maximum rainfalls of individual durations, the average amount of rainfall antecedent to the peak intensity, and the relative timing of the peak intensity. The first step in applying the method is determination of the time antecedent to the peak intensity, expressed as a dimensionless ratio. This time, t_r, divides the hyetograph into two parts, and is defined as

$$t_r = t_p/T$$

where t_p is the elapsed time from the onset of rainfall to the peak intensity and T is the total storm duration. Values of t_r are determined individually for a number of historical storms and their mean value is used for the design hyetograph. The hyetograph intensities on either side of the peak are obtained from applicable local intensity-duration-frequency relationships in the form

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

where i_{av} is the average maximum rainfall intensity over a duration t_d, and the constants a, b and c satisfy a fit to the data. Typically, one to six hours is selected as the total storm duration, T. (However, the choice of T does not affect the magnitudes of the peak rainfall intensity or the dimensionless time to peak).

Synthetic hyetographs of the Chicago method type have been derived for various assigned frequencies for a 15-year rainfall record from the station at the Royal Botanical Gardens in Hamilton, Ontario.(13) These hyetographs were adopted for the studies to be reported herein. Figure 1(a) is an example, for a "Two-Year Storm". Figure 1(c) illustrates the typical departure of variations in an actual storm from those represented in a synthetic formulation. (The computed peak discharges for Storm No. 47 in Figure 1(c) were for about a two-year frequency). Parameter values used for the Chicago method are listed in Table 1.

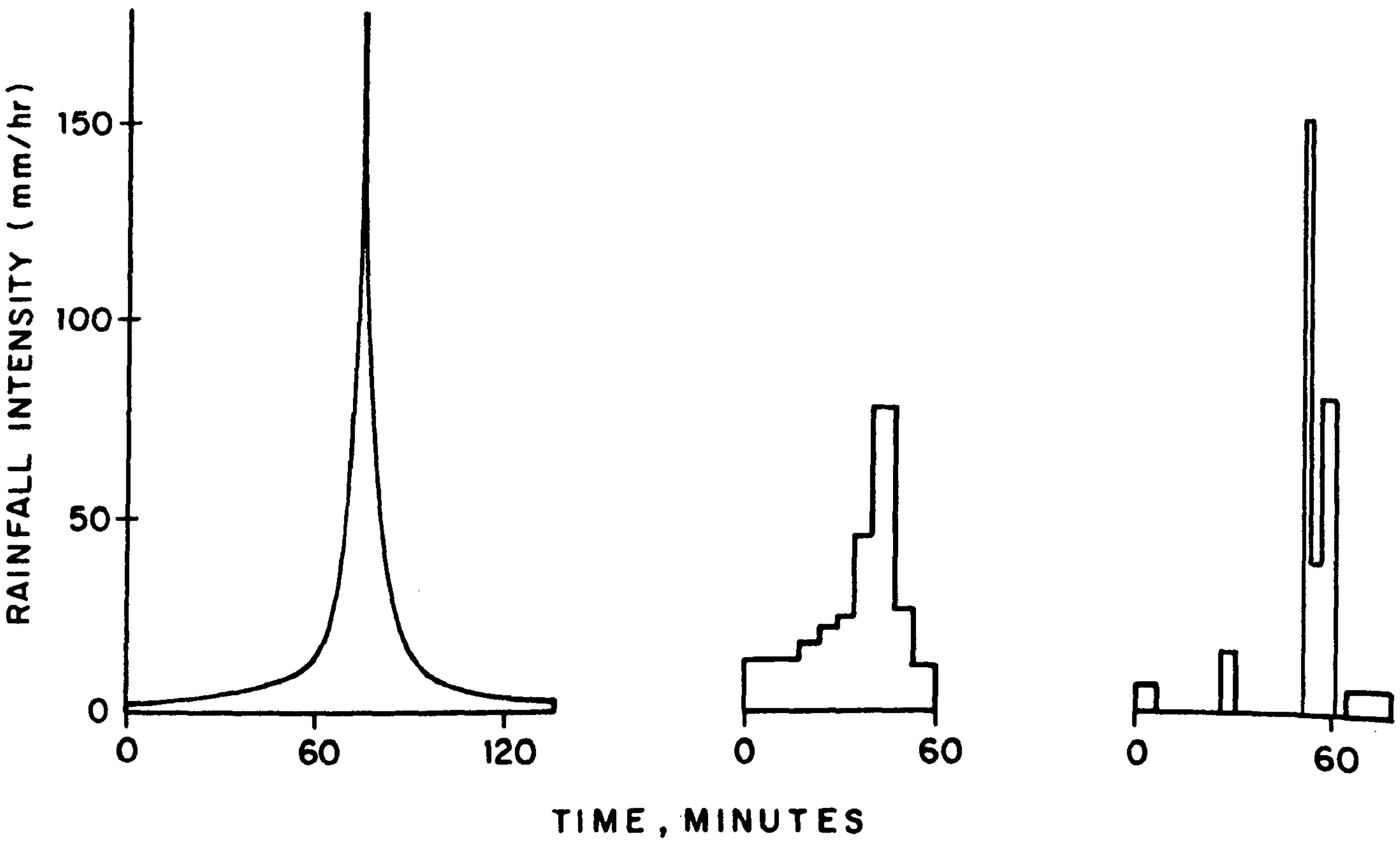
TABLE 1

Values of Parameters Used for Chicago Method

Parameters	Return Period, Years			
	1	2	5	10
t_r:	0.48	0.48	0.48	0.48
a:	22.6	29.9	43.7	55.9
b:	0.78	0.80	0.84	0.86
c:	5	6	7	8
T, min.:	180	180	180	180

Illinois State Water Survey Design Storms

In this procedure, maximum hourly rainfall depths are taken from local data or from intensity-duration-frequency relationships for various return periods. Individual hourly depths are then distributed over the selected storm duration in accordance with normalized relations developed from Illinois data.(7,14,15) For application elsewhere, actual storms are first divided into a number of groups in accordance with the relative timing of the peak intensity. Distributions over time are next determined for the predominant group of storms and their median distribution is used for the design storm.



(a) CHICAGO "2-YEAR STORM"　　(b) ISWS "2-YEAR STORM"　　(c) ACTUAL STORM No. 47

FIGURE 1 – SYNTHETIC AND ACTUAL STORM HYETOGRAPHS

As for the Chicago method, the data used to develop ISWS design storms were from a 15-year rainfall record from the station at the Royal Botanical Gardens in Hamilton, Ontario. Maximum hourly rainfall depths that had already been identified in this record(13) were used, from which values for various return periods were abstracted, Table 2.

TABLE 2

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

	Return Period, Years			
	1	2	5	10
Maximum Hourly Rainfall, mm:	22	26	33	39

About 30 heavy recorded storms, which are further described in the next subsection, were used to determine the temporal rainfall distribution. These storms were divided into three groups in accordance with the distinctive part of each storm in which the burst of peak rainfall intensity had occurred. The peak intensity for a majority of the storms occurred within the last third of their total duration. A median rainfall distribution was determined for this group and expressed as

$$R_{cp} = f(T_{cp})$$

where R_{cp} is the cumulative per cent of rainfall, T_{cp} is the cumulative per cent of storm time, and f is an empirical function. Numerical values of this distribution, which was adopted for ISWS design hyetographs, are listed in Table 3 and an example hyetograph (two-year return period) is shown in Figure 1(b). Again, there are important departures from actual storm variations, such as in the example of an actual storm in Figure 1(c).

TABLE 3

Median Rainfall Distribution of Predominant Storms
(Royal Botanical Gardens, Hamilton)

	Cumulative Per Cent of Storm Time, T_{cp}										
	0	10	20	30	40	50	60	70	80	90	100
Cumulative Per Cent of Rainfall, R_{cp}:	0	5	10	15	22	30	39	56	86	96	100

Selection of Actual Events

For analysis of certain types of urban runoff problems, particularly those related to water quality, a very strong case can be made in support of continuous rainfall-runoff-quality simulation. In some types of analysis, such as for peak flow determination, continuous simulation can be effectively approximated by using a series of single-event simulations. For the studies reported here, simulations were made for selected actual storms that would be likely to cause high runoff peak flows on urban catchments, and the findings are therefore limited to that realm of analysis.

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 4.

TABLE 4

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

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 4.

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.

Discussion

Explained above were the reasons why it was assumed to be a safe approximation to neglect the long-term effects of antecedent precipitation on runoff simulations, for the rainfall record involved. Further justification for ignoring such antecedent effects in this study is given in the next Section. However, long-term antecedent conditions indicated by the raingage record from Hamilton in Table 4 may well be unusual. Also, for small rural or very sparcely urbanized catchments, not considered here, it would probably be hazardous to neglect long-term antecedent influences.

The most important objections to the use of design storms of the type involved here are that they: attempt to summarize variegated storm patterns in a single hyetograph shape; assemble components of equal individual probability of occurrence that have a quite different joint probability; and ignore the possibility that antecedent conditions may vary considerably from storm to storm. Described in the next Section is the computer model used in the simulations, which has a single-event capability in the version employed. Therefore, it was serendipitous to be able to omit the simulation of all antecedent rainfall required in continuous simulation. That is, because minor long-term antecedent influences were indicated in the rainfall record used, it was deemed reasonable to use the single-event model without substantial modification. In what could be the more usual case where long-term antecedent conditions are more pronounced, it would be prudent to rely on continuous simulation. In sum, it must be recognized that the effect of accounting fully for the influence of antecedent precipitation on runoff for recorded storms, as opposed to its omission in design storms, was not investigated in this study. However, antecedent conditions <u>were</u> taken into account in this study, when required, by adjusting the initial detention storage and infiltration. That is, because the version of the computer model used does not maintain water balances between storms, this was done externally to set initial conditions when required. Little guidance for further refinement is provided in the literature because the effects of antecedent conditions on urban runoff are frequently discussed on the basis of conceptual models rather than on the basis of actual observations.

SECTION 3

SIMULATED PEAK FLOWS

Background

To avoid the shortcomings of the design storm approach, several researchers have proposed converting the rainfall record into a runoff record and then determining the frequencies of occurrence of various runoff flows from the latter record. Such a conversion can be performed by means of urban runoff models. In the study described here, a version of the Storm Water Management Model (SWMM) was used for runoff simulation. SWMM is essentially a single-event urban runoff model, although it has recently been run in a continuous simulation mode on a test catchment.[16] Its development was first reported in 1971 for the U.S. Environmental Protection Agency,[17] it is continuously upgraded by the U.S. EPA,[18] and it has been studied at length in Canada.[19]

The 27 top actual storms and the synthetic storms described in the preceding Section were used in SWMM simulations for an actual catchment and a group of hypothetical catchments. Simulations were performed in a single-event mode.

Actual Catchment Simulations

The test catchment, known as the Malvern catchment,[20] is a modern residential area of 23-ha with single-family housing units. The catchment is fully developed, its imperviousness is 30 per cent, and it is drained by separate storm sewers. For modeling purposes, the catchment was subdivided into ten subcatchments varying in size between 0.9-ha and 4.0-ha. The sewer system was represented by 21 pipes varying in size from 0.3-m to 0.83-m. The Malvern catchment was instrumented in 1973 and a large number of rainfall-runoff events have since been recorded there.

Before proceeding with the simulations to be reported in this subsection, SWMM was calibrated and verified for the Malvern catchment. About 25 rainfall-runoff events were available for this purpose, for which the return periods of the two highest observed runoff peak flows were about 1-year. On the average, the calibrated model underestimated observed runoff volumes by 1% and observed peak flows by 5%, with standard deviations about the means of 12% and 16% respectively. The raingage serving the Malvern catchment is located about 10-km east of the raingage at the Royal Botanical Gardens, the source of the 15-year record from which the 27 storms cited in Table 4 were obtained and from which the design storms were derived.

Design storms determined according to the Chicago method and the ISWS method were then used with the calibrated SWMM, for assigned return periods of 1-year, 2-years, 5-years and 15-years. By definition, the return periods of the resulting runoff peak flows were assumed to be identical to those assigned the design storms. For the actual storms, the simulated runoff peak flows were ranked and their frequency of occurrence determined therefrom. The results of the Malvern catchment simulations are presented in Figure 2. There are large discrepancies between the runoff peaks for the historical storms and for the design storms having the same nominal frequencies. This discrepancy will be discussed after the results from the simulations for the hypothetical catchments are presented.

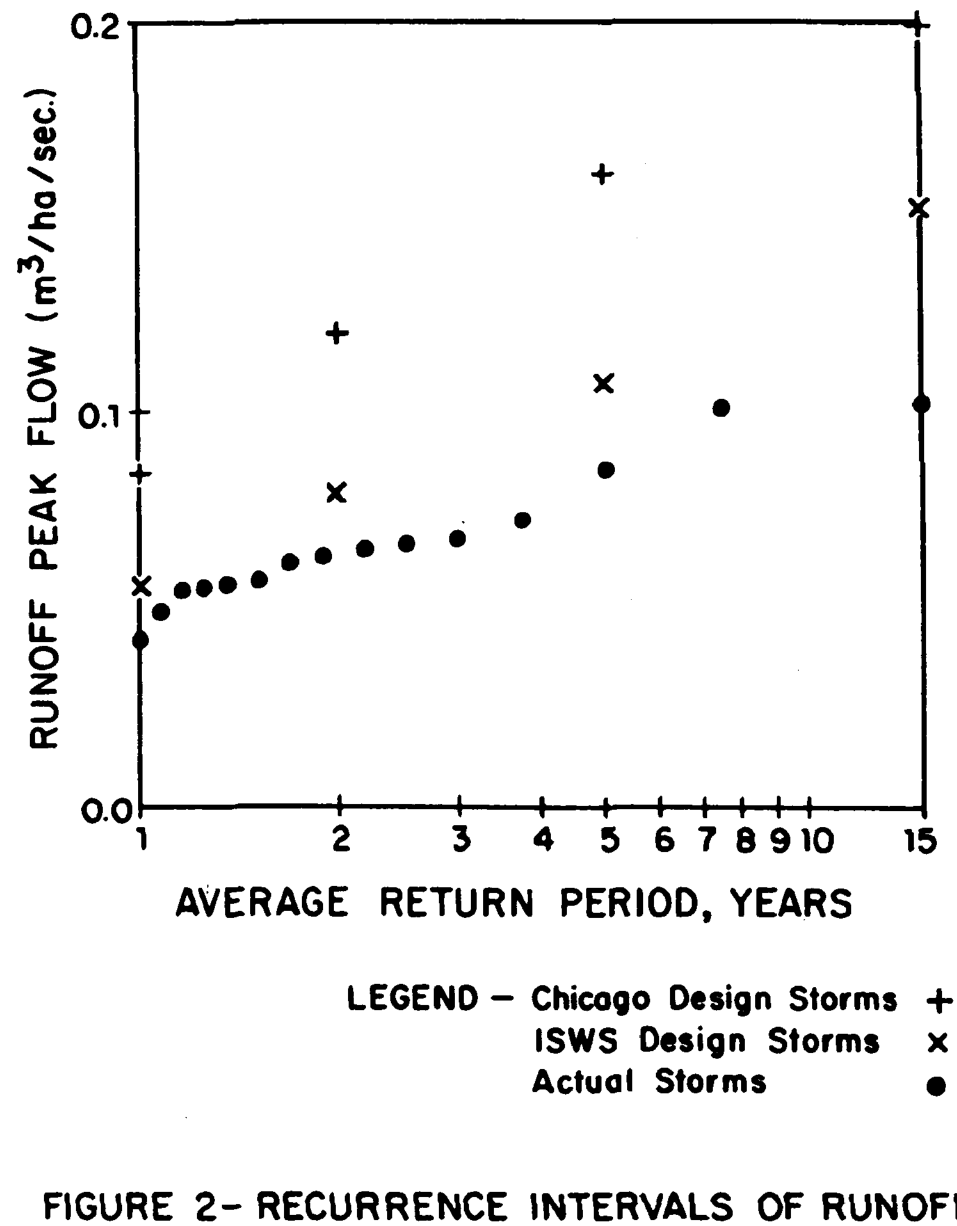


FIGURE 2- RECURRENCE INTERVALS OF RUNOFF PEAKS SIMULATED, MALVERN CATCHMENT, ACTUAL AND DESIGN STORMS

Simulation Time Step

The simulation time step was selected on the basis of previous studies on the Malvern catchment with SWMM.[20] As a part of those simulation studies, the effect of the time step on the reproduction of observed hydrographs was investigated. For high intensity storms, the best reproduction was obtained for simulations with time steps of one and two minutes. Consequently, these two time steps were adopted for all the simulations discussed in this report. Specifically, a one-minute time step was used for storms with a duration of two hours or less and a two-minute time step was used for storms of longer duration. Use of such small time steps makes possible the determination of runoff peak flows from generated hydrographs without any need for interpolation.

Hyetographs for the design storms were accordingly discretized into one or two minute intervals. For the one-hour duration ISWS design storm a one-minute interval was used, whereas a two-minute interval was used for the longer Chicago design storm, which was assigned a duration of three hours as specified by its proponents. For all simulations reported, the simulation time step was therefore the same as the time interval for the input rainfall data.

Hypothetical Catchment Simulations

Physical catchment parameters strongly influence runoff simulations and can to some extent influence the selection of rainfall inputs. In order to explore the effects of such parameters, runoff flows were simulated for a series of nine hypothetical catchments of widely varying characteristics. These catchments were patterned after some typical urban catchments located in modern residential developments in Ontario. Three catchment sizes were used: 26-ha, 52-ha and 130-ha. The drainage was maintained at about the same density for all three cases. Catchment imperviousness was set at three levels: 15%, 30% and 45%. The last two levels are typical for modern residential areas in Ontario. Because of the more extended sewer system, the lag time for the 130-ha catchment was longer by about 7 to 14 minutes than that for the 26-ha catchment.

Two types of rainfall inputs were used in runoff simulations: design storms obtained via the Chicago and ISWS methods described earlier, for four return periods; and the 27 selected actual storms.

Return periods and the associated peak runoff flows simulated for the various actual and synthetic storms are plotted in Figure 3 for the smallest hypothetical catchment (26-ha), separately for each of the three levels of imperviousness. Figure 4 is a companion illustration, for the largest hypothetical catchment (130-ha). These and similar plottings for the intermediate catchment size revealed an attenuation in peak flows per unit area with an increase in catchment size. This attenuation in peak flow was fairly consistent and represented about a 13% reduction for the largest (130-ha) catchments over the smallest (26-ha) for otherwise identical characteristics. It is conceivable that even larger differences could be encountered in practice, depending on the relation of the lag times of the catchments involved. The attenuation in peak flows with increase in catchment size is attributed to the larger concentration times noted above. The degree of this attenuation increased with the amount of imperviousness.

Comparison of runoff peaks simulated for actual and synthetic storms yielded interesting results. For all four return periods, both design storms

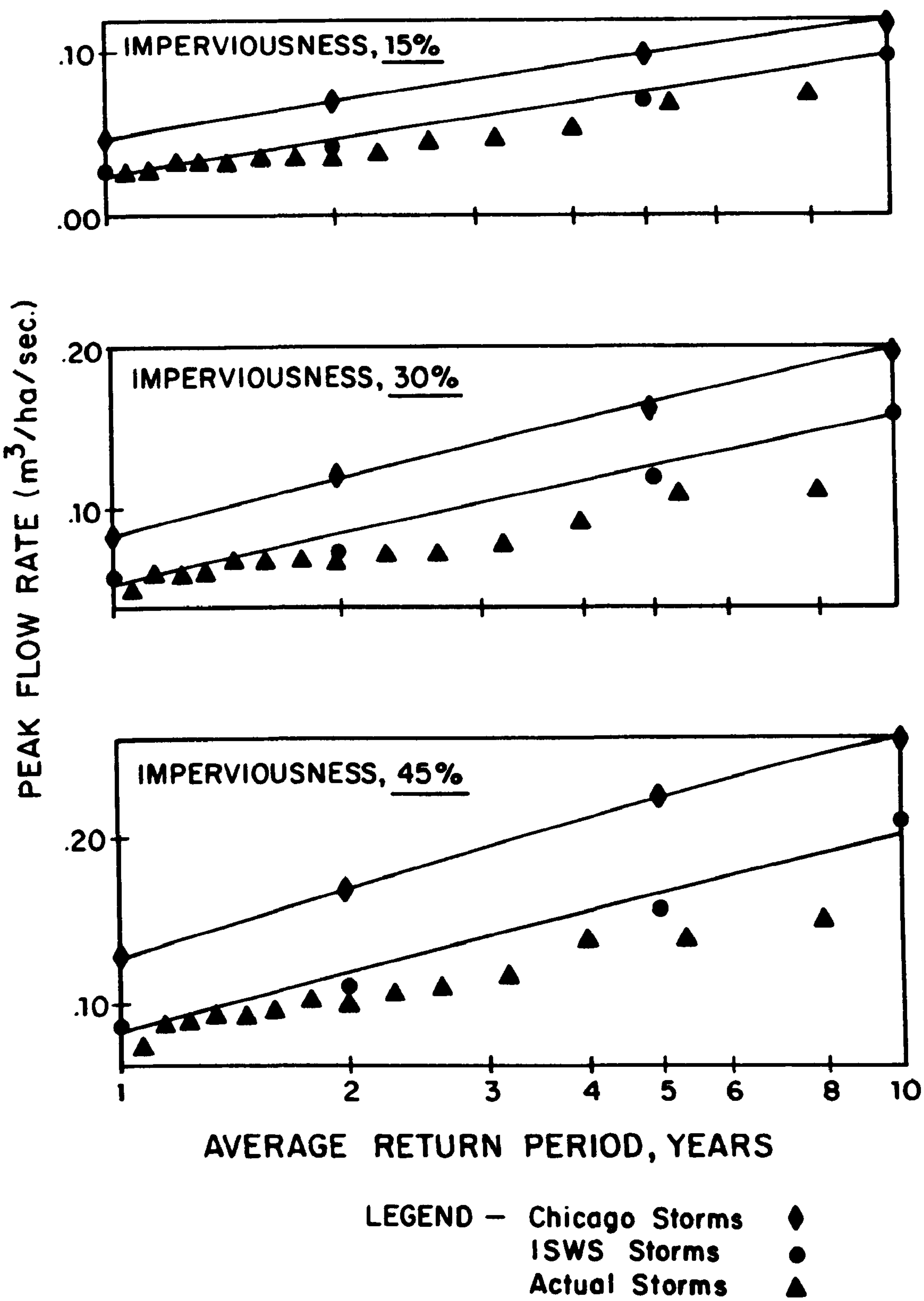


FIGURE 3 - COMPARISON OF SIMULATED PEAK FLOWS, HYPOTHETICAL 26-HA CATCHMENT

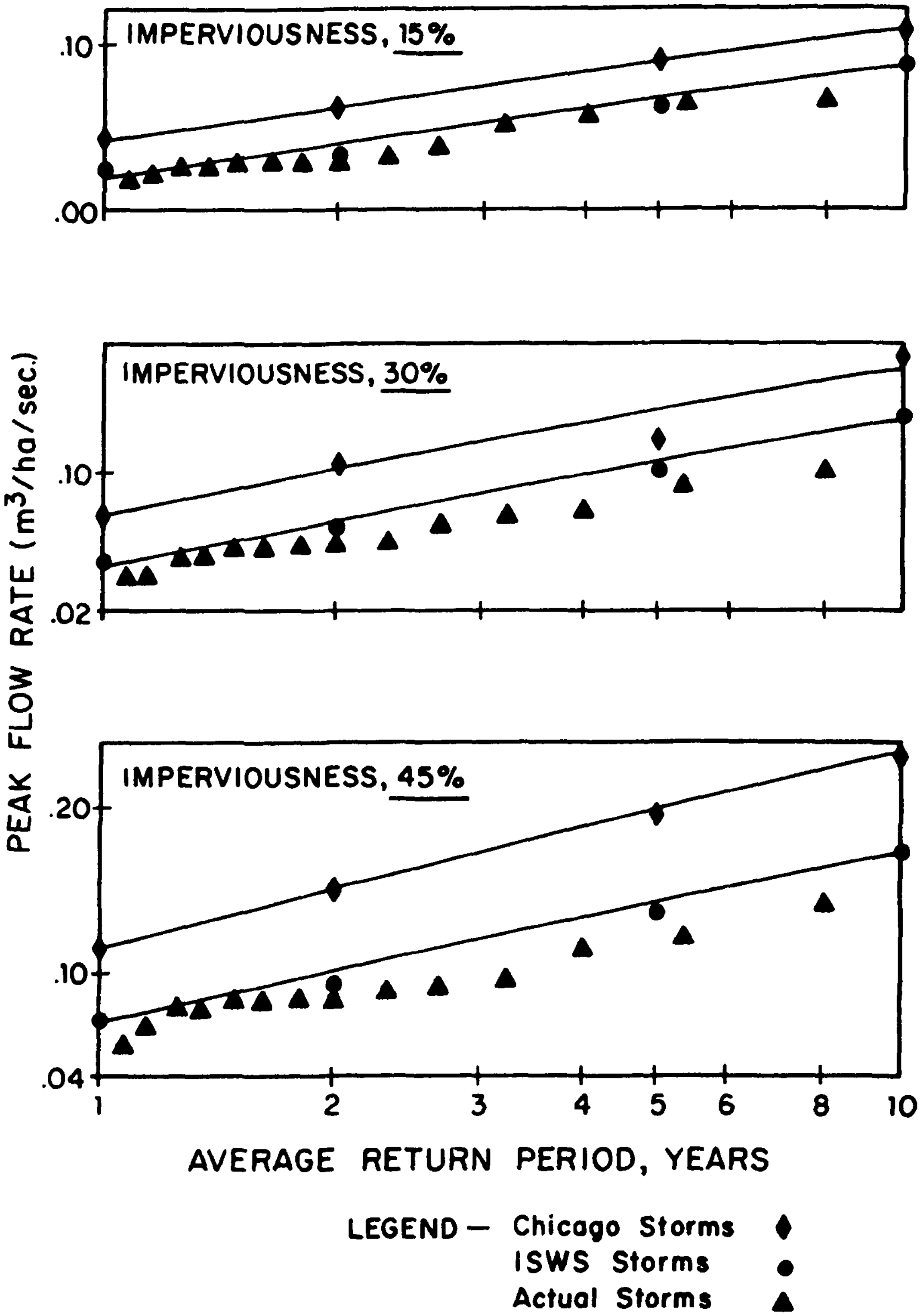


FIGURE 4 - COMPARISON OF SIMULATED PEAK FLOWS, HYPOTHETICAL 130-HA CATCHMENT

produced flows larger than those produced by the actual storms for corresponding return periods. This overestimation was particularly large for the design storms derived via the Chicago method, which produced peak flows for all three sizes of catchments that were three-fourths larger, on the average, than those produced by the corresponding actual storms. The design storms derived via the ISWS method produced better results, with simulated peak flows only one-fifth higher, on the average, than those for the corresponding actual storms.

Recall that the ISWS design storms used had a delayed pattern, dictated by the predominant characteristics of the actual storms from which they were derived. In an attempt to explore the effect of storm pattern, some simulations were also performed using the advanced pattern recommended by the ISWS for drainage design.(7) In contrast to the delayed pattern, starting at the 2-year level the simulated peak flows for the advanced pattern were lower than for the actual storms and corresponded, in general, more closely to the actual peak flows, except at the ten-year level, where they were about one-fifth below the actual storm rates. The effect of pattern shift was non-linear, as indicated below, for simulations with 30% imperviousness:

Return Period, Years	Ratio of Peak Flow for Advanced Pattern to Peak Flow for Delayed Pattern	
	26-ha Catchment	130-ha Catchment
1	0.91	0.82
2	0.84	0.80
5	0.69	0.69
10	0.61	0.62

As would be expected, storm pattern evidently has an important influence on the magnitude of catchment peak flow.

Discussion of Results

Three shortcomings affect simulated hydrographs obtained through use of Chicago design storms:

- All the values represented by a particular intensity-duration-frequency curve are implied to belong to the same storm, whereas such curves are typically obtained by a synthesis of data from a large number of storms.
- The intensity-duration curves are extrapolated beyond the smallest reported data duration of 5-minutes, yielding a peak rate exceeding the 5-minute intensity by about 60%. (However, these high intensities are reduced somewhat when the hyetograph is divided into constant time intervals).
- 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 derived from selected storms, is questionable in view of the probabilistic nature of this parameter. Analysis of the selected storms reported here yielded a mean t_r-value of 0.48 with a standard deviation about the mean of 0.32, indicating a large scatter.

While better results were obtained with the ISWS method, there is some degree of arbitrariness in the definition of these design storms, particularly in the choice of the storm duration, which affects the magnitude of rainfall intensities. A one-hour duration had been recommended by the ISWS on the basis of findings from some simulations of runoff from several urban catchments.[7] The effect of ISWS design storm duration was investigated in the present study. For the delayed pattern, when the design storm duration was reduced to one-half of an hour, the runoff peak flows increased about one-third over those obtained for a one-hour duration, such as the cases plotted in Figures 3 and 4 for a one-hour duration. When the design storm duration was raised to five hours, the runoff peak flows were much smaller than those obtained for a one-hour duration. The durations of the actual storms used here (e.g., the ones in Table 4) do not support the hypothesis of a fixed value of one-hour duration. It is evident that the comparatively better performance of the ISWS design storm for a one-hour duration reported here may be largely coincidental.

Findings from this study strongly suggest that much more attention should be given to the rainfall input than has been the normal practice. Results obtained for the two synthetic design storms differed from each other and from those for actual storms. The uncertainty in simulated runoff peaks caused by the choice of rainfall input appeared to be larger than the uncertainty inherent in the simulation process.

Recall that the actual storms used for runoff simulations were selected on the basis of the rank of peak intensities for durations of 5-, 10-, 15-, 30- and 60-minutes. As a means of investigating the efficiency of this process, correlations were examined between the ranks of peak rainfall intensities for individual durations and the associated runoff peak flows using the Spearman rank correlation coefficient. Using the values for all 27 actual storms, the correlation coefficients were larger than 0.545, which indicates a rank correlation significant at a 0.01 level of confidence.[21] The segregation of peak intensities therefore appeared to have been a good selection criterion for the identification of important historical storms which would be associated with larger runoff peaks.

Peak flow and peak storm intensity are assumed to be directly related in the Rational Method. The highest correlation coefficients obtained for simulated runoff peak flows versus peak rainfall intensities (5-, 10-, 15-, 30- and 60-minutes duration) of actual associated storms varied between 0.629 and 0.734. This means that, at best, only about half of the linear variation in the runoff peaks could be explained by the linear variation in the rainfall intensity. Evidently, other parameters of the rainfall distribution are also important in the generation of realistic runoff peak flows.

Although the results presented here are valid only for the conditions described, the methodology used in the selection of actual storms and the use of frequency graphs of runoff flows may have a general applicability. We plan to apply these methods to studies of other situations. Graphs of runoff flow frequencies, analogous to those in Figures 3 and 4, could be used to obtain quick estimates of runoff peaks from new urban developments or for checking design values.

Effects of storage reservoirs on runoff peaks were not considered in this Section. As will be shown in the next Section, such storage transposes the runoff peak flow frequency curve into a series of smaller values.

Caveat

The version of SWMM employed does not accommodate surcharging realistically in a hydraulic sense. Prior simulations of catchments of the type involved in this study revealed surcharging of some reaches at the 2-year to 5-year peak flow level. Surcharging of entire systems would be expected somewhere between the 5-year and 15-year peak flow in this type of system. Because incorporation of the WRE transport module, which accommodates surcharging realistically, would have required considerable reprogramming for the computer available, and its use would have been much more costly, an expedient was adopted instead. Conduit diameters, including those of the Malvern catchment, were artificially enlarged to sizes that would handle all flows to be simulated without surcharging. Use of this expedient naturally casts some doubt on the reliability of the results. However, because all simulations reported here employed this expedient, it is felt that the comparative results are nevertheless valid.

SECTION 4

EFFECTS OF DETENTION STORAGE

Background

Various methods of runoff control are employed in modern innovative design of urban drainage. Use of detention storage is one of the most important methods and can be used to achieve almost any degree of runoff control, provided that the costs of such facilities are not prohibitive. In Canadian urban drainage practice, use of stormwater detention ponds has become popular in recent years and many storage facilities of this kind have been constructed. Some of these ponds have a multiple-purpose use, serving not only for runoff control and reduction of drainage costs but also for recreation. These detention ponds are frequently constructed as small reservoirs with gently sloping embankments and drop-inlet spillways.

The hydrological design of detention ponds is not well established. Regardless, in urban conditions where real damage would occur as a result of flooding, detention ponds should be designed in a manner analogous to the design of flood control projects.[22] For some urban conditions, the damage would be little more than a nuisance and often a rather arbitrary decision is made to limit the probability of this nuisance to some acceptable level. In either case, however, the hydrological techniques employed should yield information adequate for the definitions of flood frequencies before and after control.[22] As discussed later, an approach based on the design storm concept does not meet this criterion.

One of the most stringent runoff controls is imposed by ordering that there shall be a zero increase in stormwater runoff due to urban development. In other words, the runoff peak flows from a developed catchment have to be restricted to their predevelopment level for rainfalls of certain frequencies of occurrence. In one case reported in the literature, such a standard was enforced for rainfalls having a 2-year or shorter return period.[23]

Simulation of Storage Effects

The concept of a zero increase in runoff is not as well defined as it would appear. Technical implications of this runoff control policy are discussed in this Section. Using historical storms, the recurrence interval of runoff peak flows was analyzed for the hypothetical catchment of 26-ha and an imperviousness of 30%, described earlier. Ten selected runoff hydrographs for this hypothetical test catchment were then routed through detention storage and, by providing a sufficient storage capacity, runoff peak flows were reduced to the predevelopment level for a range of return periods.

Characteristics of the three levels of reservoir storage employed in the simulations are listed in Table 5. In all three cases, the reservoir was assumed to be located in the vicinity of the catchment outlet.

TABLE 5

Detention Reservoir Characteristics

Reservoir Designation	Volume, cu.m.	Depth, m.	Outflow, cu.m/sec.
R-1	200	0.3	0.13
	680	0.9	0.74
	1310	1.5	0.96
R-2	500	0.3	0.13
	1650	0.9	0.74
	3020	1.5	0.96
R-3	660	0.3	0.13
	2160	0.9	0.74
	3900	1.5	0.96

Routing of runoff hydrographs through the detention reservoirs was accomplished by using the modified Puls method.(24) The outflow hydrograph peaks were then used to establish new peak flow versus recurrence interval values, plotted in Figure 5. Included also in Figure 5 are the values from Figure 2, designated in Figure 5 as "urbanized, no control". For reference, results for "non-urbanized" conditions are also given in Figure 5.

The peak outflow discharges from storage were affected not only by the magnitudes of the inflow peaks and the storage volumes, but also by the total volume of inflow and its time distribution. It can be deduced from Figure 5 that the criterion of a zero increase in runoff with urbanization is essentially satisfied for the cases of the two larger reservoirs, where the peak rates are practically identical with those for the undeveloped catchment, particularly for recurrence intervals of design interest.

Discussion

Among the various means of runoff control, only detention storage and its effects on catchment runoff were considered. By providing a sufficient storage capacity, runoff from an urbanized catchment was reduced to about the predevelopment level for a wide range of recurrence intervals (see Figure 5).

The design-storm concept was found to be of questionable use in this analysis. Multiple-peak storms may produce higher outflow peaks from storage, if a second consecutive peak arrives when the storage facility is essentially full. The outflow hydrograph of the Chicago design storm will be affected by the t_r-value. Peak runoff rates from storms with smaller t_r-values will be more attenuated than those with larger t_r-values because for smaller t_r-values the inflow peak arrives when the available storage capacity is larger than it would be for larger t_r-values. Special tests were performed using Chicago design storms with 1-year, 2-year and 5-year average return periods for all three detention reservoir cases. The resulting runoff peak flows were all higher than those indicated in Figure 5 for the historical storms ("urbanized, with storage") with departures of more than 100% for the 1-year design storm and as little as about 10% for the 5-year design storm. These results were not unexpected, as may be seen in Figure 6, which is a plot of frequency of flow volumes without storage for the related simulations (middle graph of Figure 3). In Figure 6 the

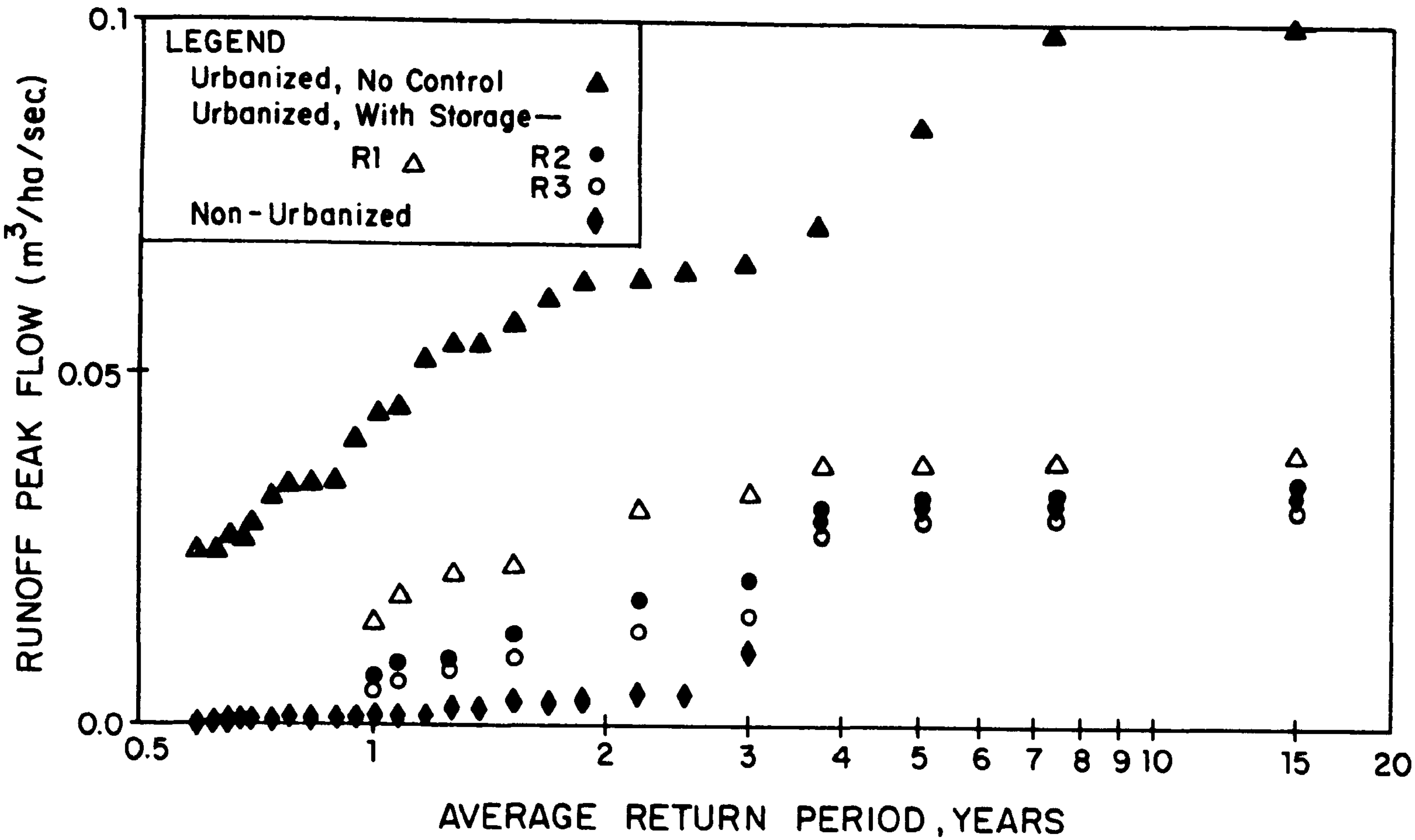


FIGURE 5 - SIMULATED EFFECTS OF STORAGE, HYPOTHETICAL CATCHMENT
(26-ha, 30% Imperviousness)

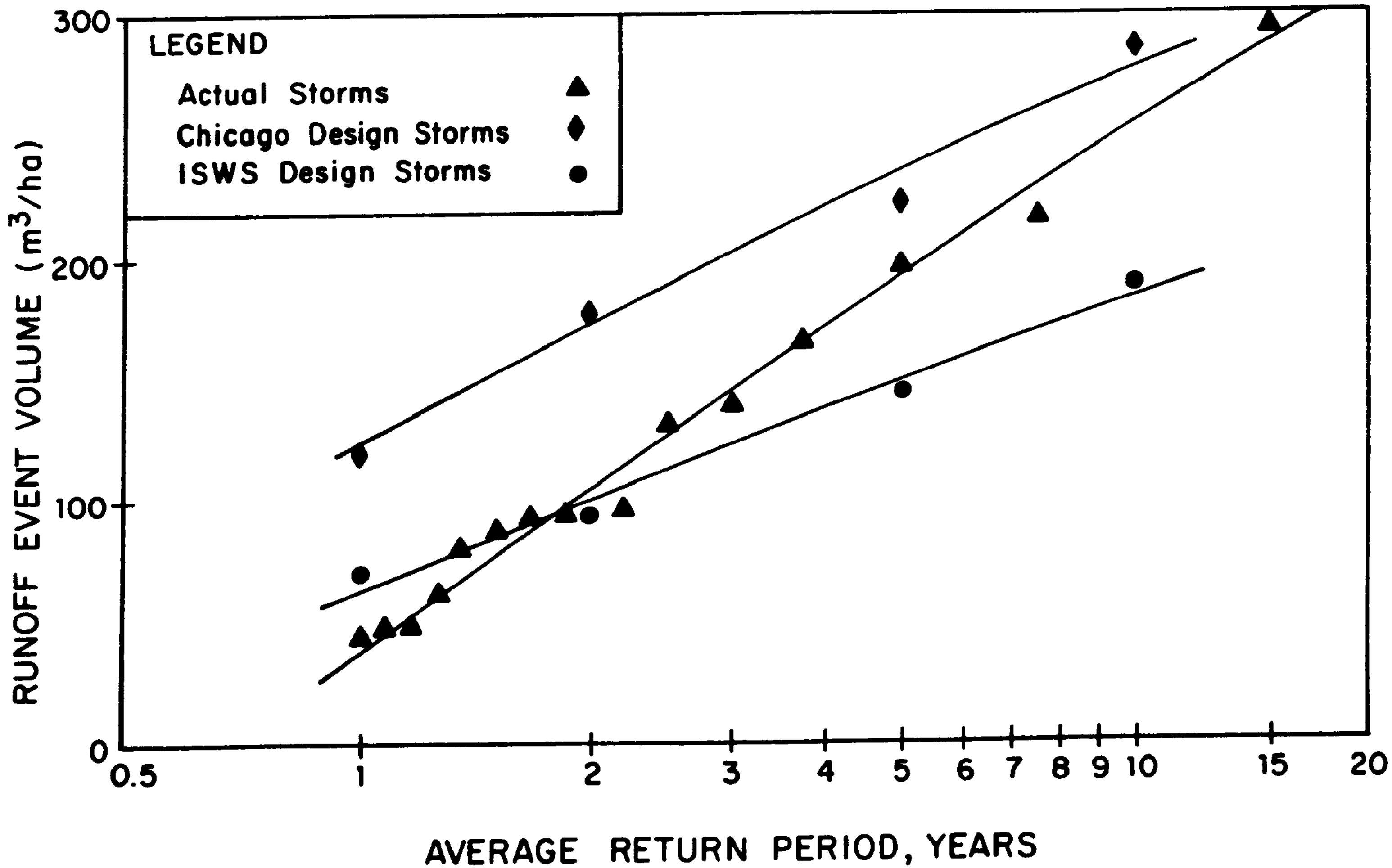


FIGURE 6 - RUNOFF VOLUMES PRODUCED BY SELECTED ACTUAL AND DESIGN STORMS
(26-ha, 30% Imperviousness)

departures in magnitude of runoff volumes between those for the Chicago design storms and those for the historical storms also decrease with rarer return period even though no routing through storage is involved.

Peak flow rates and magnitude of flow volumes are not highly correlated, as indicated in Table 6, where the top 15 peak flow rates from the 27 actual storms are associated with a seemingly random ranking of runoff volumes. During this study it was noted in about half of all cases that storm hydrographs are transformed differently by storage, with the result that frequencies of occurrence of inflows to storage and outflows from storage do not agree. Two of the most obvious examples of this change in frequencies are given in Table 7.

TABLE 6

Ranks of Selected Actual Storms with Respect to Runoff Peaks and Volumes Produced by These Storms

Storm Number	Rank of the Runoff Peak Produced by the Storm	Rank of the Runoff Volume Produced by the Storm
44	1	4
2	2	2
46	3	6
10	4	15
25	5	3
36	6	12
47	7	14
20	8	5
23	9	8
6	10	7
1	11	9
8	12	10
39	13	13
54	14	1
31	15	11

TABLE 7

Examples of Transformation of Frequencies by Storage

Storm Number (Table 4)	Average Return Period, Years	
	Without Storage	Outflow From Reservoir R3
10	4	1
54	1	8

The above indications substantiate the view that the design storm concept was found to be of questionable use in this analysis of storage effects.

The requirement of a zero runoff increase due to urban development is a political solution to flooding problems created by progressive urbanization, and under some circumstances this requirement may have little technical justification. Typically, such a requirement is specified for a design storm of a certain assumed frequency. While some control of urban runoff from urbanizing catchments is mandatory where increased runoff would contribute to flooding of downstream areas, this purpose may not be best served by the concept of zero increase in runoff due to urban development. As interpreted by various governmental agencies at the present time, this concept appears to be vaguely defined and impractical. Runoff control measures required under this policy apply to events of more or less arbitrarily specified frequency of occurrence typically expressed in terms of design storms.

Urban runoff can be controlled by means of storage to a more or less arbitrary degree, including the predevelopment level. For the analysis of storage, it is preferable to employ historical storms producing runoff events of reliable frequencies of occurrence rather than to use a design storm. The Chicago storm, in particular, is not capable of representing the true volume, timing, and multiple-peak nature of actual hyetographs.

It is recommended that the zero increase in runoff concept be replaced by a runoff policy in which the degree of runoff control is determined from a cost-benefit analysis of the control measures. The entire basin and the main stream have to be considered in such an analysis, since localized runoff control by detention and uncontrolled, poorly-timed releases of stored runoff may still lead to flooding in the main stream.

SECTION 5

WATER QUALITY SIMULATION CONSIDERATIONS

Background

Consideration of the water quality aspects of urban drainage discharges represents a new facet of urban drainage design. As "point" pollution sources are abated and become less threatening, "non-point" sources of water impairment become relatively more significant. Among such non-point sources, urban runoff has been found to be particularly important. For example, a recent study substantiates that urban runoff can convey high pollution loads, and under certain conditions can control water quality in receiving waters for extended time periods.[25]

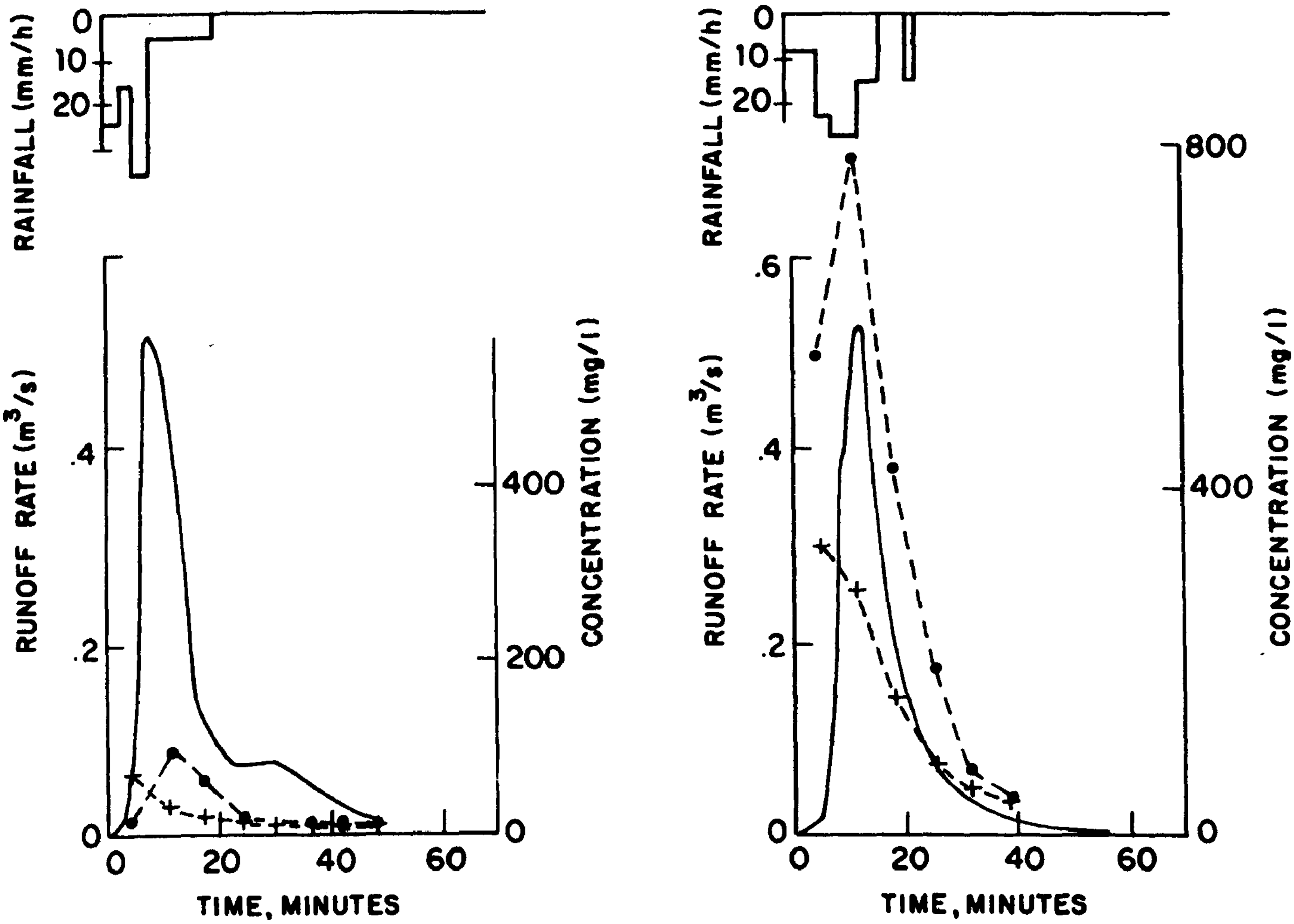
Some water quality considerations have to be recognized early in the planning of urban drainage. Such considerations are related to pollution control at the source and include the planning of land use, type of development and extent of natural drainage.

Water-quality oriented design of urban drainage is somewhat hindered by a lack of understanding of runoff-quality processes and by a lack of authoritative water quality criteria for use in such a design. Ideally, water quality criteria should be defined as receiving water criteria. In that case, a given level of water quality (e.g., pollutant concentration) has to be attained during critical storms and under critical conditions in the receiving water body. Extensive data needs and complex modeling of the environmental response retard wider application of receiving water criteria. More often, effluent control criteria are imposed in Canada. Such criteria can be stated in terms of annual allowable loads, number of events (e.g., annual average number of overflows from a combined sewer system), controls for specified events, or maximum allowable pollutant concentrations, or by prescribing specific control measures for an area or at an effluent.[26]

Although the best estimates of runoff quality are obtained from extensive local field data, such data are virtually nonexistent and quality simulation models have to be employed. Such simulation models again require precipitation input data. These data, however, differ from those used in the hydraulic design of urban drainage. Before discussing the nature of these differences, mention should be made of the relative accuracy of quantity and quality models. In each case, the uncertainty in the simulated output depends upon the uncertainties in the input and in the model description of the processes which transform inputs into outputs. Uncertainties in precipitation inputs are comparable for both cases. However, uncertainties in contaminant inputs and in the descriptions of the actual processes lead to far greater errors in simulated quality outputs than in simulated quantity outputs.

Antecedent Rainfall Effects

The magnitude of pollution loads conveyed by urban runoff events depends strongly on the initial conditions of the event, which are thought to be characterized by the amount of pollutant accumulated on the catchment surface during the antecedent dry-weather period. This point is illustrated in Figure 7, which contains runoff hydrographs and pollutographs from the Malvern catchment for two storms of similar volume and intensity of rainfall. The two hydrographs



(a) ONE-DAY DRY-WEATHER PERIOD (b) SIXTEEN-DAY DRY-WEATHER PERIOD

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FIGURE 7 - EFFECTS OF DRY-WEATHER PERIOD, MALVERN CATCHMENT

agree quite well but the storm with the longer antecedent dry-weather period, Figure 7(b), produced pollution loads about five times higher than those conveyed by the other storm, Figure 7(a). Further changes in the magnitude of event pollution loads might be introduced by street sweeping that effectively reduced the accumulation of pollutants on the catchment surface between storms.

One possible way to estimate the antecedent dry-weather period is to analyze the historical precipitation record and to derive therefrom probabilities of dry-weather durations for individual events. An example of such a relationship is given in Figure 8. For a catchment in Burlington, Ontario, a five-year precipitation record was analyzed for the duration of dry weather period considering only storms with rainfall sufficient to wash off the catchment surface (in this case, storms with rainfall larger than 2.5-mm). If the pollutant accumulation rates are known or determined from street sweeping experiments, the total pollution loads per event and their probability of occurrence can be determined from Figure 8 by multiplying the accumulation rates by the number of dry days. Note that this approach assumes that the distribution of dry periods does not depend on storm characteristics and that any antecedent storm would have completely washed off the catchment. Such assumptions appear to be acceptable when considering the large uncertainties involved in runoff quality computations.

Discussion

Under the circumstances described above, the design storm concept and single event simulation are of limited use for water-quality oriented design. Storms producing high runoff flows may produce relatively low pollution loads and vice versa. Consequently, continuous simulation of runoff quality, quality control and associated costs should be employed using historical precipitation data. Such simulations offer a good basis for the selection of the most cost-effective control alternative meeting water quality criteria. As for design peak flows, discussed earlier, continuous simulation might be reduced to indicator catchments, with abbreviated simulations indicated thereby successively applied to the more numerous other catchments in the jurisdiction involved.

In summary, water-quality oriented drainage design is a new idea which has not yet gained wide acceptance. Quality considerations related to source control have to be undertaken in the planning process. Among these considerations, one could name land use, type of development and extent of natural drainage. Precipitation data requirements for water-quality oriented design are virtually identical to those for the planning of urban drainage. Subsequently, the quality design is finalized by means of detailed simulations for selected events of which the initial conditions have been determined. The design storm concept has little application in quality design because of statistical nonhomogeneity of runoff quantity and quality events. Such nonhomogeneity is illustrated by the data plotted in Figure 7. Typically, historical rainfall events are used in quality design. Additional data, such as rainfall chemistry and atmospheric fallout rates may be useful.

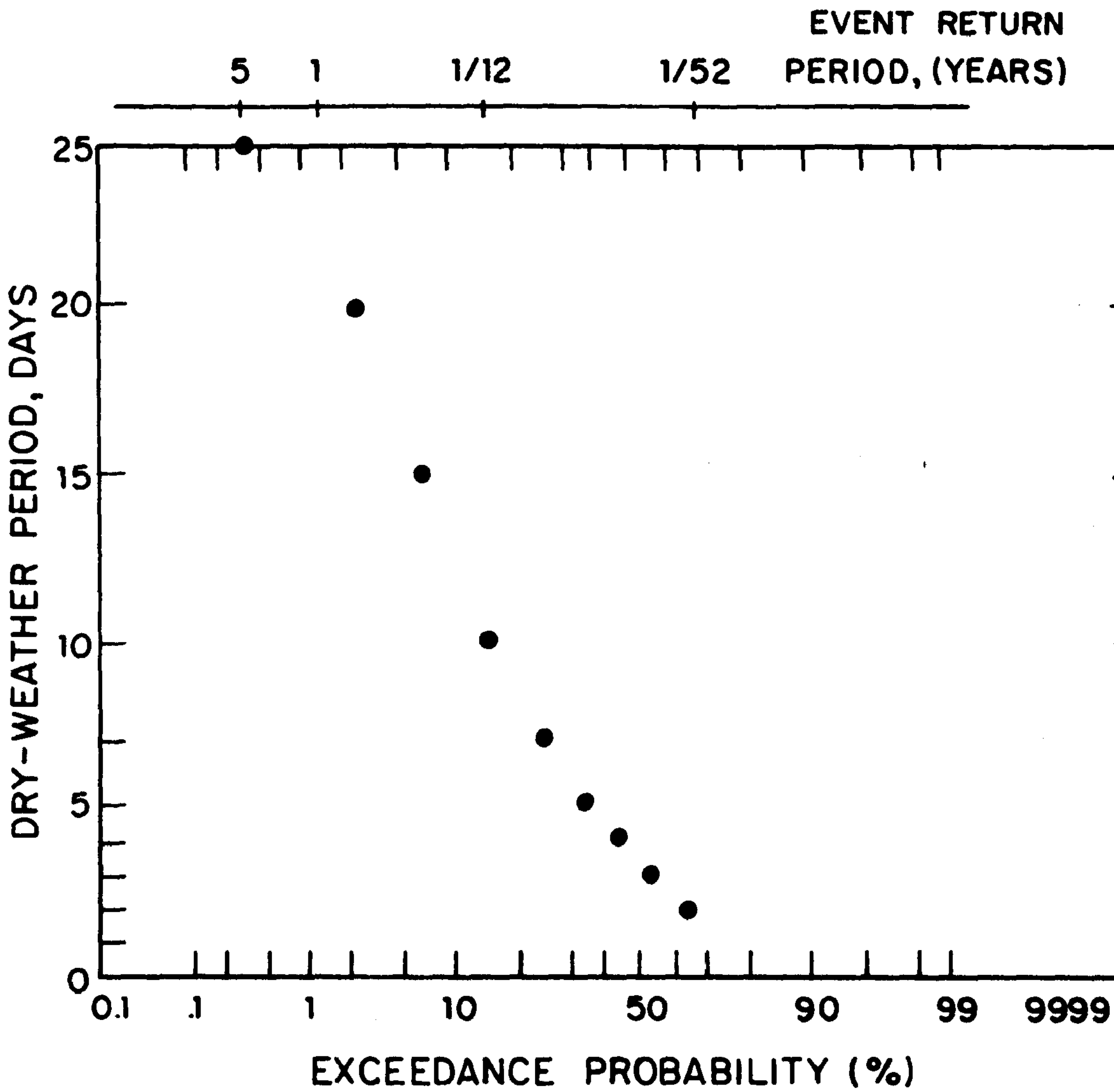

FIGURE 8- PROBABILITY OF DRY-WEATHER PERIODS, BURLINGTON, ONTARIO

SECTION 6

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*U.S. GOVERNMENT PRINTING OFFICE: 1978 620-007/3737 1-3

TECHNICAL REPORT DATA
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1. REPORT NO. EPA - 600/9 - 78 - 035	2.	3. RECIPIENT'S ACCESSION NO.
4. TITLE AND SUBTITLE Urban Runoff Control Planning		5. REPORT DATE October 1978
		6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Murray B. McPherson		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS ASCE Urban Water Resources Research Program, 23 Watson Street, Marblehead, Mass. 01945		10. PROGRAM ELEMENT NO.
		11. CONTRACT/GRANT NO. N/ A
12. SPONSORING AGENCY NAME AND ADDRESS U.S. Environmental Protection Agency, Office of Research and Development, Washington, D.C. 20460		13. TYPE OF REPORT AND PERIOD COVERED
		14. SPONSORING AGENCY CODE EPA 600/Z
15. SUPPLEMENTARY NOTES		

16. ABSTRACT

Section 208 of Public Law 92-500 (Federal Water Pollution Control Act of 1972) encourages areawide planning for water pollution abatement management, including urban runoff considerations where applicable. Areawide studies are under way or planned in just about every metropolitan area. Deadlines for initial areawide reports are not far off, and it is expected that many of the agencies preparing reports are presently resolving their projected activities beyond the current first planning phase. This report has been prepared to assist agencies and their agents that are participants in the preparation of areawide plans, from the standpoint of major urban runoff technical issues in long-range planning. Emphasized is the importance of conjunctive consideration of urban runoff quantity and quality and the need to development a factual basis that will support expected reliability of performance of proposed actions and programs. While not intended as a handbook for urban runoff control planning, this report delves into some important technical issues that are often slighted or poorly handled, such as the utilization of simulation. Recognizing that the ultimate test of any plan lies in its implementation, topics are viewed from the perspective and experience of the local government level where implementation takes place.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Urban Runoff Master Planning	Urban Hydrology Storm Water Runoff Combined Sewer Overflows	13 B

18. DISTRIBUTION STATEMENT Release to Public	19. SECURITY CLASS *(This Report)* Unclassified	21. NO. OF PAGES
	20. SECURITY CLASS *(This page)* Unclassified	22. PRICE

EPA Form 2220-1 (9-73)