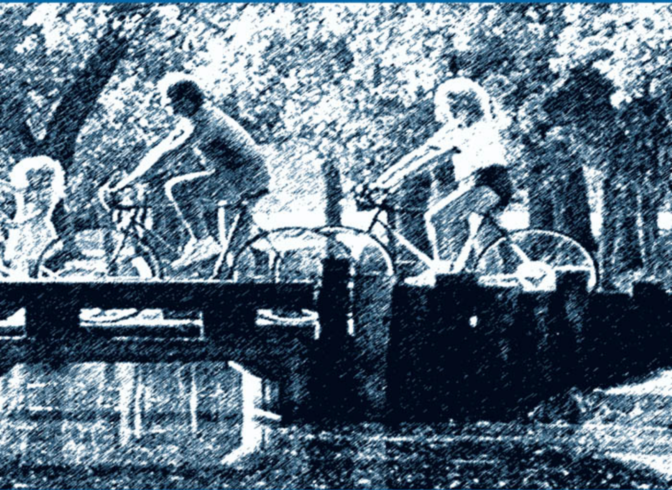
GREAT WORKS ON URBAN WATER RESOURCES (1962–2001)

FROM THE AMERICAN SOCIETY OF CIVIL ENGINEERS URBAN WATER RESOURCES RESEARCH COUNCIL

EDITED BY JONATHAN E. JONES



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GREAT WORKS ON URBAN WATER RESOURCES

(1962–2001)

FROM THE AMERICAN SOCIETY OF CIVIL ENGINEERS URBAN WATER RESOURCES RESEARCH COUNCIL

SPONSORED BY

Environmental and Water Resources Institute (EWRI) of the American Society of Civil Engineers
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EDITED BY

Jonathan E. Jones, P.E.

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Contents

Preface

Background regarding the nature of this book is provided in Chapter 1, Introduction, Overview, and Summary of Publications. The many outstanding members of the American Society of Civil Engineers and Environmental Water Resources Institute who contributed to this effort are listed in Chapter 1. The current membership of the Urban Water Resources Research Council sincerely hopes that practitioners enjoy and benefit from the greatest of the Research Council's publications over the past 40 years, nearly all of which are as timely and provocative in 2005 as when they were first published. The current members of the Research Council also call special attention to the founding members of the Research Council, including: Stifel W. Jens, D. Earl Jones, Jr., J.C. Geyer, Carl F. Izzard, and William C. Ackerman, along with the original Council Program Director, Murray B. McPherson and his deputy, L. Scott Tucker. All water resources practitioners owe a debt of gratitude to these leaders and visionaries.

Acknowledgements

The Editor in Chief, Jonathan E. Jones, P.E., sincerely thanks the following people for their invaluable contributions to the formation of the UWRRC, its long-term contributions to the profession, and/or to this effort to republish the Council's great works:

- My father, D. Earl Jones, Jr., and one of the five founders of the Council.
- The other four founders of the Council:
    - Stifel W. Jens,
    - J.C. Geyer,
    - Carl F. Izzard, and
    - William C. Ackerman.
- M. B. McPherson, Director of ASCE's Urban Water Resources Research Program for many years and lead author of many of the documents, herein.
- Scott Tucker, Assistant to M. B. McPherson and author or co-author of many of the documents, herein.
- Ms. Frankie Lane, document typing and production.
- Ms. Mary Simmerman, document typing and production.
- Michael Ports, document production and review.
- Ben Urbonas, document review.
- Don Van Sickle, document review.
- Larry Roesner, document production.
- Richard Field, document production.
- Neil Grigg, document production.
- Conrad Keyes, document production.
- Charles Rowney, document production.
- Stuart Walesh, document production.
- All officers and other members of the UWRRC from 2003 (when this project began) to 2005 (publication date).
- The Governing Board of EWRI.

Chapter 1

INTRODUCTION, OVERVIEW AND SUMMARY OF PUBLICATIONS

1.0 BACKGROUND

Over the past four decades, members of the American Society of Civil Engineers (ASCE) Urban Water Resources Research Council[1] have made extraordinary contributions to the field of urban water resources planning, design, and management. Historically, the Council has been comprised of many leading water resources engineers and scientists, and they have collaborated to prepare over 150 reports, technical memoranda, conference proceedings, and other documents on wide-ranging subjects. The purpose of this book is to republish selected great works of the UWRRC.

Many of the UWRRC's publications are as timely and provocative today (2005) as when they were first published. Indeed, readers will be struck by the foresight and perspectives of Council members from decades ago on such subjects as: drainage and flood control; stormwater quality management; water supply planning; risk assessment; public involvement and participation, and the role of the water engineer in society, to name only a few.

The founding members of the Council include:

- Stifel W. Jens
- D. Earl Jones, Jr.
- J.C. Geyer
- Carl F. Izzard
- William C. Ackermann

The original Council Program Director was Murray B. McPherson and the Deputy Program Director was L. Scott Tucker. All water resources practitioners owe a debt of gratitude to these leaders and visionaries.

2.0 UWRRC PURPOSE AND MODE OF OPERATION

The purpose of the UWRRC, as set forth in the ASCE Official Register, is:

> *To advance engineering knowledge and practice through stimulating and guiding research and assisting the financing thereof in the field of urban hydrology; to organize research projects; in cooperation with professional committees, to interpret the findings of such research; and to make available*

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[1] The following terms are used interchangeably in this book: Urban Water Resources Research Council, UWRRC, Council, and Research Council.

> *information and recommendations resulting from such research. The Council shall study continuously the needs for new information in the subject field and recommend such new research as may promise to furnish new information.*

In short, the Council has emphasized identification of research needs, the conduct of research, and transferring research results to practitioners.

Council members have regularly addressed a long recognized and recurring problem in urban water resources—using much less than we already know. The state-of-the-practice often falls too far behind the state-of-the-art. That is, the way in which we plan, design, construct, and operate our urban water systems falls short of what we know about planning, design, construction and operations. Many practitioners in public and private practice are not using the available tools and technology. Furthermore, practitioners face ever-rising expectations. Herein lies the mission of the UWRRC—searching out, gathering, packaging, and disseminating existing and newly emerging technology so that it will be used by more practitioners.

The Council seeks to accomplish the technology transfer mission in many ways. Examples include task committees leading to special reports, producing numerous technical memoranda over the years, arranging conference sessions and entire conferences, working on projects that involve collaboration with other professional societies and developing major publications and computer-based products.

An overriding theme is products—as in, producing written materials. From the beginning, every proposed Council activity must have a goal of producing products such as bibliographies, proceedings, reports, and manuals of practice.

3.0 EVOLUTION OF UWRRC ACTIVITIES AND FOCUS AREAS

3.1 Broad Perspective Adopted

Historically, the Council has focused on "big picture" issues, with major economic implications for the United States. For example, the following text is excerpted from the May 1968 issue of *Civil Engineering* regarding the UWRRC:

> *Several facets of research on urban water problems have long been neglected. This is true in spite of the fact that urban drainage works alone seldom cost less than $100 per capita, and in the United States not less than $25 billion in direct capital costs will be spent on drainage works in the next 40 years. This does not include the cost of damage to property and life from flooding. Compared to this staggering figure, funds being spent to research urban water problems are minor and almost non-existent.*
>
> *To bring attention to these problems and to begin to fill the gaps in present knowledge, a program to study and report on research and data needs for urban water problems has been developed and initiated by the ASCE Urban Hydrology Research Council (subsequently renamed the Urban Water Resources Research Council). The purpose of the ASCE project is to*

> *establish a coordinated long-range national program of research in urban water resources.*

3.2 Phased Implementation of Research Program

ASCE's Urban Water Resources Research Program was initiated and developed by the UWRRC. The basic purpose of the Urban Water Resources Research Program was to help establish coordinated long-range research in urban water resources on a national scale.

Phase I of the research program, 1967-1969, was sponsored by the Federal Office of Water Resources Research (OWRR) and the U.S. Geological Survey (USGS). There were two final reports and ten technical memoranda prepared during Phase I. The theme of Phase I was a research needs assessment.

Phase II, 1969-1971, was sponsored exclusively by OWRR. The theme of Phase II was urban water management. Phase II resulted in two reports and five technical memoranda.

Phase III, 1971-1974, sponsored by OWRR, emphasized translation of research into practice, facilitation of urban runoff research, and collaboration and participation in research of municipalities and other organizations. Three technical memoranda were completed under Phase III.

M.B. McPherson and G.F. Mangan prepared an excellent summary of the UWRRC's activities in October 1974 in a report entitled "ASCE Urban Water Resources Research Program, 1967-1974" (Technical Memorandum #25). Key quotations from that document include:

> *Activities of the ASCE Urban Water Resources Research Council (UWRRC) have been a powerful factor in focusing the attention of public agencies at all levels of government on critical research needs. It all started with the Council's first Engineering Foundation conference in 1965 where many crucial research needs were identified, particularly those in urban hydrology. Recognizing the chasm that had to be bridged, the Council formed the ASCE program in 1967 to serve as its temporary, fulltime operating arm. This made it possible to marshal the assistance of hundreds of cooperators, in addition to Council members, and to mount an uninterrupted intensive program of public service. While financial support was obtained entirely from federal agencies, major contributions to the program were made by numerous local government officials who provided ready access to the expertise of their staffs. Program products thus far include major reports and numerous technical memoranda that have significantly influenced the magnitude and direction of urban water resources research, and have provided numerous opportunities for civil engineers and kindred professionals to participate in the programs thus stimulated.*

From 1974 on, the Council's activities were not designated as "Phases", but there was no let-up in the quantity or quality of the documents, as shown in Table 1-1.

The work of the UWRRC has often had an international flavor. The Council has been anxious to learn from the successes and failures of colleagues in other countries. Various reports herein address water resources management in other countries.

3.3 Emphasis on Both Technical and Non-Technical Issues

In the course of reviewing the approximately 150 UWRRC-sponsored documents that were collected in this effort, the variety of subjects addressed by the UWRRC, and the depth with which each of these subjects was investigated, is truly remarkable. At one time or another (and often on many occasions), the Council has addressed virtually every facet of urban water resources planning, design, and management, including public and private water supply, water treatment, wastewater treatment, combined sewers, separate sewers, urban stormwater quality management, drainage and flood control, groundwater, local/state/federal regulation and risk assessment, to name only some. The Council has done a superb job of emphasizing the need for engineers to collaborate with many other professions (land planners and landscape architects, biologists, geologists, social scientists, economists, etc.) and with elected officials and citizens, and has devoted substantial effort to evaluating social, economic, and institutional aspects of projects. As stated in Chapter 7 herein, "Although all aspects of urban hydrology research continue to be of vital importance to the nation, the need to refine identifications and understandings of interdisciplinary interfaces is still of paramount importance."

Council members have consistently stressed the need for projects that are attractive and that enhance communities, that do not pose safety risks to the public and that are functional and maintainable. Indeed, UWRRC-sponsored publications have played a major role in fostering multi-purpose stormwater management facilities, nationally and internationally.

The breadth of the Council's perspective is demonstrated by the following list of "non-hydrologic aspects" defined in Chapter 8, "Systematic Study and Development of Long Range Urban Water Programs":

- Administration of works
- Economics of planning and operation
- Aesthetics of urban environment and specific works
- Financing of works
- Health and safety of population served
- Legal latitudes and constraints
- Planning and land use, metropolitan-wide
- Political impacts on planning and implementation of works

- Recreational facilities planning and operation
- Sociological interferences between a viable metropolis and development of its water resources

The Council has consistently emphasized the need to look broadly and holistically at urban water resources issues, recognizing that they are closely interrelated and can be evaluated in a systems context. In 1968, Water Resources Engineers, Inc. (WRE) of Walnut Creek, California contracted with ASCE to conduct the following task:

> *Conduct a pre-feasibility study and determine the possible effectiveness, cost, and time requirements for a comprehensive systems engineering analysis of all aspects of urban water.*

A synopsis of this excellent work is provided in Chapter 8 and is included as Figure 1-1, which depicts the urban water resources environment as an intricate network characterized by some relatively specific "locations" for water in the urban setting, and by several varieties of "transfer" elements that connect these locations to one another. These location and transfer "sub-systems" receive water from outside, hold and distribute it throughout the entire system, and then discharge it to a receiving environment or via several possible loss mechanisms.

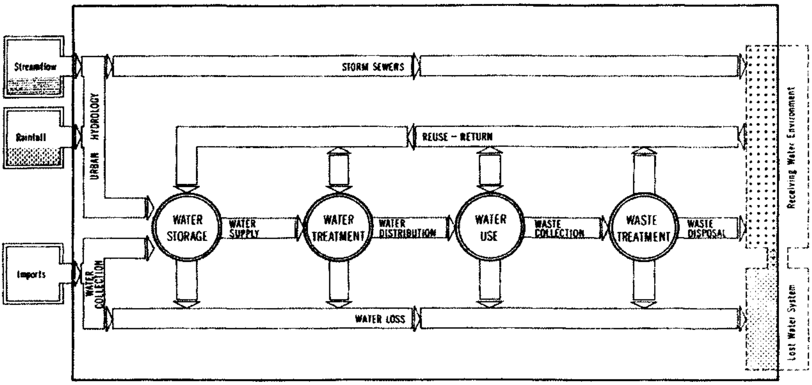


FIGURE 1-1

The major elements in Figure 1-1 are as follows. All of these elements have been addressed by the Council over time:

- *Water Location Subsystems*
    - *Water Storage*
    - *Water Treatment*
    - *Water Use*

    - *Waste Treatment*
- *Water Transfer Subsystems*
    - *Urban Hydrologic Cycle*
    - *Water Collection*
    - *Storm Sewers*
    - *Water Supply*
    - *Water Distribution*
    - *Waste Collection*
    - *Waste Disposal*
    - *Reuse and Return*
    - *Water Loss*
- *Receiving Water Systems*
- *Lost Water Systems*

Chapter 8 notes that most urban water systems have already been planned and designed and are now in operation. Also, most of them have been planned and designed in pieces or by sub-systems and, hence, are likely not operating or being operated optimally. As a consequence, the UWRRC noted that it would be worthy cause for ASCE to determine if the operation, design, and planning of these large and complicated systems could be improved through the application of advanced analysis techniques.

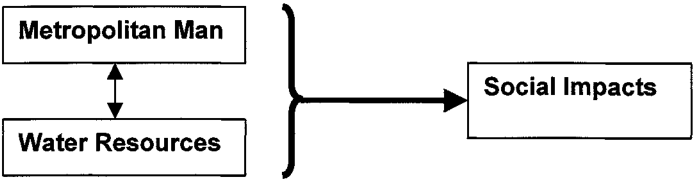
For broad assessment of technical and non-technical issues and needs associated with urban water resources, readers are referred to Chapter 7, *Water and Metropolitan Man an Engineering Foundation Research Conference* held in August 1968 in Andover, New Hampshire and chaired by Stifel W. Jens and D. Earl Jones, Jr. This extraordinary conference addressed the following major subjects, and tied them together coherently and in compelling fashion:

- Communications
- Planning
- Social impacts
- Management of water resources related to metropolitan areas
- Legal and institutional aspects
- Regulation
- Data needs
- Precipitation
- Detention storage and urban areas

- Design problems
- Systems

Many of the issues and needs identified nearly 40-years ago continue to apply today (in 2005).

Much of the work of the Council has focused on the social impacts of urban water resources management. The conceptual framework for this analysis adopted by the Council is indicated in the diagram below:



Analysis of the social impacts of water resources on "metropolitan man" was also summarized in *Water and Metropolitan Man* (see Chapter 7):

1) Who is Metropolitan Man? Various classifications that are important to identifying priority sub-groups and possible technical remedies are:
    a) Income groups;
    b) Age groups;
    c) Occupational categories;
    d) Educational levels;
    e) Ethnic composition;
    f) Inner city vs. suburbs;
    g) Ghetto poverty vs. dispersed poverty; and
    h) National geographical location.
2) What are the relevant components of urban water resources?
    a) Physical features:
        - Water supply;
        - Drainage and flood protection;
        - Sewage; and
        - Natural water geographical location (e.g., pools, fountains, etc.).
    b) Aesthetic features:
        - Water quality;

    - Visual impact; and
    - Audio impact.
  - c) Related institutional and organizational features:
    - Management methods;
    - Water use policies;
    - Weather modification (including inadvertent); and
    - Division of water-related services between public and private domains.
  - d) Reciprocal communications between metropolitan water management and relevant publics:
    - Metropolitan man's participation in the decision-making process; and
    - Broadening communication channels.

3) What are the potential impacts of urban water resources?
   - a) Health
     - Physical; and
     - Mental
   - b) Recreation
     - Impacts on physical and mental health
   - c) Economic impact
     - Land values;
     - Industrial attraction to area;
     - Direct job opportunities on water projects; and
     - Income feedback from health impacts
   - d) Aesthetic impacts
   - e) Negative impacts or penalties incurred or avoided
     - Relocation of residences and other activities;
     - Land-use conflicts (i.e., the opportunity costs of land used in water projects); and
     - Social disturbances avoided

The Council has evaluated the economic implications of urban water resources management, such as the following list of impacts of pricing policy on water use from Chapter 7:

1) Affects price as it attracts or detracts from industrial and urban expansion.
2) Affects quality of the water system and its product.

3) Encourages or discourages reuse, conservation of wastage of water.
4) Affects planning for future and future research.
5) Provides financing for non-water-related public works with consequent diversion of water resource derived funds.
6) Encourages more effective utilization of water by off-peak use through appropriate pricing schedules.
7) Indirectly affects price on fire insurance rates (through tradeoffs).
8) Restricts or encourages water use for amenities, i.e., non-life sustaining uses.
9) Sociological impacts.

Significant work by the Council has focused on detention and retention storage of stormwater in urban areas (at a micro-scale on individual lots, for development tracts and for very large regional storage facilities), including the following uses and objectives of detention facilities:

1) Reduction of flood damage
2) Downstream hydrograph control
3) Water oriented recreation
4) Water supply
5) Water quality improvement
    a) Sediment and debris removal
    b) Low flow augmentation
6) Groundwater recharge
7) Social impact
8) Minimization of storm sewer related facilities cost
9) Reduction of long-term depreciation rate
10) Floodplain encroachment

The Council has also consistently emphasized the importance of preserving natural channels as areas urbanize, and in restoring channels that have been adversely affected by urbanization. In short, protection and enhancement of public health, safety, and welfare has been a major objective of the UWRRC.

4.0 PROCESS OF RE-PUBLISHING GREAT WORKS OF THE COUNCIL

In 2003, the members of the UWRRC collectively decided that re-publishing the great works of the Council would be a fitting tribute to its founders and the many outstanding professionals who have participated in its activities since the Council's inception in 1962. Consequently, a committee was established to pursue this endeavor including:

- Jonathan E. Jones[2], Wright Water Engineers, Inc. (WWE) (Chairman)
- Richard Field, US Environmental Protection Agency
- Donald VanSickle[3], Consultant
- Neil Grigg, Colorado State University
- L. Scott Tucker, former Executive Director of the Urban Drainage and Flood Control District
- Michael Ports[4], Black & Veatch
- Ben Urbonas[5], Urban Drainage and Flood Control District
- Larry Roesner, Colorado State University
- Conrad Keyes, Environmental and Water Resources Institute (EWRI) of ASCE
- Charles Rowney, Consultant
- Stuart Walesh, Consultant

To initiate this effort, Jonathan E. Jones assembled as many of the historic publications of the UWRRC as it was feasible to locate. Current and past Council members were canvassed and asked to send their reports to WWE, which served as the clearinghouse for the effort. Ultimately, approximately 150 documents were located. (Despite good publicity and personal requests for historic documents, it was not feasible to locate all of them). These documents were reviewed for completeness and OCR-scanned by WWE, with assistance from Mr. Richard Field (USEPA). The next step was to prepare either brief summaries of each of the documents, or utilize the original abstracts/summaries; these are provided in Table 1-1. Mr. Jones was assisted by Mssrs. Urbonas and Roesner on this task, as they focused on UWRRC-sponsored conference proceedings from about 1980 to the present. The reproduced documents have received little editing, because they were of excellent quality to begin with, had typically been peer reviewed, and out of respect to the original authors. Substantial time was spent checking graphs, figures, and equations that had been scanned, to minimize error.

Far too many documents were located to enable the re-publication of all of them. Consequently, after receiving guidance from the leadership of EWRI and the UWRRC, Mr. Jones initially selected a broad range of publications to be included in this book, primarily focusing on the early years of the Council, with fewer reports

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[2] Throughout the course of this effort, Mr. Jones received outstanding assistance from Ms. Frankie Lane, Administrative Assistant, and the project was completed by Ms. Mary Simmerman, both of WWE. Mr. Jones and council members sincerely thank both Ms. Lane and Ms. Simmerman for their efforts.

[3] Member, Manuscript Review Committee for EWRI in 2005.

[4] Member, Manuscript Review Committee for EWRI in 2005.

[5] Member, Manuscript Review Committee for EWRI in 2005.

from the late 1970s and early 1980s[6]. He then provided his recommendations on key documents to other members of the committee, and asked for their input. Ultimately an EWRI review committee, comprised of Michael Ports, Ben Urbonas, and Donald Van Sickle, reviewed this manuscript prior to publication.

In many cases, it was not feasible to reproduce the entire document due to space limitations. For example, many of the reports contain appendices, and these were either deleted altogether or only excerpts are provided. In some cases, executive summaries of documents are provided. Some readers may wish to have copies of entire reports (including appendices) that are not provided in their entirety. In this case, the relevant information can be downloaded from the EWRI website, http://www.ewrinstitute.org/.

The chapters in this book are as follows:

- Chapter 1: Introduction, Overview and Summary of Publications
- Chapter 2: Engineering Foundation Research Conference Urban Hydrology Research
- Chapter 3: ASCE Combined Sewer Separation Project Progress
- Chapter 4: Study of Approximate Lengths and Sizes of Combined Sewers in Major Metropolitan Centers
- Chapter 5: Two Practical Guidance Documents On Solids Transport Velocities In Sanitary Sewers
- Chapter 6: Domestic Sewage Flow Criteria for Evaluation of Application of Project Scheme to Actual Combined Sewer Drainage Areas
- Chapter 7: Water and Metropolitan Man, an Engineering Foundation Research Conference
- Chapter 8: Response Characteristics of Urban Water Resource Data Systems
- Chapter 9: Systematic Study and Water Development of Long Range Programs of Urban Water Resources Research
- Chapter 10: The Nature of Changes in Urban Watersheds and Their Importance in the Decades Ahead
- Chapter 11: Some Notes on the Rational Method of Storm Drain Design
- Chapter 12: A Study of the Expenditures for Urban Water Services

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[6] There are many outstanding council-sponsored documents from the early 1980s to the present (see Table 1-1), but these were not included because, 1) readers are more likely to already either have them or have access to them, and 2) EWRI plans to post these on their website, where they can be downloaded.

- Chapter 13: Availability of Rainfall-Runoff Data for Sewered Drainage Catchments
- Chapter 14: Rain Gauge Networks in the Largest Cities
- Chapter 15: Combined Sewer Separation Using Pressure Sewers
- Chapter 16: Non-Metropolitan Dense Rain Gauge Networks
- Chapter 17: Availability of Rainfall-Runoff Data for Partly Sewered Urban Drainage Catchments
- Chapter 18: Systems Analysis for Urban Water Management
- Chapter 19: Prospects for Metropolitan Water Management
- Chapter 20: Feasibility of the Metropolitan Water Intelligence System Concept (Integrated Automatic Operational Control)
- Chapter 21: Metropolitan Industrial Water Use
- Chapter 22: Residential Streets
- Chapter 23: Residential Stormwater Management
- Chapter 24: Computerized Citywide Control of Urban Stormwater
- Chapter 25: Urban Hydrological Modeling and Catchment Research: International Summary
- Chapter 26: Erosion and Sediment Control
- Chapter 27: Research on the Design Storm Concept
- Chapter 28: International Symposium on Urban Hydrology
- Chapter 29: Stormwater Detention Outlet Control Structures

A partial list of UWRRC members and contributors to the UWRRC publications is provided as Table 1-2. Jonathan Jones and other members of the re-publication committee apologize to those past UWRRC members who inadvertently are not included in Table 1-2.

All members of the 2005 UWRRC sincerely hope that readers will enjoy and benefit from the republication of the following great works of the Council.

Table 1-1

Summary of Key Publications of the American Society of Civil Engineers Urban Water Resources Research Council From 1965 to 2001

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
Engineering Foundation Research Conference—*Urban Hydrology Research*	August-65		The first Engineering Foundation Research Conference on Urban Hydrology Research was held at Proctor Academy, Andover, New Hampshire, from August 9 to 13, 1965. Since urban hydrology is more than a narrow, technical, engineering subject, the conference included representation of a variety of disciplines. The first day dealt with the broad social, political, financial, legal, and overall planning aspects. The second and third days were devoted to the hydrology and hydraulics of storm drainage, and the fourth day returned to the broader considerations from the federal, state, and local viewpoints.
ASCE Urban Water Resources Research Program TM-2—*Sewage Flow Variations in Individual Homes*	February-66	M.B. McPherson	Very little data are available on flow rates for sewage emanating from individual homes, particularly for short time-intervals on the order of one minute. As an expedient, the data used for this memorandum are all water demand rate. The general concept for separation of combined sewerage systems on which the projet is based involves discharging comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing laid in existing building connections, and thence into new pressure conduits suspended in existing street sewers. Winter water demands are assumed to represent sewage flows in the absence of sewage flow data. Two samples of household water demands, recorded at one minute intervals, are used: data from six homes in Maryland for two weeks, and data from two homes in Louisville for four weeks. For each home maximum and minimum 24-hour and 60-, 15-, and 4-minute demands for each day are given. Frequency distributions of 24-hour and 50-minute flows for each sample are compared with each other and with distributions of total flows from groups of three houses and six houses. Based on routing of peak flows from nearly 500 home days of data through various pump storage combinations, a pump capacity of 10 GPM and a total storage capacity of 30 gallons are considered realistic, but not necessarily conservative, estimates for sizing of household sewage storage-pump combinations for ASCE Project applications. Pressure discharge tubing to handle expected flows and head losses for pressure building services would be ¾ to 1¼ inch I.D. Based on studies of peak flows from a six house combination it is tentatively concluded that design flows for gravity street sewers may be suitable for pressure sewer designs in preliminary feasibility studies of the ASCE Project scheme.
ASCE Combined Sewer Separation Project TM-1—*Outline Description of ASCE Project on Separation of Sanitary Sewage from Combined Systems of Sewerage*	February-66		It is estimated that from 3-5 percent of the annual flow of raw sewage from combined systems of sewerage overflows into lakes, streams, and tidal estuaries and as much as 95 percent of the sewage in combined systems at the time of a storm is emptied into these receiving waters during the progress of the storm. Separation of sanitary sewage from stormwater is considered an important need in the campaign for cleaner waters. A third of the people in the U.S. are estimated to live in communities served by

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			combined sewers. These same people comprise half of the nation's sewered population. It is feared that increasing ubranization will result in a greater rate of increase in organic loads discharged into water courses than can be handled by existing or presently planned corrective efforts. The need for separation of sanitary sewage from combined systems of sewerage and the national scope of the problem of separation is summarized in the appendix to this memorandum.
ASCE Combined Sewer Separation Project TM-2—*Sewage Flow Variations in Individual Homes*	February-67	L.S. Tucker	The nationwide cost for traditional, complete separation of existing combined sewer systems would be very high and diversion of public funds for this purpose would be in competition with projected investment in other public works and services. The basic objectives of the project are to determine the physical feasibility and limitations of the project scheme, or modifications thereto, and to arrive at measures of cost for comparison with the traditional method of separation, for evaluation of investment feasibility.
Major Appliance and Hotpoint Division Appliance Park—Sampling and Analysis of Wastewater from Individual Homes—(Task 2)	March-67	R.P. Farrell, J.S. Anderson, J.L. Setser General Electric Company	The results of the initial phase of operation of two household wastewater observation stations are described. During a three month period, household wastewater was sampled for analysis, wastewater flow rates were measured, and the behavior of components when handling wastewater under actual conditions of use was observed. Each station included a garbage grinder to macerate incoming sewage solids, a float well and level recorder, a Moyno pump and pump operation time recorder, a check valve on the pump discharge line, and fifty feet of clear plastic discharge tubing. An extensive program of sampling and analysis was carried out to completely characterize the wastewater from each home. Particulate matter in the wastewater was analyzed over an intensive seven day period to determine its exact nature in terms of particle size, density and microscopic appearance. Water consumption was measured at one gallon intervals at each house and telemetered to recording equipment at the laboratory. A set of fixture tests, during which fixtures were discharged singly and in combinations in preplanned sequences, was run at each station to obtain information on water and sewage flow patterns for fixtures. Daily peak period water demands for 37 days at one station and 48 days at the other are tabulated and frequency curves are shown. The operating history of each station is reviewed.
ASCE Combined Sewer Separation Project TM-3—*Experience with Grinding and Pumping of Sewage from Buildings*	May-67	D.H. Waller	The general concept for separation of combined sewerage systems on which the Project is based involves discharging comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing laid in existing building connections; and thence through new pressure conduits suspended in existing street sewers. Thirty-six comminutor installations in buildings are described. A description is given of the type of comminutor used in most of these installations, and of typical installation arrangements and recommended maintenance procedures. A curve displays the relative effect of a comminutor and a garbage grinder on sewage particle sizes. A typical garbage grinder is described and use of grinders for toilet wastes is reviewed. Two wet process building-waste pulping systems are described. Two machines that combine the functions of grinding and pumping are described and the pumping capability of the

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			garbage grinder is dicussed. Practice and experiment in the use of pumps, piping and backflow valves for sewage in buildings is reviewed. An appendix describes the Liljendahl vacuum sewerage system and discusses its relevancy to the ASCE Project scheme.
ASCE Combined Sewer Separation Project TM-4—*Study of Approximate Lengths and Sizes of Combined Sewers in Major Metropolitan Centers*	May-67	Dasel E. Hallmark and John G. Hendrickson, Jr.	The general concept for separation of combined sewerage systems on which the Project is based involves discharging comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing laid in existing building connections; and thence through new pressure conduits suspended in existing street sewers. In one version of the project scheme, the small pressure tubing would be laid in existing building connections and would then be connected to new pressure conduits suspended in existing street sewers. Entirely different requirements are involved for the installation of pressure conduits in walk-through and non walk-through sewers. Installations in non walk-through sewers would require at least partially remote construction techniques. Direct access would be available without any necessity for excavation only for sewers of a walk-through size. Eleven cities were contacted to obtain information on lengths and sizes of combined sewers. Based on returns from five cities, a tabulation is given of: mileage and percentage of combined sewers greater than 48 inches and percentage of sewers equal to or less than 24 inches. Sizes are based on vertical dimensions of conduits. An average of 72 percent of the sewers are smaller than 24 inches. Sizes of 54 inches and larger, which are classified as walk-through sewers, account for an average of about 15 percent of the total combined sewer mileage. Results of a storm sewer questionnaire sent out by the American Public Works Association tend to confirm this result for larger population centers. For smaller cities, the percentage is apparently smaller.
ASCE Combined Sewer Separation Project TM-5—*Pressure Tubing Field Investigation*	August-67	L.S. Tucker	The general concept for separation of combined sewerage systems on which the ASCE Project is based involves discharging comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing. TM-5 relates to special field-trial installations of tubing and conduits for the purpose of determining the nature and extent of practical difficulties that might be encountered in passing various tubing through a building's sewer, in suspending or otherwise attaching a pressure conduit in the street sewer and in making tubing-to-conduit connections. Three methods of installing pressure tubing from houses or small buildings, and of connecting the tubing with street pressure conduits, are described and discussed. For most applications installation and connection of pressure tubing and conduit in trenches, by traditional water distribution methods, is recommended. Field trials for two other methods are described. Tubing was pushed through an 86-foot-long 4-inch and 5-inch diameter building lateral, which included three 45° bends, from a specially dug pit at the upstream end into a 4-foot diameter combined sewer. The forward end of the tubing was guided by a special leader device. ¾, 1 and 1¼ inch polyethylene tubing could be pushed. Polybutylene and copper tubes could not be pushed because they buckled or crimped. A Kellems grip and swivel on the end of a rope were used to pull tubing from the combined sewer to the

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			upstream pit. ¾, 1 and 1¼ inch polyethylene and ¾ and 1 inch polybutylene could be pulled. ¾ inch copper tubing could not be pulled because of its stiffness. A connection was made from a ¾ inch polyethylene tube, which had been threaded through the building lateral, to a 2-inch PVC conduit that was temporarily installed in a shallow trench on the building side of the combined sewer. The tubing was brought out of the lateral through a no-hub-wye fitting and connected to the conduit by a corporation cock. The estimated cost of this method would exceed the cost of installing pressure tubing entirely in a new trench between the building and a pressure conduit in a trench dug parallel to the street.
Environmental Engineering Department Central Engineering Laboratories FMC Corporation Report—*Relationship of Sewage Characteristics to Carrying Velocity for Pressure Sewer*	August-67	M.F. Hobbs	Minimum carrying velocities for solidphase matter in smooth plastic 2", 3", 4", 6", and 8" pressure pipes at zero slope were studied for comminuted and uncomminuted raw sewage. The minimum velocity for scouring and the minimum velocity where depositing takes place were essentially the same. Velocities appeared to be independent of the concentration of suspended solids, fixed suspended solids, "sand" concentration, and size distribution of suspended matter and "sand" for the sewages studied. Velocities appeared to be dependent on the size distribution of fixed solids, or more likely the "sand," that acumulated on the bottom of the pipe. Carrying velocities were investigated in an 8" spiral pressure pipe and the results obtained were very erratic. Results obtained in an 8" plain plastic pipe with open channel flow appeared to agree with results found for 8" pressure pipes and with data reported in the literature. Egg shells that passed through a garbage grinder were carried in the sewage in smooth pressure pipes at lower flow rates than required for scouring the bottom sediments. Average velocities found for the various pipe sizes are given.
Conference Preprint 548—*ASCE Combined Sewer Separation Project Progress*	October-67	M.B. McPherson	This paper is a progress summary for the period from the commencement of the project in February 1966 through August 1967. The general concept for separation of combined sewerage systems on which the ASCE project is based involves pumping comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing. The tubing would connect to new pressure conduits located outside the buildings. These new, separate sanitary sewage pressure conduits would then discharge into existing interceptors that woud convey the sanitary sewage to treatment works. Stormwater alone would be carried in what were formerly combined sewers. Dr. Gordon M. Fair of Harvard University is the originator of the concept under investigation.
ASCE Combined Sewer Separation Project TM-6—*Hydraulics of a Pressurized Sewerage System and Use of Centrifugal Pump*	November-67	L.S. Tucker	The general concept for separation of combined sewerage systems on which the ASCE Project is based involves discharging comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing. TM-6 summarizes the general hydraulic requirements for both street sewers and pumping from larger buildings. It provides feasibility studies of hypothetical pressure sewerage systems in the following combined sewer drainage areas: A 330-acre residential sector in San Francisco, 160-acre residential area in Milwaukee and 60-acre commercial area in Boston. In order to keep each pressure sewer district independent of pressures in the interceptor, a regulating valve is

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			required where a trunk pressure sewer joins an interceptor. The position has been taken that pressure sewers upstream of the interceptor should be kept under positive pressure at all stages of flow. Air relief valves would still be required Curb pressures would vary from zero to 30 psi. Hydraulic gradients are illustrated for high and low flows and the effects of pressure control devices at the interceptor are illustrated and discussed. For some flat drainage areas sewage pumping would be necessary; a pressure control assembly would be needed immediately upstream, and a surge control valve would be used immediately downstream, of a lift station. For steep drainage areas, pressure control assemblies would be needed to limit maximum pressures. Centrifugal pump characteristics are discussed and information on 32 sewage and solids handling pumps is tablulated. Sewage pump characteristics are such that maximum reasonable limits on discharge rates would be greatly exceeded if variations in total dynamic head were allowed to equal curb pressure variations that are expected in some parts of a pressure sewerage system. Ordinary use of centrifugal pumps in these cases would be precluded. A possible modification of building pumping systems, by throttling pump discharge with a valve controlled to maintain a constant discharge pressure, is discussed. The use of variable speed drivers is discussed; it is doubtful that their use would be justified.
ASCE Combined Sewer Separation Project TM-7—*Minimum Transport Velocity for Pressurized Sanitary Sewers*	November-67	M.B. McPherson, L.S. Tucker and M.F. Hobbs	The general concept for separation of combined sewerage systems on which the ASCE Project is based involves discharging comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing. The required size of a street pressure conduit is governed by two considerations: 1) the magnitude of the lowest flow rate at which transport of all solids is desired, and 2) the minimum mean flow velocity needed to move all solids. The major part of this memorandum is a summarized interpretation which provides general criteria applicable to the design of pressure conduits and which sets forth tentative findings for open channel flow. The memorandum also defines design criteria used by project staff for the study of three hypothetical pressurized sanitary sewer systems. In sewage transport velocity tests, in 2-inch through 8-inch clear plastic pipes, sewage sand content was determined. Results are expressed in terms of an empirical relationship developed by Larsen, and are correlated with results of work by Craven and Ambrose. Scouring in pressure pipes requires somewhat higher velocities than depositing; no obvious difference was evident for four open channel tests. Neither comminution nor grinding particularly changed sewage sand size distribution. Size distributions of sewage sands and sands used by Ambrose and Craven are compared. Within the range of experimental data, the effect of particle size has little effect on transport requirements. A reasonably precise prediction of minimum scouring velocities is dependent upon expected total sand concentration. A linear multiple regression analysis of the sewage flow data indicates that the following variables, in order of decreasing importance, affect scouring velocities: diameter, sand concentration, sand specific gravity, median sand sieve size, and ratio of sieve size passing 90 percent to median sieve size. For preliminary hydraulic design of hypothetical pressure sewerage systems for evaluation of physical feasibility and cost attractiveness,

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			the expression $V = \sqrt{}D_{/2}$ (V is ft/sec, D is internal conduit diameter in inches) has been chosen. This relation satisfies the bulk of the data and provides a degree of conservatism consistent with the level of precision of design flow rates.
ASCE Combined Sewer Separation Project TM-8—*Domestic Sewage Flow Criteria for Evaluation of Application of Project Scheme to Actual Combined Sewer Drainage Areas*	November-67	M.B. McPherson	The general concept for separation of combined sewerage systems on which the ASCE Project is based involves discharging comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing. The basic objectives of the project are to determine the physical feasibility and limitations of the project's scheme, or modifications thereto, and to arrive at measures of cost for comparison with the traditional method of separation, for evaluation of investment feasibility. The nationwide cost for traditional, complete separation of existing combined sewer systems would be very high and diversion of public funds for this purpose would be in competition with projected investment in other public works and services. Sizing of pressure sewers must be based on attainment of minimum necessary scouring velocities at an acceptable level of frequency, such as for at least one hour each day of the year. Deposition of solids part of the time must be accepted to preclude ultra-conservative design. At the other extreme, peak sewage rates will give rise to maximum hydraulic gradients, and these in turn affect pressure requirements within the system, for booster pumping and/or pressure control. Residential sewage flow criteria are developed for use in design of pressurized sanitary sewers that form part of the ASCE Project scheme. In a typical combined sewer area, data on annual domestic water demands is the most that can be expected to be available. On the basis of a study of winter water demand data, it is concluded that refinements in projections to account for differences in demand related to numbers of dwelling occupants would be unrealistic, that estimates of mean annual domestic demand based on data from other communities would contain considerable inherent errors, and that projection of observed demands for the drainage area under study to the end of the design period is a preferred basis of design. Winter water demand data for groups of houses of varying sizes is presented in the form of ratios—to the annual average demand rates—of rates on the minimum day, maximum day, maximum hour, and maximum hour on the minimum recorded day. Data for California and the northeastern United States are presented separately, and are plotted to show how ratios to the annual average are related to numbers of dwelling units. For each region design curves are selected to represent the variation, with numbers of dwelling units, of flows for the minimum 24 hours, for the peak hour of the minimum day, and for the maximum peak hour of any day—all presented as ratios to the annual average rate.
FWPCA Contract No. 14-12-29, Tasks 7 and 9—*Report to ASCE Combined Sewer Separation Project on FWPCA Contract No. 14-12-29, Tasks 7 and 9*	December-67	Robert N. Bowen, John G. Havens	Assistance was provided in connection with special field trial installations of flexible tubing in building sewers. Materials were proposed for pushing or pulling through a building sewer and a methodology and necessary attachments and tools were recommended. Experience gained from the field trials—which are considered to have been very successful—is reviewed and used as a basis for recommendations about pressure sewerage system design and operation. Polyethylene and polybutylene tubing

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			were recommended for use inside building sewers; copper tubing could be used in open trenches. Scratching or scoring was not a serious problem in fishing tubing through a building sewer. The recommended methodology for installation was sound. A saddle type of connection is recommended for connection of pressure tubing to street pressure conduits. Experience with plowing of pressure pipe is reviewed; use of this method where paving or buried utilities are not problems should be investigated. Reference is made to standard practice for trench installations, street crossings and thrust blocking. Two methods of cleaning house pressure tubing are proposed. Six possible layouts of pressure conduits are discussed in terms of operation and maintenance of each. All six arrangements provide for routine rerouting of flow by use of a dual configuration for all conduits.
ASCE Combined Sewer Separation Project TM-9—*Peak Flows of Sewage from Individual Houses*	January-68	D.H. Waller	The general concept for separation of combined sewerage systems on which the ASCE Project is based involves discharging comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing. The specific purposes of this memorandum are to present recent data on sewage flows at two household observation stations, to use this data to estimate upper limits on pump and storage capacities for individual houses, and to examine the relationship between peak rates of sewage flow and corresponding water demand rates. The memo: 1. Reviews literature relating to flows of sewage from individual houses. 2. Describes collection and analysis of sewage flow data and related water demand data. 3. Describes the results of fixture discharge tests that were conducted at two household stations. 4. Uses the information above to develop hydrographs from maximum periods of sewage discharge. 5. Presents information on peak sewage flows for a 14-day period at a representative house and examines the relationship between peak sewage flows and related peak water demands. Sewage flows and corresponding water demands were measured at two household observation stations. Water and wastewater flows from individual fixtues and appliances were also measured. The resulting data are presented in detail and are used to estimate upper limits on pump and storage capacities for individual houses and to examine the relationship between peak rates of sewage flow and corresponding water demand rates. Literature relating to flows of sewage from individual houses is reviewed. For individual fixtures, combinations of rate, duration and frequency of discharge that will produce maximum hydraulic loading conditions are selected. The pattern of discharge of two fixtures discharged simultaneously is not significantly different from the pattern obtained by adding discharges of individual fixtures. Single fixture hydrographs are combined to produce synthetic hydrographs of peak period sewage discharge from houses. The hydrographs are used to determine pump-storage combinations for use in individual houses. Mass curves and frequency distributions, for peak sewage flows and parallel water demands at one house for a 14-day period, are presented. Frequency distributions of water demand data are compared with distributions of four other samples of data

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			and are found to be consistent. Relationships between individual house sewage flows and causative water demands, the effects of numbers of household occupants on mean demand, and the relationship of peak demands to mean demands and to numbers of fixtures and appliances, are examined.
ASCE Combined Sewer Separation Project TM-10—*An Examination of the Benefits and Disadvantages with Respect to the Disposal of Solid Wastes*	February-68	D.H. Waller	The general concept on which the ASCE Combined Sewer Separation Project is based involves the discharge of comminuted or ground sewage from buildings and/or building complexes, via relatively small pressure tubing, into new pressure sanitary sewers. The new pressure sanitary sewers would discharge into existing interceptors that would convey the sanitary sewage to treatment works; stormwater would be conveyed in what were formerly combined sewers. The ASCE Project concept envisions the installation of new systems of pressure sanitary sewers as a method of diverting sanitary sewage from existing combined sewers. The possibility of obtaining synergistic benefits by adapting the Project scheme to the disposal of solid wastes was recognized early in the Project program. The purpose of this memorandum is to examine considerations that are important to an evaluation of the feasibility and benefits of adapting the Project scheme to solid wastes disposal. Most of these considerations apply to any system of sanitary sewers. Considerations common to both open channel and pressure sanitary sewers are discussed first, followed by an examination of considerations peculiar to adaptation of the Project scheme to the disposal of solid wastes. Important considerations in an evaluation of the feasibility and benefits of adapting any sewerage system to solid wastes disposal are discussed first: the extra solids load that community refuse could add to a sewage disposal system; velocities required to move solid wastes and the effect of flow variations on sewer velocities; solid wastes separation practices and attitudes toward separation of household refuse; the need for grinding, and considerations involved in the development of a household refuse grinding device; the effects of solid wastes on sewage treatment processes and costs and benefits involved in evaluation of alternative systems for disposal of sewage and solid wastes. Considerations that are peculiar to the ASCE Project scheme are: the possibility of adapting building sewage grinder-storage-pump units to grinding and discharge of solid wastes; the need to discharge solid wastes into the system under pressure; reduced clearances in the smaller pipes of a pressure system; reduced water requirements, when compared with an open-channel system, to obtain the same velocity by flushing, and the possibility of obtaining higher velocities within the limitations of design capacity; and the possibility of greater solids deposition at low flows. Appendices provide information on: composition and characteristics of solid wastes; pertinent solid wastes research and development; results of research on transport and treatment of solid wastes in sewage disposal systems; and four systems of building solid wastes disposal.
ASCE Combined Sewer Separation Project TM-11—*Control Techniques for Pressurized*	March-68	James R. Daneker & William H. Frazel	The general concept for separation of combined sewerage systems on which the ASCE Project is based involves discharging comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing. The tubing would

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
Sewerage Systems			connect to new transmission conduits located outside the buildings. The new separate sanitary sewage street pressure conduits would then discharge into existing interceptors that would convey the sanitary sewage to treatment works. Stormwater alone would be carried in what were formerly combined sewers. Instrumentation and control of a pressurized sewerage system is well within current technology, and special designs are foreseen that can approach zero maintenance. A rubber-seated butterfly valve is recommended as the final control element. A diaphram pressure element would avoid problems due to sewage solids. A venturi type flow element or a magnetic flow meter could be used, and where accurate measurement is not a prime requirement, it is likely that a simplified version of a mass flow meter could be worked out. For control of pressure in response to flow changes, a transducer will be required to generate some characterized signal that will be the set point for a pressure controller. The transducer should incorporate a cam that can be cut in the field. System No. 1 maintains a fixed pressure upstream of the control element by modulation of the valve to correct or reduce any deviation of measured pressure from a selected set point. System No. 2 modulates the valve to maintain a specific upstream pressure corresponding to every rate of flow measured at the flow element. System No. 3 would control the start-stop sequence of a booster or lift station centrifugal pump to permit starting without surge and maintain a constant discharge pressure. For a booster station this system would vary pump speed in response to suction pressure. For nearly fool-proof fail-safe control an all hydraulic control system is recommended in preference to pneumatic, hydro-pneumatic, or electronic systems.
ASCE Combined Sewer Separation Project TM-3A—*Experience with Grinding and Pumping of Sewage from Buildings (Concluded)*	March-68	D.H. Waller	The basic objectives of the Project are to determine the physical feasibility and limitations of the Project scheme or modifications thereto, and to arrive at measures of cost for comparison with the traditional method of separation for evaluation of investment feasibility. Results of monitoring the operation of 36 comminutor installations that serve individual buildings are reported. The monitoring program, which covered periods of up to 16 months, is described. Descriptions and prior operating histories of the installations are included. Twenty-four of the installations include sewage pumps following the comminutors. Discharge mains, three to six inches in diameter, vary from less than 50 feet to more than 6,500 feet in length. Frequency of attention and maintenance is recorded and compared with manufacturers' recommendations. Twenty-five of the machines were inspected at least five times each week. Eighteen machines did not jam during the record period; average frequency of jamming for all machines was about once in six months. There appears to be no significant relationship between frequency of jamming and frequency of cleaning. Replacement or sharpening of cutting elements is more likely to be carried out as a result of damage to parts than a specific effort to maintain sharp cutting edges. Cutting elements were replaced in three machines. Rag fibers, which after passing through a comminutor tend to "rope" in a turbulent environment, and thin pieces of rubber, that are not shredded by the machine, are potential sources of trouble where projections are available where they might accumulate. An appendix contains a summary description of a system, developed at Pennsylvania State University, for

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			conservation of water in residences by recycling.
Summary Report—*Metropolitan Water Resources Research Agenda Committee Meeting*	April-68		The Office of Water Resources Research has been asked by the Federal Council for Science and Technology—Committee on Water Resources Research to develop a recommended national program on metropolitan water resources research. In response to that assignment, an Agenda Committee was organized to assist in defining the scope of a program of research on metropolitan water problems, and in formulating the approach to be followed. The committee met on April 17 and 18, 1968, at the Shoreham Hotel in Washington, DC. The job of the Agenda Committee was to outline the scope and character of the research effort. The task of specifying the research needed will be undertaken subsequently by a series of technical panels working under the guidance and direction of a Steering Committee. The objective is to have ready by November 1968, a report containing the recommendations for research that will be necessary to cope with the water resource problems of metropolitan areas. That final report will also include estimates of the funding, scheduling, and manpower requirements to accomplish the proposed research. This report summarizes the presentation and discussion of the participants at the initial Agenda Committee meeting. It also presents the Committee's judgments about the action to be taken to develop the proposed research program by November 1968.
General Electric—*Long-Term Operation of Wastewater Observation Stations (Task 2)*	April-68	R.P. Farrell	In the terminal phase of operation of two household wastewater observation stations, the stations were operated for seven months during which the principal object was collection of usage experience. Each station contained a grinder to macerate incoming wastewater solids, a wet-well, a Moyno pump and pump operation time recorder, a discharge ball-check valve, and fifty feet of clear PVC discharge tubing. The two garbage grinders were never a source of difficulty. The principal problem with pumps was wear of non-metallic rotors. The ¾ inch discharge tubing in the last month of operation of one station is attributed to a thick coating of anaerobic slime on the interior walls, which is attributed to low inflow rates, small occupancy, and extended periods of disuse. Tubing at the other station was essentially clean except for intermittent discolorations that were attributed to migration of the plasticizer out of the tubing. Operating problems that occurred only once or twice are listed and explained. Results of fixture flow tests, and information on overflows from the station wet-wells, was obtained to supplement results of earlier studies.
ASCE Combined Sewer Separation Project TM-12—Non-Mechanical Considerations Involved in Implementing Pressurized Sewerage Systems	May-68	D.H. Waller	The general concept on which the ASCE Combined Sewer Separation Project is based involves the discharge of comminuted or ground sewage from buildings and/or building complexes, via relatively small pressure tubing, into new pressure sanitary sewers. The new pressure sanitary sewers would discharge into existing interceptors that would convey the sanitary sewage to treatment works; stormwater would be conveyed in what were formerly combined sewers. The basic objectives of the Project are to determine the physical feasibility and limitations of the Project scheme or modifications thereto, and to arrive at measures of cost for comparison with the traditional method of separation for evaluation of investment feasibility.

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			The proposal, implicit in the ASCE Project concept, to install a grinder-storage-pump unit in every building, raises questions regarding: allocation of costs associated with the units; responsibility for the results of malfunction of the units; arrangements for service of the units; and willingness of owners to accept the units in their buildings. Twenty-five householders in Radcliff, Kentucky, whose houses are served by sewage pumping units, were interviewed to obtain opinions about features of the units that appeared to represent potential sources of nuisance, inconvenience, or other liabilities. Also interviewed were the superintendent of the utility operating the Radcliff sewerage system, owners of five houses in Louisville, Kentucky, at which sewage sampling stations were located, and three consulting engineering firms who have considered schemes involving the installation of sewage pumping equipment on private properties. Opinions and practices reported reflect the view that sewage pumping equipment placed on private property as a part of a public project should be purchased, installed, and serviced at public expense. Even if they were relieved of the direct costs, indirect costs, real and intangible, might discourage property owners from accepting the units. Acceptance of the units could be increased by publicly operated service organizations, community acceptance of responsibility for the results of flooding due to unit malfunction, and reduction of sewer service charges to offset costs associated with the units. The Project scheme was conceived by Dr. Gordon M. Fair of Harvard University.
ASCE Combined Sewer Separation Project TM-13—*Special Requirements for a Full Scale Field Demonstration of the ASCE Combined Sewer Separation Project Scheme*	June-68	D.H. Waller	The general concept on which the ASCE Combined Sewer Separation Project is based involves the discharge of comminuted or ground sewage from buildings and/or building complexes, via relatively small pressure tubing, into new pressure sanitary sewers. The new pressure sanitary sewers would discharge into existing interceptors that would convey the sanitary sewage to treatment works; stormwater would be conveyed in what were formerly combined sewers. This memorandum is essentially an annotated checklist of matters that should be taken into account in planning a field demonstration of the overall project scheme, described in the paragraph above. Its purpose is to delineate problems that might be encountered in planning, constructing and operating a pressure sewerage system, to detail unknowns and effects currently subject to question, and to suggest minimum required or desirable field measurements, observations and sampling for an adequate demonstration of the scheme. Some parts of this memorandum are illustrated by reference to a hypothetical installation in a 160 acre combined sewer drainage area, presently containing about 500 buildings, in Milwaukee, WI. To provide background information, an abbreviated study has been made of problems and costs of combined sewer separation on private property in the ten largest cities that have combined sewerage systems.
Report on the Second Conference on Urban Water Resources Research—*Water and Metropolitan Man*	August-68	William Whipple, Jr., et al.	Although all aspects of urban hydrology research continue to be of vital importance to the nation, the need to refine identifications and understandings of inter-disciplinary interfaces is still of paramount importance. Such refinement is fundamental to effective implementation of systematic (systems) approaches to

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			metropolitan water resources problems. This conference, therefore, had as its objective the fostering of an awareness of the metropolitan establishments of the problems by involving sociologists, political scientists, administrators, planners, engineers, etc. The conference examined the role of water in the metropolitan establishment and explored the optimum involvement of the various disciplines and pursuits. It is felt that this Conference record indicates the high interest of the urban water engineer in the social problems and impacts of metropolitan water resources and a determination on his part to welcome the necessary collaboration of the sociologist, economist, planner, administrator, lawyer, political scientist, the systems designer, and the Federal Urban Research official in the clearly necessary team approach required to most satisfactorily solve urban water resources problems. In turn the non-engineering participants in the Conference clearly evidenced their high interest in and desire to participate in the multi-disciplinary approach to metropolitan water problems.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-1—*Northwood Gaging Installation, Baltimore - Instrumentation and Data*	August-68	L.S. Tucker	This technical memorandum is a detailed report on a small urban drainage area called Northwood located in Baltimore, Maryland. Northwood was one of several urban drainage areas instrumented by the Storm Drainage Research Project, Johns Hopkins University, under the direction of Dr. J.C. Geyer, Head, Department of Environmental Engineering Science. This report was made possible through the cooperation of Dr. Geyer. Substantial assistance was given to the writer by Mr. D. Horn, last Project Engineer of the former Storm Drainage Research Project.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-2—*Oakdale Gaging Installation, Chicago- Instrumentation and Data*	August-68	L.S. Tucker	This technical memorandum is a detailed report on a small urban drainage area located in Chicago, Illinois. The drainage area was instrumented by the Chicago Department of Public Works, Bureau of Engineering, under the direction of Mr. C.J. Keifer, Chief Water and Sewer Design Engineer for the Bureau of Engineering. This report was made possible through the cooperation of Mr. Keifer. Substantial assistance was given to the writer by Mr. D. Westfall of Mr. Keifer's staff.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-3—*Response Characteristics of Urban Water Resource Data Systems*	August-68	J.C. Schaake, Jr.	This report is concerned with the collection of data. The need for these data are apparent to many who have worked with urban water resources, but different needs seem to exist for different reasons. This creates a serious problem because the financial resources to collect these data are limited.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-4—*A Critical Review of Methods of Measuring Discharge Within a Sewer Pipe*	September-68	H.G. Wenzel, Jr.	This report consists of a review and evaluation of existing devices and methods which might be used for discharge measurements in sewers, keeping in mind the data requirements and physical limitations imposed by their use in an urban pilot area. In addition, rating curves for a suggested critical flow device are presented and recommendations are made for future research which, in the author's opinion, is required.
ASCE Combined	September-68	ASCE Project	The research upon which this report is based was

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
Sewer Separation Project—*Report on Pressure Sewerage System, Summer Street Separation Study Area, Boston MA*		Staff and Camp, Dresser & McKee	performed pursuant to Contract No. 14-12-29, with the Federal Water Pollution Control Administration, Department of the Interior. As part of this contract, the ASCE is studying methods of separating combined sewerage systems in three major cities of the United States. This report is the result of a study of the 50-acre Summer Street Separation Study Area of Boston, MA. The main objective of this report is to evaluate and compare the conventional gravity separation method (the construction of a new gravity sanitary sewerage system) and pressure methods (the construction of new pressure sanitary sewerage systems consisting of pumping facilities and force mains).
Report prepared for ASCE by Brown and Caldwell Consulting Engineers San Francisco, CA—*Separation of Combined Wastewater and Storm Drainage Systems—San Francisco Study Area*	September-68	Brown and Caldwell Consulting Engineers	This report has been prepared for the ASCE. The research upon which this report is based was performed pursuant to Contract No. 14-12-29 with the Federal Water Pollution Control Administration, United States Department of the Interior. A detailed description of the research study has been prepared by ASCE and published as Technical Memorandum No. 1. Briefly stated, the objectives of the ASCE study are (1) to determine the feasibility of grinding or comminuting sanitary sewage from individual buildings or groups of buildings and conveying it to existing interceptors through a system of pressure conduits, and (2) to compare and evaluate costs and other factors for the pressure system and those for more traditional methods of separation. Work under item (1) also includes development of equipment and construction techniques applicable to the pressure system.
Report—*Systematic Study and Development of Long-Range Programs of Urban Water Resources Research*	September-68	ASCE	ASCE is assisting in outlining, developing and initiating a coordinated national program of urban water resources research. Deliberate and systematic study of urban water problems on a comprehensive basis had long been neglected. The objective of this research is to provide guidelines for initiating and expanding a program of long-range studies in urban water problems. This report covers the first year of work. Urban water resources planning and supporting research are at levels well below those for river basins and large regional complexes. Deliberate and systematic study of urban water problems from an overall point of view had long been neglected. This report covers the first year accomplishments of an ASCE project, supported by OWRR, to assist in outlining, developing and initiating a coordinated national program of urban water resources research. The project will also have significant international benefits.
Report—*Pressure Sewerage System—Summer Street Separation Study Area*	September-68	ASCE Project Staff and Camp, Dresser & McKee Consulting Engineers, Boston, MA	The research upon which this report is based was performed pursuant to Contract No. 14-12-29, with the Federal Water Pollution Control Administration, Department of the Interior. As part of this contract, the ASCE is studying methods of separating combined sewerage systems in three major cities of the United States. This report is a result of a study of the 50-acre Summer Street Separation Study Area of Boston, MA. The main objective of this report is to evaluate and compare the conventional gravity separation method (the construction of a new gravity sanitary sewerage system) and pressure methods (the construction of new pressure sanitary sewerage systems consisting of pumping facilities and force mains). CDM, in accordance with its contract with the ASCE, has conducted engineering investigations in two phases as follows:

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			1. Phase I—A study of revisions necessary on a typical building to in-house plumbing for separation of stormwater and sanitary sewage both for gravity and pressure separation and including a description of the physical problems involved and construction cost estimates for these revisions. 2. Phase II—A review of the layouts for the system of pressure separation ofthe entire study area prepared by the Project Staff and the preparation of construction cost estimates for the systems.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-5—*The Nature of Changes in Urban Watersheds and Their Importance in the Decades Ahead*	December-68	M.B. McPherson	The basic purpose of the ASCE UWRRC is to help establish coordinated long-range research in urban water resources on a national scale. The program currently consists of two major projects: "Research and Analysis of National Basic Information Needs in Urban Hydrology," sponsored by the Geological Survey; and a "Systematic Study and Development ofLong-Range Programs of Urban Water Resources Research," sponsored by the Office of Water Resources Research.
Report on Milwaukee Study Area—*ASCE Combined Sewer Separation Project*	December-68	ASCE Project Staff and Greeley and Hansen Consulting Engineers, Chicago, IL	This report has been prepared for the ASCE. The research upon which this report is based was perfromed pursuant to Contract No. 14-12-29 with the Federal Water Pollution Control Administration, Department of the Interior. This report is part of an overall research study being conducted by the ASCE to determine the feasibility of separating combined sewerage systems by using a system of pressure conduits to convey sanitary sewage from individual structures to an existing interceptor. A study area in Milwaukee, WI has been picked by the ASCE project staff. The report includes a detailed description of plumbing changes required to separate sanitary wastes from roof drains, and the work required to install a grinder-storage-pump unit in each building capable of discharging comminuted sewage to a pressure collection system located in the public right-of-way. Considerable effort was made in determining the plumbing changes required to accomplish separation of wastes within buildings, as little work has been done in this part of combined sewage separation. Two alternative layouts of a pressure sewer system have been submitted by L.S. Tucker, Deputy Project Director of the ASCE project staff. A cost estimate of the pressure layouts has been made and compared to the cost of accomplishing in-house separation and area collection of wastes by the conventional gravity sewer system.
Report—General Electirc Research and Development Center—*Advanced Development of Household Pump-Storage-Grinder Unit*	December-68	R.P. Farrell	In a modern metropolitan sewerage system, sanitary wastewater is collected in separate sanitary sewers and thereby conveyed to treatment works. Stormwater is collected in separate storm sewers and discharged into watercourses. Most of the older cities in the United States have combined sewerage systems, in whole or in part. Both sanitary sewage and stormwater are carried in combined sewers. During dry-weather conditions the entire flow is sanitary sewage, which can be fully diverted to a treatment plant. With significant precipitation, the capacity of diversion structures is exceeded and a mixture of sanitary sewage and stormwater overflows the

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			diversion barriers and discharges into receiving watercourses. Serious concern has been expressed in recent years over the pollution of waterways caused by overflows from combined sewers, and Congress has appropriated funds for research on alternative methods of reducing this pollution. A late 1967 estimate of the national construction cost for the most direct method, diverting sanitary sewage from individual buildings into new sanitary sewers, is $48 billion.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-6—*Some Notes on the Rational Method of Storm Drain Design*	January-69	M.B. McPherson	The "rational method" is the prevailing technique currently employed in determining the capacity of storm sewers in design. Improved techniques will certainly evolve from projected plans to secure data on rainfall-runoff-quality for a variety of catchment areas and to develop analytical models for process replication. The purpose of this technical memorandum is to review some of the more salient features and attributes of the rational method. It is hoped that this presentation will indicate why the rational method is no longer adequate.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-7—*A Study of the Expenditures for Urban Water Services*	February-69	LeRoy H. Clem	The ASCE has undertaken, for the Office of Water Resources Research, Department of the Interior, a contract to assist in outlining and developing a coordinated national program of urban water resources research. Present replacement value of urban facilities for public water, wastewater, and storm drainage totals $110 billion and is increasing at more than $7 billion per year.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-10—*Sewered Drainage Catchments in Major Cities*	March-69	L.S. Tucker	The USGS Project on "Analysis of National Basic Information Needs in Urban Hydrology" is primarily aimed at improvement in design and management of urban storm drainage with a predominant emphasis on sewered catchments. Data are virtually non-existent that are suitable for the analysis of rainfall runoff-quality relationships, and that have been collected with properly coordinated instrumentation in networks representing a variety of climatic, topographic and land-use conditions. A properly designed information network is essential to secure maximum transferability of analysis results across a given region of the country and from one region to another. TM-10 summarizes the study which defines the size distribution and number of sewered drainage catchments in San Francisco, Washington DC, Milwaukee, Houston and Philadelphia. Tabulation of the sizes of all sewered drainage catchments, maps showing catchment boundaries and supporting discussion are included. The size distribution characteristics of the catchments varied considerably. The distribution of sewered drainage catchment size is unique for each city. There is no evidence at hand to indicate whether each city has distribution characteristics representative of its region or that each major city in the United States would have unique characteristics. The average catchment size varies from 65 acres in Houston to 560 acres in San Francisco. The upper quartile catchment size varies from 46 acres in Houston to 455 acres in San Francisco. The upper limit of drainage catchment size representing one half of the total city area served by sewers varies from 338 acres in Houston to 2,250 acres in Washington DC.
A Study of ASCE Urban Water	March-69	L.S. Tucker	The USGS Project on "Analysis of National Basic Information Needs in Urban Hydrology" is primarily

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
Resources Research Council (UWRRC) - ASCE TM-8—*Availability of Rainfall-Runoff Data for Sewered Drainage Catchments*			aimed at improvement in design and management of urban storm drainage with a predominant emphasis on sewered catchments. Data are virtually non-existent that are suitable for the analysis of rainfall runoff-quality relationships, and that have been collected with properly coordinated instrumentation in networks representing a variety of climatic, topographic and land-use conditions. A properly designed information network is essential to secure maximum transferability of analysis results across a given region of the country and from one region to another. The purpose of TM-8 is to make available to researchers information on available rainfall-runoff data for sewered drainage catchments. The authors hope that the data would meet an immediate need for initial testing of mathematical models developed previously for natural drainage catchments. Thirteen installations for the collection of rainfall and runoff data from completely sewered urban drainage catchments are presented. The study areas included Baltimore, Cincinnati, St. Louis, Chicago, Philadelphia, New York and Washington DC.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-9—*Raingage Networks in the Largest Cities*	March-69	L.S. Tucker	Considerable attention was given to several technical aspects of storm drainage. Major goals are: to arrive at general mathematical descriptions of the rainfall-runoff process combined with a basis for quantifying pollution loadings from storm sewerage systems applicable nationwide; to develop adequate technical knowledge for reliable planning of water quality and quantity exchanges between the several urban water service functions for multi-purpose development; and to facilitate and improve the planning, design and management of drainage works. The objectives of TM-9 are to inventory the extent of rain gauge networks in the largest cities as guidelines for future data collection and to document the history of data available for the information of researchers in urban hydrology. Only networks of automatic recording rain gauges were considered. The study focused on the 20 largest U.S. cities and their surrounding areas; the number of rain gauges evaluated in any given city ranged from 1 to 162. The report comprehensively summarizes the nature of rainfall collection, analysis and interpretation methods in the varioius communities and the significance of the overall findings is discussed.
Report—*An Analysis of National Basic Information Needs in Urban Hydrology*	April-69	ASCE Staff	The *Analysis of National Basic Information Needs in Urban Hydrology* has focused on "data needs," "data devices," and "data networks." Primarily aimed at improvement in design of storm drainage, intensive study has been made of the data requirements for analyzing rainfall-runoff-quality relationships and of suitable data collection instrumentation, with consideration of the types of networks required for the collection of adequate data. Suitable data collected with properly coordinated instrumentation in networks representing a variety of climatic, topographic and land-use conditions, are virtually non-existent. There are very meager amounts of performance data with which existing or proposed storm drainage facilities can be checked or designed. Transfer of data findings between metropolitan regions is a central and primary objective. Urban water data acquisition and use would also be invaluable in facilitating the exchange of various quantities and qualities of water from one service to another, such as using stormwater for water supply. There are important implications in the use of data as part of the intelligence required for comprehensive

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			management and operation of urban water services.
ASCE Combined Sewer Separation Project TM-14—*Routing of Flows in Sanitary Sewerage Systems*	July-69	L.S. Tucker	The general concept on which the ASCE Combined Sewer Separation Project is based involves the discharge of comminuted or ground sewage from buildings and/or building complexes, via relatively small pressure tubing, into new pressure sanitary sewers. The new pressure sanitary sewers would discharge into existing interceptors that would convey the sanitary sewage to treatment works; stormwater would be conveyed in what were formerly combined sewers.
1969 Annual Convention of the Water Pollution Control Association of Pennsylvania—*Instrument Systems for Combined and Storm Sewer Research Data Acquisition*	August-69	L.S. Tucker	The continental United States is endowed with a rather fixed supply of fresh water. About 1,100 billion gallons discharge from its streams per day. Total withdrawals in 1954 amounted to some 300 billion gallons per day, and withdrawals that were later returned carrying some degree of pollution to water courses amounted to about 190 billion gallons per day. It is projected that in year 2000 total withdrawals will be about 4/5 the total stream flow and polluted returns will be about 2/3 the total stream flow. The effects and means of control of combined sewer overflows and stormwater discharges were inventoried by the APWA in 1967 in a project supported by the FWPCA. One of the several conclusions was that "there is a definite need for further research to determine the quantity and quality of overflows and the relative extent and detrimental effect of all sources of pollution" In a report to the Federal Council for Science and Technology the Committee on Pollution of the National Academy of Sciences stated that, "More research is needed to develop understanding of the whole stormwater pollution problem. This research should cover the hydrology, the hydraulics, the treatment, the effects on receiving waters and related factors."
Combined Sewer Separation Using Pressure Sewers—*Feasibility and Development of a New Method for Separating Wastewater from Combined Sewer Systems*	October-69	ASCE	This report is concerned with the separation of community wastewaters, and runoff from rainfall and snowmelt in areas presently served by combined and intercepting sewers. Separation is accomplished by withdrawing the wastewater fraction of flows from existing plumbing systems and passing it through a sequence of added systems components as follows: (1) a storage, grinding and pumping unit within each building; (2) pressure tubing fished from the unit through each existing building sewer into the existing combined sewer; and (3) pressure piping inserted in that sewer and extending to the existing intercepting sewers that carry the wastewaters to treatment and disposal works. Runoff from rainfall and snowmelt, thus unencumbered by wastewaters, is removed from the community through the residual passageways of the one-time combined sewer system, which has thus become a combination of a new pressure conduit system within an old gravity conduit system. The feasibility of this scheme of separation, the selection of available systems components and the development of required new systems components are described in this report on the basis of information drawn from 25 project reports and technical memoranda. The feasibility of storing, grinding and pumping sewage from individual residences has been established; and standard comminuting and pumping equipment will be satisfactory for serving larger buildings. Acceptable types of pressure tubing are available that can be pushed and pulled through existing building drains and sewers.

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			Pressure conduits can be suspended inside combined sewers that can be entered by workmen. There are combined sewer areas that can be separated most effectively by a version of the method investigated, but generally pressure systems will cost more than new gravity systems. New capabilities developed appear to be of potentially greater use for applications other than separation, such as new construction including utility corridors, and introduce viable alternatives for design of wastewater sewerage.
Report on the Third Conference on Urban Water Resources Research—*Urban Water Resources Management*	July-70	John R. Sheaffer, et al.	The first conference on urban water resources research (1965) probed many unknowns in the broad field of urban hydrology. Specifically, its focus was water and all of its relationships to urban man. A second conference (1968) addressed the interrelationships between man and his water resources. The conference reported herein was convened to explore broadly the may facets, dimensions, and problems of urban water resources management. This conference addressed research needs at interfaces with the total community, which remain only partially identified and which constitute generally unquantified "unknowns" that impede development of more rewarding and reliable approaches to urban water resource management organization and administration. Special attention was directed to the relationships of behavorial and institutional phenomena to physical phenomena. Approaches to and methods for stimulating and accelerating adoption of improved management practices were also given particular attention. Considerable effort was devoted to a search for ways to make design and management more responsive to social and ecological needs and demands. There was further consensus that the problems presented in efforts to manage urban water resources were in large measure bound by: (1) the consequences of professional efforts and activities with regard to the environment and the lives and lifestyles of human beings, (2) the need to obtain agreement upon the precise nature of objectives being sought, and (3) economic, institutional (including statutory), social and other constraints upon the adoption and implementation of worthwhile changes. These characteristics of the problem more or less define two major areas of data needs and research activity that must be pursued if greater benefits are to flow from our increasing technological refinement and its application. The engineer's role and the demands upon him are expanding. Today, an engineering design often must be considered unsatisfactory if it only maximizes or optimizes a client's needs or demands, or if it only satisfies some simple criterion such as a benefit/cost ratio. Today, it is growing steadily more necessary, and possibly completely necessary tomorrow, that the engineer and related scientists anticipate the consequences of designs or proposed technological changes upon others—the consequences for people and the environment upon which life depends. Successful adoption and implementation of designs and plans, moreover, are becoming increasingly dependent upon broader public concurrence in establishing and changing goals and objectives.
A Study of ASCE Urban Water Resources Research Council (UWRRC) -	January-70	L.S. Tucker	Considerable attention was given to several technical aspects of storm drainage. Major goals are: to arrive at general mathematical descriptions of the rainfall-runoff process combined with a basis for quantifying pollution

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ASCE TM-11—*Non-Metropolitan Dense Raingage Networks*			loadings from storm sewerage sewer systems applicable nationwide; to develop adequate technical knowledge for reliable planning of water quality and quantity exchanges between the several urban water service functions for multipurpose development; and to facilitate and improve the planning, design and management of drainage works. Considerable potential synergistic benefits could be obtained in comprehensive, multipurpose development of urban water, on a scale of involvement that in the aggregate could be much greater than for river basins. Whereas TM-9 documented the availability of rainfall data from networks in the largest cities, TM-11 indicates that an additional important source of rainfall data for research on metropolitan-scale storms is from dense rain gauge networks located in non-metropolitan areas, which are summarized in TM-11. The authors evaluated 13 dense recording rain gauge networks located in non-metropolitan areas of the U.S. Key information summarized in the study included: 1. Network name. 2. Network location. 3. Organization Operating Network. 4. Equipment (number and kind of recording rain gauges). 5. Chart speed. 6. Approximate size of network. 7. Approximate average distance between gauges. 8. Period of record. 9. Intensity data. 10. Form of reduced data. 11. Interpretation/significance.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-12—*Environmental and Technical Factors for Open Drainage Channels in Milwaukee*	February-70	Ted B. Prawdzik	Urban drainage systems represent large capital investments. The national replacement value of public storm and combined sewers has been estimated at $22 billion and the average annual public construction cost for urban storm drainage has been estimated at $2.5 billion for the next several years by the American Public Works Association. The AWPA also estimates that the national annual loss from urban drainage system flooding in recent years has been $1 billion or more.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-13—*Availability of Rainfall-Runoff Data for Partly Sewered Urban Drainage Catchments*	March-70	L.S. Tucker	The purpose of this technical memorandum is to identify available rainfall-runoff data for partially sewered urban drainage catchments to facilitate model development by researchers. Considerable detailed attention has been given to requirements for mathematical modeling of rainfall-runoff-quality in the first report by ASCE for OWRR. The very limited amount of available rainfall-runoff data for fully sewered catchments has been identified in an earlier technical memorandum. For the purpose of this technical memorandum a "partially" sewered drainage catchment is defined as one containing open channel water-courses together with developed secondary drainage systems consisting of street gutter drainage and/or roadside ditches and underground storm sewers.
Report—*Systems Analysis for Urban Water Management,* Prepared for the	September-70	Water Resources Engineers, Inc., Walnut Creek, CA	Expectations of private corporation management can be instructive by analogy. As industry managers seek to gain greater productivity through more integrated operations and the economies of scale, reliability

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Office of Water Resources Research, U.S. Department of the Interior			standards intensify as they try to insure the success of their enormous financial commitment; use of the computer as a routine tool in corporate decision-making is predicted; and under development is "an early-warning system for top management to see what is going into decisions rather than seeing results on financial reports at a point where it may be too late for correction." It must be clearly understood that the Steering Committee is not advocating the replacement of old techniques. Quite to the contrary, what is recommended is enhancement of management capability through the addition of new techniques which provide a greater insight with respect to the facilities and services for which managers are responsible. The objective is to maximize the value of what these managers already know. Outside specialists can assuredly assist managers in the type of development proposed, but the degree of direct involvement by upper management in the enterprise will determine its eventual success because the entire thrust of the exercise is management-oriented. The Steering Committee is of the opinion that implementation of the next phase of follow-on development proposed should materially assist the metropolitan authorities directly involved in the resolution of physical and financial expansion plans for their systems phased through the next several decades. The advantages of subsequent phases would accrue more to implementation and operation capabilities. The use of a comprehensive systems approach is advocated primarily as an overall means for organizing the technical work and management decision-making processes, and as a framework for quantification techniques (whether the latter be information handling, simulation modeling, linear programming, cost-benefit analyses, or whatever). The adoption of the proposed overview approach certainly does not preclude or minimize the necessity or importance of the multitude of special-purpose techniques in common use—these will be unaffected except where the larger brush suggests modifications or accommodations.
Report—*Prospects for Metropolitan Water Management*	December-70	M.B. McPherson	ASCE is assisting in outlining and developing a coordinated national program of urban water resources research, including the provision of guidelines for long-range studies on urban water problems. The first phase of work was concluded with the publication of two reports. This study of metropolitan water management, including an assessment of related research needs, is a part of the second phase of work which is sponsored exclusively by the Office of Water Resources Research, US Department of the Interior. The objective was to conduct a study of metropolitan water management, taking into account: expectations for the environment of future cities; necessary or desirable new administrative arrangements for future public services management; and the expected impact and interaction of emerging technology, urbanization trends, social goals and related matters.
The June, 1971 Meeting of the Sub-Group on the Influence of Urbanization, of the Working Group on the Influence of Man	April-71	M.B. McPherson	As has been generally true around the world, U.S. institutions have not kept pace with the changing character of population distribution brought about by relentless urbanization, and urban hydrology has accordingly suffered from inadequate attention. In sum, knowledge on the hydrologic effects of urbanization is fragmentary, and at a level inconsistently below its

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on the Hydrological Cycle; United Nations Educational, Scientific and Cultural Organization—*Preliminary Notes on the Hydrologic Effects of Urbanization in the United States*			importance to society. More metropolitan-scale water-balance analyses should be undertaken as a means for improving overall water resources planning and management, and follow-on inventories should be made periodically to document change and to provide a better understanding of the effects of progressive urbanization. Research should be intensified on the fundamental rainfall-runoff-quality processes of urban catchments, for better planning, design and management of drainage systems, but particularly for the purpose of assessing the true generic role and character of pollution from discharges of separate underground storm drains and from combined sewer overflows. More adequate attention should be given to the translation of research findings into forms more usable in practical applications in order to realize greater public benefits therefrom.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-14—*Management Problems in Metropolitan Water Resource Operations*	September-71	M.B. McPherson	A recent ASCE Urban Water Resources Research Program report on metropolitan water management attempted to take into account: expectations for the environment of future cities; necessary or desirable new administrative arrangements for future public services management; and the expected impact and interaction of emerging technology, urbanization trends, social goals and related matters. Management tools have also been a Program concern.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-15—*Feasibility of the Metropolitan Water Intelligence System Concept (Integrated Automatic Operational Control)*	December-71	M.B. McPherson	This report is the last of three with an emphasis on urban water management. The first attempted to take into account: expectations for the environment of future cities; necessary or desirable new administrative arrangements for future public services management; and the expected impact and interaction of emerging technology, urbanization trends, social goals and related matters. The second dealt with operating problems and had as a major purpose the framing of a background for assessment of applicability and need of the intelligence system concept. Findings from these two reports are drawn upon for some of the conclusions reached in Section 3. The present report deals predominantly with the physical capability of existing technology.
Preliminary Notes on a Plan for an Urban Water Institute for Applied Development—*Urban Water Institute*	March-72	M.B. McPherson	Research on urban water resources around the world has received miniscule attention compared with non-urban counterparts. Root causes are the involvement of all levels of government in urban water matters and the balkanized character of local water agencies in almost every metropolis. Urban water management is an activity at the local level. Governments throughout the world exhibit the rural origins of their institutions, and the reality of the fact that many have a majority of their people in urban concentrations of population still has not been taken into account. As a consequence, research conducted by national and territorial divisions (e.g., states), is framed from their perspectives and is available to urban officials as a zero-feedback input.
A Report for the AWWA Research Committee on Distribution Systems—*Water Distribution Research and Applied Development Needs*	April-72	Edward E. Bolls, Jr., Dan A. Brock, Edwin B. Cobb, Holly A. Cornell, J. Ernest Flack, Frank Holden, F. Pierce Linaweaver, Jr., Robert C. McWhinnie,	This report summarizes water distribution research and applied development needs, as suggested by the AWWA research committee on distribution systems, through interaction with the UWRRC. Representative recommendations/observations include: 1. Using a number of systems, data sensitivity analysis should be made to better determine what kind and how much data should be collected for modeling purposes.

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
		James C. Neill & Robert U. Olson	2. The time and spatial statistics of historical fire loads need evaluation on a national scale. 3. Hypothetical alternative designs of water services for segments of service districts representing residential occupancy should be prepared with the objectives of finding: the relationships between service pressures, back-up water facilities, building structural conditions fire-fighting capability as they affect fire protection and its cost; the incremental water system cost for meeting different peak rates and pressures; the relation between pressure requirements of appliances and pressure characteristics of plumbing compared with the cost of supplying those pressures, with a view towards reducing the combined costs of public services and private property; and the relation between lawn irrigation costs and reducing lawn irrigation load peaks. 4. The relationship between available pressure and required duration of lawn sprinkling should be investigated, together with an evaluation of pressure and duration desired by the public in a willingness to pay context. 5. There is wide variation in ratio of peak demand to average demand from one city to another and even between service districts within a given city. 6. Better information on flow coefficients of pipes is needed, especially for smaller cities. 7. Water leakage represents lost revenue and the relationship between methods of construction and leakage should be examined in terms of newly developed materials and practices.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-16—*Metropolitan Industrial Water Use*	May-72	L.S. Tucker, Jaime Millan & Wilford W. Burt	Management of water resources on a comprehensive basis at the metropolitan level has been frustrated by the fact that the planning, implementation and operation of facilities and services are usually fragmented in both the central cities and in their metropolitan districts. Considerable attention has been accorded the modification of institutional arrangements for facilitating the management of metropolitan water resources, but these have been almost exclusively concerned with the direct jurisdictions of local government. The role of private interests was known to be substantial, but national water statistics have failed to identify metropolitan water resources as a separate entity, dwelling exclusively on river basin, state and national totals. This report may inspire a rectification of that oversight
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-17—*Hydrologic Effects of Urbanization in the United States*	June-72	M.B. McPherson	The present challenge to the United States—and to every developed and developing nation—is to determine a more rational way of using resources so that economic growth and social progress can continue without jeopardizing the health, safety and well-being of people or endangering the Nation's security . . . . Only when confronted, in recent years, by gross pollution and threats of irreversible environmental damage have we begun to accept fully the fact that the wastes heedlessly generated by a growing, urbanized, high-production, high-consumption society exceed nature's capacity for self-renewal. . . . A new ethic has emerged which repudiates the mistakes of the past and demands the restoration and preservation of a safe, wholesome, aesthetically satisfying environment.[1] Environmental quality, in its broadest sense, has been defined as "a measure of all the physical, biological, social, economic, and cultural conditions and natural

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			beauty which relate to the habitat of man and other creatures."[2] Priority environmental quality problem areas in the U.S. are water pollution, air pollution, long-term effects of human activity on climate, solid wastes disposal, noise, pesticides, radiation, and land-uses.[3] --- [1] Department of State, U.S. National Report on the Human Environment, Department of State Publication 8588, GPO, Washington, DC, 1971, pp. 7, 17 & 18. [2] Stevenson, Albert H., "Human Ecology Considerations in Water Quality Management," in Is Water Quality Enhancement Feasible?, Specialty Conference Proceedings, ASCE, New York, 1970, pp. 66. [3] Council on Environmental Quality, Environmental Quality, GPO, Washington, DC, First Annual Report, Transmitted to the Congress August 1970.
ASCE Urban Water Resources Research Program TM-18—*Urban Runoff*	August-72	M.B. McPherson	This report deals with some of the principal aspects of surface runoff in urban areas in the U.S.
Summary from Findings, Urban Water Resources Research Program, American Society of Civil Engineers—*Better Design of Stormwater Drainage Systems*	July-73	M.B. McPherson	What is now called the ASCE Council on Urban Water Resources Research held its first Engineering Foundation Research Conference in 1965 on *Urban Hydrology Research*. Among the principal 1965 findings was a serious need for field data on rainfall-runoff-quality for sewered catchments. Mainly in response to that expressed need, the ASCE Urban Water Resources Research Program was initiated late in 1967. A national program for acquisition of needed sewered catchment hydrological data was proposed to the US Geological Survey in 1969, together with a documentaiton of its justification, but progress in implementation has been painfully slow and future prospects unfortunately are not very bright, despite the best of intentions of all concerned. As of 1969: only a very few sewered catchments had been gauged; records seldom extended over more thana very few seasons; although rainfall-runoff had been measured, for still fewer of the catchments had rainfall-runoff-quality measurements been made; runoff measurements and the rarer runoff-quality measurements had been made at outfalls but no in-system; and of observations synchronization, and too crude signal resolution. Proportionally more rainfall-runoff data were available for partially sewered areas, but the amount was nevertheless pitifully small. Since then field research sponsored by EPA and other federal agencies have extended the database somewhat. However, not much new data have been acquired on completely sewered catchments using flumes or weirs rather than stage-gauges for determining discharge. In anticipation of the possible future availability of new data, considerations for modeling sewered urban catchment rainfall-runoff-quality processes were detailed and considerations for characterizing rainfall time and spatial distributions in future research were explored. The US Geological Survey is engaged in a program of data collection and studies to serve national needs in urban hydrology, but sewered catchment activities are limited. The Office of Water Resources Research has developed a broad program of projected urban water resources research, including urban hydrology analyses. Collection of information on the quality of storm sewer discharges and combined sewer overflows, continues,

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			supported by EPA.
DRAFT Part I, International Summary—*Hydrological Effects of Urbanization: Environmental Impact*	August-73	IHD/Unesco Sub-Group on the Effects of Urbanization on the Hydrological Environment	Among the numerous studies undertaken by the IHD was an investigation initiated in 1965 by the Working Group on the Influence of Man on the Hydrological Cycle, supported by the Food and Agricultural Organization (FAO), charged with agricultural and urbanization aspects of the subject. The Co-ordinating Council in 1970 decided to form a Sub-Group on the Effects of Urbanization on Hydrological Environment, supported by Unesco, to assist the Working Group and to investigate more intensively industrial and urbanization aspects.
DRAFT Part II, National Case Studies for Selected Countries—*Hydrological Effects of Urbanization: Environmental Impact*	September-73	IHD/Unesco Sub-Group on the Effects of Urbanization on the Hydrological Environment	Chapter 1, Hydrologic Effects of Urbanization in the Federal Republic of Germany (by: Herbert Massing and Members of the State Institute for Hydrology and Water Protection Northrhine Westphalia Krefeld). Chapter 3, Hydrological Effects of Urbanization in the Netherlands (by: F.C. Zuidema, Ijsselmeerpolders Development Authority, Lelystad). Chapter 4, Hydrologic Effects of Urbanization in Sweden (by: Gunnar Lindh, Institutionen Fttenbyggnad Lund, Sweden). Chapter 6, Hydrologic Effects of Urbanization in the United States (by M.B. McPherson). Chapter 7, Hydrological Effects of Urbanization in the U.S.S.R. (by: V.V. Kuprianov, State Hydrological Institute Leningrad).
DRAFT Part III, Illustrative Special Topic Studies—*Hydrological Effects of Urbanization: Environmental Impact*	September-73	IHD/Unesco Sub-Group on the Effects of Urbanization on the Hydrological Environment	This draft of Part III has been prepared as a background document for the International Workshop on the Hydrological Effects of Urbanization, Warsaw, 8-10 November 1973. This deals with some of the principal aspects of surface runoff in urban areas of the U.S.A. Refer to the companion report, Chapter 6, Part II, for supplementary information related to the subject.
Journal of the Hydraulics Division—*Need for Metropolitan Water Balance Inventories*	October-73	M.B. McPherson	Paper of the Technical Council on Water Resources Planning and Management
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-19—Ev*aluation of Urban Flood Plains*	December-73	James E. Goddard	About 16 percent, or 16,500 square miles, of the nation's urban areas are in natural 100-year floodplains. This compares with 7 percent for the total national land area. Approximately 53 percent, or 8,800 square miles, of urban flood plains have been developed. Better national and regional data inventories are needed to evaluate the flood problem. These are essential for preparation of wise water-related programs and also for guidance in industrial development, land use planning, transportation, environmental, and other programs.
Urban Land Institute, American Society of Civil Engineers and National Association of Home Builders—*Residential Streets—Objectives, Principles and Design Considerations*	1974	Published Jointly by the Urban Land Institute, American Society of Civil Engineers and National Association of Home Builders	As many miles of residential streets may be built in the next 30 years as have been built during the entire history of the United States. Our streets have evolved from the often narrow dusty unpaved residential streets of 30 or 40 years ago to wide paved streets with curbs, gutters, sidewalks, street lighting, and landscaping. The process of converting from gravel or dirt streets to current practice has largely been the filtering down and adaptation of standards developed for highways having high volume heavy vehicle use. In an era when the national propensity was to see limitless national resources, the approach to setting standards often was "when in doubt, pick the highest standard." The purpose of this report is not to suggest that street design in the past has been "done all wrong" and should be completely redone, but rather to suggest objectives for

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			our residential streets and principles to guide us toward optimum design standards. These objectives and principles are fundamentally common sense and should be readily acceptable. What is different about the objectives and principles is that they are specifically oriented toward residential streets, and they acknowledge that residential streets differ from commercial or industrial streets and from major arterials and interstate highways. It is presumptuous to suggest only one set of design standards could be uniformly applied throughout the country. At the same time, it is legitimate to suggest each community can measure its local practices against a set of objectives and principles which do have uniform applicability. Performance measured against these objectives and principles will identify the adequacy or inadequacy of current design and this will suggest modifications needed in local practice.
Report of the Sub-group on the Effects of Urbanization on the Hydrological Environment, of the Coordinating Council of the International Hydrological Decade—*Hydrological Effects of Urbanization*	1974	M.B. McPherson	The objectives of this report are to describe the effects of urbanization together with its environmental impacts on the hydrological cycle and to recommend research needed by the managers of all types of water systems to minimize the environmental stresses. This report is primarily directed to researchers in hydrology, and a special summary is directed to water managers.
ASCE Urban Water Resources Research Program TM-20—*A Streamflow Model for Metropolitan Planning and Design*	January-74	Richard F. Lanyon & James P. Jackson, The Metropolitan Sanitary District of Greater Chicago	Mathematical models used for the simulation of urban rainfall-runoff or rainfall-runoff-quality can be divided into three distinct categories: Planning Models--Design Models--Operations Models.
Report to U.S. National Science Foundation—*International Workshop on the Hydrological Effects of Urbanization, Warsaw, 1973*	January-74	IHD/Unesco Sub-Group on the Effects of Urbanization on the Hydrological Environment	Among the numerous studies undertaken by the International Hydrological Decade was an investigation intitiated in 1965 by the Working Group on the Influence of Man on the Hydrological Cycle, supported by the Food and Agricultural Organization (FAO), charged with agricultural and urbanization aspects of the subject. The Coordinating Council in 1970 decided to form a Sub-Group on the Effects of Urbanization on the Hydrological Environment, supported by Unesco, to assist the Working Group and to investigate more intensively industrial and urbanization aspects.
ASCE Urban Water Resources Research Program TM-21—*Innovation: A Case Study*	February-74	M.B. McPherson	"Society is always taken by surprise at any new example of common sense."—Ralph Waldo Emerson. Innovation can be defined as: the act of introducing something new or novel, as in customs; a change affected by innovating; or a novelty added or substituted. The word derives from Latin and French terms for making anew or renewal. Thus, innovation can embrace the revisitation of an old idea, rephrased or reclothed for application in a new or different context. Related subjects include technology transfer, technology assessment, municipal delivery systems, municipal information systems, cost effectiveness evaluation, and public technology.
ASCE Urban Water Resources Research Program TM-22—	March-74	Noel Hobbs & Jay D. Britton Denver Board of Water	The purpose of this technical memorandum is to indicate the type of planning applications that have been made in a leading metropolitan water supply and distribution

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
Computer Modeling Applications in Urban Water Planning		Commissioners	agency.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-23—*A Model for Evaluating Runoff-Quality in Metropolitan Master Planning*	April-74	L.A. Roesner, H.M. Nichandros, R.P. Shubinski, A.D. Feldman, J.W. Abbott & A.O. Friedland	Documented is a computer model that should see extensive use in total-jurisdiction preliminary sewerage planning, if for no reason other than the simple fact that it is presently the outstanding tool available for that purpose.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-24—*Management of Urban Storm Runoff*	May-74	Water Resources Engineers and The Hydrologic Engineering Center, Corps of Engineers	On February 5-9, 1973, a training course on "Management of Urban Storm Water, Quantity and Quality," was sponsored by The Hydrologic Engineering Center, Corps of Engineers, U.S. Army, at its facilities in Davis, California. Lectures were involved with a quantity-quality mathematical model for preliminary planning called "STORM."
Proceedings of a Research Conference—*Urban Runoff Quantity & Quality*	August-74	M.B. McPherson, et al.	This conference was planned because the Urban Water Resources Research Council believed that insufficient progress was being made in developing adequate technology and data-gathering systems for urban drainage and runoff control. Public awareness of urban runoff problems is increasing, and governmental requirements for solutions are escalating. New technology which has been developed is only being sporadically applied. Therefore, the conference was directed at identifying technology gaps, research needs, and valuable new techniques which need only to be applied. The work of the Urban Water Resources Research Council, and of this conference in particular, covers topics inclusive of the interests of several of the ASCE technical divisions, and which also is of concern to disciplines other than civil engineering. Those invited to participate, although mainly civil engineers and administrators, included well-informed representatives of the social sciences and a few practicing environmentalists. The originally planned conference sessions successfully achieved most of the original objectives; but something more happened. As it proceeded, the conference developed a vitality of its own. Synergistic reactions occurred, and opinion started to coalesce as regards certain important points, not only within the original agenda, but also outside of it. While some matters discussed are still unclear, and differences among individuals still prevail, there is a much larger degree of confidence that we are collectively on the right track, as results of experience and of research findings have combined to clear the focus of understanding. There still remains the task of expressing these new understandings in words clear enough and convincing enough to explain them to others. Hopefully, these proceedings will contribute to this broader understanding.
Engineering Foundation Conference (Paper)—*Urban Runoff, Quantity and Quality*	August-74	Remarks by M.B. McPherson	Local government agencies responsible for water resources are typified by a reactive, rather than an anticipatory, planning mode. Until there are more statutory or other kinds of incentives for anticipatory planning, innovative as well as traditional concepts will probably continue to be called upon on in an ad hoc basis to meet each crisis as it arises. With all three levels of government on a reactive tack, each national shift causes perturbations through the State and then local

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			governments, and the resultant policy instabilities have a pronounced inhibiting effect on the pursuit and acceptance of innovative programs. Too many researchers are inclined to believe that publications represent an exhaustive documentation of what is known, the state of the art. Instead, there is much more excellent innovative work that goes unreported and unnoticed. The fact that numerous "new" ideas turn out to be re-warmed or repeated concepts attests to this obtrusiveness. The scale and scope of investigation truly necessary for comprehensive analysis and resolution of problems and impediments in the translation of research findings into practice, even when restricted to urban water applications, are enormous. Misinformation and lack of information can exacerbate conflicts, whereas rapprochements can be more readily reached when the public is in command of adequate, sound information.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-25—*ASCE Urban Water Resources Research Program, 1967-1974*	October-74	M.B. McPherson & G.F. Mangan, Jr.	Briefly summarized is the progress made through mid-1974, on research needs assessment, urban water management, translation of research findings into oractucem facilitation of urban runoff research, and collaboration and participation in research of local governments and other organizations. Also included are one-page abstracts for all 28 program reports and technical memoranda.
Urban Land Institute, American Society of Civil Engineers and National Association of Home Builders—*Residential Stormwater Management—Objectives, Principles and Design Considerations*	1975	Published Jointly by the Urban Land Institute, American Society of Civil Engineers and National Association of Home Builders	In the next 25 years, more money may be spent for stormwater management than has been spent for drainage during the entire history of the nation. This will amount to billions of dollars, not including funds that will be expended to maintain water quality, which also is an increasing national concern. Much, if not most, of this investment will be private funds expended during the development of land for urban uses. Ultimately, however, all costs (capital, debt and maintenance) are borne by all citizens. Public investments should be made wisely in furtherance of the quality of life so highly valued. It is the opinion of ASCE, NAHB, and ULI that the application of the Objectives, Principles, and Design Considerations contained in this report, with due regard for unique and particular circumstances and conditions found in various areas of the country, will be a significant step in the right direction. For the concepts in the report to achieve wide application, there will be a need to induce institutional changes even beyond the design professions and the regulatory institutions of governments. Changes also are necessary in the financial institutions which fund development based on their approval of a project's design, in the insuring institutions and their perceptions of the insurability of this approach, and in the legal professions in relationship to the pre-existing body of land use law regarding rights, responsibilities and liabilities. The implication of the above statement, that change may be precluded by institutional constraints, is real. The importance of encouraging application of worthwhile new approaches should provide the impetus to achieve necessary changes. It is hoped that this report will motivate creative rethinking and updating of drainage design practices. Anyone disagreeing with any part of this report is encouraged to advise the publishers of that disagreement, including their detailed reasons and alternative

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			recommendations. Such information will help guidie future revisions and enhance the document's value to our Nation.
A Study of ASCE Urban Water Resources Research Council (UWRRC) (First Draft for March 1975 Panel Review at USGS, Menlo Park, California—*Earth Science Information Needs in Local Government Water Control Master Planning*	February-75	M.B. McPherson	The San Francisco Bay Region Environment and Resources Planning Study has been conducted by the Geological Survey of the U.S. Department of the Interior under a joint sponsorship with the U.S. Department of Housing and Urban Development. The primary goal of the Study was to provide the kinds of earth science data that would be of material assistance in regional urban planning and development, particularly information regarding rock and soil conditions, slope stability, availability and quality of water, susceptibility to earthquake damage, waste disposal and storage, and locations of construction materials and other resources.
ASCE Urban Water Resources Research Program TM-26—*Effects of Urbanization on Water Quality*	March-75	Robert P. Shubinski and Steven N. Nelson	The basic content of this technical memorandum is based heavily on a paper presented at the ASCE Urban Transportation Division's Specialty Conference on "Environmental Impact" held May 21-23, 1973, in Chicago. While the Proceedings of the conference have been published recently by ASCE (*Environmental Impact*, ASCE, New York, NY, 393 pp., 1975), the original paper is not likely to be sought therein by a large enough number of urban planners. In order to reach more urban planners, Messrs. Shubinski and Nelson have rewritten and enlarged the scope of the conference paper as a voluntary contribution to the ASCE Program. Erosion evaluation criteria being used in the development of Fairfax County, Virginia, master plan supplement those outlined in this technical memorandum.
A Staff Report for the NSF-RANN Project G 42282 "Petroleum Industry in the Delaware Estuary"—*Projection of Petroleum Content of Urban Runoff*	April-75	William Whipple, Jr., Joseph V. Hunter, Shaw L. Yu	The first ten months of data gathering and analysis indicate an appreciable content of petroleum in storm sewers. If the concentration found elsewhere proves to be at about this level, the petroleum in urban runoff into the Delaware Estuary will be a source of considerably more petroleum than the effluents of the seven refineries of the estuary, after secondary treatment.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-IHP-1—*Urban Mathematical Modeling and Catchment Research in the U.S.A*	June-75	M.B. McPherson	This report is a U.S. contribution to Unesco state-of-the-art reports on urban hydrology as part of the International Hydrological Programme.
Report—*Regional Earth Science Information in Local Water Management (Earth Science Information Needs in Local Government Water Control Master Planning)*	July-75	M.B. McPherson	The major USGS-ASCE project objective was to make an assessment of the usefulness of information products of the type generated by the SFBRS, in their existing form or as modified or extended, for master planning of water resource capital improvements at the local level of government. The relationship of data needs for local government urban water management to the more than 80 products of the SFBRS is evaluated in the last part of the report, and is mostly the contribution of a volunteer ad hoc Review Panel convened for that purpose. Background for the findings on national transferability occupies the main body of the report, in which are discussed: master planning, urban water management and

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			planning interactions; selected components of urban water management; selected techniques in urban water management; and three selected case studies. A large proportion of the SFBRS type of products has been adjudged usable in other metropolitan areas, in their present form or in a more detailed form, for local government master planning. The report closes with an entreaty for more extensive and widespread use of such information.
Journal of the Hydraulics Division—*ASCE Urban Water Resources Research Program*	July-75	M.B. McPherson & George F. Mangan, Jr.	ASCE UWRRC have focused the attention of public agencies at all levels of government on critical research needs. Recognizing the chasm that had to be bridged, the Council formed the ASCE Program in 1967 to serve as its temporary full-time operating arm. Twenty-four technical memoranda of value to all water resource practitioners in the urban field have been prepared with the help of outside contributors.
Regional Earth Science Information In Local Water Management—*Study of ASCE Urban Water Resources Research Council*	July-75	M.B. McPherson	Prepared on behalf of the ASCE Urban Water Resources Research Council, the following project report on Earth Science Information Needs in Local Government Water Control Master Planning was prepared for the San Francisco Bay Region Environment and Resources Planning Study (SFBRS), U.S. Geological Survey Experimental Program designed to facilitate the use of earth-science and related information in metropolitan planning and decision-making.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-IHP-1—*Urban Mathematical Modeling and Catchment Research in the U.S.A.*	November-75	M.B. McPherson	A preliminary version of this report, dated June, 1975 was distributed to a number of program co-operators. Most of their suggestions have been taken into account in this report. The report is a U.S. contribution to Unesco state-of-the art reports on urban hydrology as part of the International Hydrological Program. ASCE took early supportive action by applying for an NSF grant to assist in two of the ten recommended projects: R1. <u>Catchment Studies Report</u>. Prepare a "state-of-the-art" report on research executed in urban catchment areas, which would include instrumentation, data acquired, analysis performed and applications. R3. <u>Mathematical Models Report</u>. Prepare a "state-of-the-art" report on mathematical models applied to urban catchment areas and dealing with, for instance, rainfall-runoff relationships and water balances, both with respect to water quantity and quality. In keeping with the findings of the Subgroup and the Warsaw Workshop, <u>modeling and catchment research for urban drainage systems is emphasized</u>. This is the subject singled out as having the largest gaps in knowledge in urban hydrology. Water quality aspects are accorded considerable attention because of the strong interest in environmental protection in the U.S.
ASCE Urban Water Resources Research Program TM-27—*Commercial Water Use (Water Use in Selected Commercial and Institutional Establishments in the Baltimore Metropolitan Area)*	December-75	Jerome B. Wolff, F.P. Linaweaver, Jr. and John C. Geyer, The Johns Hopkins University, Baltimore, MD	Priorities in Distribution Research and Applied Development Needs Committee Report. A committee report presented at the Annual Conference on June 14. Since the publication of the committee report, Water-Distribution Research and Applied Development Needs, in the June 1974 JAWWA, two subcommittees have prepared statements highlighting urgently needed work within two previously defined priority areas. Those statements are included in this article in addition to comments offered by committee members.

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
OKLA C-5228 Agreement No. 14-31-0001-4215, submitted to The Oklahoma Water Resources Research Institute—*Improving the Quality of Water Releases from Reservoirs by Means of a Large Diameter Pump*	September 1973 - March 1976	James E. Garton & Charles Rice	In temperate climates, many lakes stratify during the summer. A typical stratified lake will have a warm oxygen-rich epilimnion, a thermocline, and a colder oxygen depleted hypolimnion. High levels of iron, manganese, hydrogen sulfide, and ammonia and dissolved hydrocarbons may occur in the hypolimnion. Many efforts have been made to destratify lakes, primarily by air bubbling. These methods requre large energy inputs. A low-energy destratifier using 42-inch and 72-inch propellers to pump the water downward from the surface have been used successfully to destratify a 100 acre lake (35 feet deep) for 3 years. This project was an attempt to apply the same kind of device to Lake Arbuckle, a2400 acre, 90-foot deep lake in south central Oklahoma. A 16.5 foor aircraft propeller was used to pump approximately 200,000 gallons per minute downward. Although the lake stability index was decreased by half, a corresponding reduction in the oxygen distribution index did not occur until the fall overturn. Thus, a lake can be weakly stratified thermally and strongly stratified chemically. The fall turnover occurred about a month earlier and more completely than normal during our two years of operation. The oxygen content in the outlet waters near the pump was increased 1 to 2 mg/L during these critical summer months.
ASCE Urban Water Resources Research Program TM-28—*Household Water Use*	January-76	M.B. McPherson	Primarily addressed to researchers, the objectives of this report on household water use are definitions of: the status of information, changes in public policies and goals, and consequent needs for new information and research. Discussed are some of the more important quality, quantity and economic considerations. Demand variations are given predominant attention, for individual and multiple households. An appendix contains a listing of a unique set of water-use data for nine individual households covering a two week period at a one-minute recording interval.
ASCE Urban Water Resources Research Program TM-29—*Computerized City-Wide Control of Urban Stormwater*	February-76	Neil S. Grigg, John W. Labadie, George R. Trimble, Jr., & David A. Wismer	A comprehensive research program on automation of urban stormwater facilities has been undertaken at Colorado State University. With the support of the Office of Water Research and Technology, automation implications for individual catchments were investigated and managerial aspects of the findings were reported separately from technical project completion reports to enhance their transferability. With the support of the Research Applied to National Needs Program of the National Science Foundation, automation implications for total-jurisdiction systems were investigated, the scope of the report that follows.
A Report for Division of Advanced Environmental Research and Technology Research Applied to National Needs Program National Science Foundation Washington, DC 20550—*Urban Hydrology Research Needs*	April-76	M.B. McPherson	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. 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

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			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. In 1972, reference was made to "the continuing ambivalence of the Federal Government on the question of what its real role is in our nation's metropolitan areas," a status that has changed little since, witness the fiscal dilemma of New York City and other urban centers. All levels of government are commonly involved to some degree in urban water resources research, and the fractionalized character of local government at the metropolitan level adds to the diffusion of attention. In sum, "even the most urbanized countries still exhibit the rural origins of their institutions." Few local agencies can support hydrologic research that will yield results transferrable to other metropolitan areas, or even from one jurisdiction to another in the same metropolis. Hence, urban water resources research around the world commonly has suffered from inadequate attention and support, and from discontinuous and erratic efforts. More than a decade ago it was possible to say that there was a need for research to establish the nature of the effects of urbanization on basic hydrological processes. Today, the broad nature of these effects is only beginning to be understood.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-IHP-2—*Urban Hydrological Modeling and Catchment Research in Australia*	May-76	A.P. Aitken	This report is the second in a special series of ASCE Program Technical Memoranda for the International Hydrological Programme.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-IHP-3—*Urban Hydrological Modeling and Catchment in Research in Canada*	June-76	J. Marsalek (Canada Centre for Inland Waters Burlington, Ontario, Canada)	This endeavor originated from activities and aspirations of the Unesco Subgroup on the Hydrological Effects of Urbanization of the International Hydrological Decade which concluded in 1974. Members of the the Subgroup represented the Federal Republic of Germany, France, Japan, Netherlands, Sweden, U.K., U.S.S.R. and U.S.A.
ASCE Urban Water Resources Research Program TM-30—*Urban Flood Warning and Watershed Management Advances in Metropolitan Melbourne*	June-76	C.T. Earl, J.W. Porter, J.A. Lanaway, A.S. Alexander, R.W.M. King, G.O. Cosgriff--Melbourne & Metropolitan Board of Works and Don G. Thompson--Dandenong Valley Authority	In November, 1975 M.B. McPherson was privileged to present a paper in Canberra at the third National Symposium on Hydrology sponsored by the Australian Academy of Science. Professor E.M. Laurenson of Monash University arranged for a visit to his university, the University of Melbourne, the Melbourne & Metropolitan Board of Works and the Dandenong Valley Authority, all in metropolitan Melbourne, at the close of the symposium in Canberra. Intrigued by advances made in institutional arrangements and urban flood warning systems in Melbourne, the authors of this report were asked to contribute suitable material for an ASCE Program Technical Memorandum.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-31—	July-76	Edited by M.B. McPherson	Proceedings of a Special Session, Spring Annual Meeting, American Geophysical Union, Washington DC, April 14, 1976. Over the past few years there has been an unprecedented surge of activity in urban hydrological model development. While there has always been a lag

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
Utility of Urban Runoff Modeling			between development and application, many specialists in urban water resources matters have been concerned about what has seemed to be a snail-pace adoption of the newer tools. There is widespread agreement that what is hindering more extensive and effective use of more sophisticated techniques in local government projects is a clear definition of why their use would be more cost-effective than simpler, traditional procedures.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-IHP-4—*Urban Hydrological Modeling and Catchment Research in the United Kingdom*	July-76	M.J. Lowing	This report is the fourth in a special series of ASCE Program Technical Memoranda for the International Hydrological Programme. The prototype report was for the U.S.A. A number of contributed national reports, of which the United Kingdom report herein is the third to be received, will be issued subsequently through 1976.
A Study of ASCE Urban Water Resources Research Council (UWRRC) TM-IHP-6—*Urban Hydrological Modeling and Catchment Research in the United Kingdom*	July-76	M.J. Lowing	This endeavor originated from activities and aspirations of the Unesco Subgroup on the Hydrological Effects of Urbanization on the International Hydrological Decade which concluded in 1974. This report summarizes urban hydrologic modeling and catchment research in the United Kingdom.
ASCE Urban Water Resources Research Program TM-31—*Utility of Urban Runoff Modeling*	July-76	M.B. McPherson	Proceedings of a special session, Spring Annual Meeting, American Geophysical Union, Washington DC, 14 April 1976.
Supplement to the Technical Completion Report Project C-5228—*Demonstration of Water Quality Enhancement Through the Use of the Garton Pump*	August-76	James E. Garton & Howard R. Jarrell	A field demonstration of state-of-the-art technology was held at Lake Okatibbee near Meridian, MS. This demonstration included a review of reservoir stratification dynamics and of the development and previous research involving the Garton Pump. Upon conclusion of the "chalk talk," participants adjourned to the lake intake structure and stilling basin to observe the pump operations at first hand.
A Study of ASCE Urban Water Resources Research Council (UWRRC) TM-IHP-5—*Methods for Calculating Maximum Flood Discharges for Natural Watercourses and Urban Areas in the U.S.S.R.*	August-76	V.V. Kuprianov	This endeavor originated from activities and aspirations of the Unesco Subgroup on the Hydrological Effects of Urbanization on the International Hydrological Decade which concluded in 1974. This report summarizes methods for calculating maximum flood discharges for natural watercourses and urban areas in the USSR.
A Study of ASCE Urban Water Resources Research Council (UWRRC) TM-IHP-6—*Urban Hydrology Studies and Mathematical Modeling in the*	September-76	H. Massing	This endeavor originated from activities and aspirations of the Unesco Subgroup on the Hydrological Effects of Urbanization on the International Hydrological Decade which concluded in 1974. This report summarizes urban hydrologic studies and mathematical modeling in the Federal Republic of Germany.

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
Federal Republic of Germany			
A Study of ASCE Urban Water Resources Research Council (UWRRC) TM-IHP-7—*Urban Hydrological Modeling and Catchment Research in Sweden*	October-76	Gunnar Lindh	This endeavor originated from activities and aspirations of the Unesco Subgroup on the Hydrological Effects of Urbanization on the International Hydrological Decade which concluded in 1974. This report summarizes urban hydrologic modeling in catchment research in Sweden.
A Conference Report—*Guide for Collection, Analysis, and Use of Urban Stormwater Data*	November-December-76	William M. Alley	The objective of this report is to present guidelines for designing a data-collection program to generate, within the context of available resources, data of sufficient breadth, detail and representativeness to define urban-runoff problems, to calibrate and verify urban-runoff models, and to monitor the effectiveness of "solutions" to urban-runoff problems. Rising project complexity, increasing potentials for investments required, and continually changing "rules of the game" such as the 1972 Amendments to the Federal Water Pollution Control Act, the flood insurance acts, more extensive land-use planning, the environmental movement, etc., are all resulting in an ever-increasing need for more field data. Local governments are essentially on their own for the acquisition of local urban-runoff field data. Documentation of the methodology for acquiring and analyzing urban-runoff data are scattered throughout the literature and guidelines for developing a complete "data collection package" are not presently available. This publication attempts to fill that gap. The emphasis is not on instrumentation theory, laboratory analytical techniques or model theory, but rather on guidelines that can be used by local governments for an approach to data collection that is based on the purposes for which the data are to be used. Five separate, but interdependent topics are discussed: data utilization, data analysis, network planning and design, instrumentation, and data collection. Summaries of these chapters are presented in Table 1.
Article Reprinted from EOS, Volume 57, No. 11, November 1976—*Urban Water Resources*	November-76	M.B. McPherson	Urbanization growth.
Engineering Foundation Conference—Easton, MD "NETWORK DESIGN" (emphasis on temporal factors)—*Instrumentation and Analysis of Urban Stormwater Data, Quantity and Quality*	November-76	M.B. McPherson	The purpose of this presentation is to supplement or compliment the coverage in the workshop report draft issued November 28 to participants, with regard to temporal aspects of data network design. This workshop has its origins in the International Hydrological Programme (IHP). Project 7 of the IHP, *Effects of Urbanization on the Hydrological Regime and on the Quality of Water*, contains five sub-projects, one of which is *Research on Urban Hydrology* (sub-project 7.1, for which the speaker is the rapporteur for Unesco). Included as the initial effort of sub-project 7.1 is a series of national case studies, which are being assembled as ASCE Program Technical Memoranda in a special IHP series (processed to date are reports for the USA, Australia, Canada, United Kingdom, USSR, Federal Republic of Germany, Sweden and France). In addition,

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			sub-project 7.1 calls for the preparation of information manuals on urban water-data collection, analysis and use, with the first such manual emphasizing catchments. Having failed to obtain financial support for an international symposium to develop a catchment manual, and recognizing the urgent need for such a manual in the US, the ASCE UWRRC turned instead to the development of the present workshop. An international accent has been maintained by the inclusion of national reports in the workshop program for Canada and Sweden.
ASCE Urban Water Resources Research Program TM-IHP-8—*Urban Hydrological Modeling and Catchment Research in France*	November-76	M. Desbordes & D. Normand	This report has been prepared on behalf of the French National Committee for the International Hydrological Programme as a contribution to IHP Project 7, "Effects of Urbanization on the Hydrological Regime and on Quality of Water." Reviewed in this section are the present status of research on urban hydrology in France and plans for future research in that field.
ASCE Urban Water Resources Research Program TM-IHP-9—*Urban Hydrological Modeling and Catchment Research in Norway*	December-76	Nils Roar Selthun	This report is a Norwegian contribution to the UNESCO state-of-the-art report series on urban hydrology (IHP Subprojects 7.1 and 7.2) The report has been prepared under a project on urban catchment research, Project PRA 4.2, financed by the Norwegian Ministry of Environmental Affairs and administered by the Hydrological Division of the Norwegian Water Resources and Electricity Board.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-IHP-10—*Urban Hydrological Modeling and Catchment Research in the Netherlands*	January-77	F.C. Zuidema	In spite of a continuous growth in the population of the Netherlands (13 million in 1970 with 15.6 million expected in 2000) and an increasing population density (384 inhabitants per square km in 1970, 403 in 1975, 434 in 1985, and 463 in 2000), development in urban water resources research has been rather tardy. However, there is an enormous diversity among urban hydrological problems, which are solved adequately. For example, while only a few urban catchment studies are going on, mathematical models are used for different goals and at different levels: models for rainfall-runoff, water quality, comprehensive urban water and water resources management.
Paper—*The Design Storm Concept*	February-77	M.B. McPherson	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 floodplain 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, and there is evidence that the usage of such storage is on the rise. However, local detention storage has been only occasionally employed as part of overall flood mitigation, as in Denver, Coloardo and Fairfax County, Virginia. Detention storage is recognized as one of the principal means for abatement of pollution from stormwater discharges and combined sewage overflows. There are opportunities in new land development to incorporate detention storage at the ground surface. 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

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			requirements for them is greater. Recognized very early was the need for some form of automatic control because of system complexity and the need to manipulate flows in order to insure their containment as a means for reducing overflows.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-IHP-11—*Urban Runoff Research in Poland*	February-77	Pawel Blaszczyk	Earlier research in Warsaw on the influence of combined sewerage schemes on the Vistula River, as well as a survey of the area conducted as part of a general project for development of sewerage and drainage in Warsaw, clearly showed the need for investigation of the relations between rainfall and the runoff from drainage catchments in order to meet the requirements for planning and designing sewerage and drainage schemes.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-IHP-12—*Urban Hydrological Modeling and Catchment Research in India*	May-77	S. Ramaseshan & P.B.S. Sarma	This report summarizes the state of the hydrology practice in India. Because of the low priority for urban hydrology in India, and scanty information dealing with mathematical models and urban hydrologic research, the scope of this report is extremely limited.
A Study of ASCE Urban Water Resources Research Council—*Urban Runoff Control Planning*	June-77	M.B. McPherson	The growth impact of projected areal enlargement of urban areas 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.
A Study of ASCE Urban Water Resources Research Council (UWRRC)—*Urban Runoff Control Planning*	June-77	M.B. McPherson	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. 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.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-IHP-13—*Urban Hydrological Modeling and Catchment Research: International Summary*	November-77	M.B. McPherson & F.C. Zuidema	Twelve national reports for the International Hydrological Programme (IHP) have been compiled on the state-of-the-art in urban catchment research and hydrological modeling, with particular attention given to underground conduit systems. Summarized in this report are their principal commonalities, together with particularly noteworthy observations or advances reported for individual countries.
ASCE Urban Water Resources Research Program TM-32—*Nomographs for 10-Minute Unit Hydrographs for Small Urban Watersheds*	December-77	William H. Espey, Jr.; Duke G. Altman & Charles B. Graves, Jr.	Unit hydrographs have been developed from rainfall-runoff field data for completely sewered drainage catchments in Louisville, KY and Atlanta, GA. 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.
ASCE Paper—*Urban Runoff Control, Quantity*	March-78	M.B. McPherson	Presented at American Public Works Association Urban Drainage Workshop, Omaha, Nebraska March 14, 1978. This paper summarizes the main points that have been covered in presentations to areawide planning agencies

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
and Quality			regarding urban runoff management.
Proceedings of the Research Conference—*Water Problems of Urbanizing Areas*	July-78	William Whipple, Jr., et al.	This conference grew out of continuing concern by various members of the Universities Council on Water Resources and of the Urban Water Resources Research Council, ASCE, during 1977 that the water problems of urbanizing areas were not being given sufficient attention. Originally, the conference was intended to give principal attention to water quality and environmental problems, covering other water problems primarily as interfaces. However, the sponsoring agencies urged broadening of the scope; and, accordingly, water supply and flood control problems also were given attention. This change was desirable, but it considerably increased the scope of coverage of the conference. Please note the distinction between "water problems of urbanizing areas" and "urban water problems." The latter has been the subject of considerable research and is a problem of substantial economic importance. The subject of this conference is intended to be more focused, concentrating on the unique problems of those areas undergoing the process of urbanization. Our theme this week is "Water Problems of Urbanizing Areas." This concept is significantly different from the urban water programs and conferences of the past few years, in that we are emphasizing metropolitan regions rather than central cities, and we are considering a dynamic process rather than a condition. In treating of environmental pollution in urbanizing areas, we will be thinking of the planning processes which may limit and control development, to the aim that areas not yet fully developed may be able to preserve their environmental amenitieis. In the past, some of us tried to recognize the broader planning aspect by stressing water problems of metropolitan rather than urban areas; but this concept was too fine a distinction and never had much appeal except to theoreticians. The term "urbanizing areas" brings out the dynamics of the situation, where the problems are most acute and the necessities for better management most apparent.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-33—*Research on the Design Storm Concept*	September-78	Jiri Marsalek	This Technical Memorandum is Addendum 4 of a 1977 ASCE Program report on "Urban Runoff Control Planning." Addendum 1, "Metropolitan Inventories," and Addendum 2, "The Design Storm Concept," were appended to the latter report. Addendum 3 was the first of several additional, individual Addenda to be released over the period 1977-1979.
A Study of ASCE Urban Water Resources Research Council (UWRRC) TM-34—*Testing of Several Runoff Models on an Urban Watershed*	October-78	Jess Abbott	Six models, plus two variants of one and a variant of another, were tested in this study, with the objective of making a preliminary evaluation of their relative capabilities, accuracies and ease of application. A detailed comparison of the many capabilities and features of these models was beyond the scope of the study. For four of the models, plus two variants of one of them, the primary performance criterion was the degree to which simulated values matched observed daily and monthly runoff volumes for the 5.5-square mile Castro Valley Watershed near Oakland, California.
A Study of ASCE Urban Water Resources Research Council (UWRRC) - ASCE TM-35—	November-78	Van-Thanh-Van Nguyen, M.B. McPherson & Jean Rousselle	The objective of this empirical study was to explore the feasiblity of tracking storms for combined sewer system automatic control applications. Data used were for the largest depth events in a 27 month record of hourly rainfall from a network of 18 gauges in the Montreal

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
Feasibility of Storm Tracking for Automatic Control of Combined Sewer Systems			(Canada) region.
Journal of The Water Resources Planning and Management Division—*Urban Runoff Control Master Planning*	November-78	M.B. McPherson	The thesis of this paper is that rational planning requires conjunctive consideration of the quantity and quality aspects of urban runoff within a comprehensive, multiple-use framework. Explored are some of the more obvious arguments with reference to land-use factors, economic evaluations, flood control considerations, performance simulation, and metropolitan water resource inventories. There appear to be very few master plans extant that have integrated water quantity management with water quality management. Given the institutional constraints in metropolitan areas, perhaps the most that can be expected is adoption of an integrated, comprehensive, or systems approach to urban runoff planning. However, such an approach is subtle in a documentary sense, and more rational conjunctive master planning may be in progress than external indications seem to imply.
Paper—*Reviews of Geophysics and Space Physics*	November-78	M.B. McPherson	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 management utilizes the social and biological sciences as well as the physical sciences. The fact that the term "urban hydrology" gained currency less than two decades ago reflects the lag in the recognition that America became an urban nation over half a century 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. Because the period 1975-1978 was dominated by attention to quantity and quality of urban runoff, that emphasis will be preserved here. However, one of the most important trends of the period was a growing acceptance of a perception of water in urban areas as a totality, as a resource [Kohlhaas, 1975; Dendrou et al., 1978a]. This integrated view is exemplified in a prognostication of needs for urban water models by Sonnen et al. [1976].
ASCE Urban Water Resources Research Program TM-35—*Feasibility of Storm Tracking for Automatic Control of Combined Sewer Systems*	November-78	Van-Thanh-Van Nguyen, M.B. McPherson & Jean Rousselle	Addendum 6 of the ASCE Program report "Urban Runoff Control Planning Addendum 5 of the ASCE Program report "Urban Runoff Control Planning," June 1977
Urban Land Institute, American Society of Civil Engineers and National Association of Home Builders—*Residential Erosion and Sediment Control—Objectives, Principles and Design*	1978	Published jointly by Urban Land Institute, American Society of Civil Engineers and National Association of Home Builders	This report covers some of the basic concepts for protecting land and water resources against the detrimental impacts of erosion and sedimentation during residential construction. The objectives, principles, and design considerations are complex individually and collectively on a variety of levels; many detailed considerations have, of necessity, been omitted. The following are some of the basic concepts generally found to be useful:

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
Considerations			Erosion and sediment movement and deposition have both beneficial and detrimental effects. Sediment movements should not be permitted at rates or in quantities which will cause significant residual damage. Under ideal conditions any change in the nature or amount of sediment leaving a site as a result of construction should maintain or improve environmental quality when compared to pre-construction conditions. While these ideal conditions cannot always be achieved, the importance of environmental quality to man's long-term welfare and survival means that sound judgment must be exercised in establishing allowable rates of change from those expected in the natural cycle. Strong emphasis needs to be placed on "natural" engineering and land planning techniques, which will not only preserve and enhance natural features of the land, both on and off the site, but protect them. There are techniques which use and improve the natural processes taking place at a construction site, during and after the actual construction period, rather than ignoring or replacing them with artificial systems. There must be increasing recognition that each site has its own set of natural resources, land use limitations, envirionmental conditions, and occupancy requirements. These factors and their inter-relationship vary from site-to-site within a community, and variations in design standards will be required for achievement of optimum off-site protection. There must be continuing recognition of a balance of responsibilities and obligations between individual land owners and the public for the protection of the environment from the adverse impacts of excessive erosion and sedimentation. It must be understood that significant immediate and long-term expenditures for the construction and maintenance of this protection will be incurred by individual homeowners and the community. A balance must be struck in determining the acceptable ranges of damage, in order to avoid restricting housing availability and choice.
Paper—*Urban Water Balances Considering Conservation*	Post 1978 (per references)	M.B. McPherson	Much of this presentation is necessarily a recapitulation of parts of three earlier papers: *Conservation in Household Water Use; Measures for Municipal Water Conservation,* and *Water Conservation: Response.* (Includes figures that are editorial modifications of illustrations prepared in 1975 and 1976 respectively, that have been reintroduced several times since.
A Study of ASCE Urban Water Resources Research Council (UWRRC) TM-36—*Introduction to Social Choice Theory for Environmental Decision Making*	February-79	Philip D. Straffin, Jr.	Chapter 2, "Power in Decision Making Bodies," and Chapter 3, "Voting Methods for More Than Two Alternatives," will be of immediate interest to anyone involved with environmental decisions. The author has crystalized the principles of relevant mechanisms that have heretofore been dispersed widely over the technical literature of the social sciences.
14th Annual Henry M. Shaw Lecture in Civil Engineering—*Challenges in Urban Runoff Control*	March-79	M.B. McPherson	Under Section 208 of the Federal Water Pollution Control Act of 1972, areawide planning for water pollution abatement management has been undertaken in a majority of metropolitan areas. Urban runoff considerations are an issue in most of these areas. Supported by a National Science Foundation grant, the ASCE Program assembled a report in 1977 intended to assist agencies and their

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			agents preparing areawide plans, from the standpoint of major urban runoff technical issues in long-range planning. The report plus two new addenda were published in a single volume by EPA in 1978. Under a renewal grant from NSF, on-site elaboration of the principles presented in the 1977 report, and their adaptation to local conditions, have been provided in seminars, workshops and conferences hosted by areawide planning agencies. The main points made at the first of such visits were summarized in a paper on *Urban Runoff Control, Quantity and Quality* presented at an APWA Urban Drainage Workshop, Omaha, Nebraska, March 14, 1978. The number of visits has since doubled, and the paper which follows is a substantial enlargement of the Omaha paper that takes into account the more recent visits and related experiences. The dozen visits made so far have been at widely separated urban centers across the country, ranging from Maine to Nevada to Texas to Florida. This ASCE-NSF project wil be concluded in a few months. A presentation for the Hydraulics group of the Boston Society of Civil Engineers Section of ASCE in Cambridge, Massachusetts, on January 31, 1979, afforded an opportunity to test the credibility of the more recent portions of this paper.
ASCE Urban Water Resources Research Program TM-37—*Challenges in Urban Runoff Control (Planning, Implementation & Simulation)*	August-79	M.B. McPherson	The ASCE program assembled a report in 1977 intended to assist agencies and their agents preparing areawide plans, from the standpoint of major urban runoff technical issues in long-range planning. Included with the report were two addenda on "Metropolitan Inventories" and "The Design Storm Concept." The report, plus two additional separately released addenda, was published in a single volume in 1978.
Proceedings of Two Special Sessions, Spring Annual Meeting, American Geophysical Union, Washington, DC, 28 May 1979 - ASCE TM-38—*International Symposium on Urban Hydrology*	September-79	Edited by M.B. McPherson	This Technical Memorandum is a supplementary report for a project supported by the NSF.
ASCE Urban Water Resources Research Program TM-39	October-79	M.B. McPherson	Proceedings of a Special Session, Spring Annual Meeting, American Geophysical Union, Washington, DC, May 29, 1979
Draft Paper—*Introduction to Digital Recording of Rainfall by Radar*	September-79	M.B. McPherson	The intended audience of this first draft is primarily the civil engineer interested in automatic control of either combined sewer systems or separate storm sewer systems or in flood warning systems for urban streams. It is widely believed that a telemetered raingauge network would be required for real-time control or warning. The most advanced applications would include predictions of rainfall on individual catchments, two or three hours in advance. The means for such rainfall prediction is digital radar recording, a capability that has emerged only very recently. Following is an attempt to summarize the state of the art in radar rainfall measurement and prediction from the viewpoint of potential applications in urban

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			water resources operations. This draft has been prepared under support of USEPA Grant No. R806702010 awarded to ASCE for a project on *Feasibility of Automated Combined Sewer Systems Allied with Metropolitan Flood Warning Systems.* In its final form, it will be a chapter in the final project report, around mid-summer of 1980.
Draft Paper—*Metropolitan Flood Warning Systems*	December-79	M.B. McPherson and William G. DeGroot	The role of the first author in the preparation of this first draft has been supported under USEPA Grant No. R806702010 awarded to ASCE for a project on *Feasibility of Automated Combined Sewer Systems Allied with Metropolitan Flood Warning Systems.* In its final form, it will be a chapter in the final project report, around mid-summer of 1980. Distributed earlier to EPA-ASCE project cooperators was *Introduction to Digital Recording of Rainfall by Radar*, draft of September 28, 1979 which will become another chapter in the final project report. The third major text chapter will be on *Automated Combined Sewer Systems*, and will feature findings from a study underway at Ecole Polytechnique of Montreal. It is generally agreed that complete automatic control of combined sewer systems will require the use of special new radar capabilities. In a given metropolitan area, such a special radar installation backed by calibrating telemetry-connected raingauges could also serve as part of a metropolitan flood warning system. A central thesis of the EPA-ASCE project is that the attractiveness of automatic control of urban runoff may be expected to increase as the territory served becomes larger and the number of functions served increases. Thus, the purpose of this presentation is to review prospects from a metropolitan flood warning system perspective, focusing on leading-edge initiatives in the Denver, Colorado area.
EPA Grant No. R806702—*Integrated Control of Combined Sewer Regulators Using Weather Radar*	June-80	M.B. McPherson	Abatement of pollution from combined sewer overflows has been regarded by some localities as requiring the addition of extensive new storage, transportand treatment facilities to their systems. The central purpose of the study reported herein was to explore the technical feasibility of some less costly measures that might prove adequate for other communities with less stringent abatement requirements, namely the possibility of reducing the extent of overflows from conventional combined sewer systems that typically divert some stormwater via regulators to interceptor sewers. The means for increasing the abatement efficiency of such systems was postulated as the real-time, integrated operation of all interceptor regulators from a central computer. Dynamic regulators are normally actuated on the basis of local control whereby the interceptor flow stage in the immediate locality of a given regulator is maximized. When no in-line or other in-system storage can be called upon, no advantage over local automatic dynamic regulator control can be obtained by integrated operation of regulators unless expected flowrates to the interceptors can be estimated in advance of their actual occurrence, and unless an operational bias is introduced which either minimizes overflows from only some of the regulators on an interceptor or favors the timing of overflows from all of the outlets, such as during the initial storm period. Computations for an actual system covering a summer season indicate that, even when a time-varying pollutant parameter is accounted for, the reduction in mass of a

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			pollutant entering the receiving water under integrated operation of regulators may be only marginal. Consideration is given to the potentials for exploiting in-line storage induced in collector and outfall sewers to gain greater flexibility with integrated regulator operation. As part of an introduction to digital recording weather radar for non-meteorologists, newly available rainfall tracking, measuring and predicting capabilities of such radars are described. Use of these capabilities would be required to achieve maximum effective integrated regulator control. A thesis of the study was that the attractiveness of an integrated regulator control system would be limited if it was necessary to dedicate a radar mostly to that service. Accordingly, auxiliary uses for radars in metropolitan areas are given consideration, particularly for urban flood warning systems.
Conference Proceedings of the International Symposium on *Urban Hydrology*	May-Jun-83	A.R. Moodie, et al.	This Symposium, the second in what we expect will be a series of exchanges of information in urban hydrology on a national basis, was held as a follow-on to a symposium held in Washington, DC, in 1979, at which national reports on the current status of urban hydrology were presented. The participants agreed to meet again in 1983 to present national progress reports including assessments of what advances have taken place, what progress has been made, and what research needs remain unfulfilled. This Symposium is the result, and includes progress reports from all the original participants, as well as status reports from Japan, Venezuela, Switzerland, Finland, and Nigeria, who were not represented in 1979.
U.S. Environmental Protection Agency, Municipal Environmental Research Laboratory, Research and Development—*Project Summary Integrated Control of Combined Sewer Regulators Using Weather Radar*	April-81	M.B. McPherson	In this study, the possibility of reducing the extent of overflows from combined sewer systems was studied. In general, when no in-line or other in-system storage is used, integrated regulator operation has no advantage over local automatic-dynamic regulator control unless (1) expected flow-rates to the interceptors are estimated before they occur, and (2) an operational bias is introduced that either minimizes overflows from only some of the regulators on an interceptor or favors the timing of overflows from all of the outlets, such as after the initial storm period. A review of the capabilities of digital recording weather radar indicates it has the best potential for estimating rainfall needed for flowrate predictions. Other possible uses for such radars in metropolitan areas were considered, particularly their use as part of urban flood warning systems. The possibility of inducing in-line storage in collector sewers to gain greater flexibility with integrated regulator operation was also considered.
Proceedings of the Conference on—*Stormwater Detention Facilities*	August-82	L. Scott Tucker, Ben Urbonas, William DeGroot, et al.	The objective of this conference was to explore issues, define available technology, identify problems, and identify research needs with regard to the planning, design, operation, and maintenance of stormwater detention facilities. This was accomplished by bringing together the leaders in the field, and thereby merging the researcher, the technical practitioner, and the institutional practitioner to explore the issues. The conference concentrated more on the quantity aspects of stormwater detention than on quality. This is not to diminish the importance of the quality aspects of detention, but, as a practical matter, it was necessary to limit the subject matter so that issues could be explored in depth. At the same time, quantity and quality issues cannot be totally separated. As a result, a meaningful

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			portion of the conference was directed at the use of detention for stormwater quality enhancement. The conference was organized into five major areas. These are: 1) Recent developments, 2) quantity issues, 3) quality issues, 4) institutional issues, and 5) research needs workshops.
Conference Proceedings—*Emerging Computer Techniques in Stormwater and Flood Management*	Oct-Nov-83	Stuart Smith, et al.	Conclusions of the conference were as follows: 1. The rate of price decrease of computer-compatible field instrumentation means much better temporal and spatial resolution is possible for model applications. 2. Models and design applications must adapt to this data environment (e.g. calibration will become a routine part of design). 3. Database management systems are critical for the analysis of the vast quantities of input data becoming available, and for the output from continuous-modeling systems). 4. Color graphics and interactive digital input devices together form the preferred man/machine interface. 5. Modular structured FORTRAN '77 (and updates) will remain our high-level programming language of choice. 6. Serious consideration should be given to UNIX-based operating systems (for 8-bit, 16-bit, and 32-bit and networked systems. 7. When acquiring a microcomputer, buy the cheapest system that will meet your immediate needs only, considering hardware, software, instrumentation, implementation, maintenance and design-time turnaround. 8. Buy immediately. 9. Undergraduate education must adapt to the evolving personal microcomputer environment. 10. University researchers must be given state-of-the-art 32-bit personal computers with hard disks and color graphics (say about $50,000 each). This will ensure that research activities at the graduate level will be applicable when completed. 11. Engineers should receive better training in software engineering. 12. Utilities such as compilers, editors, and the like should be made more engineer-user oriented. The American Society of Civil Engineers should actively influence standards (this presumably would apply equally to the Canadian Society of Civil Engineers!) 13. Hardware and software is evolving too rapidly to hold conferences such as this only once every 3-4 years, especially over the next few years. The next conference should be held within one year at a central location. 14. Distribution of a conference tape is a good idea, and all speakers at the Conference should be required to contribute the software reported in their papers.
Proceedings of an Engineering Foundation Conference—*Impact and Quality Enhancement*	June-86	Ben Urbonas and Larry Roesner, et al.	This book contains the papers presented at the Engineering Foundation Conference, "Urban Runoff Quality," held June 22-27, 1986 in Henniker, New Hampshire. Topics covered include data needs and collection technology, pollution sources and potential impacts on receiving waters, institutional issues, effectiveness of best management practices, detention,

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
Technology			retention, and wetlands. A workshop on research and future activities needs is summarized.
Proceedings of an Engineering Foundation Conference on Current Practice and Design Criteria for Urban Quality Control—*Design of Urban Runoff Quality Controls*	July-88	Larry Roesner, et al.	This book describes current practice in the design of pollution controls for urban runoff. The contents comprise the proceedings of an Engineering Foundation Conference held in July 1988, titled *Current Practice and Design Criteria for Urban Runoff Water Quality Control.* The papers are concerned with the pragmatic, functional design and maintenance of devices that have been demonstrated to work in the field. These include: wet and dry detention ponds, infiltration devices, sedimentation tanks, swirl concentrators, wetlands, and source controls. Three related sessions address receiving water responses to urban runoff, the Water Quality Act of 1987, and institutional issues related to implementation of urban runoff quality control. These proceedings bring together much of the collective knowledge of American and European technology in the subject area. Many of the authors are internationally recognized in their field.
Proceedings of an Engineering Foundation Conference—*Urban Stormwater Quality Enhancement—Source Control, Retrofitting, and Combined Sewer Technology*	October-89	Stuart G. Walesh, et al.	These proceedings contain the papers presented at the Engineering Foundation Conference on *Urban Stormwater Quality Enhancement—Source Control, Retrofitting and Combined Sewer Technology,* held October 22-27, 1989, in Davos Platz, Switzerland. Session topics include institutional issues; stormwater regulations and standards; on-site detention, infiltration and percolation; inlet controls; planning; real-time control; in-systems control; treatment; rehabilitation; and research needs.
Proceedings of an Engineering Foundation Conference—*Stormwater NPDES Related Monitoring Needs*	August-94	Ben Urbonas, et al.	These proceedings *Stormwater NPDES Related Monitoring Needs,* consist of papers presented at the Engineering Foundation Conference held in Colorado, August 7-12, 1994. The Conference brought together 90 experts in the field of urban stormwater management to discuss the current state of the U.S. Environmental Protection Agency's Non-point Pollution Discharge Elimination System (NPDES) regulations related to discharges of urban stormwater, and the monitoring requirements under those regulations. The objective was to summarize the current state of stormwater monitoring with respect to meeting these regulatory requirements, and to lay out an agenda for the future. Technical sessions included: 1) An overview of stormwater monitoring needs; 2) locating illicit connections; 3) system runoff characterization; 4) NPDES compliance monitoring; 5) policy and instutional issues of NPDES monitoring; 6) BMP monitoring for data transferability; 7) monitoring receiving water trends; and 8) stormwater and best management practice (BMP) monitoring. A major conclusion reached by the conferees was that existing monitoring requirements will not yield the information necessary to determine impacts on the environment or to evaluate the effectiveness of BMPs.
Proceedings of an Engineering Foundation Conference—*Effects of Watershed Development and Management on Aquatic Ecosystems*	August-96	Larry Roesner, et al.	These proceedings, *Effects of Watershed Development and Management on Aquatic Ecosystems,* comprise papers presented at the Engineering Foundation Conference held in Snowbird, Utah, August 4-9, 1996. The conference brought together 75 experts in the fields of environmental sciences and engineering to discuss problems, solutions, and issues associated with the preservation, maintenance, and re-establishment of ecosystems in urban areas. Technical sessions included: 1) Approaches to expanding monitoring beyond water quality; 2) advances in using toxicity bioindicators in

REPORT TITLE	DATE PUBLISHED	AUTHOR(S) or EDITOR(S)	ABSTRACT
			urban aquatic ecosystem assessment; 3) effects of watershed development in hydrology and aquatic habitat structure; 4) impacts of watershed development of aquatic biota; 5) consequences of watershed development on stream morphology; 6) watershed development effects in arid and semi-arid regions; and 7) management and institutional issues. The papers and presentations demonstrate that management of urban water resources embodies a diverse set of scientific disciplines and engineering issues. But they also reveal that no template currently exists for integrating these disciplines and issues into municipal programs for urban water resource management. These proceedings do, however, establish the basis for building an integrated approach to development of sustainable urban water resource programs that embody the appropriate scientific and engineering considerations, along with the institutional structure to implement the program.
Proceedings—*Sustaining Urban Water Resources in the 21st Century*	September-97	A. Charles Rowney, et al.	These proceedings, *Sustaining Urban Water Resources in the 21st Century,* contain papers presented at the Engineering Foundation Conference held on September 7-12, 1997, in Malmo, Sweden. This conference brought together 90 international experts in fields including engineering, sciences, planning, land development, and landscape architecture to discuss problems, issues, and solutions associated with the development of stormwater management programs that truly sustain and enhance the urban water environment. Most technical sessions focused on case studies where a municipal or private development project resulted in clearly demonstrable improvement in an urban water resource. Other papers addressed the science and technology of urban runoff flow and quality management practices. These proceedings establish a basis for development of sustainable urban water resources program that embody the appropriate scientific, engineering, cultural and institutional structures to implement the programs.
Proceedings of an Engineering Foundation Conference—*Linking Stormwater BMP Designs and Performance to Receiving Water Impact Mitigation*	August-01	Ben Urbonas	*Linking Stormwater BMP Designs and Performance to Receiving Water Impact Mitigation* consists of papers presented at the Engineering Foundation Conference held in Snowmass, Colorado, August 19-24, 2001. It brought together professionals of many disciplines to discuss and debate the linkages between various BMP designs and their performance and ability to mitigate receiving water impacts of urbanization. Specific areas addressed included urban watershed trends; regulatory and institutional perspectives; what is known about impacts of urbanization on receiving waters; BMPs and linkages to in-stream integrity; need for and examples of in-stream controls and habitat enhancements; policy issues related to zero and de-minimus impact development policies; design for sustainable water resources; information and monitoring needs to evaluate impacts mitigating potential of BMPs; and experience and science outside the United States.

Table 1-2

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[7] Research failed to provide UWRRC rosters over time. Consequently, this list includes authors of UWRRC publications, task committee members, and UWRRC-sponsored conference attendees who participated in written products from the conference. The Republication Committee regrets the omission of any members. Companies and titles listed in this table are as they appeared in the relevant documents, and they may not be current (as of 2005).

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Chapter 2

ENGINEERING FOUNDATION RESEARCH CONFERENCE ON URBAN HYDROLOGY RESEARCH

August 9-13, 1965
Proctor Academy
Andover, New Hampshire

1.0 INTRODUCTION

The first Engineering Foundation Research Conference on Urban Hydrology Research was held at Proctor Academy, Andover, New Hampshire, from August 9 to 13, 1965. It was organized and co-sponsored by the Urban Hydrology Research Council of the American Society of Civil Engineers.

Since urban hydrology is more than a narrow, technical, engineering subject, the conference represented a variety of disciplines. The first day dealt with the broad social, political, financial, legal, and overall planning aspects. The second and third days were devoted to the hydrology and hydraulics of storm drainage. The fourth day returned to the broader considerations from the federal, state, and local viewpoints. Friday morning summarized the conference.

2.0 MAJOR FINDINGS - NON-ENGINEERING

Major threads clearly noted throughout the conference were:

1) The high economic, legal, social, political, and technological importance of urban stormwater drainage in the many problems posed by the continuing urbanization of our society. In the next 35 years we will, in effect, have to duplicate the entire infrastructure of the urban United States.
2) Research covering the problems and needs of the non-engineering aspects of urban storm drainage is as urgent and necessary as that required for the engineering problems and needs of urban storm drainage.
3) Cooperation and coordination of the several disciplines involved must be promoted and improved. The engineer, city planner, lawyer, administrator, politician, economist, sociologist, financier, and land developer each have a proper interest in urban storm drainage.
4) There is need for leadership and support for work in the field of urban hydrology research. The American Society of Civil Engineers, through its Urban Hydrology Research Council, has great opportunity in this regard. The Engineering Foundation, in sponsoring conferences such as this, can stimulate and bring out leadership and sustain it by similar conferences dealing in depth with various facets not possibly treated thoroughly in an initial, broadly conceived program.

5) The new Federal Cabinet Department of Urban Affairs could provide a logical "home" for research and development support in urban hydrology and for intelligent application of known information.

6) Planning can relate land-use to resource management and thus include urban hydrology in its work.

7) Stormwater should be considered a "resource out of place." Its handling, storage, use, and disposal must consider its quality as well as its quantity and rates at which gathered, transmitted, used, or disposed of. There is need for "fresh" thinking. This is exemplified by the proposal to develop underground (rock tunnel basins) storage and possibly associated pumped storage peak power generation in Chicago. Other examples include recharge of groundwater aquifers, treatment and reuse of stormwater, use of floodplains for both storing water and recreation, and consideration of the aesthetics of design. In other words, multi-purpose planning should be used.

8) Legal considerations need research to develop solutions to the lack of uniformity in existing state law, which is divided and often mingled among common, civil and "reasonable use" rules. The law can provide means for minimizing conflicts with: a uniform drainage code; a uniform trial procedure; and enabling legislation for effective organizing, governing, administration, and financing of drainage districts.

9) Organizationally and administratively, the smallest desirable drainage unit in any case should include the entire physical watershed. Drainage, as with many other urban problems, needs to be handled at the proper level of organization. Research appears desirable as to when the political and administrative organization should be at the national level, megalopolis, the multi-state-basin, states, counties, cities, drainage or sewer districts and basin administrations within these political units.

10) The problems of equitable cost and benefit sharing emphasized the widely varying practices concerning cost and benefit-sharing between state and local governments, between urban and suburban communities and between the developer and the local public agency. Critical study of present practices and research into improved hydrology and the economics involved could reduce the present chaos and increase public satisfaction with the results.

The great difficulty in dollar evaluation of the intangible benefits (or the damages avoided) has resulted in no realistic cost-benefit studies having been made in urban storm drain design. As a result, it is suggested that the economic approach might well be that increasingly evident in other areas of public expenditure involving significant intangible benefit items, namely, a determination of the goal desired by the public and development of a least-cost solution. For example, a properly informed public could be presented with several alternatives. Starting with a just-adequate design, there would be presented successively more complete solutions, including provisions for multiple-use of the stormwater, open space, recreation, floodplain zoning, aesthetics, etc. Their willingness to select something more than a high frequency of overcharge recurrence, i.e., a just-adequate design might more properly reflect the intangibles and unquantifiable items. Some conferees hold the opinion that it still is desirable to attempt to evaluate and quantify intangible benefits; research how to do so is indicated. Also strongly suggested are the needs for psychological and social research in methods of understanding public attitudes and gaining public support.

3.0 MAJOR FINDINGS - ENGINEERING

The foregoing all bear on the non-engineering aspects of storm drainage. The following summarizes the engineering matters:

1) For realistic determination of inputs to a storm drainage system and performance of the actual drainage system, analyses of related rainfall-runoff phenomena over a wide range of conditions are necessary. A comprehensive, well-formulated data collection program is a high priority need. The objectives (which depend upon what will be done with the data ultimately) as now visualized, include data for straight statistical (multiple regression or multi-variate) analysis; unit hydrograph or linear systems studies; and non-linear systems studies. Areas gauged should include not only the small, relatively simple ones, but also the large, interconnected, complex systems. Knowledge of storm pattern and its effects is considered among our most important research areas. A systematic attack on the problems of rain gauge catch and other factors, which are necessary to understanding storm behavior, should be undertaken. Dense gauging networks need to be established in urban areas from which it may be possible to derive constants to be used in a generating function that could be sent through a "black box" which would turn out either storms or hydrographs.

   High quality data need to be collected by or under the supervision of a single organization, over a wide range of urban conditions in a similar manner, preferably in digital form, and processed and reported in a similar manner through a storage and retrieval center. Data presentation should include precipitation and runoff volumes for each storm, hydrograph peak and precipitation intensities for each storm, and selected important hydrographs for each watershed.

   To collect satisfactory rainfall-runoff data, instrumentation needs research, particularly to give accurate information distinguishable to a fraction of a minute from the very small areas. More reliable and accurate flow gauging devices for use in existing storm drainage facilities are desirable. Research should be undertaken to develop a practical means of reflecting antecedent precipitation (or changes in soil moisture or infiltration capacities).

   The foregoing suggests the desirability of a pilot study to develop the best criteria for selecting an existing urban area to be gauged, determining the significant parameters to be measured (physical characteristics of the watershed and the collecting system of drains and appurtenances), and developing the most practical, suitable, and economic instrumentation.

Following such a pilot study that might take two to three years, a variety of urban areas (geographically, climatologically, hydrologically well distributed) would be requested to cooperate under central direction. Related rainfall-runoff, water quality and any other data found desirable in the pilot study should be gathered in a central repository and be readily available to researchers.

2) To the collected data there should be applied all types of research approaches: (a) theoretical; (b) experimental; (c) applied; and (d) statistical. The theoretical approaches seek fundamental understanding of the physical processes or the natural laws that affect rainfall, the input and runoff, and the output. Lacking theoretical approaches based on fundamental principles, and lacking urban rainfall-runoff data (especially for short time intervals and small areas) to develop overall empirical relationships, experimental approaches are being pursued. Experiments are designed to test hypotheses, ranging from fundamental understanding, to fluid flow in rigid pipes, to strictly empirical relationships and calibrations. Because of the numerous classes of hypotheses (microscopic and macroscopic, linear and non-linear, deterministic and probabilistic, static and dynamic, hydrologic and hydraulic, etc.), many types of experiments are required. Some are being initiated, such as simulation models on electronic computers, analog models either electrical or hydraulic laboratory models, field prototype experiments or field experiments using man-controlled rainfall. No single experiment could be designed on an all-purpose basis. The urban drainage system can be considered as two parts: (a) the hydrologic part (essentially the hyetograph of rainfall as modified by the various abstractions, interception, evaporation, depression storage, infiltration), and (b) the hydraulic part (the overland flow, gutter flow, channel or conduit flow and their attendant storage effects). Knowledge concerning the hydraulic behavior of drainage systems is more advanced than the hydrologic behavior of inlet areas. There is need for developing an adequate body of urban hydrologic and hydraulic experimental data.

The applied research approach, particularly the so-called rational approach to storm drain design (first developed in 1889), aims at producing visible results quickly. Empiricisms conditioned by judgment, guide the practical application of the widely used (and generally criticized) "rational method." For small areas there are differences of opinion as to the adequacy of the method. Some feel it is adequate if properly interpreted C (coefficient) values are used. Most generally agree that the "rational method" is inadequate for large areas.

Another approach is the statistical, which, in its purest form, bypasses understanding, including rainfall-runoff relationships, by relating magnitudes of runoff to expected frequency. Statistical procedures are powerful tools for guidance of experiments, interpretation of data and testing of hypotheses. Such statistical approaches must be based on time-invariance and prodigious amounts of data to define the sewer flow-frequency relationships for many combinations of climatic regions and urban characteristics.

3) An initial, high priority, early objective of research is an improvement in current storm drain design methods and techniques, since large investments are continually being made in urban storm drainage; whereas, much of the desirable research will not produce practical results for some time to come. Obsolete practices should be discouraged.

4) Hydraulic structures in storm drainage systems need study from both a structural and hydraulic point of view, such as manholes, junctions, transitions, inlets, outlets, culverts, pipes, or conduits, energy dissipaters, channel linings, etc.

5) Successful urban drainage depends on realistically routing the hydrologic inputs into and through the conveyance system. Among the hydraulic research needs are:

    a) Stability of open channels under erosive velocities.

    b) Supercritical velocity problems, such as energy dissipation, cross-waves due to sinuosity, and junctions.

    c) Water-holding techniques to delay runoff and diminish peaks (roof-top holding, lawn retention, parking lot pondage, underground detention, basins, etc.).

    d) Unsteady flow in shallow channels.

    e) Improved highway and street gutter efficiencies (what is a reasonable and safe width of flow? optimum cross-slope? etc.).

    f) Handling of drainage and erosion problems during construction.

    g) Erosion and sedimentation problems: research is needed on critical tractive force, bed and suspended load characteristics, manner in which sediments are laid down in pipes and structures, effect of sediment on transport capacity of pipes, local scour and flow patterns, effects of erosion on hydraulic structures, design criteria relative to erosion and sediments. Primary need is for collection of data and field observations to identify the problems.

    h) Water quality variation with time position in the hydrograph of combined flows, i.e., does only the first flush present a pollution problem? Could holding basins for the first flush minimize polluting storm overflows?

    i) Research needs in storm flood routing by groups of problem(s):

        - Comparison of physical and mathematical waves. The research need is to determine how well the existing quasilinear hyperbolic partial differential equations of mass conservation and momentum for gradually varied unsteady free surface flow (Saint-Venant equations) approximate the physical waves under various conditions.

        - Branching-type and closed-type systems of storm drainage. The dendritic system is most frequently used where topography permits good grades. The "closed system" involves thoroughly interconnected drains, usually in lowland, flat areas. There is need for research from a consideration of unsteady free-surface movement through these two systems in order to answer the question: whether and under what conditions is a closed system more feasible than the branching system?

- Dynamic discontinuity in sub-surface storm drainage evidenced by secondary waves and pulsations is worthy of investigation. Manholes, junctions, and air circulation are principal sources of disturbance.
- Comparison between small laboratory models and prototypes is desirable, since many secondary problems (secondary currents, pulsations, propagations of small disturbances, air movements at the top of drains during maximum flow) are not easily investigated on small-scale models.
- There is need for developing simplified methods of flood routing based on the two partial differential equations.
- In addition to the need for research in the hydraulics of junctions of large storm drains (manholes not included in this use of the term "junction"), research is required in applying the computer for solving wave propagation in a system of a large number of storm drains and their junctions, especially with backflows.
- Initial and boundary conditions for integration of unsteady free-surface flow equations should be studied to determine how to incorporate these conditions into solutions of the two partial differential equations.
- While analog computers for flood routing, based on the simple input-storage-output approach have been tried, and many are operational, there is need for research and development of analog computers based on the two De Saint-Venant partial differential equations. Each large and complex storm drainage system, either existing or planned, should be currently studied on an individually designed analog computer.

6) The conference did not discuss the problems of cold weather, but research is desirable on those problems associated with storm drainage, such as the effects on drainage of anchor ice, frazil ice, etc.

As above indicated, there is strong need for a broad, comprehensive research program in a wide variety of the problems associated with urban hydrology. The Gordon-type conference under the sponsorship of the Engineering Foundation and the Urban Hydrology Research Council of the American Society of Civil Engineers, in the attractive and favorable environment of New Hampshire's summer charm, resulted in a maximum of interchange of ideas among the eighty-three conferees. It is expected that from this, the first conference on urban hydrology, specific study and fruitful research will develop.

Chapter 3

ASCE COMBINED SEWER SEPARATION PROJECT PROGRESS

October 16-20, 1967
ASCE National Meeting on
Water Resources Engineering
New York, New York

1.0 INTRODUCTION

This paper is a progress summary for the period from the commencement of the project in February 1966 through August 1967. (Please see Chapter 15 for later summary of this research project.)

The general concept for separation of combined sewerage systems on which the ASCE project is based involves pumping comminuted or ground sewage from individual buildings and/or building complexes through relatively small pressure tubing. The tubing would connect to new pressure conduits located outside the buildings. These new, separate sanitary sewage pressure conduits would then discharge into existing interceptors that would convey the sanitary sewage to treatment works. Stormwater alone would be carried in what were formerly combined sewers. Dr. Gordon M. Fair of Harvard University is the originator of the concept under investigation.

The project is wholly supported by the Federal Water Pollution Control Administration (FWPGA), Department of the Interior. For the first eight months, support was under Demonstration Grant WPD 104-01-66. Since then, support has been under Contract No. 14-12-29. Mr. George A. Kirkpatrick is FWPGA Project Officer on the contract and provides valuable technical support.

The ASCE Urban Hydrology Research Council (predecessor of the UWRRC) assisted significantly in the pre-proposal stage of the project. A steering committee provides general guidance and direction for the project staff at bi-monthly meetings. Dr. Gordon M. Fair is Chairman, and the other members are: Mr. Vinton W. Bacon; Dr. Morris M. Cohn; Dr. John C. Geyer; Mr. Richard Hazen; Mr. Martin Lang and Mr. S.W. Steffenson. ASCE Executive Secretary, W.H. Wisely, is legally responsible for contractual arrangements and closely follows the general progress of the project. ASCE Research Manager, Donald C. Taylor, provides continuous administrative supervision. Through the good graces of Dr. Fair, the project staff office is located at Harvard University. Staff members are Mr. Donald H. Waller; Mr. L. Scott Tucker and the writer, who have worked a total of 45 man-months on the project through August 1967.

General information on the ASCE Project is given in "Outline Description of ASCE Project on Separation of Sanitary Sewage from Combined Systems of Sewerage," Technical Memorandum No. 1, Combined Sewer Separation Project, ASCE, 345 E.

47th Street, New York, NY 10017, February 21, 1966; and *Civil Engineering*, November 1966, pp. 79-80.

2.0 PROJECT MANAGEMENT APPROACH

The basic objectives of the project are to determine the physical feasibility and limitations of the project scheme, or modifications thereto, and to arrive at measures of cost for comparison with the conventional method of separation, for evaluation of comparative investment attractiveness. The underlying issue is the feasibility of a pressurized sanitary sewerage system. Little in the way of helpful precedent exists. Equipment assemblage of compatible materials, methods of installation and maintenance, design criteria, and even fundamental data, mostly must be developed from scratch.

From the outset, the project management approach has been to maximize assistance from organizations and individuals who have developed a mastery of specialized subjects relating to the project. Suffice it to say that various levels of assistance are being provided by over 100 individuals from almost 50 organizations, including officials of municipal water pollution control agencies, representatives of major manufacturers, members of consulting firms and staff members of trade and professional associations.

The superb and extraordinary cooperation and assistance being received is clearly a consequence of ASCE responsibility for the project. Most of the people involved are members of ASCE and have a sincere desire to be of personal help to the Society. Because ASCE gave generous publicity to the project in its own publications and literally saturated all other engineering media with information, nearly all technical people, regardless of specialty, were well aware of the project scheme and goals when first approached.

It has been carefully planned that each major facet will have been sufficiently investigated so that the results will be of use in solving associated problems. For example, basic criteria and devices for pressurized sanitary sewerage systems in new construction will be resolved as a bonus feature without diverting from the central mission of the project on combined systems. Everyone involved is determined that the "spin-off" benefits of the investigation will be as great as practicable within limitations of time and funds.

3.0 POTENTIAL ADVANTAGES AND LIABILITIES

Potential, obvious advantages of the project scheme include preclusion of stormwater and most groundwater infiltration from interceptors and treatment plants, thereby effectively increasing the hydraulic capacity of these facilities. While traditional separation effects a diversion of stormwater from interceptors and treatment works, groundwater infiltration continues to burden the sanitary sewage flow throughout the system. Further, there may be interference with traffic and commerce for a pressurized sanitary sewerage system.

Last year (1966), water pollution control officials were interviewed in the 11 largest U.S. cities having fully combined or partly combined sewer systems. One of the objectives of the interviews was to elicit viewpoints on potential liabilities of the project scheme, and the city officials volunteered a number of important opinions, suggestions and positions, all of which have substantially contributed to the reference background of the ASCE project. A summary of the most relevant, most frequently made comments may be of interest.

Complete, conventional separation of a combined sewerage system is a clearly definable objective only when the combined sewers are of adequate capacity for at least contemporary storm drainage requirements. In several cities there are extensive areas served by combined sewers where storm drainage capacity is deemed inadequate. Although in many cases some degree of inadequacy arises from design standards upgraded since World War II, it is evident that new land use practices, not anticipated when the sewers were designed, are a major contributing factor. In these instances, the question of separating sanitary sewage is inextricably bound with needs for stormwater flooding relief.

Virtually all officials noted that the ASCE project scheme, as in conventional separation, would not have any beneficial effect on the quality of stormwater discharged to receiving watercourses after conversion from combined to separate storm sewer service. Current national interest in abatement of pollution from sewerage systems has extended from concern for diversion of all sanitary sewage to treatment plants to include reduction in pollution from storm drainage. The reservations of city officials on conventional separation and directly comparable alternatives are echoed in *Environmental Pollution, A Challenge to Science and Technology Report*, of the Subcommittee on Science, Research, and Development to the Committee on Science and Astronautics, U.S. House of Representatives, 89th Congress, Second Session, 1966:

> *The separation of storm and sanitary sewers has been recommended but recent evidence indicates that the contamination from streets, sidewalks, and city surfaces would make the runoff from rains quite a pollutant to receiving waters, even if it did not contain sewage. Therefore, the very expensive reconstruction of city sewer systems would not yield comparable increases in quality in the receiving water.* (p. 13)
>
> *Evidence is now accumulating that the separation approach would have very little benefit for the quality of receiving waters even if it could be accomplished.* (p. 28)

Combined sewer outfall detention basins with treatment capability are of current paramount interest because they have the potential of reducing the pollution from stormwater overflows without reconstruction of sewers, at possibly much less attendant costs.

Officials in one city noted that, little by little, major cities are gradually being rebuilt, separate sewers are generally provided in nearly all new construction, and in another 50 years or so, most existing combined sewer systems essentially will have been separated. The virtue of a crash separation program under the constraints of prevailing and foreseeable land use was, therefore, questioned.

Several officials recognized that the problems for which solutions are sought have not been adequately defined.

> *More research is needed to develop understanding of the whole stormwater pollution problem. This research should cover the hydrology, the hydraulics, the treatment, the effects on recovery waters, and related factors.* (National Academy of Sciences, National Research Council, Publication 1400, Washington, DC, 1966, p. 170)

One version of the ASCE project scheme incorporates the discharging of ground sanitary sewage from individual buildings and building complexes through relatively small pressure tubing laid in existing building connections, and then, into new pressure conduits suspended in existing street combined sewers. The separate sanitary sewage pressure conduits would then discharge into existing interceptors that would convey the sanitary sewage to treatment works. Although some city officials conceded that feasible and acceptable methods and devices might be developed for attaching pressure conduits inside walk-through size combined sewers, virtually all expressed firm reservations on non-walk-through size sewer applications. Although development of suitable installation techniques for non-walk-through sewer applications was recognized as a formidable problem, objections were mostly with respect to maintenance, repair and replacement. Particularly: the possibility that inserted conduit and tubing would aggravate accumulation of debris; the potential interference of the intruded services with free movement of sewer cleaning devices; and remote accessibility of conduit, tubing and connections for repair, connecting new building services and inevitable long-term replacements. Although major skepticism was with regard to pressure conduits in non-walk-through sewers, several were also opposed to the presence of ¾-inch, to 1¼-inch I.D. plastic tubing in a building sewer because it was anticipated that clogging from roof debris would be accentuated, and the tubing would interfere with cleaning and tree root cutting devices.

Several officials were concerned about ownership of building grinder-storage-pump units, and enumerated some advantages and disadvantages of municipal versus private ownership. Repeatedly, reservations were stated on the attainability of adequate homeowner acceptability, and the related aspects of maintenance and provisions for overloading conditions. In particular, serious concern was expressed on adequate prevention of backflow into buildings and protection from overflow in buildings from household grinder-storage-pump units during power outages.

Several cities have made considerable progress on their FWPCA facilities demonstration grant projects since they were visited in 1966 by the ASCE project staff. Accounts of such work in Chicago, Detroit, Milwaukee, Minneapolis, New York, Boston and Cleveland are summarized in a recent paper, "Current Developments in Separate versus Combined Storm and Sanitary Sewage Collection and Treatment," by John J. Cannon and Larry Streck, School of Public Health, the University of Michigan, Ann Arbor, Michigan, a paper presented on June 19, 1967 at the 42nd Annual Conference of the Michigan Water Pollution Control Association, at Boyne Mountain Lodge, Michigan.

4.0 GRINDING AND PUMPING

Relatively small building service tubing or conduit are required for a sanitary sewerage system where pressurization is attained within individual buildings. Because of the relatively small diameter of such conveyances compared with gravity sanitary sewers and anticipated need for system control valves and service fittings, all of which would tend to instigate solids blockages, it is currently considered that particle size reduction of sewage solids prior to pressurization is a mandatory requirement.

Commercially available comminutors appear suitable for large buildings, using standard sewage lift pumps with wet wells for pumping modulation. Thirty-six existing comminutor installations serving large buildings and building complexes are being monitored to obtain a record of their operating history. These installations were located with the assistance of the Chicago Pump-Hydrodynamics Division of the FMC Corporation. A full year of record will have been accumulated on most of these by late 1967. Interim findings are included in "Experience with Grinding and Pumping of Sewage from Buildings," by D.H. Waller, ASCE Combined Sewer Separation Project, Technical Memorandum No. 3, May 1, 1967, and terminal findings are to be reported in a subsequent technical memorandum. Findings include: "Average frequency of jamming for all machines is about once in six months." "Sharpening or replacement of cutting elements at most installations is infrequent."

Late in 1966, the General Electric Company, in cooperation with the ASCE project, initiated operation of two special household observation stations in Louisville, KY. Both stations incorporate a grinder, storage tank, pump, about 50 feet of small diameter tubing (¾-inch and 1-inch I.D.), a composite sampler, devices for determining rates of incoming sewage flow, and a special remote-reading water use device mounted on the meter in the house. Extensive data were acquired over a period of several weeks, including laboratory analyses of effluent sewage, sewage flow rates and water demand variations. The installations are described in "Sampling and Analysis of Wastewater from Individual Homes," by R.P. Farrell, J.S. Anderson and J.L. Setser, General Electric Major Appliance and Hotpoint Division, Report 67-MAL-3, Appliance Park, Louisville, KY, March 24, 2967. Supplementary information on sewage grinding is included in ASCE project Technical Memorandum No. 3, cited above.

The Johns Hopkins University made available water demand data for individual homes originally obtained by the U.S.D.A. (see Chapter 6, herein). Analysis of this data, plus four weeks of water demand data from each of the two G.E. Louisville observation stations, is reported in "Sewage Flow Variations in Individual Homes," by L.S. Tucker, ASCE Combined Sewer Separation Project, Technical Memorandum No. 2, February 24, 1967. The data on sewage flows from the G.E. stations have not been completely reduced, but a technical memorandum is forthcoming that will correlate sewage flow and water demand rates.

Late in June 1967, the R&D Center of General Electric at Schenectady, NY, commenced work on a subcontract with ASCE for advanced development of a special household grinder-storage-pump unit. The unit will be designed suitable for assembly line, large volume manufacture, under which conditions the fob cost is hoped to be less than $250 per unit. One design version being considered would incorporate the grinding features of a commercial garbage grinder. Among the functions to be provided is a secure backflow prevention capability. Optimal delivery pressure range would be 5 to 35 psi. Assuming a successful development program, about a dozen prototype units are to be delivered for a field demonstration in June 1968.

The duration of the field demonstration would be about one year. About a dozen prototype household units, perhaps augmented by a comminutor-pump or other larger facility, will be installed in residences on one side of a street in the same block. All units will be manifolded to a common pressure pipe running parallel with the street. A special automatic control valve at the pipe terminus will be programmed to vary the street sewer pressure with time, in accordance with expected variations for a pressurized system. Trouble-detection sensors and operation-logging devices will be provided. The major purpose of the field demonstration is a shakedown proving of the units as a conditional prelude to their later use in any full-scale demonstration by others.

One version of the project scheme is to insert tubing in each building sewer, between the grinding-storage-pumping unit and the combined sewer in the street, and connect the tubing to a pressure conduit suspended in the combined sewer. Several facets of this possibility have been explored or are currently under investigation.

Small tubing, ¾-inch through 1¼-inch I.D., was both pushed and pulled through a building sewer in Washington, DC, with extensive help from the District Department of Sanitary Engineering. Polyethylene, polybutylene and soft copper tubing were employed. The soft copper was completely unsatisfactory and the polybutylene could be pulled, but not pushed, through the building sewer. The polyethylene tubing was successfully both pushed and pulled through the 86-foot length of 5-inch C.I. building sewer, which had three 45° ells. For the pushing tests, the tubing was passed from inside the building towards the street sewer. For the pulling tests, advantage was taken of a 4-foot combined sewer in the street, which permitted free access at both ends of the building sewer.

Connecting tubing to pressure conduits suspended in walk-through sewers is an entirely simpler matter than for non-walk-through sewers. The extent of each existing size range is, therefore, important. The Portland Cement Association made a special study for the project without charge: "Study of Approximate Lengths and Sizes of Combined Sewers in Major Metropolitan Centers," ASCE Combined Sewer Separation Project, Technical Memorandum No. 4, May 1, 1967 (see Chapter 4 for copy of this study). The general findings were that an average of about 85% of the total length of combined sewers in major cities have an interior clear height of 48 inches and less; an average of about 72% of the total length of combined sewers in major cities have an interior clear height of 24 inches and less; and for smaller cities, the proportion of larger sewers is even less.

In August 1967, the Johns-Manville Company initiated work under a subcontract with ASCE to develop a hanger system for suspending a pressure conduit, or conduits, in a combined sewer. Development of suitable materials and techniques for attaching the hangers and provision for resisting thrust from unbalanced internal pressures are among their major objectives. After the hanger system has been developed and proven structurally competent, it is planned for others to make a field installation to demonstrate the durability of the hanger assembly and the restrained pressure conduit.

Under a subcontract between the University of Illinois and ASCE, Dr. J.M. Robertson is conducting laboratory and analytical studies of a general nature, which will give a reasonable indication of surcharged capacity reduction occasioned by the presence of a pipe inside a combined sewer. Preliminary results indicate, for example, that under surcharged conditions a 12-inch O.D. pressure conduit in a 6-foot I.D. combined sewer would result in a decrease in flow capacity of slightly less than 5% with the conduit located against the sewer, and slightly more than 12% with the conduit suspended at the sewer centerline (the limiting distance the conduit can be away from the sewer wall). The results of these studies will be of importance in planning the field installation mentioned in the preceding paragraph.

Returning to non-walk-through size sewers, at the time of this writing (October 1967), the ASCE project staff has not been able to identify a feasible, practicable method for installing a pressure conduit in non-walk-through combined sewers and effecting simultaneous connections of tubing from individual buildings to the pressure conduit without extensive deep excavation. However, one alternative under consideration is to lay the pressure conduit in a shallow trench excavated to a depth just below the frost line, with a similar shallow trench normal to the conduit trench to carry the connecting tubing from the building. The conduit trench might be located behind the curb line or, in the extreme, in the street cartway. Construction would closely conform to traditional water distribution system construction for which there is an abundance of precedent. Other possibilities are being sought.

Tentatively, a dual pressure conduit configuration appears desirable, one conduit in each side of the street where local service connections must be accommodated, to facilitate cleaning, maintenance and rehabilitation without interfering with service upstream. The shallow trench-type of pressure conduit installation is immediately compatible with a dual configuration.

The National Sanitation Foundation, under a subcontract with ASCE, has studied fittings, connections, materials, cleaning techniques and other aspects of tubing and conduits for a pressurized sewer system. The Foundation advised and assisted in the pushing and pulling of tubing in the Washington field trials, above, and in a special installation in Washington of a tubing and conduit connection.

As a part of the determination of physical feasibility and cost attractiveness of the project scheme, three pressurized sanitary sewer systems are being designed. These are hypothetical pressure systems, for study purposes only. The use of actual drainage areas provides a realistic basis and a degree of sampling for evaluation. Considerable detailed data are being made available on the following combined sewer study areas: a 60-acre commercial area in downtown Boston (see Appendix A), a 160-acre residential area in Milwaukee (Appendix B), and a 330-acre residential area in San Francisco (Appendix C). The Public Works Departments of the three cities have made these studies possible through their extensive assistance. The consulting firm of Camp, Dresser and McKee has provided considerable supplementary physical data on the Boston study area under a service contract with ASCE. For the San Francisco study area, the consulting firm of Brown and Caldwell will evaluate plumbing revision considerations on private property, review pressure sewer system designs prepared by the project staff and prepare comparative estimates of cost for pressure systems versus conventional separation, under a service contract with ASCE. Similar assistance will be sought for the other two study areas.

Design criteria for minimum transport velocities of sewage in pressure pipes (see Chapter 5 for more information) was not available. Special research was conducted at the Central Engineering Laboratories of the FMC Corporation in Santa Clara, CA. Raw sewage, with and without particle-size reduction by comminution, was passed through a 2-inch, 3-inch, 4-inch, 6-inch and 8-inch I.D. pipe. Limited tests were also made with an 8-inch spiral corrugated pipe and exploratory open channel tests were made with the plain 8-inch pipe. Extensive observation indicated, rather conclusively, that the material in the sewage which was the last to be scoured and the first to be deposited was predominantly sand. For all tests, the sewage was salted with ground egg shells, but these were always moved at lower mean flow velocities than the sand, which was in low concentrations, viz., 8 to 78 ppm. No discernable difference was noted in the minimum transport velocities for comminuted and uncomminuted sewage. The minimum scouring velocities tended to be greater than the maximum depositing velocities, but the difference was small and almost inconsequential. As a first approximation, the minimum transport velocity is $\sqrt{D/2}$, where D is the interior pipe diameter in inches (plain pipe, full flow), for a velocity in feet per second; e.g., the minimum velocity for a 4-inch pipe is approximately 1.0 fps. More precise values require accounting for sand concentration, and comprehensive reports in preparation will set forth pertinent details, including correlation with research findings obtained elsewhere on transport of sand in pipelines.

For the two residential combined sewer study areas, use has been made of data from The Johns Hopkins University. The scope of this work is outlined in "Summary

Report on the Residential Water Use Research Project," by F.P. Linaweaver, Jr., J.C. Geyer and J.B. Wolff, J. AWWA, March 1967, pp. 267-282. Raw, hourly water demand data for a non-lawn-sprinkling period for groups of 44 to 410 homes were generalized for the Milwaukee and San Francisco regions. Envelope curves developed from the data are being used as follows: the lowest peak hour rate of any day for a given number of residences is combined with the minimum transport criteria, described above, to size street pressure conduits; and the highest peak hour rate of any day for a given number of residences is taken to estimate maximum hydraulic gradients. Data from individual homes, described earlier, was useful in establishing lower limits of the design curves. For both the Milwaukee and San Francisco systems, local winter season domestic water demand data are used as a base of reference. For the Boston commercial land use combined sewer study area, local field data from the firm of Camp, Dresser and McKee are being used, augmented by selected data from "Water use in Selected Commercial and Institutional Establishments in the Baltimore Metropolitan Area," by J.B. Wolff, F.P. Linaweaver, Jr., and J.C. Geyer, The Johns Hopkins University, Baltimore, MD, June 1966.

A pressurized sewer system requires main sewer controlled valves to hold pressures along an upstream system reach within prescribed limits. Mr. J.R. Daneker of Builders Iron and Foundry, Providence, RI, has volunteered to write a design manual for the project on control systems for pressurized sewer systems, pending delineation of operational specifications, which will arise from the studies of the three systems by the project staff.

Arrangements are being sought for a simulation of system performance by computer. Both a gravity and a hypothetical pressure sanitary sewer system would be replicated. Available data on water demand variations in individual homes would be applied to metered winter quarter mean domestic water demands for an area of perhaps 500 dwellings. Output would include frequencies of different levels of flow rates at various points throughout the hypothetical systems.

5.0 OTHER TASKS

In addition to the preceding work, terms of the contract between ASCE and FWPCA include: (a) evaluation of non-mechanical problems, such as private ownership, maintenance and homeowner acceptability of household grinder-storage-pump units; (b) specifications for a possible future demonstration installation by others in an entire combined sewerage drainage area; (c) consideration of solid wastes disposal as an augmentation of the project scheme; and (d) submission of subcontractor reports, project technical memoranda, and a final report.

The possibility of obtaining synergistic benefits by augmenting the project scheme to include solid wastes disposal was recognized early in the program. Research at the University of Pennsylvania is closely related, with a focus on transport of ground solid wastes slurry in a pipeline. Work on the solid wastes aspect has been started.

While consideration has been given to non-mechanical problems and planning for a large demonstration system, there are no significant findings to report at present.

Before the project is completed, about two-dozen subcontractor reports and project technical memoranda will have been written. These will constitute appendices to the final report.

The scope and scale of the national combined sewerage problem deserves mention. The following section suggests the importance of research on potential alternatives to conventional separation of combined sewerage systems, including the project undertaken by ASCE.

6.0 SEPARATION COSTS

Washington, DC, is the largest city in the U.S. involved in an active, traditional combined sewerage separation program. Their actual experience costs were included in "Pollutional Effects of Stormwater and Overflows from Combined Sewer Systems," Public Health Service Publication No. 1246, 1964. The writer found that some major cities had been using per capita or per-square-mile costs for Washington in lieu of detailed local evaluations to arrive at separation cost estimates.

Some cities report estimated total construction cost, others report the aggregate of estimated total construction cost plus debt service, and only a few give a breakdown by categories. Such a disparity in accounting makes comparisons and generalizations extremely difficult.

The general impression is that estimated construction costs for new sanitary sewers and ancillary public work have more often been arrived at from more reliable and realistic data than costs for separating plumbing and drains on private property. From those cost estimates which were based on reasonable field reconnaissance of representative sample areas it appears that the construction cost of separating plumbing and drains on private property might typically be in the range of 40% to 60% of total construction costs, or roughly half. This range is mostly a reflection of different, historical building plumbing codes.

Many combined sewers and even entire combined sewer systems are of inadequate capacity for storm drainage service in some cities. In those instances, segregation of separation costs related to storm drainage flooding relief from those specifically assignable to sanitary sewage conveyance would be extremely difficult.

The 1964 USPHS report states that:

> *These rather crude manipulations of data indicate that total separation costs could amount to $25-$30 billion, or even more.*

On the basis of details from some of the major cities visited, it appears that total construction cost could quite conceivably be as high as $50 billion, with about half of this cost for public sewerage construction and about half for modifications of plumbing and drainage on private property. Any reasonable allowance for debt service and escalation of costs over the period of construction would certainly be on the order of the total construction costs, for a possible grand total of as much as $100 billion. The above very crude estimates are in terms of major cities, and costs per capita or per square mile for smaller cities may well be less, which would temper these estimates downward somewhat.

Rather than invest extensive time and funds on the preparation of comprehensive cost estimates for conventional separation of combined sewerage facilities, most major cities are concentrating efforts on investigations of alternative schemes. Costs for traditional separation are, thus, commonly obtained only for subsystems where alternative means are under investigation, to serve as a basis for indicating effective expected reduction in cost of the alternative over conventional separation.

The objective of a recent American Public Works Association Research Foundation contract with the Federal Water Pollution Control Administration is to obtain detailed information on the combined sewers of approximately 900 communities by onsite personal interviews. The APWA report to FWPCA is due October 15, 1967. Undoubtedly, this report will contribute towards a more precise evaluation of the estimated national cost for traditional separation of combined sewerage systems.

7.0 CLOSING REMARKS

The work of the ASCE project is truly a team effort, with over a hundred people involved. Whatever progress can be claimed is collectively creditable to the project Steering Committee, the ASCE headquarters administrators, the FWPCA officers administering the project for the government, the scores of engineers and others representing cooperating organizations and the project staff.

The writer takes this opportunity to acknowledge sincerely and respectfully the superb, outstanding and invaluable help of the many people who are providing assistance or helping the project in some way; and their number seemingly increases every week. Only where one individual of an organization is a principal team member has his/her name been cited in the paper, for the sake of brevity. Complete acknowledgments will be included in formal project reports on individual phases of the work, and in the final report.

In closing, it is appropriate to cite two reports on related subjects: "Stormwater Runoff from Urban Areas, Selected Abstracts of Related Topics," by Gordon G. Robeck and staff, FWPCA Robert A. Taft Sanitary Engineering Center, Cincinnati, OH, April 1966; and "Separating Storm and Sanitary Sewers in Urban Renewal," House Report No. 1648, 89th Congress, 2nd Session, 32nd Report by the Committee on Government Operations, June 23, 1966.

Chapter 3, Appendix A

REPORT ON PRESSURE SEWERAGE SYSTEM—SUMMER STREET SEPARATION STUDY AREA, BOSTON, MASSACHUSETTS

September 1968
ASCE Combined Sewer Separation Project
New York, New York

1.0 INTRODUCTION

The research upon which this report is based was performed pursuant to Contract No. 14-12-29, with the Federal Water Pollution Control Administration, Department of the Interior. As part of this contract, the American Society of Civil Engineers (ASCE) is studying methods of separating combined sewerage systems in three major cities of the United States. This report is a result of a study of the 50-acre Summer Street Separation Study Area of Boston, Massachusetts. The main objective of this report is to evaluate and compare the conventional gravity separation method (the construction of a new gravity sanitary sewerage system) and pressure methods (the construction of new pressure sanitary sewerage systems consisting of pumping facilities and force mains).

To complete this objective, present and future sanitary sewage flows have been estimated by the ASCE Project Staff. Based upon these estimated flows, the Project Staff prepared three sewer system designs.

Camp, Dresser & McKee, in accordance with its contract with the ASCE, has conducted engineering investigations in two phases as follows:

Phase 1: A study of revisions necessary on a typical building to in-house plumbing for separation of stormwater and sanitary sewage, both for gravity and pressure separation, and including a description of the physical problems involved and construction cost estimates for these revisions.

Phase 2: A review of the layouts for the system of pressure separation of the entire study area prepared by the Project Staff and the preparation of construction cost estimates for the systems.

An undistributed preliminary report on Phase 1 was submitted to the ASCE on February 23, 1968. Its contents have been included in the present report.

In accordance with the letter agreement of February 6, 1968, Camp, Dresser & McKee, in addition to the above described work, has incorporated descriptions of Project Staff design work into its report under Phase 2. The concept of hanging pressure sanitary conduits within existing combined sewers also has been evaluated.

2.0 SUMMARY

Chapter III of this report has been written by Mr. D.H. Waller of the ASCE Project Staff and reviewed by Camp, Dresser & McKee. It describes flow variations in heterogeneous commercial areas of large cities similar to the Summer Street Separation Study Area in Boston. The flow data developed have been used for the design of pressure systems, and are considered reasonable by Camp, Dresser & McKee.

Chapter IV was prepared by Camp, Dresser & McKee and had been submitted as the Phase 1 preliminary report. The problems of separating the combined plumbing in buildings within the study area are covered in detail. The plumbing of a typical existing building, the proposed plumbing for building separation with a gravity sanitary sewerage system; and the proposed plumbing scheme for building separation with a pressure separation scheme are described. Construction cost estimates for separation schemes are presented. The gravity separation cost is estimated at $10,000 per building, and the pressure separation cost is estimated at $20,000 per building. With about 200 buildings to be separated, the cost for the gravity separation is $2,000,000, and the cost of pressure separation is $4,000,000. These costs have been modified slightly to reflect the addition of pressure control valves on each pressure discharge connection.

Designs of pressure separation systems are presented in Chapter V, prepared by Mr. D.H. Waller of the ASCE Project Staff. Average and peak sanitary flows developed for each loading point are tabulated. Based on these flows, three designs were developed. These designs are for two-pipe systems with no pumping station, and one and three main pumping stations. The designs utilize pipes ranging from 3-inches to 14-inches in size in the Summer Street area.

Cost estimates for the components of each ASCE Project Staff design and the conventional system design prepared by Camp, Dresser & McKee are presented in Chapter VI. An alternative scheme using a one-pipe system with no main pumping station has also been evaluated and its cost estimated. The use of main pumping stations to reduce required pipe sizes in the system did not reduce the estimated costs. The total cost estimates of all systems are summarized below.

Systems	Estimated Cost
Gravity	$4,700,000
Pressure – Two pipes without any main pumping station	$6,400,000
Pressure – Two pipes with one main pumping station	$6,630,000
Pressure – Two pipes with three main pumping stations	$7,000,000
Pressure – One pipe without any main pumping station	$6,200,000

All of the above costs are for construction (including engineering and contingencies) only. While the gravity separation system would have minimum annual maintenance cost, the pressure systems would require considerable operating and maintenance cost, because the flows from each building would have to be pumping into the street force mains. Based on reports and data furnished by the ASCE Project, appreciable annual maintenance costs also would be required for pressure systems.

The concept of hanging pressure force mains inside large existing combined sewers has been investigated. Estimates indicate that construction of pipes inside combined sewers would be more costly than the construction of a two-pipe system of comparable size, located in the same streets. In addition, the capacity of the existing combined sewer would be considerably reduced, and maintenance difficulties of both the combined sewer and the hanging pipe system would be severe.

Chapter 3, Appendix B

REPORT ON—MILWAUKEE STUDY AREA

December 1968
ASCE Combined Sewer Separation Project
New York, New York

1.0 SUMMARY AND CONCLUSIONS

A study area north of the business district in Milwaukee, Wisconsin, has been picked by ASCE to study separation of sanitary sewage from stormwater in combined sewerage systems. Construction of a pressure sanitary sewer system has been compared with a conventional gravity system. Particular attention has been given to the changes required and associated costs to accomplish in-house separation.

It is estimated construction of a separate sanitary sewer system in the Milwaukee study area would cost $3.2 million for a pressure system and $2.2 million for a gravity system. This represents a cost, per structure, of $3,800 and $2,600 respectively for the pressure and gravity systems. The cost per acre is approximately $20,000 for a pressure system and $14,000 for a gravity system. Major components of total construction cost are summarized as follows:

	Pressure System		**Gravity System**	
	Cost	**%**	**Cost**	**%**
Collection System	$ 594,000	18.4	$ 1,055,000	48.0
In-House Plumbing	$ 1,214,000	37.6	$ 1,140,000	52.0
Provision of Installation of Grinder-Pump Units	$ 1,417,000	44.0	--	--
Total	$ 3,225,000	100.0	$ 2,195,000	100.0

The cost of in-house plumbing changes would be only slightly higher for a pressure system. The collection system cost for the pressure system would be only about 9/16 that for a gravity system, but this is much more than offset by the added cost of providing and installing grinder-pump units. Even though pressure system components were developed to lesser unit cost levels as a consequence of market demand, it is likely that the costs for pressure systems would significantly exceed the costs for gravity systems.

Aside from cost considerations, in our opinion it would be unwise to embark upon major pressure system projects until experience with pilot installations has demonstrated that operation and maintenance difficulties are not of serious consequence. Although for the Milwaukee test area, the pressure system is estimated to cost much more than the gravity system, it is possible that in other situations the cost relationship will be different.

Chapter 4

STUDY OF APPROXIMATE LENGTHS AND SIZES OF COMBINED SEWERS IN MAJOR METROPOLITAN CENTERS

May 1, 1967
ASCE Combined Sewer Separation Project
Technical Memorandum No. 4
New York, New York

1.0 INTRODUCTION

This study was conducted to establish reasonable estimates on conduit sizes and lengths handling combined wastes. The limited data collection and analysis was performed by Dasel E. Hallmark, Hydraulic Engineer, and John G. Hendrickson, Jr., Water Resources Engineer, Portland Cement Association. This study program was conducted as part of the ASCE Combined Sewer Separation Project in an effort to better delineate basic project goals.

Although the data reported are limited, it is our opinion that they reflect the basic trends of wastewater collection systems. As such, basic designs, construction details, and costs of separating storm and sanitary conduits can be estimated with a greater degree of reliability using these data than was heretofore possible.

2.0 SCOPE OF SURVEY

A letter questionnaire was mailed to 11 major population centers having combined sewers.

In the survey, data were requested on the vertical dimension of the conduits used and mileage of each size conduit. These data were requested regardless of construction material or cross section. An analysis of these data provides the basis of this report. The purpose of obtaining the vertical dimension and lengths of each size conduit is to define the limits of walk-through and non-walk-through sewers. The condition of size greatly modifies the various techniques of converting combined systems to separate ones.

3.0 DATA ANALYSIS

A total of 11 cities were contacted for sizes and lengths of their combined wastewater systems. The data were studied and combined into Table 4-1.

TABLE 4-1

Approximate Lengths and Sizes of Combined Sewer Collection Systems in Major Metropolitan Cities

City	Population 1960 Census	Total Miles Sewer Conduits	Mileage Equal or Less than 48 inches	Mileage greater than 48 inches	Percentage Equal or Less than 48 inches	Percentage Greater than 48 inches	Percentage Equal or Less than 24 inches
New York	7,781,984	3,704	--	--	--	--	--
Chicago	3,550,404	3,595	3228.7	366.3	89.9	10.1	81.6
Philadelphia	2,002,512	1	--	--	--	--	--
Detroit	1,670,144	2,893	2109.0	784.0	73.0	27.0	55.0
Cleveland	876,050	1	--	--	--	--	--
Washington, DC	763,956	1,750	--	--	--	--	--
St. Louis	750,026	1,116	979.3	137.1	87.7	12.3	68.2
Milwaukee	741,320	5502	512.4	37.6	93.2	6.8	74.9
San Francisco	742,855	870	729.4	140.6	83.5	16.5	78.0
Boston	697,197	1,361	--	--	--	--	--
Pittsburgh	604,332	1	--	--	--	--	--

[1] Not available

[2] Study based on ¼ total system

Of the 11 cities, four did not have the requested information readily available. Washington, DC, Boston and New York were able to supply only the gross length of sewers and size range of conduits used. Five major cities supplied an adequate listing of sizes and lengths to develop Table 4-1.

A size of 54 inches in the vertical dimension was selected as the minimum desirable size to classify as a walk-through sewer. A design change of manholes often occurs at the walk-through size to accommodate personnel entry. Therefore, the column for mileage equal to or less than 48 inches (vertical dimension) contains the non-walk-through conduits. As an average, 85% of the combined sewers for large metropolitan areas are equal to or less than 48 inches in the vertical dimension.

Also of primary interest is the percentage of conduits equal to or less than 24 inches. These sizes would constitute the bulk of the collection lines from houses or apartments to the larger trunk lines. An average of 72% of the combined sewers fall into this size category. Also, an average of 13% of the sewers range in size greater than 24 inches but less than 54 inches. The percentage in each size above 24 inches is rather evenly divided and mostly in the common pipe sizes of 30 inches, 36 inches, 42 inches and 48 inches. Some old odd-shaped sewers were reported as still serviceable.

The percentage of walk-through sewers (54 inch and greater) averages about 15% of the total sewer mileage. However, this figure will vary depending on many factors such as availability of outlets, wastewater treatment facilities, topography, and metropolitan spread.

As the conduit size increases above 48 inches, the percentage of sewer mileage varies from 1% to 4% for each common pipe diameter size up to about 10 feet. Several of the larger conduit shapes reported were arch, box, or multiple conduits and in this report were analyzed for size category as having a vertical dimension and area equivalent to circular pipe.

Due to the time limit on this study, no correlation could be made to population, topography, and other factors that have a controlling influence on design. We believe the data in Table 4-2, and as discussed in this report, to be representative of combined wastewater systems for large cities.

A storm sewer questionnaire sent out by the American Public Works Association produced information on the length and size distribution of combined sewers from 36 cities in the United States. Population of these cities ranged from 41,000 to over 3,500,000. Total mileage reported ranged from one mile to over 4,000 miles. Additional cities reported size distribution figures but it was assumed that these were estimates since no lengths were given.

For those reporting actual mileage, the averages of the reported percentages in three size ranges and for three population groups are given in Table 4-2.

Table 4-2

Average Percentages of the Total Mileage of Combined Sewers in Three Size Ranges

Population Range	**Average Percentages of Total Mileage**		
	21 Inches or Less (%)	**24 to 60 Inches (%)**	**60 Inches & greater (%)**
100,000 or less	64.5	30.0	5.5
100,001-500,000	49.3	37.9	12.8
Over 500,000	61.1	27.1	11.8

These data tend to confirm the conclusion that around 15% of the total mileage of combined sewers is in the walk-through size (54 inches or larger) for major population centers. It is interesting to note that this percentage is apparently less for smaller cities (100,000 or less). This should be expected since the average area of runoff would be smaller and would not generate the hydraulic need for as great a mileage of larger size sewers.

Chapter 5

CHAPTER 5 REPUBLISHES TWO PRACTICAL GUIDANCE DOCUMENTS ON SOLIDS TRANSPORT VELOCITIES IN SANITARY SEWERS

RELATIONSHIP OF SEWAGE CHARACTERISTICS TO CARRYING VELOCITY FOR PRESSURE SEWERS R-2598[1]

Summary

Minimum carrying velocities for solid phase matter in smooth plastic 2-inch, 3-inch, 4-inch, 6-inch, and 8-inch pressure pipes at zero slope have been studied for comminuted and uncomminuted raw sewage. Data from the comprehensive testing of the 3-inch pressure pipe indicated that the minimum velocity for scouring and the minimum velocity where depositing takes place were essentially the same. The velocities appeared to be independent of the concentration of suspended solids, fixed suspended solids, "sand" concentration, and size distribution of suspended matter and "sand" for the sewage studies at FMC's Central Engineering Laboratories waste treatment facilities. However, the velocities appeared to be dependent upon the size distribution of fixed solids, or more likely the "sand," that accumulated on the bottom of the pipe and was the most difficult to scour and the first to deposit.

Carrying velocities were investigated in an 8-inch spiral pressure pipe and the results obtained were very erratic. The scouring velocity, in general, was much greater than the associated minimum depositing velocity, which was due to the build-up or trapping of solids in the spirals of the pipe.

Carrying velocities were investigated in an 8-inch plain plastic pipe in open channel flow and the results obtained appeared to be in general agreement with the results found for the 8-inch pressure pipes and with data reported in the literature.

Observations of the movement of solids in the smooth pressure pipes have shown that egg shells that passed through a garbage grinder were carried in the sewage at lower flow rates than required for scouring the bottom sediments and, consequently, were not the basic criteria for determining minimum carrying velocities.

The permanent and experimental facilities were adequate for making this sewage velocity investigation. The raw sewage studied in the experimental test pipes was found to compare very favorably with samples of sewage obtained in the city sewer. One exception was the lack of large frail suspended matter in the test facility because all sewage passed through a centrifugal pump in order to be lifted to the experimental site.

The average velocities found for the various pipe sizes are as follows in Table 5-1.

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[1] August 1967, Environmental Engineering Department, Central Engineering Laboratories, FMC Corporation

TABLE 5-1

Summary of Average Velocities for Scouring and Redepositing in 2, 3, 4, 6, and 8-inch Pressure Sewer Pipes in an 8-Inch Open Channel Sewer Pipe

Pipe Size Inches	Incipient Velocity (ft./sec.)	
	Scouring	Redepositing
2	0.846	0.826
3	0.930	0.915
4	1.252	1.014
6	1.212	1.025
8 (plain)	1.414	1.315
8 (spiral)	1.46	0.90
8 (plain-open channel)[1]	1.41	1.22

[1] Slope equal to 0.00062, at approximately quarter-full to third-full flow.

MINIMUM TRANSPORT VELOCITY FOR PRESSURIZED SANITARY SEWERS[2]

Summary

The ASCE Combined Sewer Separation Project is essentially a feasibility study of pressurized sanitary sewerage systems, wherein sewage is pumped from individual buildings into street transmission conduits (ASCE Conference, Preprint 548 October, 1967). The size of a street conduit is dictated by the magnitude of the lowest flow rate at which transport of all solids is desired, coupled with the minimum mean velocity required to move all solids. Because insufficient information existed on minimum transport velocities for full pipe flow, and as described in Section 1.0 above, tests were conducted at the Central Engineering Laboratories of the FMC Corporation in Santa Clara, California, using 2-inch through 8-inch horizontal clear plastic pipe.[3]

Sewage analyses included determination of pH, COD, grease, total solids, total volatile solids, suspended solids, settleable solids, and specific gravity and amount of settleable solids on each of 11 sieve fractions, the last in terms of both suspended and volatile content. Analyses were also made of sieve fractions of special samples

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[2] November 16, 1967, ASCE Combined Sewer Separation Project, Technical Memorandum No. 7, New York, New York

[3] "Relationship of Sewage Characteristics to Carrying Velocity for Pressure Sewers." Central Engineering Laboratories, FMC Corporation Report R-2598, August 1967.

collected from the bottom of the test pipe, representing the portion of solids that settled out most readily.

As opposed to the largest particles, with a predominantly volatile content, the bulk of the smaller sieve fractions for the bottom samples had specific gravities above two and a small volatile content. Microscopic examination of the latter fractions indicated that the predominant mineral involved was sand. Because no meaningful correlation could be obtained between minimum transport velocities and sewage characteristics analyzed in the initial series of tests, sewage sand content was also determined in the final tests, including specific gravity and amount of each sand sieve fraction "... sand has the highest specific gravity of materials commonly found in sanitary 112 sewers"[4]

Laursen[5] correlated the minimum sand transport velocity findings of Craven[6] for closed conduits and Ambrose[7] for closed conduits and open channel flow in terms of an empirical relation that can be expressed as:

$$K = \frac{Q}{\sqrt{g(S_S - 1)D^{\frac{5}{2}}}}\left(\frac{S_S(10^6)}{C}\right)^n$$

where Q is the rate of flow in cfs, g is 32.2 feet per second, S_S is the sand specific gravity, D is the internal pipe diameter in feet, C is the sand concentration in ppm by weight and K is a dimensionless constant. Although Laursen used n=1/3, it was found that n=1/5 gave a fit at least as good and was more compatible with the data for sewage flow.

All appropriate full conduit flow data from Ambrose and Craven and the sewage tests are summarized in Table 5-1, and values of K for n=1/5 are plotted versus C in Figure 5-1. Ambrose and Craven determined the largest velocity for impending deposition, whereas for the present tests the smallest velocity for total removal of solids from the invert was also measured. These velocities are termed herein as depositing and scouring, respectively. The curve for scouring in Figure 5-1 is assumed to meld with the curve for depositing at about C = 100 ppm. Scouring requires somewhat higher velocities than depositing, and the following are some coordinates of the upper curve in Figure 5-1:

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[4] Raths, C.H. and R.F. McCauley. "Deposition in a Sanitary Sewer." Water and Sewage Works, May 1962, pp. 192-197.

[5] Laursen, E.M.. "The Hydraulics of a Storm-Drain System for Sediment-Transporting Flow." Iowa Highway Research Board Bulletin No. 5, June 1956.

[6] Craven, J.P.. "A Study of the Transportation of Sand in Pipes." Ph.D. Dissertation, State University of Iowa, 1951.

[7] Ambrose, H.H.. "The Transportation of Uniform Sand in a Smooth Pipe." Ph.D. Dissertation, State University of Iowa, 1952.

C, ppm	K	C, ppm	K
10	2.86	80	2.15
20	2.30	100	2.32
30	2.02	150	2.48
47	1.88	$\geq$200	2.5
60	1.92		

Use of the curve for scouring gives velocities within 10% of the 12 observed values of Table 5-1; and use of the curve for depositing gives velocities within 10% for 20 of the 22 observed values of Table 5-1, and within 20% for all 22.

Because sewage grinding in individual buildings is considered a mandatory requirement in advance of pressurization for the ASCE project, both comminuted and uncomminuted sewage was employed in the tests. Neither comminution nor grinding particularly changed sewage sand size distribution, and the total sand concentration would be unaffected by size reduction. It is, therefore, not surprising that no discernable differences could be found between comparable velocities for uncomminuted and comminuted sewage.

For conveyance of ground garbage in a gravity sewer, eggshell particles (S=2.3) were the constituent most resistant to movement.[8] Ground eggshells were added in all sewage tests, but particles always moved at a lesser mean flow velocity than for the sewage sand.

A series of four tests were conducted with a special clear plastic helically corrugated 8-inch pipe made available by Kaiser Aluminum and Chemical Sales, Inc. Observed velocities were quite erratic as a consequence of difficulties encountered in interpreting solids movement. Values of K for scouring were in approximate agreement with those in Figure 5-1, but the depositing values averaged about a third smaller than for the straight walled pipe. Full flow characteristics for transport of sand in a 12-inch helically corrugated pipe were investigated by Garde[9], but he did not observe limiting scouring and depositing conditions and concentrated on sand bed movement. Practically all of the data obtained by Craven, and over half of Arabrose's, were for tests on bed movement.

Four exploratory open channel flow tests were made on the straight walled 8-inch pipe on a slope of 0.00062, with sewage depths ranging from about 1/4- to 1/3-full and C values ranging from 10 to 478 ppm. Data for 54 open channel tests by Ambrose with D=0.465 feet and C>100 ppm, tempered by the exploratory sewage test results, were reasonably correlated using n=1/5. The results are summarized in

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[8] Cosens, K.W. and E.J. Haneman. "Sewer Velocities Required for Kitchen-Ground Wastes." The American City, January 1949, pp. 102-103.

[9] Garde, R.J. "Sediment Transport Through Pipes." MS Thesis, Colorado State University, October 1956.

Figure 5-2, where Y is the depth of flow and hence $\frac{Y}{D}$ is the relative depth. The limiting values of K at full flow are consistent with the depositing curve in Figure 5-1. Curves for the scouring condition might fall slightly to the right of those in Figure 5-2, but no obvious difference between the two conditions was evident for the four open channel sewage tests.

Considerably more research is needed to verify Figure 5-2 for $C < 100$ ppm and resolve any differences between scouring and depositing. In the meanwhile, use of Figure 2 on a tentative basis leads to some interesting inferences.

An 8-inch sanitary sewer is normally the minimum size used, and the design flow rate typically would be accommodated at a mean velocity of 2-feet per second flowing half full.[10] The upper end of the solid curve in Figure 5-3 meets these specifications, with velocities at lesser depths being for the same sewer slope and friction coefficient as for half full. The maximum sewage sand concentrations that can be conveyed at the velocities for this curve are, according to indications from Figure 5-2:

y, inches	**C, ppm**
4	170
3	180
2	160
1	100

For comparison, curves for minimum sewage sand transport velocities for C=45, 100 and 500 ppm, computed using Figure 5-2, are traced in Figure 5-3. Median sieve sizes $\bar{d}$, are listed in Table 5-2. Size distributions for the sewage sands with the largest and smallest median size are graphed in Figure 5-4 (the curves for the other sewage sands fall between these two), together with those for sands used by Ambrose and Craven. Laursen stated that, within the range of experimental data, the effect of particle size had little effect on transport requirements. This conclusion appears applicable to still smaller sized sewage sands.

Laursen carefully noted that "determination of the equilibrium condition in all cases involved the judgment of the observer." Despite the readily acknowledged subjective nature of judging when transport velocities were reached in the sewage tests, results presented should provide encouragement and initial orientation for further research.

This research by and for ASCE was wholly supported by the Federal Water Pollution Control Administration, Department of the Interior, pursuant to Demonstration Grant WPD 104-01-66 and Contract No. 14-12-29.

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[10] "Design and Construction of Sanitary and Storm Sewers." ASCE Manual of Engineering Practice No. 37, 1960.

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The exploratory open channel flow data of this project has not been tabulated in this memorandum because judgment is required for its interpretation. Reference 1 should be consulted for details.

For convenience, the portion of Reference 5 data for open channel flow depositing velocities, is included in Tables 5-2 and 5-3.

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TABLE 5-2

Summary of Closed Conduit Test Results

Internal Pipe Diameter, D (feet)	**Solids Phase**	**Median Sand Sieve $\bar{d}$ Size, (mm)**	**Sand Content**		**Velocity (fps)**		**K, n=1/5**	
			S_S	**C (ppm)**	**Depositing**	**Scouring**	**Depositing**	**Scouring**
CEL/FMS Sewage Data								
0.165	Comminuted	0.03	2.60	9.0	0.801	0.864	2.67	2.88
0.165	Comminuted	0.07	2.14	78.0	0.851	0.827	2.10	2.04
0.248	Comminuted	0.04	2.24	43.6	0.657	0.784	1.43	1.71
0.248	Uncomminuted	0.17	2.25	22.5	0.863	0.900	2.14	2.24
0.331	Uncomminuted	0.17	2.31	7.8	1.03	1.23	2.69	3.21
0.331	Comminuted	0.17	2.38	11.8	0.997	1.27	2.35	2.99
0.494	Uncomminuted	0.07	2.27	44.9	1.21	1.24	1.84	1.89
0.494	Comminuted	0.07	2.03	27.6	1.14	1.09	2.08	1.99
0.663	Comminuted	0.07	2.19	23.0	1.42	1.46	2.19	2.25
0.663	Uncomminuted	0.08	2.28	26.3	1.41	1.42	2.06	2.07
0.663	Comminuted	--	2.28	40.1	1.23	1.52	1.65	2.04
0.663	Uncomminuted	--	2.39	25.8	1.27	1.36	1.80	1.93
Ambrose Data								
0.465	Sand	0.58	2.65	83.3	1.87		2.35	
0.465	Sand	0.58	2.65	144	2.17		2.44	
0.465	Sand	0.58	2.65	295	2.65		2.57	
0.465	Sand	0.58	2.65	530	2.94		2.55	
0.465	Sand	0.58	2.65	930	3.36		2.60	
0.465	Sand	0.58	2.65	2,090	3.74		2.47	
0.465	Sand	1.62	2.62	555	2.81		2.43	
0.465	Sand	0.25	2.65	533	2.76		2.35	
Craven Data								
0.167	Sand	0.25	2.65	3,820	2.84		2.77	
0.167	Sand	0.25	2.65	4,600	2.37		2.23	

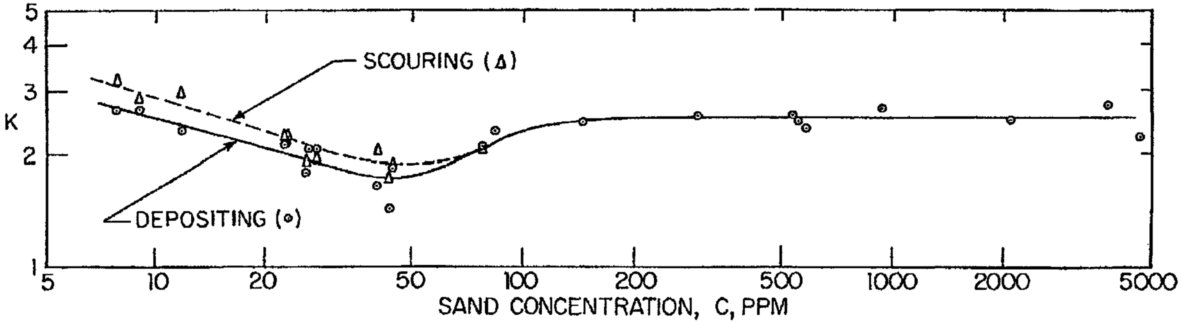

FIGURE 5-1. Variation of K for n=1/5 with sand concentration for full pipe flow.

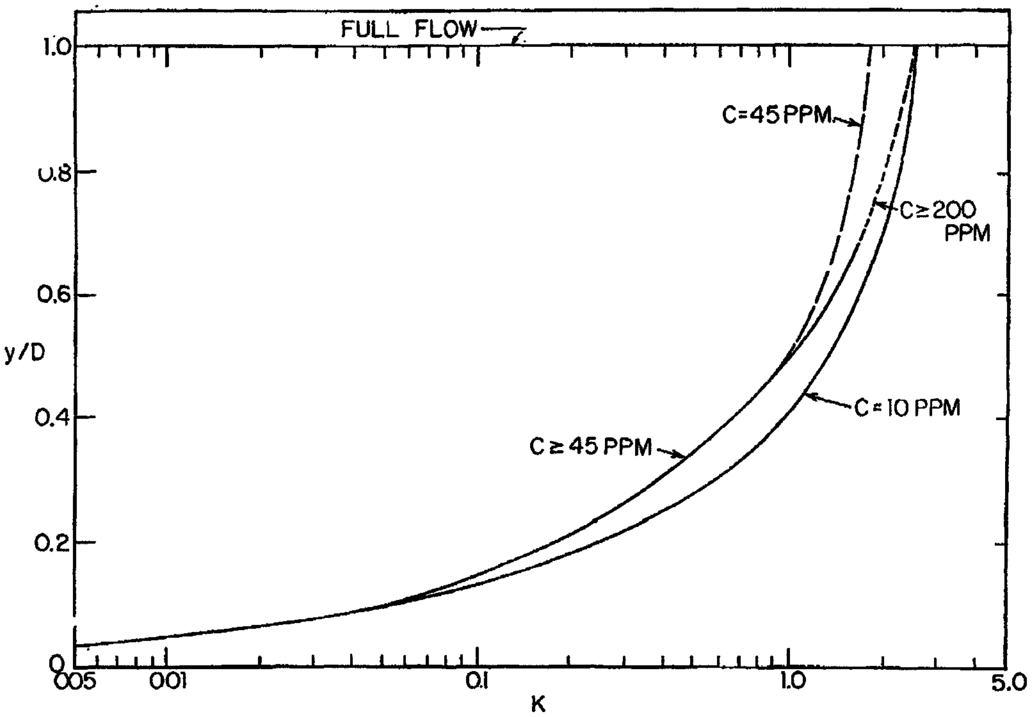

FIGURE 5-2. Variation of K for n=1/5 with relative flow depth at a given sand concentration for depositing condition.

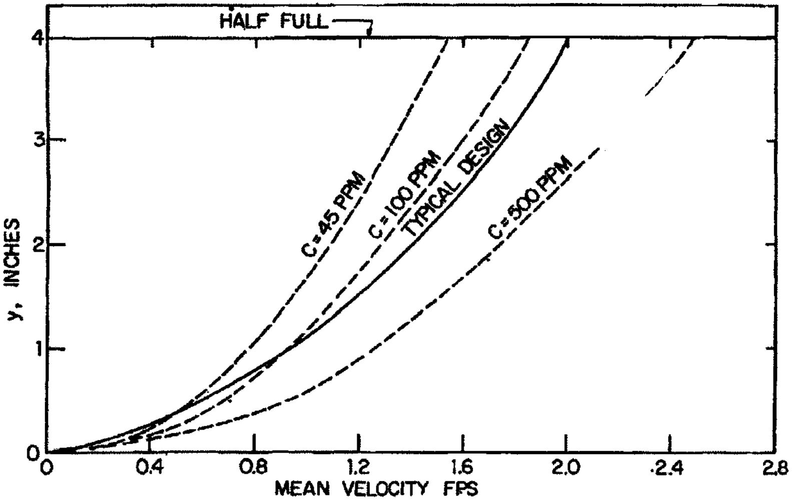

FIGURE 5-3. Approximate minimum transport velocities for various depths in an 8-inch gravity sanitary sewer.

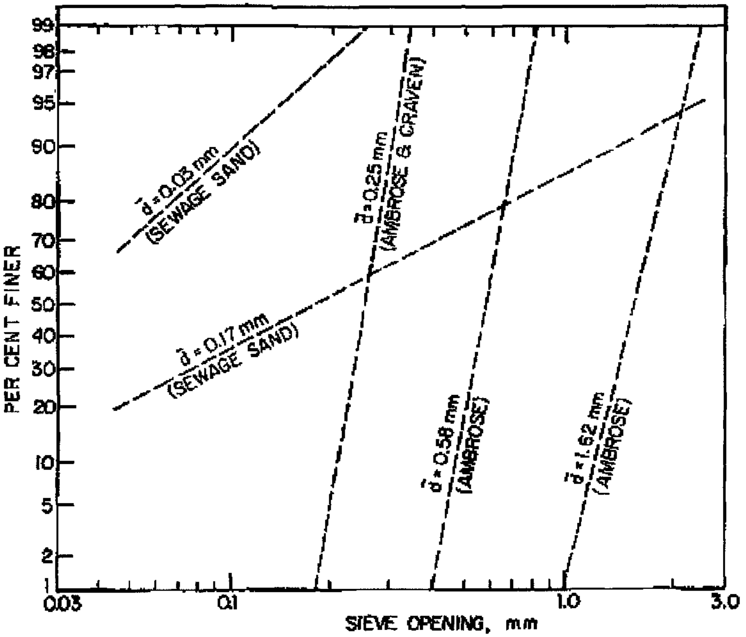

FIGURE 5-4. Sand size distributions.

TABLE 5-3

Open Channel Flow Data by Ambrose
D=0.465 ft.; S_S=2.65 (Sand)

y/D	C (ppm)	Depositing Velocity (fps)	y/D	C (ppm)	Depositing Velocity (fps)
$\bar{d}$ = 0.58 mm			0.407	2,650	3.08
0.750	106	1.83	0.256	5,300	2.92
0.630	133	1.78	0.142	13,200	2.72
0.366	265	1.78	0.095	26,500	2.45
0.234	530	1.66	0.663	2,650	4.19
0.129	1,320	1.57	0.405	5,300	3.88
0.086	2,650	1.41	0.224	13,200	3.52
0.055	5,300	1.36	0.142	26,500	3.40
0.032	13,200	1.22	0.600	5,300	4.74
0.764	185	2.05	0.460	7,950	4.37
0.561	265	2.04	0.327	13,200	4.14
0.346	530	1.92	0.211	26,500	3.83
0.271	795	1.65	$\bar{d}$ = 1.62 mm		
0.194	1,320	1.72	0.045	13,200	1.47
0.120	2,650	1.74	0.105	13,200	2.11
0.069	5,300	1.96	0.127	2,650	1.60
0.045	13,200	1.47	0.234	13,200	3.31
0.850	331	2.60	0.288	2,650	2.56
0.559	530	2.66	0.359	530	1.84
0.305	1,325	2.28	0.695	2,650	3.97
0.198	2,650	2.10	$\bar{d}$ = 0.25 mm		
0.127	5,300	2.00	0.045	13,200	1.47
0.075	13,200	1.76	0.097	13,200	2.33
0.755	662	2.91	0.116	2,650	1.82
0.448	1,320	2.72	0.213	13,200	3.77
0.286	2,650	2.50	0.277	2,650	2.61
0.190	5,300	2.22	0.316	622	2.17
0.101	13,200	2.23	0.588	766	2.90
0.655	1,320	3.40			

Chapter 5, Appendix 1

(L.S. Tucker)

A stepwise linear multiple regression analysis was made using six variables obtained from observations of the ten complete scouring velocity test runs in Table 5-1. The six variables were scouring velocity in feet per second (V), internal pipe diameter in feet (D), sand concentration in ppm (c), sand specific gravity (S_S), median sand sieve size in $(\bar{d})$ mm, and ratio of sand sieve size at which 90% of particles are smaller to median sand sieve size (f). The data for the ten scouring velocity test runs is given in the following table:

Run	**Scouring Velocity, V (fps)**	**Internal Pipe Diameter, D (feet)**	**Sand Concentration, C (ppm)**	**Sand Specific Gravity, S_S**	**Median Sand Sieve Size, $(\bar{d})$ mm**	**Sieve Size at which 90% of Sand is smaller—Median Sand Sieve Size, feet**
1	0.864	0.165	9.0	2.60	0.03	37
2	0.827	0.165	78.0	2.14	0.07	56
3	0.784	0.248	43.6	2.24	0.04	23
4	0.900	0.248	22.5	2.25	0.17	1
5	1.23	0.331	7.8	2.31	0.17	11
6	1.27	0.331	11.8	2.38	0.17	2
7	1.24	0.494	44.9	2.27	0.07	3.8
8	1.09	0.494	27.6	2.03	0.07	3
9	1.46	0.663	23.0	2.19	0.07	36
10	1.42	0.663	26.0	2.28	0.08	3

The general relationship $Y = cx_1^a x_2^b x_3^c x_4^d x_5^e$ was assumed to exist between the variables. The above relationship was converted to log form so that a linear multiple regression analysis could be made. The results of the analysis are as follows:

1) When the dependent variable and all five independent variables are considered, the regression equation is:

$$V = 0.880 D^{+0.461} C^{+0.0025} S_S^{+1.258} \left(\bar{d}^{+0.173}\right) f^{+0.0503}$$

The multiple correlation coefficient R is 0.976 and R^2 is 0.952. The actual scouring velocities versus the predicted velocities from the above equation are as follows:

Run	**Actual Scoring Velocity (ft./sec)**	**Predicted Scouring Velocity (ft./sec)**	**Percentage that Predicted Velocity Varies from Actual (%)**
1	0.864	0.840	-2.8
2	0.827	0.781	-5.6
3	0.784	0.865	+10.3
4	0.900	0.953	+5.9
5	1.23	1.27	+3.3
6	1.27	1.21	-4.7
7	1.24	1.22	-1.6
8	1.09	1.04	+4.6
9	1.46	1.49	+2.1
10	1.42	1.42	0

Thus, the predicted scouring velocities for the ten runs are within ± 10.3% of the actual scouring velocities.

2) When the dependent variable and four strongest independent variables are considered (S_S omitted) the regression equation is:

$$V = 2.66\left(D^{+0.407}\right)\left(C^{-0.0781}\right)\left(\bar{d}^{+0.119}\right)\left(f^{+0.0413}\right)$$

The multiple correlation coefficient R is 0.952 and R^2 is 0.905.

3) When the dependent variable and three strongest independent variables are considered (S_S and f omitted) the regression equation is:

$$V = 2.44\left(D^{+0.378}\right)\left(C^{-0.0790}\right)\left(\bar{d}^{+0.0617}\right)$$

The multiple correlation coefficient R is 0.933 and R^2 is 0.870.

4) When the dependent variable and two strongest independent variables are considered (S_S, f, and $\bar{d}$ omitted) the regression equation is:

$$V=2.22\ D^{+0.393}\ C^{-0.0935}$$

The multiple correlation coefficient R is 0.921 and R^2 is 0.848.

5) When the dependent variable and the strongest independent variable are considered (S_S, f, $\overline{d}$ and C omitted) the regression equation is:

$$V=1.653\ D^{+0.391}$$

The multiple correlation coefficient R is 0.872 and R^2 is 0.760. The actual scouring velocities versus the predicted velocities from the above equation are as follows:

Run	**Actual Scoring Velocity (ft./sec)**	**Predicted Scouring Velocity (ft./sec)**	**Percentage that Predicted Velocity Varies from Actual (%)**
1	0.864	0.816	-5.6
2	0.827	0.816	-1.3
3	0.784	0.959	+22.5
4	0.900	0.959	+6.7
5	1.23	1.07	-13.0
6	1.27	1.07	-15.7
7	1.24	1.25	+0.8
8	1.09	1.25	+14.7
9	1.46	1.41	-3.4
10	1.42	1.41	-0.7

Thus, the predicted scouring velocities for the ten runs are within + 22.5% of the actual scouring velocities.

6) The simple correlation coefficients (zero order correlation coefficients) for the six variables are as follows:

	V	D	C	$\overline{d}$	F	Ss
V						
D	0.8722					
C	-0.2829	0.0162				
$\overline{d}$	-0.0683	-0.3712	-0.6401			
f	0.3996	0.2000	-0.2861	-0.1536		
Ss	-0.3122	-03691	0.1510	0.1063	-0.6299	

Chapter 6

DOMESTIC SEWAGE FLOW CRITERIA FOR EVALUATION OF APPLICATION OF PROJECT SCHEME TO ACTUAL COMBINED SEWER DRAINAGE AREAS

November 17, 1967
ASCE Combined Sewer Separation Project
Technical Memorandum No. 8
New York, New York

1.0 INTRODUCTION

The ASCE Combined Sewer Separation Project scheme is concerned with pressurized sanitary sewer systems (see Chapter 3 for overview). Sizing of pressure sewers must be based on attainment of minimum necessary scouring velocities at an acceptable level of frequency, such as for at least one hour each day of the year. Deposition of solids part of the time must be accepted to preclude ultra-conservative design. At the other extreme, peak sewage rates will give rise to maximum hydraulic gradients, and these in turn affect pressure requirements within the system, for booster pumping and/or pressure control. Drainage areas to be examined for hypothetical application of the Project scheme are located in the northeastern portion of the United States and in northern California; this discussion is concerned only with data for those locations. The Department of Environmental Engineering Science at the Johns Hopkins University, Baltimore, Maryland, in the summer of 1966 concluded a five-year research project on "Residential Water Use," and findings from that project form the major reference material used. Two papers have been published describing the Johns Hopkins project. [1,2]

2.0 USE OF WATER RATE DATA

Reliable data on variations of sewage flow rates would be the obvious choice for attaining the stated objectives. For the limited amount of sanitary sewage flow data available for modest sized service areas, application of subjective judgment is often required to abstract groundwater "infiltration" or stormwater that migrates into sanitary sewers. In the absence of comprehensive, reliable domestic sewage flow data, water demand data for one to over 400 homes acquired by the Residential Water Use project at Johns Hopkins has been exploited, together with other data deemed immediately useful.

In residential areas, during the grass-growing season considerable water is used for lawn irrigation, and there is a substantial disparity between water demand and sewage flow for individual dwellings at that season. Conversely, during the winter season in northern latitudes practically all water delivered to a home leaves as a carrier of

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[1] Wolff, J.B. "Peak Demands in Residential Areas" *Journal of the American Water Works Association*, Volume 53, No. 10, pp. 1251-1260 (October 1961).

[2] Linawearer, Jr., F.P. and J.C. Geyer. "Use of Peak Demands in Determination of Residential Rates" *Journal of the American Water Works Association*, Volume 56, No. 4, pp. 403-410.

household (domestic) sewage. Water use data for winter months has been utilized herein under the assumption that water demand rates and volumes are closely representative of those for sanitary sewage emanating from the same buildings, ignoring any disparities arising from storage effects in water distribution piping, household plumbing and sewers.

In the absence of suitable sewer gauging data, the most field data that can be expected for any typical combined sewer drainage area is the mean water demand during winter months (non-lawn irrigation, non-air conditioning season), and then only where individual buildings are metered and meter reading frequency is at least quarterly. That is, in the absence of suitable gauging data, the most that can be expected is to obtain estimated mean annual sewage rates for individual buildings, which are combinable into estimated drainage area sector mean annual sewage rates. Therefore, for any criteria to be useful they should be in terms of mean annual rates. For the foregoing reasons, data have been interpreted in terms of multiples of mean annual rates for ASCE Project application.

3.0 PER CAPITA CONSUMPTION

For a given drainage area, the number of residents in each building, or at least in each census tract and block, may be determined from data for the 1960 Census. The extensive effort required to determine residency of individual buildings would perhaps be justified if per capita water use was significantly related to number of occupants, in a definite form of dependence. Regardless, such information would be reliable only in application to conditions prevailing in 1960, would not necessarily apply for 1967, and surely would have to be modified in characterizing the end of the design period, say 25 years or more hence.

Ignoring the preceding reservations, winter per capita water use information was studied for the purpose of determining the effect of number of building occupants on mean water demand.

In Table 6-1 are listed values from a U.S. PHS Taft Center report[3] and a Johns Hopkins report.[4] Although results of analyses for ten county areas were included in the Taft Center report, these were for winter months varying between November, December, and January and December, January, February, and March. In order to put the data on a common basis, values listed separately in the Taft Center report for January, February and March were averaged and appear in Table 6-1, but monthly values were given only for six of the ten areas. The gallons per capita per day entries, enclosed in parentheses and indexed with an asterisk, are values given via equations in the references from fits to actual data for variable winter month periods.

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[3] R.E. Thomas and T.W. Bendixen, "Domestic Water Use in Suburban Homes." Final Report to the F.H.A. from U.S. PHS Taft Sanitary Engineering Center, Cincinnati, Ohio, June 29, 1964.

[4] "Report on Phase One, Residential Water Use Project," Department of Environmental Engineering Science, The Johns Hopkins University, Baltimore, Maryland, October 1963.

TABLE 6-1

Winter Water Demand
(see text for explanation)

Month	Taft Center Report							Johns Hopkins	
	J-F-M	J-F-M	J-F-M	J-F-M	J-F-M	J-F-M	Winter	Winter	Fit
State (County)	NJ (Bergen)	TX (Dallas)	TN (Knox)	CA (Santa Clara)	NC (Mecklenberg)	TX (Nueces)	Ten Areas	Pine Valley MD	Five Areas MD
2 people									
gpd	83	180	102	81	120	178	142		
gpcd	42	90	51	40	60	89	71	70	
gpcd*	(61)	(100)	(54)	(60)	(62)	(74)	(71)		(80)
3 people									
gpd	158	251	133	169	149	186	178		
gpcd	53	84	44	56	30	62	59	72	
gpcd*	(52)	(81)	(44)	(54)	(49)	(54)	(60)		(61)
4 people									
gpd	210	286	152	228	172	201	232		
gpcd	53	72	38	57	43	50	58	52	
gpcd*	(48)	(71)	(39)	(50)	(43)	(44)	(55)		(51)
5 people									
gpd	210	314	169	216	200	218	262		
gpcd	42	63	34	43	40	44	66	39	
gpcd*	(45)	(65)	(36)	(48)	(39)	(38)	(51)		(44)
6 people									
gpd	278	348	191	304	217	234	292		
gpcd	46	58	32	51	36	39	49	--	
gpcd*	(44)	(61)	(33)	(47)	(37)	(34)	(49)		(39)
Total No. Homes	97	495	90	21	207	56	21-495	120	
Total Record	1955-60	1955-59	1946-58	1949-57	1950-58	1948-53	Varies	1959-62	Varies

For the ten county areas considered in the Taft Center report: "Mean winter water use by county ranges from 40 to 71 gpcd with a weighted mean of 56 gpcd." For the five Maryland areas of the Johns Hopkins report, weighted mean winter water use ranged between 50 and 65 gpcd with a "weighted average annual domestic use for all areas (of) 56 gpcd." The overall trend of values in Table 6-1 supports the generally held view that per capita winter (or domestic) demand decreases with number of occupants per home. However, the spread of values is interesting:

No. of Occupants	**Range of Values, Six Counties, Table 6-1 (gpcd)**
2	90 -40 = 50
3	84 -30 = 54
4	72 –38 = 34
5	66 –34 = 32
6	58 –32 = 26

County	**Range of Values, 2-6 People, Table 6-1 (gpcd)**	**Demand for 3 People Minus Demand for 5 People, Table 6-1 (gpcd)**
Bergen	53 –42 = 11	53 –42 = 11
Dallas	90 –58 = 32	84 –63 = 21
Knox	51 –32 = 19	44 –34 = 10
Santa Clara	57 –40 = 17	56 –43 = 13
Mecklenberg	60 –30 = 30	30 –40 = (10)
Nueces	89 –39 = 50	62 –44 = 18

The range of values from county-to-county for any given number of occupants is generally greater than the range of values in individual counties for two to six occupants per dwelling. Further, the range in weighted mean winter water demand for the ten counties, 31 gpcd, is on the order of the range in values for individual counties with two to six occupants per dwelling.

On the assumption that the data presented are reasonably representative of prevailing demand variations in various communities, the following assertions are deemed appropriate: (1) estimates of mean winter water demand for a given small area (say, for up to 1,000 homes), based on observed data from other communities, may contain inherent errors on the order of + 30%; and (2) estimates of winter per capita demand in terms of number of occupants per dwelling unit could be on the order of + 30% in error. Should the reader feel that these are extravagant assertions because they are merely approximations of apparent extremes, it is important to note that the entries in Table 6-1 are themselves averages of a number of observations, obscuring further variations for individual homes and from year to year. For example, for Santa Clara, California from Reference 3:

No. Occupants	Item	January	February	March
3	No. of home months	35	35	35
	Mean Use, gpcd	52	54	63
	Standard Deviation, gpcd	20	17	30
5	No. of home months	31	32	32
	Mean Use, gpcd	41	44	45
	Standard Deviation, gpcd	24	14	15

Changes in building occupancy over the projected design period for sewer systems can be anticipated only subjectively. While mean annual domestic demands might be projected with a reasonable degree of confidence for expected future land use, at best, the projections could be made realistically only for whole blocks or sub-areas in a system.

From the foregoing presentation it is reasonable to conclude that refinements in projections to account for differences in demand related to expected number of occupants per dwelling unit would not be realistic, estimates of mean annual domestic demand based on data from other communities would contain considerable inherent errors, and projection of the latter to the end of the design period would imply acceptance of rather specious criteria.

Instead, the preferred basis would appear to be projections of observed mean annual domestic demand for the drainage area of the system to be analyzed, with the detail available governing the degree of refinement to which projections would be made. If the sewer system is served by a fully metered water system, and meter readings are amenable to direct usage, an acceptable estimate of mean annual sanitary sewage flow can be obtained by using water demand meter readings for non-lawn sprinkling months. Although these may be fairly reliable indicators of present sewage flow, their projection will be subjective, and use of refinements, such as allowances for differences in number of occupants per dwelling, does not appear to be at all justified.

4.0 DESIGN FLOW CRITERIA

In the preceding section it was argued that the best and most realistic measure of sanitary sewage flow would be based on estimated mean annual domestic water demand from meter readings. In any event, the preferred format for presentation and usage of data on demand variability from diverse communities is in terms of variations about their respective average annual domestic water demands.

A Johns Hopkins University report[5] was practically the exclusive reference used. The "Residential Water Use Project" had analyzed winter month water demand data ("domestic") obtained from sectors of water distribution systems serving up to 410 homes, and ratios of observed peak day and peak hour domestic demands to average annual domestic demands are listed in Reference 5.

Study areas for investigation of the feasibility of the ASCE Project scheme are located in cities in the northeastern quadrant of the United States and in Northern California. Reported ratios of peak domestic demands to the average annual were much higher for the southwest and west than for either the southeastern or northeastern quadrants. Consequently, two groups of data were segregated from the total available, one for water systems in the northeastern quadrant and one for the state of California.

The northeastern data are summarized in Table 6-2 and the ratios listed therein are plotted in Figure 6-1. Also included are ratios for 1, 3, and 6 dwelling units, obtained by extrapolation of lines of fit to a common expected mean recurrence interval of 1/180 days, from a previous Technical Memorandum.[6] The latter are for individual homes in the English Manor subdivision served by the Washington Suburban Sanitary District, also cited in Table 6-2, for a group of 309 homes.

In Table 6-3 are summarized the ratios for California, and these are plotted in Figure 6-2.

The entries enclosed by parentheses in Tables 6-2 and 6-3 were obtained from the original tabulations of hourly water demand data, made available through the generosity of the Johns Hopkins Residential Water Use Project. Other than the English Manor-USDA entries in Table 6-2, all open entries in Tables 6-2 and 6-3 are from Reference 5.

Rates and ratios in Tables 6-2 and 6-3 are for observed values, without extrapolation, over the number of days of data indicated. Ratios plotted in Figures 6-1 and 6-2 are for only those cases where data for about one or more months of observation were available. While it would be possible to array the raw data, make suitable fits and extrapolate these to a common expected mean recurrence interval, the extensive time

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[5] Linawearer, F.P., Jr., J.C. Geyer and J.B. Wolff. June 1966. "Report V, Phase Two, Final and Summary Report on the Residential Water Use Project." Department of Environmental Engineering Science, The Johns Hopkins University.

[6] Tucker, L.S. February 24, 1967. "Sewage Flow Variations in Individual Homes." Technical Memorandum No. 2, Combined Sewer Separation Project, ASCE, 345 E. 47th Street, New York, NY.

required for such elaboration was not regarded as justifiable and the observed values were used without manipulation or statistical fitting.

TABLE 6-2

Northeastern U.S. Variations in Domestic Water Demand

Northeastern U.S. Location/Area	No. of Dwelling Units	Winter Rates (December, January and February)									No. Full Days of Data
		(a) Avg. Ann.	(b) Min. Dly.	(c) Max. Dly.	(d) Peak Hrly	(e) Peak Hr. of Min. Day	(b) ÷ (a)	(c) ÷ (a)	(d) ÷ (a)	(e) ÷ (a)	
Des Moines, IA											
Patricia Park	325	182	(167)	242	425	(354)	(0.92)	1.33	2.34	(1.95)	106
Clive	307	165	(162)	195	405	(275)	(0.98)	1.18	2.45	(1.67)	33
Washington Suburban Sanitation District											
Palmer Park	395	175	(171)	219	431	(273)	(0.98)	1.25	2.46	(1.56)	13
Glenmont	129	173	(176)	216	409	(298)	(1.01)	1.25	2.36	(1.72)	11
English Manor	309	224	(190)	276	536	(482)	(0.85)	1.23	2.39	(2.15)	28
NW Branch Est.	124	262	(274)	290	542	(542)	(1304)	1.11	2.07	(2.05)	2
Baltimore, MD											
Country Club Park	289	189	(154)	243	442	(367)	(0.82)	1.29	2.34	(1.94)	39
Pine Valley	210	204	(148)	272	555	(326)	(0.73)	1.33	2.72	(1.59)	225
Campus Hills	179	251	(249)	323	602	(471)	(0.99)	1.29	2.40	(1.87)	163
Hampton	44	157	(133)	216	457	(277)	(0.85)	1.38	2.91	(1.76)	90
Philadelphia, PA											
Normandy	410	220	(262)	331	595	(407)	(1.19)	1.50	2.70	(1.85)	53
Benton Street	200	252	(182)	303	565	(413)	(0.72)	1.20	2.24	(1.64)	32
Philadelphia Suburban											
Downeast	287	212	--	262	477	--	--	1.24	2.25	--	(0)
St. Albans	137	195	(166)	242	596	(356)	(0.85)	1.24	3.06	(1.82)	44
Dogwood Lane	113	228	(188)	322	713	(380)	(0.82)	1.41	3.13	(1.66)	131
English Manor-USDA (from Technical Memorandum No. 2)	(1)						0.40	2.40	13.0	2.0	--
	(3)						0.70	1.85	7.9	2.0	--
	(6)						0.87	1.45	5.5	2.0	--

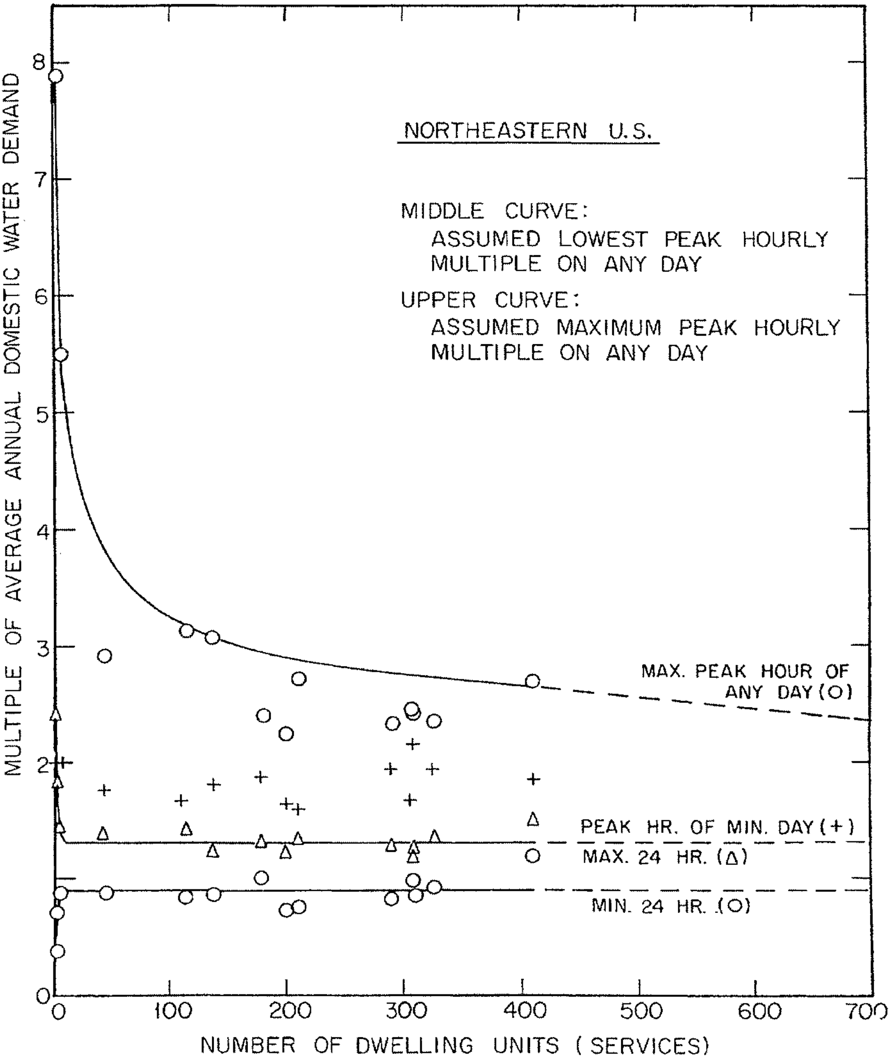

FIGURE 6-1. (T.M. No. 3).

TABLE 6-3

California Variations in Domestic Water Demand

California Location/Area	No. of Dwelling Units	Winter Rates (December, January and February)									No. Full Days of Data
		(a) Avg. Ann.	(b) Min. Dly.	(c) Max. Dly.	(d) Peak Hrly	(e) Peak Hr. of Min. Day	(b) ÷ (a)	(c) ÷ (a)	(d) ÷ (a)	(e) ÷ (a)	
East Bay											
San Lorenzo	81	233	(210)	452	1,227	(386)	(0.90)	1.94	5.27	(1.66)	180
Creekside Acres	143	295	(214)	610	1,635	(377)	(0.73)	2.07	5.54	(1.28)	151
Burton Valley	137	282	(269)	474	1,242	(559)	(0.96)	1.68	4.40	(1.98)	88
Chabot Park	295	297	(253)	700	1,665	(562)	(0.85)	2.36	5.61	(1.89)	76
San Diego											
Rancho Hills	112	215	(176)	294	705	(337)	(0.82)	1.37	3.28	(1.57)	151
Rancho Hills <u>Sewage</u>	(110)	(239)	(135)	(376)	(754)	(273)	(0.59)	(1.64)	(3.28)	(1.19)	171
Ruffin Road	259	234	(199)	377	1,338	(374)	(0.85)	1.61	5.72	(1.60)	75
Muirlands	66	344	(265)	598	1,547	(558)	(0.77)	1.74	4.50	(1.62)	168
Muirlands <u>Sewage</u>	(71)	(336)	(243)	(467)	(846)	(549)	(0.72)	(1.39)	(2.52)	(1.64)	80
Helix Irrigation District											
El Cajon	187	194	(177)	322	912	(317)	(0.91)	1.66	4.70	(1.63))	35
Lemon Grove	235	223	(173)	379	924	(374)	(0.78)	1.70	4.14	(1.68)	166
California Water and Telephone											
Minot Avenue	63	150	(126)	335	940	(256)	(0.84)	2.23	6.27	(1.71)	77
Sacramento											
Golf Course Terrace	134	248	(285)	559	1,090	(479)	(1.15)	2.25	4.40	(1.93)	15

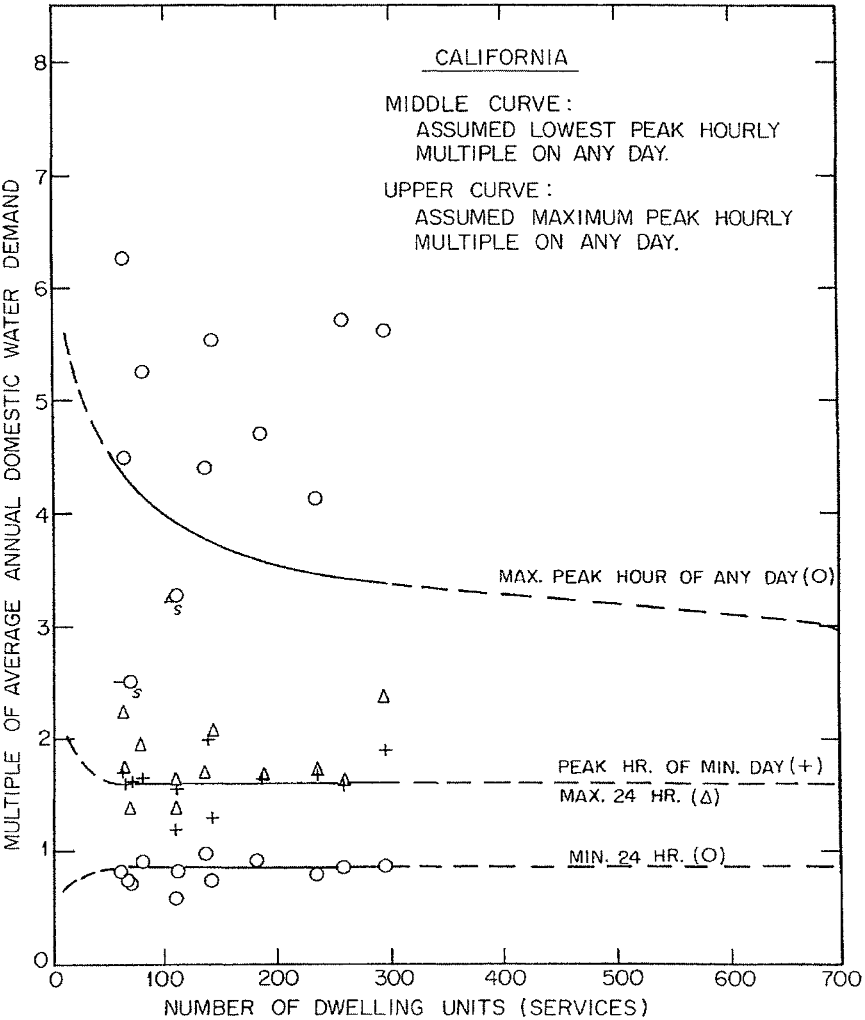

FIGURE 6-2. (T.M. No. 3).

Similarly, the actual minimum daily peak hourly demand ratio was not obtained because of time limitations, and was approximated, instead, by using the more readily identified peak hour of the minimum daily demand of the record. The latter is generally higher than the true minimum daily peak hourly demand, which was found by checking a few cases to be close to the maximum daily demand. Hence, the middle curves in Figures 1 and 2 are regarded as representing the lowest peak hourly domestic demand ratio for any day over the period of record.

For preliminary design of pressure sewers in evaluation of the ASCE Project scheme, the flow rate obtained from the middle curve of either Figure 6-1 or 6-2 applied to the average annual present and estimated future domestic demand will be used in conjunction with minimum solids transport criteria (to be reported in a later Technical Memorandum) for sizing of street pressure sewers. The implication is that for at least one hour of every day (actually, of the record period, which is less than a year for all data presented) all solids would be moved; and conversely, some deposition would occur some hours of a number of days. More conservative criteria would be to use the minimum daily rate, but the consequence would be higher velocities in smaller sewers at peak loading, with associated steeper hydraulic gradients. The criteria adopted for sizing pressure sewers is, therefore, not on the conservative side with respect to steepest hydraulic gradients under maximum loadings.

The ratio for the maximum, or largest, peak hour of any day in the record to the annual average needs no explanation. However, some explanation of the curves for these data is necessary. For the northeastern U.S. data of Figure 6-1 it may be noted that the curve has been passed through the points for the English Manor-USDA individual household data and practically envelopes the remaining peak hour points. This seeming conservatism was exercised because ratios for observed sewage ratios for California (Figure 6-2) were of that order, and having already adopted non-conservative criteria for sizing pressure sewers, it was thought that the peak hour curve should be in the conservative direction.

It should be noted that the time interval for demand data observations used by the Residential Water Use Project was one hour, so that evaluation of peaks for shorter intervals of time from that data is not possible. In connection with the multiple-home data: "The domestic use includes all the normal uses within the home. Investigation has shown that these uses are almost always less than 50-minutes in duration." (Linaweaver, et al., 1966). Returning to the preceding discussion of the peak hourly ratio, it is evident that the upper curve in Figure 6-1 is not conservative considering that higher ratios than for the peak hour can commonly occur, over shorter time intervals.

Upon checking, it was found that the rates and ratios in Table 6-2 from Reference 5 were in complete agreement with the raw data. However, the rates and ratios for the peak hour in Table 6-3 from Reference 5 (open entries) were generally lower, suggesting that some correction had been applied, perhaps to allow for winter season lawn irrigation. Also, it should be noted that the Figure 6-2 peak hour ratios for the two sewage gaugings were significantly lower than for corresponding water demands, which had been already adjusted downward. It is difficult to believe that peak domestic water demands in California homes are as much higher than those in northeastern homes, as implied in Figure 6-2 versus Figure 6-1, and this seems to be supported partially by the disparity in sewage and water ratios for the two study areas cited. In consideration of the possibility that the peak hourly ratio points in Figure 6-2 were too high, the upper curve of Figure 6-2 was constructed as a multiple of its middle curve in direct proportion with the multiple of the upper to middle curves of Figure 6-1. While there is considerable uncertainty about the validity of the upper curve in Figure 6-2, at least it is consistent with that in Figure 6-1, but could conceivably be much less conservative.

As part of a sewerage research project at the Johns Hopkins University, sanitary sewage was gauged for seven residential areas located in a total of four cities. The number of sewer services for only five of the gauged areas are given in the final report[7], and one of these is in Florida. From log-normal frequency distribution plots given in the final report, the ratios of peak domestic sewage flow rate for each day to the average daily domestic sewage flow rate are approximately as follows:

Location	**Number of Services**	**Ratio of Peak Flow to Average Daily for Observed Mean Recurrence Interval**	
		1/100 Days	**99/100 Days**
Valley Wood, Baltimore, MD	45	5.5	1.9
Pine Valley, Baltimore, MD	200	3.2	1.7
Springfield, MO	480	3.4	1.2
Falcon, Anaheim, CA	140	4.2	1.7

Compared with the middle and upper curves of Figures 6-1 and 6-2 both sets of values in the above table are higher. However, the 1/100 ratios compared with the 99/100 ratios are proportionally little more than the respective upper curves compared with the middle curves of Figures 6-1 and 6-2 for the same number of dwelling units. This is to say that were the ratios of the above table used for design of pressure sewers, rather than Figures 6-1 and 6-2, larger sewer sizes would result, but hydraulic gradients would be only slightly steeper. It must be noted, nevertheless, that although there is no suitable observed data covering mean recurrence interval peaks of 1/365 days or rarer, or 365/365 days or commoner, greater extremes than in Figures 6-1 and 6-2 are inevitable and the spread between the middle and upper curves of these two figures is, therefore, hardly conservative.

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[7] Geyer, J.C. and J.J. Lentz. (about 1963). "An Evaluation of the Problems of Sanitary Sewer System Design, Final Report of the Residential Sewerage Research Project of the Federal Housing Administration Technical Studies Program. The Johns Hopkins University, Baltimore, MD.

Discussion has thus far centered on household domestic sewage flows. Little information is available on groups of commercial buildings, such as hotels, restaurants and other large buildings. Some useful detailed data are available for individual buildings in this classification.[8] However, considerable difficulty would be encountered in integrating information given on individual commercial establishments and institutional buildings in the use of Figures 6-1 and 6-2 for mixed service areas.

Water use recorder data obtained between the summer of 1964 and the spring of 1965 for a total of 42 business establishments or institutions in Baltimore were analyzed in the "Commercial Water Use Research Project."[8] Although the number of days of usable hourly demand data from recorders ranged between only 1 and 65 days, long-term averages were obtainable from quarterly billing data supplied by the Baltimore Bureau of Water Supply. Because interest was restricted to peak water demands, some of the peaks and means reported include an unknown portion of lawn irrigation demands. Restricting consideration to establishments that usually have comparatively small lawn areas, the ratios for peak hour demand proposed in the report for design applications are as follows:

Type of Establishment or Institution	**Ratio of Peak Hour to Annual Water Use; for Design**
High-rise apartments	2.6
Office buildings, general offices less than 10-years old	3.6
Department stores	2.9
Commercial laundries and dry cleaners	3.1
Conventional restaurants	3.6
Barber shops	4.4
Beauty salons	2.5

Compared with the upper curve of Figure 6-1 for groups of homes, most of these ratios are not particularly high. In a study of demands for a hypothetical community of 100,000 persons with 28,000 dwellings, synthesized from demand hydrographs given in the report, including those for a representative number of institutions and commercial establishments, it was found that for a typical winter day (no lawn sprinkling) the commercial demands during the peak hour (8) constituted only a fifth of the total. Presumably, this relative magnitude would also hold approximately for the peak hour of a year, discounting lawn sprinkling. At any rate, Figures 6-1 and 6-2

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[8] Wolff, J.B., F.P. Linawearer, Jr., and J.C. Geyer. June 1966. "Water Use in Selected Commercial and Institutional Establishments in the Baltimore Metropolitan Area." Department of Environmental Engineering Science, The Johns Hopkins University, Baltimore, MD.

of this report are hardly applicable to mixed types of developments because they are based on data for households only.

A nationally used manual[9] is being revised and this first revised edition is scheduled for printing late in 1967. The final draft of the revised edition may include an upward revision of some of the values shown in a graph that replaces that given on page 27 of Reference 9. The new graph is based on observed sewer flows, corrected for infiltration, and on water records, for municipalities and sectors of municipalities in New England with mean domestic sewage flow rates of about 0.1 mgd or greater. For a completely residential area of 400 homes, the lower limit of data would be for a mean annual sewage flow of about 250 gallons per dwelling. For 0.1 mgd mean annual sewage flow, the ratio of the minimum 24-hour flow to the mean annual is about 0.3, the ratio of the maximum 24-hour flow to the mean annual is about 3.0, and the ratio of the peak on the maximum day to the mean annual is about 5.6. The first of these is much lower and the latter two are much higher than given by the curves in Figures 6-1 and 6-2 for about 400 dwellings. However, Figure 6-1 does not include any data for New England. More importantly, the revised manual graph is for heterogeneous land-occupancy and not purely for household flows. It seems evident that Figure 6-1 may be completely inappropriate for application to mixed-occupancy areas in New England.

Aside from any consideration of groundwater and foreign surface water, it must be noted that a gravity sewer system generally has more leeway for overloading without dire consequences than a pressure sewer system. Gravity sanitary sewers designed to flow half-full at the design rate can carry essentially twice the design rate without surcharge, and in many systems some surcharge with even higher rates can occur without damage to property. In comparison, a pressure sewer has a singular operating characteristic, with the hydraulic gradient rising roughly in proportion to the square of the flow, as for a surcharged gravity sewer. As a consequence, unless extremely conservative criteria are employed in designing pressure sewers, some allowance must be made to accommodate the occurrence of occasional pressures exceeding the design criteria, in selecting booster pumping equipment and/or pressure control valves.

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[9] "Design and Construction of Sanitary and Storm Sewers." ASCE Manual of Engineering Practice No. 37, 1960.

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Chapter 7

WATER AND METROPOLITAN MAN, AN ENGINEERING FOUNDATION RESEARCH CONFERENCE

August 11-16, 1968
Proctor Academy
Andover, New Hampshire

FOREWORD

This conference was co-sponsored by the Urban Hydrology Research Council of the American Society of Civil Engineers. The campus of Proctor Academy is a traditional home of these conferences and afforded an attractive location for both intensive informal discussion and recreation.

Co-chairmen of the conference were Stifel W. Jens (Reitz and Jens Consulting Engineers) and D. Earl Jones, Jr. (Federal Housing Administration), Chairman and Secretary, respectively, of the Urban Hydrology Research Council, which became the UWRRC.

In 1965, a similarly sponsored conference dealt with the subject: "Urban Hydrology Research." That conference dealt broadly with urban hydrology research needs, their economic importance to the nation, mechanisms for fulfilling the needs, and recommended strongly that additional conferences explore specific aspects of the overall subject. This 1968 conference was responsive to that recommendation.

Although all aspects of urban hydrology research continue to be of vital importance to the nation, the need to refine identifications and understandings of interdisciplinary interfaces is still of paramount importance. Such refinement is fundamental to effective implementation of systematic (systems) approaches to metropolitan water resources problems. This conference, therefore, had as its objective the fostering of an awareness of the metropolitan establishments of the problems by involving sociologists, political scientists, administrators, planners, engineers, etc. The conference examined the role of water in the metropolitan establishment and explored the optimum involvement of the various disciplines and pursuits.

The current funded urban water resources research underway with ASCE guidance were reported upon, and plans for its continuance and supplementation were presented.

This was a "working conference." There were opportunities for extensive floor discussion and for the presentation of related information and ideas. There were detailed problem considerations by Work Groups and floor discussion and evaluation of Work Group findings.

By these means, the conference was directed toward the interrelationships of the water resource with every thread of metropolitan activity.

1.0 WATER AND METROPOLITAN MAN - CONFERENCE RECOMMENDATIONS

Section 1.0 will summarize the most salient recommendations. Priorities are not implied by the order of listing. The first 11 items for which research is briefly discussed represent the findings of Work Groups.

In an excellent and succinct "Summary of the Conference," that was presented at the last session (see Section 2.0, below), General William Whipple, Jr., discussed the "most significant ideas and thoughts which appear to underlie" the research recommendations of this *Second Engineering Research Foundation Conference on Urban Water Resources Research*.

1.1 Communication

The patent interdisciplinary character of urban water resources problems, initially presented as the first day's sessions, highlighted the urgent need for clearer definitions of concepts, words and terms that are mutually understandable by social scientists, engineers, and other planning team representatives. Research might point towards a glossary of terms of the various disciplines as such terms might occur in the interfaces of urban water problems, with more in-depth explanation than required or found in an conventional thesaurus.

Research in the improvement of communication between the disciplines and the public is suggested by the need for answers to questions such as:

1) Why do certain groups speak out on problems while others remain silent? What motivates the public?
2) What are basic public behavior patterns, with priorities often set by the public?
3) What intelligence mechanisms are available to administrators, and how are these to be selected and used?

1.2 Planning

Studies of existing successful urban water resources plans and efforts should be undertaken to identify those plans and their common ingredients that most fully satisfy categorized social needs, and would suggest better urban water resources planning goals and guidelines. Such case studies should include among their objectives: political and financial structure; engineering problems and solutions; legal constraints and encouragements; deficiencies; operations and maintenance requirements; reasons for successful implementation; time required to produce results; and, applicability to other areas. The major research goal is how to have the water resources planning best fit into the overall urban plan in all its aspects, not only to satisfy the physical human needs, but also make the urban area a better place in which to live.

1.3 Social Impacts

First, it is necessary to learn what impacts (and in what amounts) are generated by currently feasible urban water resources engineering works. Some of these impacts are on: health; recreation; economics (land values; attraction of industry; direct job opportunities on design; construction and operation of water projects; and, income feedback from health impacts); aesthetic values; relocation of residences and other activities; land-use conflicts; and, avoidance of social disturbances.

With the social impacts on those metropolitan societal subgroups most likely to benefit therefrom, the following types of projects appear to have high potential as research undertakings:

1) Physical engineering projects:
    a) Ways of more fully utilizing established water distribution systems to provide recreation and other social benefits;
    b) Injection into storm drainage criteria: recreation, open space, aesthetics;
    c) Street cleaning and sanitation;
    d) Public multi-purpose water use;
    e) Reuse of urban waters, including treatment of stormwater for recreation.
2) Institutional-organizational changes:
    a) Broadening engineering criteria and outlook to include social objectives;
    b) Pricing of water—both by cost related to use and to ability to pay;
    c) Housing codes and public controls over private activity;
    d) Participation in decision-making process by power structures and under-involved groups;
    e) Weather modification, intentional and inadvertent;
    f) Translation of new engineering ideas into the planning process.

It is recommended: that the above projects be undertaken as case studies, some with demonstration projects, to determine, qualitatively and quantitatively, the social impacts; that these projects be undertaken soon with a sense of urgency to meet expressed national goals of improving the urban environment; and, that the program be on-going with follow-up on the initial projects. In terms of the high potential payoff and low cost relative to existing programs of technical research, these procedures seem justified.

1.4 Management

Study the formulation of methods or directions in selecting the proper criteria as guidelines in establishing the local (or regional) water resource management unit; i.e., economic, physiographic, or political boundaries. Develop ways to insure the implementation of necessary and proper coordinated urban water resources development at all levels of government, including the avoidance of the obstruction of selfish interests, e.g., empire building, personal prestige, vested interests, etc. Make comparative studies of current urban water resources management practices, both here and abroad.

Conduct research on the basic impacts of pricing policy on water use as it: affects industrial and urban expansion; encourages or discourages reuse, conservation or wastage of water; provides financing for non-water related public works; encourages more effective off-peak water use through appropriate pricing schedules; indirectly affects fire insurance rates through tradeoffs; restricts or encourages water use for the amenities; and, as it impacts on sociological patterns.

1.5 Legal and Institutional Aspects

There is a great need to codify much of the common law relating to water. The roots of common law bearing upon water are in part in the great lack of sound hydrologic knowledge at the time such common law originated and developed. With today's much better understanding of hydrology (even though much still remains to be known), codification of water law could bring it in consonance with current understanding of the physical phenomena involved. Cooperative research by lawyers, engineers, economists and sociologists is strongly recommended.

The foregoing is one of the four recommended areas of research in water uses, institutions, finances and laws.

Water uses in the metropolitan area cover the entire spectrum and research might to advantage indicate how they might be within the jurisdiction of a model water resources institution with identification of the legitimate interests and responsibilities of other agencies.

Research by case studies of existing water resources institutions would give an understanding of their strengths and weaknesses. This should give guidance to whether or not, and under what circumstances there should be: a) an overall Metropolitan Water Resources Board responsible for all water programs; or b) an Advisory Water Resources Council composed of members from all the various agencies responsible for single water functions or programs (often from each of several political subdivisions); or c) a single Water Resources Coordinator, or "Czar."

Research should critically study the current financing of water facilities and operations. For example, while the sale of water is a long-standing practice with a record of apparent success, should present practice be continued? Can, or should, the water department bear the cost of other programs as it has and does provide water for fire fighting? What new resources for revenue are available?

Completion of these research tasks should lead to suggestions for several possible types of water resources institutions and sets of laws which can serve as guides to be adapted to fit specific urban areas in various parts of this country.

1.6 Regulation

Defining regulation as formal or informal action of a public or private entity for the purpose of moving decision-making toward the attainment of desired objectives leads to the following research needs:

1) For better understanding of the potentialities for mutually enhancing interaction; e.g., between planning and regulation.
2) Possibilities of obtaining better multiple-use of water.
3) Investigation of character and level of jurisdiction of organizations both geographically and functionally.
4) Relative effects of various constraints and inducements, including economic efficiency and other criteria (public and private) for implementing regulation.
5) Feasibility of designing a model for evaluating the suitability of engineering and economic regulatory tools to attain objectives.

1.7 Data Needs

Recognizing that data consists of information and not just numbers, and that needs are both specific and of a general nature, such data must: 1) reflect the measured quantity; 2) be justifiable in terms of both use and economics; and, 3) match the information requirements of both methods of analysis and the problems to be solved.

The research needs for new data for urban areas should focus on:

1) Identification of which data are transferable.
2) Data quality deterioration through population or land-use change.
3) Society's willingness to act on instrumenting water resource projects.
4) Variability of water use (qualitative and quantitative) by socio-economic groups and by land-use variations. Collect data on use patterns and possible future changes; establish minimum requirements.
5) Level of acceptable practices imposed by public desires.
6) Degree of influence that can be achieved through public relations approaches.
7) Sensitivity of system and sub-system models to accuracies in basic data.
8) How non-hydrologic data affect hydrologic designs.
9) Quantification of benefits and costs both internal and external to the system.
10) Data on post-project evaluations; e.g. public opinion of adequacy of engineering for use in quantification of benefits.

The Work Group further recommended that a program of pilot studies be initiated for purposes of investigating the aforementioned data needs. This latter affirms the continuing work that it is expected will proceed as a result of the USGS-ASCE program reported upon Tuesday at the Conference.

1.8 Precipitation

Research is particularly needed in developing stochastic models of precipitation, both in time and space; including thunderstorms and cyclonic models, and perhaps others. Method of use should be stressed, either in rainfall-runoff models or in design methods for small urban basins. There is also need for data collection and research on both dry fallout and chemical quality of rainfall in the urban environment.

1.9 Detention Storage

Possible research on the following items would provide a better understanding of the design, operation, and maintenance and social community and aesthetic gains in small detention reservoirs:

1) Techniques for optimizing design of detention systems and their multi-purpose features.
2) Improvements in techniques to determine hydrologic inputs.
3) Effects upon upstream and downstream channel stability.
4) Effects upon upstream and downstream ecology.
5) Evaluation of property enhancement or depreciation.
6) Determination and allocation of costs and benefits.

1.10 Design Problems

The research proposed by this Work Group focuses on urban design problems influenced by hydrologic processes in the broad scope of this Conference. They concluded that the following topics included areas of needed research:

1) Preservation of natural watercourses under the effects of urbanization. This research should stress the social values in the aesthetic and sometimes recreational potentials in visible, accessible natural waterways.
2) Continuing research to improve our knowledge of the effects of meteorological and physiographic factors on peak flows. Even though there is on-going research in these matters, it is included for emphasis.
3) Development of engineering and socio-economic criteria for selecting design discharge frequencies for various elements of drainage systems. An effort should be made to optimize benefits due to flood prevention.
4) Develop methodologies of storm drainage design to achieve the most balanced, efficient contributions of *all* elements of the system. This study should include consideration of the potential benefits of all types of detention storage.

5) Study existing storm drainage systems to determine their deficiencies and how they may be corrected. Consideration should be given to underground or surface detention storage where practicable.
6) Investigate the potentials of using inter-connectors between sub-systems to allow for mutual relief (through remote-controlled gates) in instances of extreme but localized rainfall.
7) It was recognized that attention is needed to relate design of all types of urban facilities to the broad interaction of metropolitan man with water in its many uses, settings and contexts. It is recommended that research be conducted in the engineering, architectural and social science fields that will lead to design guides and criteria for achieving public identification with and satisfaction from the presence of water in metropolitan areas.

1.11 Systems Analysis

The Work Group, because of the very considerable attention given the subject during the first two days of the Conference limited its activities to discussions of the following: 1) a definition of the term "systems analysis"; 2) the applicability of mathematical models to urban water system; 3) problems associated with simulation and significant figures; and, 4) recommended research areas pertaining to systems analysis. Here, only the latter will be presented as "Recommended Research Areas" (indicative rather than comprehensive):

1) Quantification and/or ranking of intangibles such as attitudes and aesthetic values.
2) Sensitivity analysis to determine the relative importance of:
    a) Particular forms and types of data; and
    b) Individual process within the system.
3) Improved methods for linking the hydrologic and social sub-systems.
4) Improved mathematical descriptions of the various urban sub-systems and their interrelationships leading to the development of a comprehensive model.

1.12 General Comments

Dr. Leonard Dworsky, Chairman of the Committee on Water Resources Research in the President's Office of Science and Technology (Washington, DC) in addressing the Conference indicated that the large federal funding of water resources research will emphasize the social aspects of urban water as well as the technological. It is felt that this Conference record indicates the high interest of the urban water engineer in the social problems and impacts of metropolitan water resources and a determination on his part to welcome the necessary collaboration of the sociologist, economist, planner, administrator, lawyer, political scientist, the systems designer, and the Federal Urban Research official in the clearly necessary team approach required to most satisfactorily solve urban water resources problems. In turn, the non-engineering participants in the Conference clearly evidenced their high interest in and desire to participate in the multi-disciplinary approach to metropolitan water problems.

At the conclusion of the Conference, there was a unanimous vote for a *Third Urban Water Resources Research Conference.* There was also a heavy majority (about 2.5 to 1) in favor of the Work Group format with more pre-planning of subjects and choice of personnel for each Group. It was suggested that more public officials be encouraged to attend.

William Whipple, Jr.
Brig. General (Ret'd)
Rutgers—The State University
Water Resources Research Institute
New Brunswick, New Jersey

2.0 SUMMARY

In summarizing the Conference, I will not cover the recommendations as to research that the Conference has just arrived at, which would be quite superfluous. I will confine myself to an impression of the most significant ideas and thoughts that appear to underlie those conclusions.

The heading of the first day's program—"Metropolitan Areas and their Water Resources," is so broad an umbrella that almost anything could have been covered under it. It was devoted mainly to the methodology of interdisciplinary water resources analysis. Practically every speaker expounded upon this theme. The opening talk by John Woods outlined a planning methodology referred to as systems analysis that was advocated as a new and evolutionary way to solve complex water resources problems. The approach appears to be generally valid, but the principles used do not seem to be particularly new. The main device relied upon, the integration of different disciplines into a common approach, was made easier in this case by the fact that all of those concerned were employees of the TRW Company. In usual experience, it is not sufficient that the team should include a member of a certain discipline; what is essential is that the member should closely represent the viewpoint of a particular organization or interest. Therefore, the members of a final decision-making Group must be inter-agency or inter-interest as well as interdisciplinary, and often each individual must report back to a particular group. The federal agencies have worked on techniques of such coordination for about 20 years; and no great breakthrough in approach appears available.

It has not been established that all processes of coordination in decision-making form a part of systems analysis. Mike Sonnen gave us a totally different concept. According to him, systems analysis is a body of methodologies through which a system can be analyzed mathematically to produce a final optimal answer. He believes that this process would not be interdisciplinary at all. Under his concept, all intangible values are too quantified, in economic terms; the systems engineer being supposedly competent to do this in spite of the usual convention by which "intangible" defines values that economists consider cannot be measured.

Most of the other formal and informal talks about systems analysis (or general economic analysis, which our speakers apparently considered as equivalent) were somewhere between these positions. Generally, most of the conferees do not consider that all intangible and social values can be measured; they agree with Wade Andrews that the attitudes of the public are not always economically rational; and, that powerful social forces and attitudes must be given weight, but cannot be expressed in dollar values.

The majority of the conferees consider that interdisciplinary and inter-interest coordination is essential for resolving most complex problems of resources development, whether or not such a process is designated a part of systems analysis. Preferably, such coordination goes through a first stage prior to the engineering part of the study; inevitably such a process of coordination occurs afterwards, whether so intended by the engineers or not. Agreement would be desirable as to what part of the planning and decision-making process should be referred to as systems analysis, what methods of systems analysis are most useful, and what principles or criteria systems analysis should apply. There seems to be a clear implication that processes such as those described as systems analysis (not necessarily referred to by that name) represent the modern approach to water resources planning and decision-making. There seems to be an even stronger consensus that unless they are most carefully conducted, with strict attention to sound basic principles, systems analysis can produce appalling errors.

Among related items, Paul Walters pointed out that "either . . . or" dichotomies are going out of fashion in favor of a continuous spectrum of ends and means. Dick Michaels stated that engineers should no longer plan independently of the social process that they are supposed to serve. However, informally he agreed that if the social sciences proceeded to establish proper concepts and procedures, engineers could proceed with much more of their work without falling into error. Jack Schaeffer pointed out that planners no longer produce a plan; if successful, they influence public decisions and programs; if unsuccessful, they prepare justifications for decisions already made by others. John Wilkenson called for research into principles of analysis, including time variables now unrealistically extended toward distant horizons. Gene Eaton recommended research into evaluation of externalities.

In all of the emphasis on methodologies of systems analysis, there was relatively little discussion of economic principles to guide such analysis. I have a primary interest in a small book,[1] "Economic Principles of Water Resources Analysis." Chuck Howe spoke of the desirability of further increasing our ability to measure what formerly were considered intangibles; in particular, he stated that it is desirable to measure the economic benefits of water quality. This point is of enormous importance. Water quality is undoubtedly the primary problem of most metropolitan areas, and unless its benefits are measured, the systems analysis must incorporate an assumption rather than an objectively determined value, and the entire optimization process will be largely deprived of its validity. The perfecting of refined methodologies of system analysis cannot give really important results in planning until progress is made on determining economic benefits of water quality.

Our talk favored great consideration of secondary benefits to show the value of irrigation projects. Apparently, the speaker lumped into this concept of secondary benefits, both indirect benefits that are properly included, and secondary benefits proper that are seldom allowable on a national basis. Another speaker from the floor

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[1] Prepared for OWWR by Brig. General William Whipple, Jr. and available through The Water Resources Research Institute of Rutgers. The State University; New Brunswick, New Jersey.

said he always allowed 50% for indirect benefits as a matter of course; still another spoke of cases in which the indirect and secondary benefits have been added in an amount necessary to justify the project with a good B/C ratio. Time did not permit adequate discussion of this matter; but such assertions may indicate the need for research into the use of indirect and secondary benefits to justify government projects—or, perhaps not research but a process of communication and education is called for.

The Conference was favored by clear and detailed accounts of progress of research under ASCE sponsorship, headed up by Murray McPherson. These projects are funded by the U.S. Geological Survey and the Office of Water Resources Research, respectively. The first, on "National Basic Information Needs in Urban Hydrology" is largely technical in nature. It is mostly related to rainfall and storm drainage, with some reference to water quality. Probably, because storm sewers and drainage are not glamorous, and are usually backed by either a federal or state agency, the deficiencies in data and in experimental verification of theoretical formulae are astounding. One of the most pressing needs is for simple reliable methods of determining flow in sewers, in order that even the simplest hydrologic studies may proceed.

The OWRR project under Murray calls for "Systematic Study and Development of Long-Range Programs of Urban Water Resources Research." This tremendously broad scope of program is reflected in the variety of work reported on. David Dawdy was concerned with mathematical modeling of rainfall-runoff relationships and of storm overflow damage. Al Kaltenback was concerned with many matters including water- based recreation and water quality. The views of Paul Walters, Mike Sonnen and John Wilkenson largely related to systems analysis as stated above. Obviously, these progress reports could only give a partial idea of what to expect from the completed project, but it seems probable that major contributions to the evaluation of research needs will result.

Some of the most interesting and informative discussions of the Conference came in informal contacts, or in the Work Groups. These discussions are represented by the quality of the recommendations of the Work Groups, which it would be superfluous for me to summarize again. There is only one point that I would like to emphasize, because it was not included in the Committee Report.

The metropolitan area outside of a large city (including the watersheds that supply that city) is normally made up of numerous towns and townships (or corresponding subdivisions) within which the responsibility for water supply, storm drainage, water quality control and land-use planning is fragmented to a very large degree. In the disposition of wastes, optimum plans of disposal seldom can be adopted, either because the optimum layout of works would extend across political subdivision boundaries, or because the favorable effects of high standard treatment occur to subdivisions other than those required to pay them. Water supply is similarly hampered. As Leonard Dworsky said, within the last generation, school districts have been consolidated across the country in order to allow organization of more effective control of schools. Perhaps a similar process will occur for certain aspects of water supply and waste disposal. An engineering forum is certainly not the best place to recommend the fundamental reorganization of our system of government. However, it seems entirely suitable to recommend research into the present institutional handicaps to proper water and waste disposal in metropolitan areas and study how, within larger concepts of government, progress can be made to remove these handicaps.

Let me close by congratulating the organizers of the Conference and all participants on the high level of discussion and the agreeable atmosphere that has been maintained throughout.

Gene E. Willeke
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Menlo Park, California
Report of Monday, August 12, 1968 Sessions

3.0 METROPOLITAN AREAS AND THEIR WATER RESOURCES

3.1 The Scope of the Problem

The systems analyst considers in harmony the various disciplines that relate to urban water resources and seeks to clarify the nature of the interfaces. In this process, one of the outputs is a determination of the extent to which one component or person in a role affects other parts of the system. A multi-disciplinary approach poses certain threats and problems, including role conflicts and the status of the engineer, for example, *vis a vis* other disciplines.

Urban planning is characterized by being multi-purpose, using multiple means, in an on-going process of interrelating various aspects of a plan. The planner's role has been interpreted differently: as a collator; a justifier of projects; or a provider of data (in which cases he is doing less than he should); as a coordinator and conceptualizer (in which cases he fulfills his proper role). The planning process no longer results in a master plan. It is a goals-oriented process that deals with uncertainty. It is a strategy rather than a blueprint.

Among the most pressing legal problems in metropolitan water resources is the upstream-downstream situation. One emerging solution is to use the concept of natural vs. unnatural runoff to force the upstream owner to pay his proper share of the costs. There is a great need to codify much of the common law relating to water. The adversary approach has hampered progress and should be replaced with a systems approach.

Natural processes collide with the social system. Technology assumes its proper role in adapting the natural system to meet the human needs expressed by society. The various disciplines can either complement or conflict with each other. Different objectives cause tensions in maximization processes because of the variable degree to which persons are benefited. Because resources are limited relative to needs and desires, rational criteria for selection of projects are needed. These criteria require, in turn, the quantification and/or identification of impacts, direct and indirect.

Social welfare has elements of intangibility. It depends on the values held by people. They are relatively stable, though of varying strengths. Choices between values often give rise to apparently irrational behavior, judged by some standards. At the present time in the federal establishment, consideration of public values and social needs is considered essential.

3.2 Initial Definition of the Water Resources Sub-system

Systems analysis is distinct from planning in that the former is a body of methodology applied in a particular way to problems. Planners use systems analysis as one item in their bag of tools. They perform a quasi-political function. The engineer's role is to provide a set of alternatives (given certain public objectives) that can be returned to the interdisciplinary arena for feedback, interaction and decision. This process breaks down if planners tell people what they need and do not ask what they want. The planner at times occupies the role of an initiator or change agent. Communication is an important problem in the process.

Engineers as administrators, designers or planners do not usually have an adequate system of intelligence (in the military sense) to determine the needs of society. Yet, they have values that are often impressed upon society. At the present time, technology fights society and vice versa. Determining the desires of society is complicated by the fact that there are multiple interests. The changing pattern of the economy and society often leaves us with decisions for which the implications are unclear. Our time perspectives are sometimes too long to be useful in such a situation, but other times, they may be too short to be more than stopgaps.

L. Scott Tucker
American Society Of Civil Engineers
Cambridge, Massachusetts
Report Of Tuesday, August 13, 1968 Sessions

4.0 ASCE URBAN WATER RESOURCES RESEARCH PROGRAM

An *Engineering Foundation Research Conference on Urban Hydrology Research* needs was held in Andover in 1965. The central recommendation of the 1965 Conference was for the acquisition of much more information and data on urban storm drainage, rainfall-runoff-quality relationships. A research proposal was prepared by the ASCE Urban Water Resources Research Program director and the ASCE research manager, designed to implement some of the central recommendations of the 1965 Conference. The proposal was presented to several government agencies for funding, which after considerable alteration, resulted in viable contracts between ASCE and the USGS and ASCE and the OWRR. The objective this session was to report the findings, to date, of the two contracts.

The bulk of the work on the two projects is performed by volunteer specialists in the form of three Task Groups on the USGS project and three Task Committees on the OWRR project. All six have met twice since the projects commenced in the fall of 1967. Members participate under their own name, but care is taken to insure that meeting findings are community views and that no direct quotation of policy of the participant's agencies or organizations can be inferred. This is accomplished by the project director writing meeting findings under his name. A man from each group and committee reported at the Conference on their particular group's or committee's main findings.

In addition to the ad hoc Group and committee reports, summaries of four systems analysis pre-feasibility studies under the OWRR project were presented. Also, two special one-man studies in progress for the USGS project were summarized.

4.1 USGS Project

The objective of the USGS project is to provide an analysis of national basic information needs in urban hydrology. The three Task Groups consisting of volunteer specialists were as follows: Data Needs, Data Devices, and Data Networks. The following is a summary of the Task Group findings, to date, plus progress reports of the two special one-man studies.

1) *Data Needs*—The Task Group was charged with determining what hydrologic data are needed that would be meaningful and helpful to designers, planners, researchers, and operating agencies. The data that are needed are the necessary information to collage the quantitative and qualitative factors of precipitation, runoff, sewage flow, groundwater and stormwater overflow; and to determine how these factors are affected by land-use, land management, topography, subsoil conditions and other contributing influences. The immediate goal should be to make hydrologic data available to national and

metropolitan centers. Concurrently, a few pilot stations should be installed on specific land-use areas in disparate climatic and topographic regions to obtain intensive hydrologic information. The long-range goal based on research and scientific studies should be the development of mathematical models usable by metropolitan centers. The models should be capable of providing designers, planners, and governmental agencies day-to-day and long-range answers regarding the effects of urban hydrology and water quality.

2) *Data Devices*—The Task Group was charged with advising on the most efficient means of collecting the needed information identified by the Data Needs Group. They formulated the following recommendations that apply to all devices in general: 1) susceptibility to vandalism must be carefully considered in the selection and development of an instrument package; 2) susceptibility of instruments and data transmission equipment to malfunction caused by lightning must be considered in their design; and, 3) thorough calibration of all devices planned for use is needed together with scientific evaluation of response, precision, bias, operating peculiarities, special installation requirements and related factors.

   The basic precipitation gauge should be a tipping-bucket gauge, with a 1/100 inch bucket, calibrated for intensity. Heaters should be incorporated to insure winter operation. A Parshall flume with stilling well was recommended as the first choice for gauging flow at an outfall. Flumes were recommended for gauging flow at an outfall. Flumes were recommended for gauging flow at locations in the system where hydraulically practical. Parshall or similar type flumes built into reconstructed inlets were recommended for gauging flow at inlets.

   Regarding water quality, the Task Group recommended that the initial pilot installations be automatically sampled with a battery of bottles. Temperature, DO, pH and conductivity could be recorded using on-site electro-chemical sensors. Automated wet chemistry should be a goal, but probably won't be used in the field at the beginning.

   Data from an urban water intelligence system should be recorded on a real time basis. To accomplish this, telemetering data to a control center was recommended. Because of the complexity of the required logic, a computer-based telemetry system was recommended. Digitization of the data should be accomplished at or before reaching the control center.

3) *Data Networks*—The Data Networks Group was charged with performing a pre-feasibility study of a data network to be designed on the basis of modern sampling techniques that would: a) permit efficient acquisition of useful information; 2) permit optimal transferability of observational data; and, 3) identify and define factors to be included in network sampling. The Task Group divided networks into the following three broad groups:

   a) Scientific networks on sub-drainage systems of a given metropolis for research purposes;

   b) General purpose networks of a given urban area for operation and planning; and
   c) National networks of metropolitan regions for transferability of data.

   The Group recommended the establishment of a number of pilot research areas that ultimately could be developed in general metropolitan networks. A primary goal is to maximize transferability of findings across the nation. The Group felt that a need exists for an organized carryover in data reporting by the agency responsible for completion of current and future FWPCA demonstration projects.

4) *Special one-man study by H.G. Wenzel*—This project arose from the discussion of the Data Devices Task Group. The study involves a review and evaluation of devices that may be suitable for measuring discharge within a sewer pipe.
5) *Special one-man study by J.C. Schaake*—This project also arose from the discussions of the Data Devices Task Group. The purpose of the work was to investigate the expected performance of various types of system components. Goals are to develop a mathematical model describing the accuracy and reliability of potential instrument systems and to identify desirable and economically attainable response characteristics.

4.2 OWRR Project

The charge of the OWRR project is for a systematic study and development of long-range programs of urban water resources research. The objective of the project is to provide guidelines for initiating and expanding a program of long-range studies in urban water problems. The work was performed by three specialized Task Committees called the Methods of Analysis, Damage and Storage, and Non-Hydrologic Aspects Task Committees. Two pre-feasibility studies were made to determine the possible effectiveness, cost and time requirements for a comprehensive systems engineering analysis of all aspects of urban water. Also, two pre-feasibility studies were made on a general economic analysis of costs and pricing parameters of all aspects of urban water. The following is a summary of the Task Committee findings, plus summaries of the four pre-feasibility studies.

1) *Methods of Analysis*—The charge of this Committee was to conduct a state-of-the-art study of mathematical models and related simulation methods potentially usable for analyzing urban rainfall-runoff-quality processes. The study will address the following: 1) model uses and model requirements; 2) modeling the loss function in urban hydrology; 3) modeling surface runoff in urban hydrology; 4) modeling water quality in urban hydrology; and, 5) evaluating models.
2) *Drainage and Storage*—The charge of this Committee was to study requirements for assessment of drainage damage and explore alternatives to direct storm runoff such as the utilization of recharge basins or other storage schemes. Among others, the group indicated there is a need to evaluate

damages, a need to evaluate the role of storm drainage in the expenditures of urban government, and a need to investigate the legal, political and jurisdictional roadblocks to effective action in environmental management. The Group also recommended a study of the efficiency of existing drainage facilities, and discussed potentials for storage exploitation.

3) *Non-Hydrologic Aspect*—The charge of this Committee was to study political, economic, legal and social problems related to urban water management with recommendations for further work. It was the Committee's opinion that immediate research needs were for more basic information and data rather than for more analytical tools, and a clearer definition of words and terms that are mutually understandable by engineers, social scientists and other planning team representatives. The following subjects were discussed at length: 1) administration of work; 2) economics of planning and operation; 3) aesthetics of urban environment and specific works; 4) financing of works; 5) health and safety of population served; 6) legal latitudes and restraints; 7) planning and land-use; 8) political impacts on planning and implementation; 9) recreational planning and operation; and, 10) sociological inferences between a viable metropolis and development of its water resources.

4) *System Analysis Pre-feasibility Studies*—Representatives from each of the four organizations making pre-feasibility studies briefly reported their findings. Economic Systems Corporation, and Water Resources Engineering, Inc. each made an engineering system analysis; and North American Rockwell and Arthur D. Little, Inc. each made an economic systems analysis. It was indicated that a systems analysis of the engineering and for economics of urban water and all its aspects would indeed be feasible. The estimates of costs of such systems analyses ranged from $265,000 to $15,900,000, and time requirements ranged from 20-5/12 man-years to 465 man-years. All four organizations indicated a great need for data. Although it was difficult to measure the potential effectiveness of a systems analysis, it was indicated that better optimization of water management functions would probably result.

5.0 COMMUNICATIONS

The Committee Membership comprised a variety of disciplines, including engineers, chemist, resource economist, educators and engineering administrators, which presented an occasional problem in communicating between ourselves. However, by proximity, the members overcame this barrier, demonstrating that this can be done.

Our basic assignment included four separate questions, and the Committee thoughts are summarized below according to each question.

1) *How to improve interdisciplinary communications.*

   First, we agreed that there is a problem in interdisciplinary communication, as demonstrated by events at the conference itself. This language barrier, however, is not unique to urban hydrology, but exists in many other fields, in many cases to a greater extent than has been observed at this conference.

It was agreed that a glossary of terms would be useful, particularly in future conferences, but some difficulty is expected in defining a number of terms, such as "systems" so that more in-depth explanation than that found in a conventional thesaurus will be required. To provide a start for this glossary, the Work Group asked all the conferees to provide lists of terms that had been used at this conference and that need definition.

It is felt that this barrier extends not only to words, but also to organization of our basic data. The consulting engineer, for example, in attempting to develop data for a particular geographic area, is confronted with the task of reducing to a common base, data based on census tract, river basin, economic region, etc. Our educational system can play a major role in overcoming this problem, and there is some trend toward this, but there remains much to be done in educating the various disciplines as to each other's language. The Committee feels that the communication problem between disciplines can be broken into two phases:

a) The need to provide a proper environment for the various disciplines to exchange ideas, and

b) Convincing ourselves of the need to understand the meaning of words used by the other disciplines.

   The old adage that the listener hears only what he wants to is particularly true in this case.

To solve this problem of interdisciplinary communication, the following ideas were suggested:

a) A research project to study the development of uniform technology in urban water resources;

b) The use of local meetings of professional societies as a forum for other disciplines to present their ideas;

c) Convincing all concerned that communication between disciplines will pay dividends to the individual; and,

d) Preparation and distribution of "reading lists" of popular books in each discipline so that others can become educated in the concepts and terms used.

2) *How can the public surely, wisely and without panic be advised of flooding and other water hazards?*

After discussing some of the mass media techniques, such as TV spot announcements used in alerting the public, the Committee questioned first, whether there is any *sure* way for a citizen to be warned, and second whether warnings issued without some type of panic connotation could be effective. Accordingly, this question was revised to read "Is there an alternative to the use of the panic technique in realistically advising the public of flooding and other water hazards"? In this sense, the Group uses the word "panic" as a connotation of a crisis needing immediate action, i.e., to achieve a goal, the public must be concerned over the problem and not merely know that the problem exists. The use of a crisis, however, cannot be overdone, or it will create a credibility gap similar to the little boy who cries "Wolf"!

The need to alert the public crosses many problem areas, including health and safety. Water quality standards for bathing waters, for example, are often set on the basis of where people will swim, rather than firm scientific bases. To alert people to the hazard of pollution, therefore, it may be necessary to create "panic" by such actions as beach closings.

Often, the specialist can recognize the existence of a need, but the community does not. How, then, can the specialist persuade the community to take the necessary action? Here, the sociologist can be of value in determining how social changes can be accomplished, and provide knowledge in selecting proper "change agents," or means of developing awareness, introducing ideas, and initiating the desired changes.

Social changes can be initiated through selection of an "initiating set," or group of citizens with influence and a following, who are persuaded of the need for the change. This set can then diffuse the idea to "diffusion sets" who spread the concept until public awareness is achieved.

There is a need for a new "social invention," a method or system to bring together the specialist and the public through the initiating sets, by participation in advisory groups, etc. The big job is to keep this influential, yet small, part of the public aware of our programs.

3) *What can be accomplished by "stream associations"?*

Both formal and informal stream associations exist, in some manner or other. Advisory Groups created by certain interstate commissions have been shown to be of value. Existing Groups, such as Soil Conservation Districts, can serve as a starting point for creation of such associations, but there is a need to ensure adequate urban representation.

Given proper guidance, these associations can be a very dynamic political force, but a tremendous amount of time and effort may be required of agencies and engineers in dealing with these groups before they are effective.

4) *How can communications between public and the disciplines be improved?*

The Committee feels that a major problem in communication between the public and the discipline, as represented by the administrator, is that the administrator either doesn't wish to communicate, because he feels he knows the answers, or is unable to understand the public, due to a lack of knowledge of social mores, etc.

We recognize that there have been many cases where such a lack of communication has caused serious slow downs in public works activities, but there is some question as to the type of communication that is required to meet the problem. The intelligence setup established in small town government, by which means local leaders determine desires and values of the public, breaks down in the complex urban society. One basic need is to replace this intelligence mechanism, but there is concern over identifying cases where such techniques as Survey Research and Public Hearings would be of value as opposed to cases where such methods present more problems than are solved. Establishment of an intelligence network may be hampered by the failure of the politician to know the true leaders, to know what sector of society certain groups really represent, or to identify the appropriate public that must be communicated with. The Committee recognizes that in the urban society there are multiple publics, each with its own distribution of interests and values.

Research ideas suggested by the Committee's discussion of this question are:

- Why do certain groups speak out on problems, while others remain silent? What motivates the public?
- What intelligence mechanisms are available to administrators, and how are these to be selected?
- Basic public behavior patterns, together with priorities that would be set by the public.

5.1 Questions from the Floor

1) Did the Committee discuss the problem of false warnings?

   Yes. We felt that this depends on the particular case. In some circumstances, false warnings may be worse than no warning at all, while in other cases, exaggeration may be a very necessary technique.

2) Does the report recognize the reasons for posting "Beach Closed" signs as a safeguard against litigation?

   Not specifically. As noted in the report, we discussed the use of beach postings as a means of alerting the public to the need for certain actions.

3) One area that could be investigated is the creation of courses in education to bring engineering concepts to non-engineers.

4) The language barrier requires an effort by all to understand each other's terminology. Combined with this effort, there should be a movement to solve the problem by reverting back to the basic English language insofar as possible.

6.0 PLANNING

6.1 Introduction

It is an acknowledged fact that a crisis exists in our big cities today. This has been caused to no small extent, by the lack of planning in all aspects of city life, poor organizational structure, by unwieldy, archaic, and unresponsive management, and by outmoded tools of government and planning. City management is big business, spending billions of dollars, yet it is run as in colonial days. It would have "gone out of business" years ago if it had been competing on the open market. The civil unrest erupting in our big cities today is a sign that they are failures and perhaps socially bankrupt.

1) *Research* is needed on city government to develop organization that can cope effectively and efficiently with modern problems in various-sized communities.
2) *Research* is needed to develop modern tools for the planning process to supersede the costly and sometimes disastrous trial and error methods of today. If officials could know to some degree beforehand what impact various courses of action would have on the whole economic, financial and social face of the urban area, intelligent decisions could be made. With limited funds, natural resources and assets, new methods or tools must be evolved that can evaluate the competing demands for these assets in order to put them to optimum use. For example, in the final analysis, the use of high-priced core area land for a small lake may not only satisfy aesthetic and recreational needs, but rising valuation of surrounding real estate may more than offset the loss of tax base on the inundated land.
3) *Research* is needed on how the water resource plan can best fit into the overall urban plan in all its aspects, not only to satisfy the physical needs of the inhabitants, but to make it a better place to live. In addition, the water resource plan could actually affect or at least influence desirable land-use.

6.2 Inventory and Analysis of Existing Successful Urban Water Resource Projects

The urban water resource plan should fit into an overall comprehensive plan for urban development. The plan should present sufficient flexibility in terms of alternatives to adjust to the changing needs of the urban society. While every urban region is unique in its features and needs, some general guidelines could be used in the preparation of a successful water resource plan of any urban area.

1) Research is needed to inventory existing successful water resource plans and efforts throughout the country and in foreign countries and to identify those plans most completely fulfilling social needs.
2) Analysis should be made of those projects to determine what particular aspects provide, or are most responsible for, the social need satisfaction. Case

studies could show their political and financial structure, engineering capabilities, legal requirements, deficiencies, operation and maintenance requirements, reasons for successful implementation, time required to produce results, and applicability to other areas.

3) The common ingredients that can be identified with the satisfaction of categorized social needs could be determined and urban water resources planning guidelines and goals could be developed based on these findings to aid and supplement existing guidelines.

4) Taxation policies at all levels of government can be a two-edged sword with both beneficial and detrimental effects. For example, in a rapid-growth area, when land is assessed for its potential use rather than its present use, it becomes financially impossible to continue farming, or continue as a riding academy when subdivisions are springing up all around. It is just a matter of time when property taxes will force the owner to sell or subdivide.

   Research: Are present taxing policies at various levels of government bringing about the most desirable results in the water resources field, or could others be devised that would serve us better?

5) *Land Reservation*—The public too often finds itself in the uncomfortable position of having to pay premium prices for land for various public uses. In many cases, the rising land values were brought about by public investments such as roads, extension of utility lines, and development of parks.

   Research—What techniques are available or could be developed to reserve or option land for future acquisition at the original "raw land" price so that the public rather than the private owner can benefit from the original public expenditure?

6.3 Storm Drainage

The next few items are on storm drainage because this area of water resource planning has received the least attention until fairly recently.

1) *Social Benefits to be Derived from Planning for Natural-Type Drainage Channels*—Natural-type waterways within an urbanizing area are too often deepened, straightened, lined and sometimes put underground. A community loses a natural asset when this happens. In addition, slow-flowing channels are speeded up causing greater downstream peak flows and higher-cost drainage works downstream. In addition, the opportunity for recharging the aquifer is much diminished.

   Research is needed to identify urban social benefits that can readily be obtained by developing greenbelts and recreation areas as a secondary drainage waterway use, even when channel rectification work is required to increase capacity. Bicycle paths, foot paths, and riding trails are a few uses that can be developed even when a narrow channel is dictated by economic necessity.

2) *Storm Runoff Detention*—Storm drainage systems are costly. The high per capita cost often encourages developers and urban officials to "turn the other way" when new developments are being planned.

   Research—New and imaginative techniques are needed to reduce the high cost of drainage and yet provide better protection. The most direct and simple method of reducing high-runoff peak flow from urban areas is to detain a portion of the runoff at or near the point where the precipitation falls. Research is needed on zoning ordinances and building codes so as to permit, encourage and require onsite ponding on roofs, parking lots, in residential yards and swales, and wherever else water can be detained at less cost.

3) *Major Drainage Considerations*—An urban area storm drainage system designed to collect the 2- or 5-year recurrence period runoff will be overtaxed when its design storm is exceeded. The waters will then seek their own path overland, often causing significant and costly damages to improvements situated unwisely. Urban planners too often ignore *major* surface drainage system needs. While there are few engineering problems in designing such systems, it would be useful to:

   a) Inventory the numerous examples of large storms over-taxing the convenience drainage system causing waters to flow down design-forgotten major drainageways, resulting in damage to property.

   b) Develop better communication with municipal officials and planners to provide an immediate awareness of this pressing problem.

6.4 Implementation

The most imaginative and desirable water resource plan is a failure if it cannot be implemented. The entire interdisciplinary team is again needed here, as well as politicians, legislators, public relations experts and others.

1) The federal government, which often holds the purse strings, is extremely influential in setting policy. Regulations have not always attained federal goals. It is well known that in spite of great expenditures on flood control works, the damages from floods are increasing each year at an alarming rate. The requirement for a 25-year flood channel through residential developments to obtain F.H.A. financing has often simply resulted in ditching what was formerly an attractive natural channel that could have served the dual role of parkland and floodplain.

   Research is needed on what the results of present policies in the field of water resources have been, and what changes in old or development of new regulations would be recommended to obtain desired goals.

2) On the state level, constitutions and legislation often present obstacles to the ideal water resource plan. In Colorado, for example, an attempt is now being made to formulate a new water code that will permit optimum use of the state's surface and groundwater supply. On the other hand, enabling

legislation is sometimes needed, to foster, for example, water resources planning on a basin-wide or regional basis as is done in the Ruhr Valley of Germany in the field of water pollution control.

Research is again needed here to identify such legal obstacles and develop a workable legal framework.

3) At the county and city level there are zoning regulations, subdivision regulations, floodplain regulations, and water and sewer policies that have a far-reaching effect on the entire urban environment. In some cases, in fact, these are so effective that the urban areas have been able to create the desired environment and attain their goals virtually without federal or state aid.

   Research would be valuable to develop model codes for local governments, with new techniques such as zoning, police power, eminent domain, easements. The Planned Unit Development technique, for instance, trades density and cluster development for open space, permitting natural drainage courses to remain untouched, and cheaper utility service because of the shorter lines.

4) In some cases, a water resources plan is put to the supreme test at the polls. This is necessitated when the plan requires a bond issue for instance. Many good plans have failed at this point in spite of substantial engineering documentation.

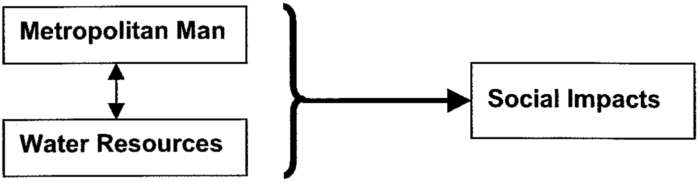
   Research would be valuable in analyzing actual successes and failures. Case studies could analyze the role of citizen's action groups, define the power structure, attitude influencing, public relations, influence of news and other communications media. Recommendations could be made to serve as guidelines of positive steps that can be taken, pitfalls to be avoided and include suggestions such as the importance of developing a palatable package proposal that will appeal to different segments of the electorate.

In conclusion, we realize that we have barely scratched the surface of this vast subject, that there are many other concepts to be investigated and questions to be posed.

7.0 SOCIAL IMPACTS

7.1 Overview of Task Group's Work

The problem area assigned to this Task Group was "the social impact of water resources on metropolitan man." It was initially decided that it was necessary to define what was meant by *metropolitan man, urban water resources*, and *social impacts*. The conceptual framework was conceived to be as indicated in the diagram below:



i.e., man interacting with the water-related environmental to produce certain social impacts.

As a beginning, the various, potentially relevant sub-sets of metropolitan man, the components of metropolitan water resources, and possible types of impacts were inventoried. These inventories are listed in the following section of this report. It was decided to consider a sequence of technically feasible urban water projects or related management practices as potential research areas for discovering what types of social impact actually are generated by the projects and to pursue the quantification of these impacts as far as possible. The particular urban water projects were selected on the basis of what the task group perceived to be priority sub-sets of the urban population that had pressing problems to which the Nation's attention is currently turned.

The overall framework within which this set of proposals is intended to fit is that first, it is necessary to learn what impacts (and in what "amounts") are generated by currently feasible urban engineering works. When such information is available, it will be possible for higher authorities to determine the relative desirability of water-related projects, *vis a vis* other major types of expenditure (e.g., education, transportation, subsidizing industry in urban areas). It may turn out that water resource projects are poor or very good tools for the solution of certain urban problems, but more study is needed before judgments can be made.

In the deliberations of the Task Group, it was considered important not to interject the intuitive notions of the group regarding what impact might be forthcoming from different types of projects, nor to interject the Group's own value judgments regarding how the various types of impacts rank in importance. It was possible to select certain priority sub-groups of society on the basis of widely recognized current problems and stated national priorities and to recommend the study of the impacts of types of projects that seemed most likely to have beneficial impacts on those priority sub-groups.

The Task Group recognizes and recommends that the findings of any of the recommended research that may be carried out be made available and injected into the urban planning process, possibly through the provision of advisors to the design teams working on the model cities programs or similar programs.

The following section lists the inventories of sub-groups of metropolitan man, water resources in the metropolitan area, and types of impacts perceived by the Group.

7.2 Analysis of the Social Impact of Water Resources on Metropolitan Man

1) Who is Metropolitan Man? Various classifications that are important to identifying priority sub-groups and possible technical remedies are:
   a) Income groups;
   b) Age groups;
   c) Occupational categories;
   d) Educational levels;
   e) Ethnic composition;
   f) Inner city *vs.* suburbs;
   g) Ghetto poverty *vs.* dispersed poverty; and
   h) National geographical location.
2) What are the relevant components of urban water resources?
   a) Physical features:
      - Water supply;
      - Drainage and flood protection;
      - Sewage; and
      - Natural water geographical location (e.g., pools, fountains, etc.).
   b) Aesthetic features:
      - Water quality;
      - Visual impact; and
      - Audio impact.
   c) Related institutional and organizational features:
      - Management methods;
      - Water use policies;
      - Weather modification (including inadvertent); and
      - Division of water-related services between public and private domains.
   d) Reciprocal communications between metropolitan water management and relevant publics:
      - Metropolitan man's participation in the decision-making process; and
      - Broadening communication channels.

3) What are the potential impacts of urban water resources?
    a) Health
        - Physical; and
        - Mental
    b) Recreation
        - Impacts on physical and mental health
    c) Economic impact
        - Land values;
        - Industrial attraction to area;
        - Direct job opportunities on water projects; and
        - Income feedback from health impacts
    d) Aesthetic impacts
    e) Negative impacts or penalties incurred or avoided
        - Relocation of residences and other activities;
        - Land-use conflicts (i.e., the opportunity costs of land used in water projects); and
        - Social disturbances avoided

7.3 Establishment of Priorities in Research

The delineation of certain priority sub-groups in metropolitan society led to the selection of priority research topics. The societal sub-groups perceived by the Task Group to have problems of high priority are:

1) Inner-city poverty groups;
2) Occupational groups of high unemployment who may be particularly susceptible to water-related employment; and
3) Education levels that might benefit particularly from the employment and training effects of water projects.

From a consideration of these groups and available technology, the following types of projects appear to be of high potential pay-off as research undertakings. It is to be emphasized again that the objective is to study the *impacts* forthcoming from these projects (both qualitatively and quantitatively), not the engineering aspects themselves (some of which are, of course, quite trivial).

1) Physical engineering undertakings:
    a) Ways of utilizing more fully established water distribution systems and providing recreation and other social benefits. Example: application of sprays to fire hydrants;

   b) Storm drainage criteria for recreation, open space, aesthetics;
   c) Street cleaning and sanitation;
   d) Public multi-purpose water use, e.g., street pools;
   e) Aesthetic improvements of present channels, streams, etc.; and
   f) Reuse of urban waters, including treatment of stormwater for recreation.

2) Institutional-organizational changes:
   a) Possibility of broadening engineering criteria and outlook to include social objectives. This should include restudy of *arbitrary* engineering design standards;
   b) Pricing of water: both by cost related to use, and by ability to pay (e.g., different price structure in ghetto area);
   c) Housing codes and public controls over private activity;
   d) Participation in decision-making process by power structures and under-involved groups;
   e) Weather modification, intentional and inadvertent; and
   f) Translation of new engineering ideas into the planning process, e.g., through case studies/demonstration projects.

7.4 Special Recommendations and Procedures

1) That the above projects be undertaken as case studies, some with demonstration projects, to determine, qualitatively and quantitatively, the social impacts.
2) That these projects be undertaken soon with a sense of urgency to meet expressed national goals of improving the urban environment.
3) That the program be on-going with provision for follow-up on the initial projects.
4) That these procedures seem justified in terms of the high potential pay off and low cost relative to existing programs of technical research.

8.0 MANAGEMENT OF WATER RESOURCES RELATED TO METROPOLITAN MAN

1) Areas for Potential Improvement in Metropolitan Water Resources Management
   a) Increased emphasis on long-range planning and the development and application of better projection methods.
   b) Develop guidelines for local inter-jurisdictional management of water resources in metropolitan areas, to include but not limited to:
      - Consolidation of water resources related responsibilities.

- Avoidance of further fragmentation of agencies that have water resource related activities.
- Definition of responsibilities.
  - a) Program recommendations for proposed legislation that would define responsibilities of political subdivisions and involved agencies (both public and private) as related to water resource management.
  - b) Urgent need for coordination of local, state and federal agencies in the area of water resources management with the objective of defining responsibilities, promoting effective public relations and encouraging competent leadership.
  - c) In studies on improvement of water resources management, private utilities, investment bankers and private business should be consulted for guidance and experience in areas of finance and management, as may be applicable to the water management field.
  - d) Consideration should be given to the formulation of methods or directions in selecting the proper criteria as bases for establishing the local water resource management unit, i.e., economic, physiographic, or political boundaries.
  - e) Research should be initiated for the investigation (and development if necessary) of methods for more rapid utilization of technological advances and management procedures.
  - f) Research is needed for the development of new and "proper" fiscal management as related to water resources, i.e., effective sources of revenue, allocation of revenue to needs.
  - g) Develop ways to insure the implementation of necessary and proper water resource development at all levels of government over the obstruction of selfish interests—empire building, personal prestige, vested interests, etc.
  - h) Make comparative studies of present water resource management practices here and abroad.

2) Basic Impacts of Pricing Policy on Water Use
   - a) Effect of price as it attracts or detracts from industrial and urban expansion.
   - b) Affects quality of the water system and its product.
   - c) Encourages or discourages reuse, conservation or wastage of water.
   - d) Affects planning for future and future research.

e) Provides financing for non-water-related public works with consequent diversion of water resource derived funds.

f) Encourages more effective utilization of water by off-peak use through appropriate pricing schedules.

g) Indirect effect of price on fire insurance rates (through tradeoffs).

h) Restricts or encourages water use for amenities, i.e., non-life sustaining uses.

i) Possible sociological impacts.

3) To improve Contradictions Among Diverse Government Programs

Examine and define the nature and extent of conflicts in public agency programs as related to water resources management and seek methods of reducing or eliminating such conflicts.

4) Questions and Comments During Discussions

Whipple......................	Comments seem primarily to water supply, how about recreational use of reservoirs (water supply reservoirs)?
Jens	Consider management of water resources as single unit with attempts to make use of excess water—storm and sanitary sewage.
Herr............................	How far can government go in departmentalization of water?
Linsley	How can per capita use be reduced? What benefits accrue from this? Conflicts result from misunderstanding of scientific basics—this can be resolved.
Whitman.....................	Fragmentation is not necessarily critical provided top management forces lower level of management. Should study how top management can bring about these coordinations.
Howe..........................	Basis of establishing rate structures should be explored considering various objectives.
Willeke.......................	Communications—power goes to those who assume it. Informal structure more important than formal since it is more realistic. Lower echelon doesn't communicate with equals in other agencies unless backed and encouraged by superiors.

Wade Andrews...........	Need research into who actually makes decisions. Too much dependence on formal charts.

5) Legal and Institutional Problems

   Goal to provide nicer place to live but retain individual rights. People do things, not institutions.

9.0 LEGAL AND INSTITUTIONAL ASPECTS

The Work Group first established that the goal was that the water resources of an urban area should be used as effectively as possible to provide a nicer place to live—while still preserving the rights of others.

In order to implement this goal it was felt that a (or several) model(s) water resources institutions (councils, agencies, districts, or what-have-you) operating under model law(s) should be developed as suggested guides for adaptation to specific urban areas.

In order to accumulate a background for the development, research is needed in the four relatively independent areas of uses, institutions, laws, and finances.

While it would be premature to suggest what form these institutions should take and what institutional arrangements are necessary, it is clear that one form and one arrangement probably would not serve all communities. Indeed, institutions may be required at several levels; regional, state, interstate, and national. Furthermore, the institutional arrangements to be considered must include not only the relationships between water resource institutions at various levels, but also the relationship of a water resource institution to its peer institutions having an interest in water, and also the relationship of a water resource institution to the usual governmental entities.

A model water resource institution(s) should be so organized that it can effectively plan and manage the urban area's water resources including at least water supply, waste treatment, and pollution control, drainage and flood control, and such other responsibilities as may be proper. Thus, water-based recreational facilities may or may not be included. However, these aspects of water can also be of interest to other agencies such as parks, health, planning, welfare; and, these other agencies need a mechanism so their needs and desires can be voiced and given due consideration. An advisory council of agencies concerned with water resources would be such a possible mechanism. In turn, the water resources institution might be represented on similar advisory councils for other agencies such as planning.

Laws may need modification to permit a model water resources institution to fulfill its responsibilities. However, it might be pragmatic to adapt the institution to present law insofar as feasible.

The four backgrounding research projects (or topical areas) recommended are:

1) The identification of water uses or programs that might be within the prerogative of a model water resources institution and the identification of legitimate interests and responsibilities of other agencies.

   For example, urban water uses are sometimes now considered to include:

   a) Water supply;
   b) Waste treatment;
   c) Pollution control;
   d) Drainage;
   e) Flood control;
   f) Navigation;
   g) Fire protection;
   h) Greenbelts;
   i) Swimming;
   j) Fishing;
   k) Boating; and
   l) Lawns and gardening.

2) A critical analysis by case study of existing water resources institutions seeking an understanding of their strengths and weaknesses.

   For example, is it more effective to have:

   a) An overall Metropolitan Water Resources Board responsible for all water functions; or
   b) An Advisory Water Resources Council composed of members from all the various agencies responsible for single water functions from each political subdivision; or
   c) A single Water Resources Czar; or
   d) A single Water Resources Coordinator.

3) A critical analysis by case studies of existing law as it relates to water resources institutions.

   For example, why is the law the way it is? Is there a lack of enabling legislation, does the law restrict unduly the authority and responsibility of such institutions? How can the law aid in the development of such institutions?

4) A critical review of the financing of facilities and operations as related to water uses and programs.

   For example, the sale of water is a practice of long standing with a record of success. However, should present practice be continued in the future?

> Can the water department bear the cost of other programs as it provides water for fire fighting? What new resources for revenue (or finance) should be tapped?

With the completion of these research tasks, the background should be accumulated that should lead to suggestions for several possible organizations of water resources institutions and sets of laws that can serve as models, or guides, to be adapted to fit the particular situations in different urban areas in different states.

The ideal researcher for these suggested projects would be competent in several fields such as an engineer-lawyer for the critical analysis of law as it relates to water resources institutions. Otherwise, teams of consultants will be required to do the kind of jobs envisioned.

The results of the on-going research (and the final suggested model institutions and laws) should be published widely to reach all interested groups; e.g., law journals, engineering journals and the Commissioners on Uniform State Laws.

10.0 REGULATION

10.1 Definition

"Regulation" in the context of the questions addressed to this Work Group is considered to mean: Formal or informal action of a public or private entity for the purpose of moving decision-making toward the attainment of desired objectives. Regulation may be *operative at any level* of the public or private sectors. (E.g., in the public sector: local subdivision, municipal, state or national government, and in the private sector: tract developer, bond-holders consortium.)

Desired objectives may be preponderantly hydrologic such as management of storm runoff affected by hydrologic processes, or the objectives may be preponderantly non-hydrologic such as urban configuration, control of waterborne disease, recreation development, and ghetto improvement. In a good many cases, the objectives may be an intimate mixture of hydrologic and non-hydrologic objectives.

10.2 Regulations (as above defined) are Needed

1) To secure adequate and effective management of water resources, resources related toward water, and related facilities:
    a) For storm runoff;
    b) For erosion control and sediment production (especially in new developments);
    c) For water quality protection, e.g., sewer separation; and
    d) For compatibility with river basin plans.

2) To deal with emerging new problems consequent to coalescence of urbanized areas into metropolitan belts and metropolitan regions of large geographic area and huge populations, e.g.:
    a) Trans-basin water supply;
    b) Waste management especially protection of estuaries and lakes; and
    c) Recreation development.
3) To secure amenity or other social benefits:
    a) For support of the metropolitan area plan with respect to distribution and configuration of land occupancy and use;
    b) For realization of recreation, open space, and aesthetic enhancement potentialities;
    c) For local and/or regional economic flood plain development; and
    d) For alleviation of the conditions of the urban poor and unemployment, ghettos.

10.3 Effective Regulation

1) Regulation as thus identified, includes as a principal function (per Sheaffer) bringing about optimal coordination of often multiple and disparate efforts so as to enhance their effectiveness in the attainment of common or public purposes. Such coordination may be operative through, for example:
    a) Large scale federal effort (creative federalism), e.g. HUD programs, FHA loan standards, FWPCA treatment plant construction grants, etc.;
    b) Public state, county, municipal programs, e.g., open space and clean river restoration projects;
    c) Design and operation of systems of facilities; and
    d) Guidance of private development and by private financing, e.g., fire insurance underwriters and bondholder's inducements or penalties, and price mechanism.
2) It is necessary that the requirements be engineeringly feasible, economically practicable, especially as to the availability of funding, and socially acceptable—but each of these criteria may have to be re-examined from time-to-time in the light of the rapidly changing conditions of the metropolitan environment and of technological capability and economics.
3) In the United States, the establishment and administration of public (i.e., government or quasi-government) regulations requires appropriate representation of those affected by the regulation.

10.4 Research Needs

1) Research for better understanding of the potentialities for mutually enhancing interaction:
    a) Planning and regulation;
    b) Technological improvements and regulation; and
    c) Regulation and economic conditions (economic growth and development).
2) Possibilities for attaining better multiple uses of water (rooftop and surface retention for use in air-conditioning systems, retention of surface runoff for use in recreational ponds).
3) Investigation of:
    a) Character and level of jurisdiction for organizations both geographically and functionally for establishment and administration of the various types of regulation in different circumstances; and
    b) The advantages and disadvantages of combining within a single organization regulatory responsibilities for other functions such as:
        - Regulation for storm runoff in central cities versus in major metropolitan belt;
        - Similar for recreation;
        - Possible combination of regulatory function and water system responsibilities; and
        - Same for water regulatory functions and building codes.
4) Feasibility of designing a model for evaluating the suitability of engineering and economic regulatory tools to attain objectives.
5) Relative effectiveness of various constraints and inducements, including economic efficiency and other criteria (public and/or private) for implementing regulation.
    a) Identification of costs and benefits of regulation, including externalities; and
    b) Design of regulation to comprehend externalities (for example, the effectiveness of various combinations of public and private financing).

11.0 DATA NEEDS

Data consists of information, not just numbers. Data needs are both specific and of general nature. These data must possess the following characteristics:

1) Reflect the measured quantity;
2) Match the information requirements of both methods of analysis and the problems we must solve; and

3) Be justifiable in terms of both economics and use.

11.1 Research Needs in New Data for Urban Areas

1) Identification of data that can or may be transferable.
2) Quality deterioration through population or land-use change.
3) Society's willingness to act in instrumenting water resources projects.
4) Variability of water use (qualitative and quantitative) by socio-economic groups or by land-use variations. Establish minimum requirements and attempt to collect data on use patterns and possible future changes.
5) Level of acceptable practices imposed by public desires.
6) Degree of influence that can be achieved through public relations approaches.
7) Sensitivity of system and sub-system models to accuracies in basic data.
8) How non-hydrologic data affect hydrologic designs.
9) Quantification of benefits and costs, both internal and external, to the system.
10) Data on post-project evaluations. Public opinion of adequacy of engineering for use in quantification of benefits.
11) Maximizing the value of information from data gathering procedures.
12) Framework for establishing the priorities of our information needs. Data needs would stem from these information requirements.

11.2 Supporting Data

Data source book containing reference to all water resource data, not just hydrologic data; e.g., census, projections, etc.

11.3 Event versus Fixed Time Recording

Would depend upon information requirements of data.

11.4 Data Format Requirements

Advocate urban environmental data centers for regional assemblage of all environmental data.

11.5 How to Achieve Regional Centers

Primarily local region responsibility. Perhaps through a Large-City Mayors' Council, or Regional Planning Groups using assistance of:

1) University;
2) Private research organizations;

3) Consultants; and
4) State governmental agency.

11.6 Justification of Data Need

1) Necessary input to system; and
2) Would allow better understanding of the complex interaction of the many subsystems.

11.7 Recommendation

That a program of pilot studies be initiated for purposes of investigating the aforementioned data needs.

12.0 PRECIPITATION

Two points should be considered as assumptions throughout our considerations:

1) It is understood that probability concepts should underlie all studies in precipitation; and
2) Design applications should be kept in mind and stressed in all studies of precipitation in the urban environment.

Facets of precipitation that should be considered:

1) Amount
    a) Intensity; and
    b) Duration.
2) Time distribution—most important for small basins.
3) Areal distribution—most important for large basins (including storm movement).
4) Variability in each of the above. (Sampling error.)

Research is particularly needed in developing stochastic models of precipitation, both in time and space.

1) Thunderstorm models;
2) Cyclonic models;
3) Other models?; and
4) Method of use should be stressed, either in rainfall-runoff models or in design methods for small urban basins.

Where do we stand?

1) Time distribution at a point for a storm has been studied extensively. Autocorrelation in time has not been studied sufficiently.
2) Areal distribution and cross-correlation has not been stressed sufficiently.
3) Including both time and area distributions in one model is deficient.
4) Utility of stochastic modeling has not been investigated critically.
    a) Is modeling of area distribution for urban watersheds necessary?
    b) Is a stochastic model necessary, or can other methods suffice?
        - I.e., translation of rainfall records.
        - Abandon rainfall-runoff models entirely?

The errors that result from the use of a point rainfall record as a measure of basin rainfall should be studied insofar as their effect on simulation results. (Instrument measuring errors excluded.)

1) Effect in the rainfall model.
2) Effect in the modeling results (relative importance).
3) Comparison of different methods of making rainfall records "representative." (Sensitivity of results to methods of removing bias.)

The question of representativeness of given rainfall records in respect to rainfall on a basin of interest.

1) General regional variation; and
2) Local effects—orographic, etc.

The above has considered only quantity of rainfall and its relation to drainage design. In addition:

1) Need both data collection and research on both dry fallout and chemical quality of rainfall in the urban environment;
2) Snowfall affects the urban environment:
    a) Design of sewer inlets and manholes for easy snow removal; and
    b) Removal of snow by chemical methods as opposed to alternative methods. "Cheapest" method may actually be the most expensive;
3) Highway design as applied to trafficability and safety during rainfall. Planning effect—how can research in rainfall be used to design highway crowning, for instance?

13.0 DETENTION STORAGE IN URBAN AREAS

The Work Group met and had two lively sessions. It became evident in the first part of the first session that the size of the detention storage pond had to be limited for this study. We tried several ways to limit the size according to drainage area, but this was purely arbitrary. We finally decided to use the word small and call the study a study of small detention reservoirs.

Also, it became desirable to make the following table of the types of storage considered in this report.

12.1 Types of Storage

1) Focal Point (reservoirs)
    a) Dry
    b) Permanent pool
2) Dispersed
    a) Storm sewers
    b) Soil and stratigraphy
    c) Roofs
    d) Backyard swales
    e) Channel and floodplain

With the types of storage shown above, the next step was to consider the use and objectives.

Again, the Group felt that to place them in tabular form was the best approach.

Table 7-1

Uses and Objectives

	Focal Point		Dispersed
	Dry	Wet	
1. Reduction of flood damage			
2. Downstream hydrograph control			
3. Water oriented recreation			
4. Water supply			
5. Water quality improvement a. Sediment and debris removal b. Low flow augmentation			
6. Groundwater recharge			
7. Social impact			
8. Minimization of storm sewer related facilities cost			
9. Reduction of long-term depreciation rate			
10. Floodplain encroachment			

The above table was completed for some typical examples using symbols[2] to designate positive, negative, or not applicable situations.

With the completion of the two above tables, and with the study and discussion that went with the development of the tables, it became evident that there was a lack of knowledge and experience with the conception, design, operation and maintenance of small detention reservoirs. The Work Group believes that possible research on the following items would provide a better understanding of the design, operation, maintenance, and community betterments of small detention reservoirs.

13.1 Possible Research Subjects

1) Techniques for optimizing design of detention systems and their multi-purpose features.

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[2] The discussion of the entire Conference after presentation of the Work Group Report resulted in a general opinion that the table as presented in the Conference Report should not include the Work Group's pluses, minuses and "not applicable" designations, on the ground that such evaluations "were in some instances controversial, and not of important significance, except for each specific project. The Work Group agreed to the omission of its "evaluations" in the table.

2) Shoreline management for blue-green concepts (water level fluctuations, types of vegetation, vector control, permitted use, etc.).
3) Responsibility for management of access and use of detention storage area.
4) Evaluation of property enhancement or depreciation in a community.
5) Determination and allocation of costs and benefits.
6) Stormwater quality in wet ponds.
7) Improvements in techniques to determine hydrologic inputs.
8) Effects upon upstream and downstream ecology.
9) Effects upon upstream and downstream channel stability.

14.0 DESIGN PROBLEMS

14.1 Introduction

The Committee endeavored to scan the range of urban design problems that are influenced by hydrologic processes in the context of the broad scope of this Conference. Attention has been focused on seven topics, each of which included one or more areas of needed research or investigation. The discussion included a consideration of the development of entirely original concepts for the removal of stormwater. No completely new system was indicated, but a number of innovations are included in the recommendations for research.

14.1.1 Preservation of Watercourses Under the Effects of Urbanization

It is desirable to keep natural waterways visible and accessible because they constitute valuable aesthetic and sometimes, recreational resources. This involves both the protection of the stream itself, and the elimination of the encroachment by buildings in the adjacent floodplains. The problems arise because of indiscriminate unregulated land development and/or because increased runoff rate due to urbanization requires increased streamflow capacities. Alleviation of the problem can be brought about in part by zoning ordinances that will prevent misuse of the streams and adjacent land. However, if the capacity of the streams must be increased, proper design methods should be developed to maintain, insofar as possible, the social benefits derived from the presence of the watercourse. Design decisions may involve the choice between a closed conduit and an open channel. The open channel is, of course, to be preferred if it can be designed in such a manner as to maintain a pleasing appearance. The use of a deep channel with steep side slopes reduces greatly its aesthetic value and may create an attractive hazard for children. Consideration should be given to the use of detention basins to reduce the design discharge. The detention basin might also serve to remove sediment or for recreational purposes. Public understanding is vital and alternative methods and costs should be brought clearly to the attention of the affected people.

14.1.2 Research Needs

1) Develop criteria for establishing zoning ordinances for the purpose of maintaining, insofar as possible, the aesthetic benefits of watercourses.
2) Develop optimum channel designs taking into consideration operating efficiency, maintenance, safety and aesthetic and recreational values.
3) Carry out research in public attitudes toward the objectives of increased safety and aesthetic values, both among people living near the waterway, and among those living farther away.
4) Attempt to quantify the benefits of the aesthetic, recreational and safety factors, insofar as possible. If quantification of all factors proves to be impossible, develop a scale of social values to assist in decision-making.

14.1.3 Continuing Research

Continuing research to improve our knowledge of the effect of meteorological and physiographic factors on peak flows, with special consideration of the effects of urbanization. It was recognized that there is on-going research in this area, but because of its importance in most aspects of design, this topic was included in the list.

14.1.4 Development

Development of engineering and socio-economic criteria for selecting design discharge frequencies for various elements of drainage systems. An effort should be made to optimize benefits due to the prevention of flooding.

14.1.5 Investigation

Investigate the design of storm sewer systems to achieve the most efficient contributions of all elements of the system.

This investigation should include an examination of the potential benefits of all types of storage. Elements of the system such as inlets and storm conduits should be kept in balance. The functioning of total drainage systems during storms exceeding the design capacity should be studied and their effect on the environment and essential public services should be determined. An investigation of the efficiency of individual elements of the system should not be overlooked.

14.1.6 Deficiencies

Determine the deficiencies in the existing storm sewer system and determine methods for alleviating their inadequacies. Consideration should be given to the use of underground storage, and, hopefully, other procedures that are not now part of conventional design practices.

14.1.7 Benefits

Investigation of the benefits of using inter-connectors between storm sewer systems to allow for mutual relief in case of extreme but localized rainfall. Under special conditions, similar advantages might be obtained from cross-connection in surface drainage systems.

14.1.8 Design

As a Design Group, it was recognized that attention is needed to relate design of all types of urban facilities to their effects on the broad interaction of urban man with water in many uses, settings, and contexts. The results of urban design efforts may play as significant a role in an individual's or society's psychological, social and cultural well-being as in their economic success and physical comfort. Furthermore, the existence of water and waterbodies plays an important role in establishing a water-oriented image and identity for some cities (Chicago, San Antonio, Annapolis) whereas in other urban areas, inhabitants and visitors are hardly conscious of the presence of water. Specific interrelationships between urban man and water that are directly affected by design include:

1) Man's access to waterfronts and water.
2) Visibility of waterbodies in urban areas.
3) Use of water for unique forms of urban transportation.
4) Inhabitation of urban areas by desirable forms of wildlife and vegetation.
5) Use of audiovisual properties of water created by rapid flow, fountains, falls and other interesting hydraulic configurations.

Man's relationship with water in the urban environment is affected not only by design of hydraulic structures, but also by design of roads, highway bridges, buildings, docks, playgrounds, parks, etc.

Furthermore, the engineering profession can exercise its influence with communications and advertising media to promote closer identity between urban man and the water resources.

In consideration of these and other socio-hydrologic relationships, it is recommended that research be conducted in the engineering, architectural and social science fields that will lead to the development of design guides and criteria for achieving public identification with and satisfaction from the presence of water in urban areas.

The possibility that new ideas can be obtained from a study of design procedures outside of the United States should not be overlooked.

15.0 SYSTEMS

During the course of the Conference, the subject of systems analysis received considerable attention. Therefore, the Systems Work Group limited its activities to a discussion of the following four items on which questions had been raised:

1) Definition of the term "systems analysis."
2) Applicability of mathematical models to the urban water system.
3) Problems associated with simulation and significant figures.
4) Recommended research areas pertaining to systems analysis.

15.1 Systems Analysis Definition

The goal of systems analysis is to aid the attainment of a solution that is either a satisfactory response or a satisfactory final decision. Consequently, systems analysis may be considered as either a straightforward or repetitive process for finding satisfactory solutions to a problem. It requires:

1) System definition through description of physical, economic or other boundary conditions; assembly of necessary data; and formulation of pertinent relationships;
2) One, or a body, of analysis methodologies; and
3) Production, through the analysis techniques employed, of solutions or responses to conditions specified by the input data.

Systems analysis may or may not receive inputs from various disciplines and may or may not produce tentative decisions as outputs of the methodology. Facility for feedback from the initially derived responses or decisions to gain improved or updated multi-disciplinary inputs, system definition, or analysis methods may be necessary. It is important to note that final decisions are made outside the systems analysis process.

15.2 Applicability of Mathematical Models to Urban Water Systems

The two most promising applications of mathematical models to urban water systems are:

1) The description of the many possible ways that physical components of the urban water environment can react to postulated stimuli or loads. This promises greater understanding of individual phenomena and also makes possible sensitivity analysis of the various phenomena;
2) The solution of mathematically formulated objectives to aid the decision process, such as minimization of costs, maximization of net benefits, minimization of water losses, and similar possible urban goals; and
3) The use of simulation models to describe constraints for optimization models.

Examples of components of the urban water environment that have received some systems analysis research attention include:

1) Water storage and distribution systems;
2) Urban storm sewer systems;
3) Water and waste treatment systems; and
4) Quality variations in receiving waters.

Other components that (at least in the experience of the Group members) appear worthy of more attention than in the past include:

1) Reuse (return) systems;
2) Water loss systems (consumptive and non-consumptive);
3) The water use system as its subcomponents change water quality and quantity; and
4) All the subsystems as they affect water quality.

15.3 Problems Related to Simulation and Significant Figures

Simulation of complex systems can involve long computational routines. There are two problems concerned with significant figures in such a modeling procedure:

1) In the mechanical procedure of computation, errors may be propagated and either magnified or damped out. If they are magnified, the number of significant figures in the results will decrease as computation continues.
2) The complexity of the model should conform to the level of significance of the data.

If results with only one or two significant figures are desired, a highly complicated simulation model may not be justified. An example of this is the determination of peak runoff rates as a function of return period.

The first problem above is a common consideration in numerical analysis. For some computation schemes, the mathematical stability and convergence properties can be determined analytically. For most simulation schemes, an empirical testing of the system is all that can be done to examine stability and convergence. Since the nature of the scheme, with respect to stability, will usually be different in different ranges and combination of ranges of the model variables and parameters, complete assurance of stability is rarely possible.

Tests should be included in complex systems to examine values of variables for ranges outside those normally expected.

The second problem requires that the sophistication of the model be geared to the sophistication of the data. However, all computations should be made with sufficient accuracy to preserve the number of significant figures in the input data. The relationship between model and data significance also implies that efforts at improving data accuracy should be directed toward those data that are limiting the number of significant figures of the model output. The results of the simulation should be presented in a manner that suggests the number of significant figures in the data.

In collecting data for the evaluation of a simulation model, it may be convenient to have intermediate or interior data as well as input and output data. Testing or comparing interior data points can give additional verification and provide reference points for computer checks of the stability of calculations.

No specific research by engineers appears necessary on the effect of significant figures on simulation modeling. However, anyone involved in modeling complex systems should investigate the significant figures in the output to be expected from the range of errors in parameter and input variable values. A computer will print out data as if it had eight significant figures even if there are no significant figures.

15.4 Recommended Research Areas (Indicative Rather than Comprehensive)

1) Quantification and/or ranking of intangibles such as attitudes and aesthetic values.
2) Sensitivity analysis to determine the relative importance of:
    a) Particular types and forms of data; and
    b) Individual processes within the system.
3) Improved methods for linking the hydrologic and social subsystems.
4) Improved mathematical descriptions of the various urban subsystems and their interrelationships, leading to the development of a comprehensive model.

Chapter 7, Appendix D[1]

REFLECTIONS ON THE RELATIONSHIPS BETWEEN SOCIAL SCIENCE AND HYDROLOGICAL ENGINEERING

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As a social scientist who was asked to address a large group of distinguished engineers, it is appropriate for me to tell you that I deem it an important and stimulating opportunity to do so. Furthermore, it is incumbent upon me to report that I have always found my many associations and collaborations with members of the engineering profession to be highly rewarding and critically important to my own professional endeavors. When one stops to consider the number of transformations worked upon the world by engineers and the rapidity with which those transformations have occurred, he must inevitably acknowledge the great influence this group has on social relationships. Engineers are without question among the foremost agents of social change. Only through the most narrow technological interpretations can we conclude that social problems are not, in part, engineering problems as well.

There is, then, no excuse for the failure of engineers and social scientists to collaborate in their research efforts. Engineering facts have deep social consequences. The urban ghetto is today as much an engineering problem as it is a social problem. In many ways, it is an engineering problem because it is a social problem. Some of the solutions to social problems will require engineering solutions. To put it another way, the manner in which we have constructed our engineering feats, and they have been very large indeed, has been partially responsible for the type of social system we now possess and the directions it has taken. Moreover, what we do with our engineering systems in the future will determine the type of society in which we will live just as surely as will the things that we do politically and economically. Thus, as engineers, you are more than just technicians selling your skills to bidders. You play a very large role, along with others to be sure, in determining the quality of the American society and its culture.

The responsibility about which I have just spoken points up several of the most important areas of research for collaboration between social scientists and engineers. Above all else, we need to know how engineering feats and technological transformation affect and direct the social system. Presently, we possess only the crudest and most impressionistic generalizations about the ways in which social change relates to technological change, as well as how these two types of change interact and affect each other. We need more than just casual historical surveys and impressionistic assessments.

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[1] Other appendices are not included due to space limitations.

Among the most critical research needs are determined efforts to find means and ways to link engineering systems to social systems in research designs. This must be done in a manner that enables us to test for consequences, especially crucial outcomes. Briefly, we must begin now if we are ever to begin determining how elements of engineering systems affect, relate to, or interact with elements of social systems. This challenges us to undertake interdisciplinary systems approaches. Engineers and social scientists must develop models and research designs through which we can test empirically for those linkages or relationships. I realize fully that many social scientists and engineers alike will call me to task for being what they believe is utopian and unrealistic. I do not share their conservative outlook. I am confident that research into such questions will be undertaken within the next decade, and I am convinced the yields will afford greater opportunities, as well as a richer culture for all of us to share and enjoy. I am also convinced that those people who begin making attempts to relate engineering and social systems now will be ahead of the game, and that they will be richly rewarded for their efforts. The road will be a difficult one that is full of frustrations and disappointments. But no road to discovery is a very easy one.

Successful collaborations will depend upon a new type of social scientist who recognizes that collaboration with engineers affords him as great, if not a greater, opportunity to affect the social outcomes than he would have if he remained in his traditional molds and pursued the predominant orientations he now displays. Moreover, successful interdisciplinary relationships will require engineers to appreciate the underdeveloped state of many of the social sciences. Engineers will have to recognize the non-parametric conditions of the social science data. This means that engineers will be required to work more with stochastic models than some are now inclined to do. However, it will also challenge social scientists to search for parameters to their systems so that they can develop parametric models that will be adaptable for relating social elements to the elements of engineering parametric models.

Furthermore, in these endeavors, as well as in others involving the social sciences, engineers must be mindful of the softness of much of the social science data. They must also remember that this softness makes the generalizations more suspect, but no less important. In this regard, social scientists, and many engineers also, must resist the old temptation to collect that data that are easily obtained rather than collect data that are more difficult to obtain but more essential and important. Social scientists also need to develop new, and refine old, statistical techniques. In this endeavor, engineers can be of great assistance to the functioning social scientists, eventually helping to overcome the very frustrating tendency for much of the social data to interact. The dilemma of interacting data is what keeps much of the data soft, for scientists continually find that they have more unknowns that they have equations for solving them. They are, in short, under-identified in their equations. It poses the need for more sophisticated statistical measures.

Where then can the social scientist be of the greatest aid to the engineer? I would suspect that he could help most by obtaining research yields that indicate where and why some of the finest engineering programs meet defeat in the social, political, and economic decision-making milieus. I know from experience that this is most frustrating to many urban hydrologists, for example. I am certain that we will find that answers to this dilemma involve more than easy impressionistic assessments such as better public relations or "selling" rather than "telling." These measures will help in many instances, but they are not sufficient to meet rapid developments in the engineering fields such as urban hydrology. We need to know the impacts that urban hydrology has on urban and rural decision-making systems and how decisional processes can be penetrated to achieve progressive goals. If this is to be done, one of the first things social scientists must undertake is development and refinement of measures for quantifying social values, social costs, and social benefits. Further, we must develop measures of the impacts of water services, types of management, policy formulation, and community life in general. Certainly, we need intensive investigations of water communication systems. We need to know how certain water systems, their publics, and their decision-makers communicate with each other. We also must begin studying the ways in which researchers and members of the various academic disciplines communicate with each other and their water publics, policymakers, and managers. Another area of study and analysis involves experimentation through the employment of simulation wherein engineering models are linked to social models. Here again, stochastic models will be of higher utility than parametric ones, due to the present condition of social data. For example, it is now quite possible to simulate model water institutions operating under model water laws in a specific environment that is loaded into the simulator.

To my way of thinking, the dictum that we cannot measure many of these things is no longer tolerable. We simply have to do it. Human ingenuity has conquered more frustrating problems in the past and it will do so in the future. Without further adieu, please permit me to merely suggest just a few problems to which both engineer and social scientists can begin lending their mutual efforts at the present time. Hopefully, I can whet a few appetites and we can expand this list, both as this Conference progresses and during the years to come.

1) We need a great deal more information on how legal systems relate to and affect both engineering and/or social systems generally. In so doing, we need to search for the opportunities afforded under existing laws as we do for the restrictions imposed by law.
2) We need to search out the effects of both public and private decisions, and we need to define more rigorously the responsibilities of each segment of the public and private decision-making arenas.
3) We need to discover ways for consolidating water resource administration.
4) Above all, we need to study and analyze planning and planning behavior thoroughly and meticulously.

5) We certainly need to develop criteria for preserving engineering, safety, and aesthetic values.
6) We need to know more about the formulation and behavior of public, and decision-making coalitions.
7) We need to know the roles and the impacts of those roles of urban hydrology in the model city programs.
8) We need to know more about what motivates publics and their decision-makers.

These are just a few of the needs outlined for research collaboration between social scientists and engineers. I hope to raise more as the conference proceeds. However, these alone should stimulate someone here. Hopefully, at least, I have raised a few challenges.

Chapter 8

RESPONSE CHARACTERISTICS OF URBAN WATER RESOURCE DATA SYSTEMS

August 1968
ASCE Combined Sewer Separation Project
Technical Memorandum No. 3
New York, New York

1.0 SUMMARY

This report is concerned with the collection of data. The need for these data is apparent to many who have worked with urban water resources, but different needs seem to exist for different reasons. This creates a serious problem because the financial resources to collect these data are limited.

At a recent IBM symposium on Air and Water Resource Management, Professor R.K. Linsley[1] chaired a session on Planning and Data Systems. His message to that Conference encompassed two points. First, that the computer, which allows us to do things more rapidly than in the past, also allows us to do things differently than we did formerly. The second point, which partly follows from the first, is that computer applications may impose new data requirements so that new conceptual approaches to data collection may be needed.

I believe that data should be collected to satisfy some definite purpose. Without a purpose, there is no basis for comparing alternative data systems. With a purpose, there is a basis, although it may be difficult to rank alternative systems if the objective is not well posed.

The danger exists today of collecting too much data, although this may seem absurd when the deficit is so great. Automatic data recording and information processing systems are so totally more efficient at this task than humans that it becomes easier to collect more data than to decide if these data are worth collecting.

An assumption is implicit here. If some data are not worth collecting, something else must be more worthwhile. In the United States, we have economic criteria to help decide if one thing is worth more than another, so it seems reasonable to apply that measure to data systems. Large data systems, once they get started, are hard to stop or to change significantly. This is because a data system is not only a technological system, it also is a social system. It involves the people who run it, and everyone else for whom it is run. Therefore, the benefits that any large data system might provide should be weighed against the costs of that system at the time the system is designed.

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[1] Linsley, R.K. October 1967. "Chairman's Remarks." Proceedings IBM Scientific Computing Symposium on Water and Air Resources Management.

The task of estimating the potential benefit of urban water resources information is formidable, if not impossible. It would seem presumptuous of anyone to pretend to have accomplished this task adequately. However, not knowing the exact benefits does not preclude us from seeking that information, nor does it exempt us from applying economic criteria to our data systems.

For the present, I think these systems should be designed to meet some definite measurement requirement at minimum cost, or to give maximum information content for a given budget constraint. Which of these may be preferred is irrelevant because they are essentially equivalent from the Economic and Operations Research points of view. In the context of these objectives, optimal schemes for gauging have been discussed by Fiering,[2] Matalas,[3] Eagleson,[4] and Gunnerson.[5]

Application of these new methods to plan new data systems imposes the need to consider system response characteristics as they relate to specific measurement objectives. The types of system components that might be included in a pilot system to initiate an open watershed program for urban areas have been the subject of this special study. These are classified into three groups of instrumentation: for rainfall, for runoff, and for water quality. Separate investigation of each of these has been made and is reported below. In the final section on water quality, an optimization scheme is suggested for sampling water quality. Although it is based on tenuous assumptions, the model should be developed further, and its implications for water quality data systems should be explored.

As part of the final design phase of a pilot data system, alternative instrument systems should be simulated so that opportunities to alter the response characteristics may clearly be anticipated. The value of a computer instrument system simulation model should not be limited to the design phase; it should extend to studies of data characteristics as well. For example, the model could be used to devise measures of accuracy for specific future research problems. Also, suggested system improvements could be tested and the benefits from these improvements to other measurement objectives could be quantified.

2.0 RAINFALL

Rainfall measurements usually are made with weighing-type or tipping-bucket-type rain gauges. Both of these measure the total volume of rainfall accumulated up to a given time. This measurement is made with a definite precision that is determined by the size of the tipping-bucket or by the weighing and recording system of the weighing-type of gauge.

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[2] Fiering, M.B. 1965. "An Optimization Scheme for Gauging." *Water Resources Research.* Volume 1, No. 4.

[3] Matalas, N.C. October 1967. "Optimum Gauging Station Location." *Proceedings IBM Scientific Computing Symposium on Water and Air Resource Management.*

[4] Eagleson, P.S. 1967. "Optimum Density of Rainfall Networks." *Water Resources Research.* Volume 3, No. 4.

[5] Gunnerson, C.G. October 1967. "Optimizing Sampling Intervals." *Proceedings IBM Scientific Computing Symposium on Water and Air Resource Management.*

The basic variable to be measured is rainfall intensity, but measuring devices usually measure the integral of this variable so that the recorded variable must be differentiated. This step of differentiation introduces an error term in the expression for the rainfall intensity.

2.1 Rainfall Measurement Error Model

Let P (t) be the instantaneous rainfall intensity. Let the average rainfall intensity over the previous interval, T , be

$$\mathrm{I}(\mathrm{t}, \mathrm{T}) = \frac{1}{T} \int_{-T} \mathrm{P}(\mathrm{t})\, \mathrm{dt} \tag{1}$$

Let the measured value of I (t, T) be î (t, T). The ratio

$$\mathrm{r}\,(\mathrm{t}, \mathrm{T}) = \frac{\hat{\imath}\,(\mathrm{t}, \mathrm{T})}{\mathrm{I}\,(\mathrm{t}, \mathrm{T})} \tag{2}$$

is a random variable.

For measuring devices to be acceptable, the expected value of r(t, T) should be near 1.0 for all values of T and patterns of rainfall P(t). Because the integral of P(t) is measured, it seems likely that the expected value of r(t, T) is near unity for typical rainfall patterns.

The variance of this ratio must also be considered in the design of an instrument system. The square root of the variance is the standard deviation. The ratio of the standard deviation to the mean is the coefficient of variation.

Because the coefficient of variation is directly related to the percent error in estimates of I(t, T), the coefficient of variation of the ratio, r(t, T), has been used throughout this study as a surrogate for the accuracy of alternative measuring systems.

The statistical model for î(t, T) is

$$\hat{\imath}(\mathrm{t}, \mathrm{T}) = I(t, T) * r(t, T) \tag{3}$$

in which r (t, T) is a random variable with mean μ and variance σ^2. The coefficient of variation of r(t, T) is

$$C_V = \frac{\sigma}{\mu} \tag{4}$$

because

$$\mu \quad \sim 1 \tag{5}$$

it follows that

$$C_V \sim \sigma \tag{6}$$

2.2 Criteria for Rainfall Measurement

Three specific rainfall measurement objectives have been studied to investigate the response characteristics of various instrumentation systems. Different objectives arise from different needs for rainfall information for different research purposes. Each objective imposes unique requirements on the instrumentation system. The objectives considered here are to measure:

1) The *T-minute average rainfall* intensity at a single site so that the coefficient of variation, C_V, of the ratio of the measured average to the true average is less than x.
2) The *instantaneous rainfall intensity at a single site* so that the coefficient of variation, C_V, of the ratio of the measured intensity to the true intensity is less than x.
3) The *T-minute average rainfall intensity over an area* so that the coefficient of variation of the ratio of the measured average to the true average is less than x.

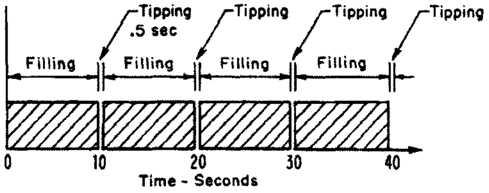
The decision variables in the design of the instrumentation system are: 1) the size of the tipping-bucket, and 2) the recording scheme used to record the signals from the tipping-bucket.

Two basic recording schemes have been considered here. The first is to record the time at which the bucket tips each time it tips. This information is then processed (offline) to estimate the average rainfall intensity. The second is to record the number of tips in a counter and to sample the counter at equally spaced intervals of time. Tipping-bucket sizes considered are .001, .005, .01, .05, and .10 inches.

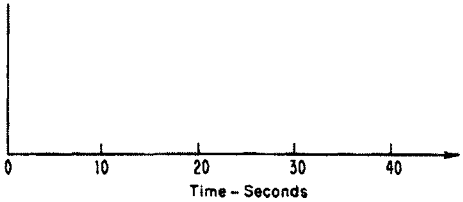
2.3 Response Characteristics of a Tipping-Bucket Rain Gauge

The instrumentation requirements to meet objectives 1 and 2, above, can be determined from a study of the response of a tipping-bucket rain gauge to a sinusoidally varying instantaneous rainfall input.

It is not valid to assume that a tipping-bucket exactly measures the rainfall it receives, and this assumption becomes less valid with increasing rainfall intensity. A cycle takes place that is a sequence of two events: filling and tipping. The time to fill is a function of rainfall intensity. It also takes time for the bucket to tip after it is full. The cycle of events is shown in Figure 8-1, where the time to tip is assumed to be 0.5 seconds. In that figure, the actual rainfall intensity at the gauge is 3.79 inches per hour, and the bucket size is .01 inch. The sequence of signals that would be sent by the rain gauge is also shown. Because of the delay introduced by the tipping of the bucket, a correction factor must be applied to the apparent rainfall intensities recorded from the gauge.



Timing of Filling - Tipping Sequence



Pulse Signals from Rain Gauge

FIGURE 8-1. Tipping-Bucket Rain Gauge Timing

Assume that it takes 0.5 seconds for the bucket to tip so that any rain falling when the bucket is tipping is not measured. The expected volume of rain, in inches, occurring in b seconds, as a function of rainfall intensity is

$$\Delta = \frac{I * b}{3600} = \frac{I}{7200}$$

7

The time, in seconds, required for S inches of rain to fall, as a function of rainfall intensity is:

$$\tau = \frac{3600S}{I}$$

8

The time required for a bucket to tip when the rainfall intensity is I is

$$\mathrm{T} = \tau + b \approx \tau + 0.5$$

9

During which time the amount of rain

$$\mathrm{V} = \mathrm{S} + \Delta$$

10

has fallen.

The uncorrected, measured rainfall intensity is

$$i = \frac{3600S}{T}$$

11

The actual intensity is

$$I = \frac{3600S + B * I}{T}$$

12

so that

$$I = \frac{3600S}{T-b} \approx \frac{3600S}{T-.5}$$

13

in the time interval, T, between tips.

If the record is read so that there are n tips during a sampling interval of duration Δt, a different scheme is needed. Assume the actual intensity is constant during a sampling interval, Δt, and the number of tips of an S-inch bucket is n. The average interval between tips will be

$$T = \frac{\Delta t}{n}$$

14

so that

$$I = \frac{3600S}{\frac{\Delta t}{n} - b} \approx \frac{3600S}{\frac{\Delta t}{n} - .5}$$

15

and

$$I = \frac{3600S * n}{\Delta t - b * n} \approx \frac{3600S * n}{\Delta t - .5n}$$

16

Note, Δt is measured in seconds.

The ratio of the actual rainfall intensity, I, to the uncorrected intensity, i, is

$$\frac{I}{i} = 1 + \frac{I * b}{36} \approx 1 + \frac{I}{72}$$

17

so, the theoretical percent error at 7.2 in/hr is 10%. The actual volume of rain falling during time T with a bucket volume, S, is

$$V = \frac{S*T}{T-b} \approx \frac{S*T}{T-.5}$$

18

It is of some interest to study the response of a tipping-bucket gauge to the instantaneous intensities applied at that gauge. The test pattern imposed on the gauge is a sinusoidal variation shown in Figure 8-2. It is completely determined by three parameters: the average intensity, P; the relative amplitude of oscillation, a; and the wavelength, W. The maximum rainfall intensity is

$$P_{\max} = (1+a)P$$

19

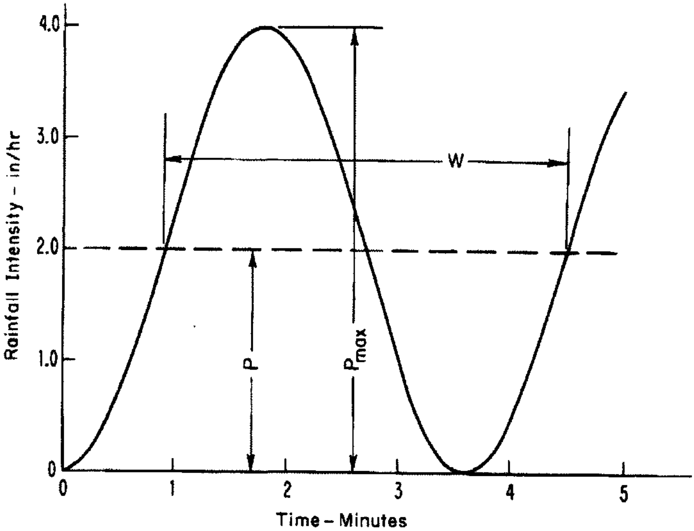


FIGURE 8-2. Sinusoidal Rainfall Pattern

Any measure for comparing the apparent intensity pattern recorded by the rain gauge with the actual test pattern must be factitious. The one chosen here is the coefficients of variation of the ratio of the recorded intensities to the actual intensity. Typical input/response pairs are illustrated in Figures 8-3 and 8-4 for sampling schemes 1 and 2, respectively.

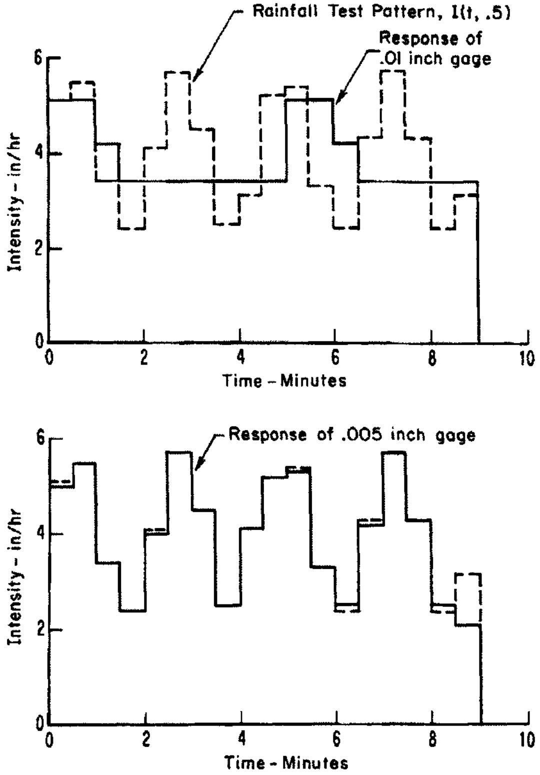


FIGURE 8-3. Response of a Tipping-Bucket Rain Gauge to Sinusoidal Rainfall Input, Sampling Scheme 1

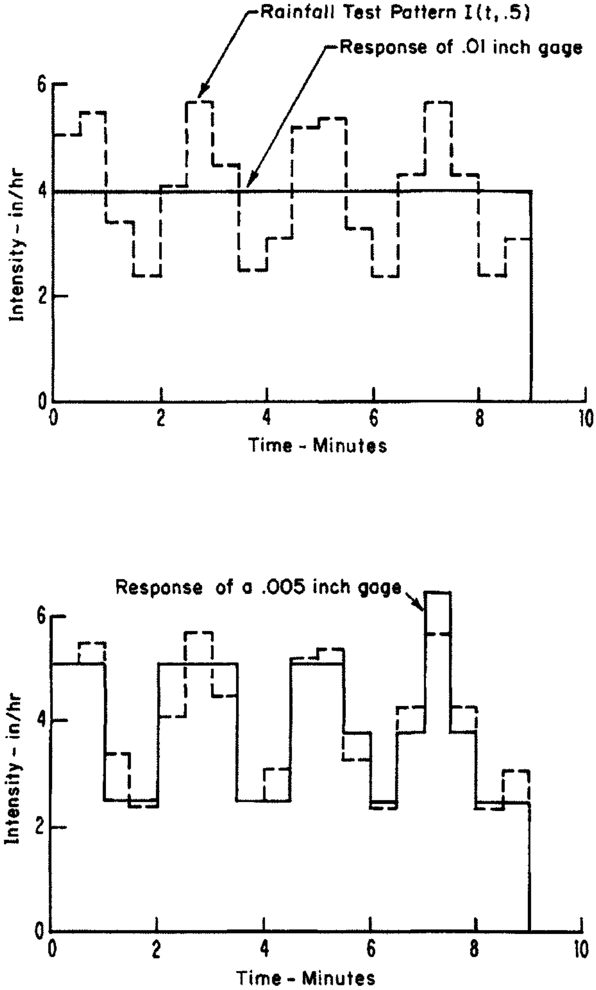


FIGURE 8-4. Response of a Tipping-Bucket Rain Gauge to Sinusoidal Rainfall Input, Sampling Scheme 2

A regression equation to estimate the coefficient of variation as a function of the rainfall variables and decision variables was obtained. The relations are

$$CV1 = 5.0 \frac{S^{0.686} A^{0.632}}{T^{0.607} P^{0.635} W^{0.024}} \qquad 20$$

for sampling scheme 1, and

$$CV2 = 3.1 \frac{S^{.529} A^{.298}}{T^{.610} P^{.505} W^{.066}} \qquad 21$$

for sampling scheme 2. The multiple correlation coefficients were .941 and .948, respectively.

An important result of this analysis is that the measurement error for a tipping-bucket rain gauge is essentially independent of the frequency of oscillation of the rainfall intensity. On the other hand, it is most sensitive to the amplitude of that oscillation.

Using these relations, a measurement system can be designed to meet the first two objectives. To meet the third objective, the following additional study of random rainfall processes was made.

2.4 A Stochastic Rainfall Process

Rainfall intensities during intense convective storms are widely variable for short durations from one location to another and from one instant to the next. These storms are unstable and the instantaneous rainfall intensities are not constant from place to place.

At a single location, the rate of rainfall at any instant is not likely to be the same as the average for the vicinity around that location. However, over a long period of time it seems reasonable to expect that total volumes of rain at different neighboring locations during uniform storms should be nearly equal.

TABLE 8-1
Coefficient of Variation of the Ratio of the Measured Rainfall Intensity to the Actual Intensity

P	A	W	T	S	CV1	CV2
1.93	0.80	3.11	0.25	0.001	0.0273	0.1361
1.93	0.80	3.11	0.25	0.005	0.1148	0.2963
1.93	0.80	3.11	0.25	0.010	0.2712	0.3935
1.93	0.80	3.11	0.25	0.050	0.6186	0.6314
1.93	0.80	3.11	0.25	0.100	0.8015	0.8003
1.63	0.12	1.83	1.00	0.001	0.0052	0.0159
1.63	0.12	1.83	1.00	0.005	0.0075	0.0780
1.63	0.12	1.83	1.00	0.010	0.0480	0.1777
1.63	0.12	1.83	1.00	0.050	0.2665	0.3788
1.63	0.12	1.83	1.00	0.100	0.2681	0.2975
1.18	0.81	3.97	1.00	0.001	0.0282	0.0435
1.18	0.81	3.97	1.00	0.005	0.0748	0.1252
1.18	0.81	3.97	1.00	0.010	0.1078	0.1540
1.18	0.81	3.97	1.00	0.050	0.5155	0.6206
1.18	0.81	3.97	1.00	0.100	0.6228	0.6307
3.92	0.10	2.79	1.00	0.001	0.0068	0.0136
3.92	0.10	2.79	1.00	0.005	0.0194	0.0320
3.92	0.10	2.79	1.00	0.010	0.0134	0.0581
3.92	0.10	2.79	1.00	0.050	0.1434	0.3357
3.92	0.10	2.79	1.00	0.100	0.1726	0.4062
5.96	0.43	0.65	0.10	0.001	0.1570	0.1652
5.96	0.43	0.65	0.10	0.005	0.0476	0.3433
5.96	0.43	0.65	0.10	0.010	0.0661	0.3211
5.96	0.43	0.65	0.10	0.050	0.4205	0.4425
5.96	0.43	0.65	0.10	0.100	0.5984	0.5883
2.12	0.26	3.75	0.50	0.001	0.0117	0.0533
2.12	0.26	3.75	0.50	0.005	0.0474	0.1169
2.12	0.26	3.75	0.50	0.010	0.0425	0.2829
2.12	0.26	3.75	0.50	0.050	0.2515	0.2669
2.12	0.26	3.75	0.50	0.100	0.3050	0.2657

P = Average rainfall intensity (in/hr)
A = Relative amplitude of oscillation of rainfall intensity
W = Wavelength of oscillation (min)
T = Sampling interval (min)
S = Size of Tipping-Bucket (in)
CV1 = Coefficient of variation, sampling scheme 1
CV2 = Coefficient of variation, sampling scheme 2

TABLE 8-2
Coefficient of Variation of the Ratio of the Measured Rainfall Intensity to the Actual Intensity

P	A	W	T	S	CV1	CV2
2.18	0.95	3.56	1.00	0.001	0.0239	0.0418
2.18	0.95	3.56	1.00	0.005	0.685	0.166
2.18	0.95	3.56	1.00	0.010	0.1759	0.1395
2.18	0.95	3.56	1.00	0.050	0.7543	0.6400
2.18	0.95	3.56	1.00	0.100	0.8818	0.8578
4.24	0.79	2.34	0.25	0.001	0.0379	0.0667
4.24	0.79	2.34	0.25	0.005	0.0613	0.1744
4.24	0.79	2.34	0.25	0.010	0.1237	0.4258
4.24	0.79	2.34	0.25	0.050	0.4860	0.5221
4.24	0.79	2.34	0.25	0.100	0.8294	0.8227
0.22	0.59	1.06	0.25	0.001	0.1812	0.4720
0.22	0.59	1.06	0.25	0.005	0.6814	0.6877
0.22	0.59	1.06	0.25	0.010	1.1809	1.1326
0.22	0.59	1.06	0.25	0.050	9.9999	9.9999
0.22	0.59	1.06	0.25	0.100	9.9999	9.9999
4.02	0.45	2.23	0.50	0.001	0.0152	0.0322
4.02	0.45	2.23	0.50	0.005	0.0313	0.0820
4.02	0.45	2.23	0.50	0.010	0.0841	0.1378
4.02	0.45	2.23	0.50	0.050	0.3632	0.5585
4.02	0.45	2.23	0.50	0.100	0.2628	0.3177
5.31	0.12	0.81	1.00	0.001	0.0052	0.0146
5.31	0.12	0.81	1.00	0.005	0.0101	0.0346
5.31	0.12	0.81	1.00	0.010	0.0244	0.0566
5.31	0.12	0.81	1.00	0.050	0.1339	0.2714
5.31	0.12	0.81	1.00	0.100	0.0225	0.2538
2.82	0.96	2.09	0.10	0.001	0.1895	0.3304
2.82	0.96	2.09	0.10	0.005	0.5897	0.6294
2.82	0.96	2.09	0.10	0.010	0.8916	0.9342
2.82	0.96	2.09	0.10	0.050	1.6332	1.6313
2.82	0.96	2.09	0.10	0.100	1.8690	1.8695

P = Average rainfall intensity (in/hr)
A = Relative amplitude of oscillation of rainfall intensity
W = Wavelength of oscillation (min)
T = Sampling interval (min)
S = Size of Tipping-Bucket (in)
CV1 = Coefficient of variation, sampling scheme 1
CV2 = Coefficient of variation, sampling scheme 2

If the intensity at a specific site is not always exactly equal to the average for the neighborhood, but varies randomly about that average, the time required to accumulate a given volume of rain is also a random variable. The expected time is a function of the average intensity, but the actual time for a specific volume at a specific location is a random variable.

There are numerous ways to represent rainfall as a random process, but not very much effort has been directed to studying rainfall with that approach. A random process may be defined as an ensemble of sample functions. A sample function is a realization of the random process.

Here a sample function is the sequence of rainfall intensities that occur at a single location during a storm. The collection of all storm events at one site, for all time, is a possible definition of a random process.

Another random process is defined by the ensemble of all rainfall records for the same storm over a small watershed. All locations would never be gauged, even in the smallest catchment, so this random process can only be assumed to exist. Also, it is assumed here that any sample function of this random process is equally likely to occur at any location. This is not precisely valid because the expected rainfall intensity may actually vary from place to place. For very small areas, the assumption should be nearly true. The average overall sample function is the average rainfall intensity over the area.

A specific random process is used in this study to test the response characteristics of a tipping-bucket rain gauge and the response of a rain gauge network. The particular random process used here may not actually exist. Moreover, rainfall intensities may not be as variable as is assumed by the particular process used here. However, it seems likely that any system designed to respond favorably to this random process should also work at least as well in practice.

Assume that the time required to receive a given volume of rain during a storm is a random variable. The probability of receiving that volume increases as the time over which it is collected increases. Consider a storm with steady, uniform expected rainfall intensity. Assume that the probability of having received the given rainfall volume increases with time and approaches 1. 0 exponentially. That is

$$F(T) = 1 - e^{-\lambda T} \qquad 22$$

in which T is the time required to fill the bucket of a tipping-bucket rain gauge, and λ is a function of the rainfall intensity, P, and the volume of the bucket, S. Further, λ is the average rate at which the bucket is expected to tip, so that

$$\lambda = \frac{P}{60S}$$

23

where the units are P (in/hr), S (in), and λ (tips/min). The mean time between tips is

$$\tau = \frac{1}{\lambda} = \frac{60S}{P}$$

24

The probability of receiving n tips in a period of T minutes is

$$P_n(T) = \frac{(\lambda T)^n e^{-\lambda T}}{n!}$$

25

which is the Poisson distribution. This is widely used in Queuing Theory to represent the distribution of arrivals at a service facility. In this case, the arrival times of bucket tips are assumed to be exponentially distributed, which leads to a Poisson distribution of numbers of tips for a given waiting time, T.

The probability of receiving n tips in T minutes when the rainfall intensity is P in/hr is tabulated in Table 8-3 for several combinations of parameter values. The table shows that the probability of receiving less than .01 of an inch of rain in any 0.1-minute period is 0.959 when the average rainfall intensity over the area is 0.25 in/hr. Values for a .01-inch bucket, as well as for a .10-inch bucket are tabulated for the same combinations of average rainfall intensity and sampling interval.

TABLE 8-3
Probability of N Tips of an S-Inch Tipping-Bucket During Any T-Minute Period When the Mean Rainfall Intensity is P-Inches Per Hour Intensity

S	P	T	0	1	2	OVER 2
0.01	0.25	0.1	0.959	0.040	0.000	0.000
0.01	0.25	0.5	0.812	0.169	0.018	0.001
0.01	0.25	1.0	0.659	0.275	0.057	0.009
0.01	0.25	5.0	0.125	0.259	0.270	0.346
0.01	1.00	0.1	0.846	0.141	0.012	0.000
0.01	1.00	0.5	0.435	0.362	0.151	0.052
0.01	1.00	1.0	0.189	0.315	0.262	0.234
0.01	1.00	5.0	0.000	0.002	0.008	0.989
0.01	4.00	0.1	0.513	0.342	0.114	0.030
0.01	4.00	0.5	0.036	0.119	0.198	0.647
0.01	4.00	1.0	0.001	0.008	0.028	0.962
0.01	4.00	5.0	0.000	0.000	0.000	1.000
0.01	16.00	0.1	0.069	0.185	0.247	0.498
0.01	16.00	0.5	0.000	0.000	0.000	1.000
0.01	16.00	1.0	0.000	0.000	0.000	1.000
0.01	16.00	5.0	0.000	0.000	0.000	1.000
0.10	0.25	0.1	0.996	0.004	0.000	0.000
0.10	0.25	0.5	0.979	0.020	0.000	0.000
0.10	0.25	1.0	0.959	0.040	0.000	0.000
0.10	0.25	5.0	0.812	0.169	0.018	0.001
0.10	1.00	0.1	0.983	0.016	0.000	0.000
0.10	1.00	0.5	0.920	0.077	0.003	0.000
0.10	1.00	1.0	0.846	0.141	0.012	0.000
0.10	1.00	5.0	0.435	0.362	0.151	0.052
0.10	4.00	0.1	0.936	0.062	0.002	0.000
0.10	4.00	0.5	0.717	0.239	0.040	0.005
0.10	4.00	1.0	0.513	0.342	0.114	0.030
0.10	4.00	5.0	0.036	0.119	0.198	0.647
0.10	16.00	0.1	0.766	0.204	0.027	0.003
0.10	16.00	0.5	0.264	0.351	0.234	0.151
0.10	16.00	1.0	0.069	0.185	0.247	0.498
0.10	16.00	5.0	0.000	0.000	0.000	1.000

Table 8-3 is useful to show that different numbers of tips can be received during a sampling interval. This is true even if the average rainfall intensity is constant over the area. Because the instantaneous intensities vary randomly, the rain gauge must respond accordingly.

The expected number of tips, the standard deviation, and the coefficient of variation are tabulated in Table 8-4 for the same combination of average rainfall intensities, etc., as in Table 8-3. The expected number of tips is easily verified by multiplying P by T, then dividing by S and adjusting the units. However, the values in Table 8-4 were computed from equation 22. The expected number is

$$\overline{N} = \sum_{n=0}^{\infty} n P_n(T)$$

26

The standard deviation of n is computed from

$$\sigma_n = \sqrt{\sum_{n=0}^{\infty} n^2 P_n(T) - \overline{N}^2}$$

27

Because the expected number of tips is easily computed by less sophisticated methods, the most valuable information in Table 8-2 is the assortment of values of the coefficient of variation. These are computed by

$$C_V = \frac{\sigma_n}{\overline{N}}$$

28

and they measure the variability of rainfall intensities in terms of the average intensity over the catchment.

If the goal is to measure the average rainfall intensity, over the catchment, the results in Table 8-4 are sobering. For instance, it would seem unwise to attempt to measure the average rainfall intensity over an area, for a time interval of 0.1 minute with a .01-inch tipping-bucket if the average rainfall intensity were only 0.25 in/hr. (The expected volume of rain would be only .00042 inch.) This is confirmed by the large coefficient of variation, 4,899, given in the table.

TABLE 8-4

Number of Tips of an S-Inch Tipping-Bucket During Any T-Minute Period When the Mean Rainfall Intensity is P-Inches Per Hour

S	P	T	Expected Number	Standard Deviation	Coefficient Variation
0.01	0.25	0.1	0.042	0.204	4.8990
0.01	0.25	0.5	0.208	0.456	2.1909
0.01	0.25	1.0	0.417	0.645	1.5492
0.01	0.25	5.0	2.083	1.443	0.6928
0.01	1.00	0.1	0.167	0.408	2.4495
0.01	1.00	0.5	0.883	0.913	1.0954
0.01	1.00	1.0	1.667	1.291	0.7746
0.01	1.00	5.0	8.333	2.887	0.3464
0.01	4.00	0.1	0.667	0.816	1.2247
0.01	4.00	0.5	3.333	1.826	0.5477
0.01	4.00	1.0	6.667	2.582	0.3873
0.01	4.00	5.0	33.333	5.773	0.1732
0.01	16.00	0.1	2.667	1.633	0.6124
0.01	16.00	0.5	13.333	3.651	0.2738
0.01	16.00	1.0	26.666	5.163	0.1936
0.01	16.00	5.0	133.332	11.545	0.0866
0.10	0.25	0.1	0.004	0.065	15.4919
0.10	0.25	0.5	0.021	0.144	6.9282
0.10	0.25	1.0	0.042	0.204	4.8990
0.10	0.25	5.0	0.208	0.456	2.1909
0.10	1.00	0.1	0.017	0.129	7.7460
0.10	1.00	0.5	0.083	0.289	3.4641
0.10	1.00	1.0	0.167	0.408	2.4495
0.10	1.00	5.0	0.883	0.913	1.0054
0.10	4.00	0.1	0.067	0.258	3.8730
0.10	4.00	0.5	0.333	0.577	1.7320
0.10	4.00	1.0	0.667	0.816	1.2247
0.10	4.00	5.0	3.333	1.826	0.5477
0.10	16.00	0.1	0.267	0.516	1.9365
0.10	16.00	0.5	1.333	1.155	0.8660
0.10	16.00	1.0	2.667	1.633	0.6124
0.10	16.00	5.0	13.333	3.651	0.2738

However, it does seem wise to use a .01-inch tipping-bucket if the average rainfall intensity is 4 inches per hour and the sampling interval is one minute. The accuracy of doing this is again indicated by the coefficient of variation, 0.3873. This means that 1/3 of all one-minute time intervals would indicate 38.7% more, or less, rain than the true average rate. If the averaging time is extended to 5 minutes, the coefficient of variation is reduced to 0.1732. Similarly, an averaging time of one minute, for 5 gauges, also gives a coefficient of variation of 0.1732.

At this point a note of caution is needed because the random rainfall process, from which these results were deduced, was assumed. The assumed process may not accurately represent actual rainfall events. However, it seems unlikely that the real process is any more variable than the assumed process, so that all of the results indicated here describe the responses of specific measuring systems to inputs that are not necessarily equivalent to those actually encountered.

Data from tipping-bucket rain gauges for two storms recorded in Baltimore by the Johns Hopkins Storm Drainage Research Project were analyzed to study the statistical distribution of times between tips of a .01 inch bucket. These data were recorded at a chart speed of 1.5 inches per minute, so the time at which a tip occurred was easily read to the nearest second. The data for these storms are presented in Tables 10.5 and 10.6.

According to the postulated random process, the arrival times of tips are exponentially distributed according to equation 22 so the density function is:

$$f(\tau) = \lambda e^{-\lambda\tau} \tag{29}$$

The parameter λ is a function of the rainfall intensity that varies during a storm. Because λ varies, a scheme is needed that is independent of λ to test if the random process postulate is valid.

TABLE 8-5
Rainfall at Hamilton Hill During a Storm Between 6/25/65 and 7/2/65

i	τ_i	τ_{i+1}	τ_{i+2}	U_i
1	37	22	32	.685
4	32	39	39	.416
7	29	11	29	.725
10	19	17	16	.578
13	22	17	15	.688
16	14	17	16	.424
19	15	16	17	.455
22	15	16	15	.484
25	13	14	13	.481
28	11	12	10	.500
31	9	7	10	.529
34	11	11	13	.458
37	9	8	9	.529
40	9	9	9	.500
43	13	11	9	.650
46	19	16	10	.731
49	13	11	9	.650
52	9	10	9	.474
55	9	8	9	.529
58	13	22	32	.241
61	53	28	27	.964
64	26	38	56	.277
67	55	48	50	.561
70	19	18	15	.578
73	16	16	19	.457
76	13	7	6	1.000
79	8	8	9	.471
82	8	7	7	.571
85	5	7	8	.333
88	5	5	6	.455
91	7	7	7	.500
94	7	10	10	.350
97	16	14	21	.429

TOTAL 17.664/33 = .535

TABLE 8-6
Rainfall at Montebello During a Storm Between 8/2/65 and 8/5/65

i	τ_i	τ_{i+1}	τ_{i+2}	U_i
1	118	111	108	.539
4	84	59	98	.535
7	158	175	89	.598
10	67	127	138	.253
13	56	28	29	.982
16	21	28	15	.488
19	21	21	52	.288
22	10	14	9	.435
25	13	9	12	.619
28	14	14	8	.636
31	10	9	11	.500
34	11	14	19	.333
37	29	24	28	.558
40	23	30	28	.397
43	35	33	43	.461
46	42	45	46	.462
49	61	54	70	.492
52	64	86	82	.381
55	101	101	124	.449

TOTAL 9.404/19 = .495

Several statistics were devised, and the following was selected. In a given rainfall record, let the times between tips be the sequence (τ_1, τ_2, τ_3, τ_n) where there are a total of n + 1 tips. Let the statistic, U, be defined by

$$U_i = \frac{\tau_i}{\tau_{i+1} + \tau_{i+2}} \tag{30}$$

and let U_i be computed for every third value of i beginning with unity.

Assuming that λ is constant for the time $\tau_i + \tau_{i+1} + \tau_{i+2}$, it can be shown for

$$y_i = \tau_{i+1} + \tau_{i+2} \tag{31}$$

that the density function of y is

$$f(y) = \lambda^2 y e^{-\lambda y} \tag{32}$$

and for

$$U_i = \frac{\lambda_i}{y_i} \tag{33}$$

that the density function of U is

$$f(U) = \frac{2}{(U+1)^3} \tag{34}$$

which is independent of λ ! The expected value of U is

$$\varepsilon\{U\} = \int_0^\infty \frac{2U dU}{(U+1)^3} = 1 \tag{35}$$

which is surprising because the ratio of the expected values of τ_i and y is ½. The variance of U is infinite.

If the actual distribution of times between tips is less variable than this exponential process, the average observed value of U would be less than 1 and could approach ½. The average values of U for the data in Tables 10.5 and 10.6 are 0.535 and 0.495. This supports the conclusion that the actual rainfall process at a single site is less variable than the random process used here to test the response of rainfall measuring systems.

2.5 Rain Gauge Network Response

Objective 3 is to measure the T-minute average rainfall intensity over an area so that the coefficient of variation of the ratio of the measured average to the true average is less than x. The decision variables are the size of the tipping-bucket, the recording scheme, and the number of rain gauges.

To design the network, the impact of the decision variables on the coefficient of variation must be investigated. An initial attempt at this was made as part of this study. The results, which are presented here, are tentative and should be restudied in much more detail than could be done in the limited time available. The random process assumed here is probably much more variable than the real process, so that considerable savings of instrumentation costs could be achieved by more detailed study of available rainfall data.

A sinusoidally varying average areal rainfall intensity was assumed to occur uniformly throughout an area. The expected instantaneous rainfall intensity at each gauge was assumed to be equal to the instantaneous average value for the area. This expected instantaneous intensity was used to compute the parameter λ for the assumed random process at each gauge. These random processes were assumed to act independently of each other. The actual rainfall intensity at each gauge was a random function of λ, and the random variations from gauge to gauge were uncorrelated.

The responses of the rain gauges were recorded by each of the two sampling schemes described above. According to each of these schemes, the average rainfall intensity over the area was estimated. The ratio of the estimated average to the true average was computed for each sampling period. These ratios were used to compute an average ratio, the standard deviation, and a coefficient of variation. In Tables 10.7 and 10.8, the coefficients of variation are presented for various combinations of storm and rain gauge parameter values.

The average values of the coefficient of variation for all tests were .289 and .298 for sampling schemes 1 and 2, respectively. These values are much larger than the values that were obtained above for the rain gauge response to a deterministic rainfall input. Those corresponding values were .118 and .218, which clearly indicate the first sampling scheme is more accurate than the second. It seems that the random variations, which have been added to study the network response, mask out the individual gauge characteristics. Presumably, this effect could be explained, but that will require more careful study than could be given here.

TABLE 8-7
Coefficient of Variation of the Ratio of Measured Rainfall Intensity to the Actual Intensity at Several Gauges

P	A	W	T	S	N	CV1	CV2
0.73	0.86	0.63	0.10	0.010	5	0.9946	1.0128
4.01	0.90	1.91	0.10	0.005	1	0.7035	0.8013
2.28	0.93	1.15	0.10	0.050	3	0.3012	1.3004
0.15	0.15	2.86	0.10	0.050	1	0.0000	0.0000
1.17	0.55	2.48	0.50	0.010	3	0.6767	0.6459
2.77	0.15	3.23	0.25	0.005	3	0.3703	0.3965
4.68	0.97	3.25	0.25	0.100	5	1.3112	1.3368
4.56	0.53	1.74	1.00	0.005	5	0.1108	1.1070
5.45	017	3.40	1.00	0.010	3	0.1841	0.1594
1.93	0.34	0.93	0.50	0.050	1	0.6798	0.8537
4.72	0.70	0.97	0.50	0.010	3	0.3100	0.3050
4.95	0.49	2.27	1.00	0.050	1	0.5333	0.5795
5.95	0.47	3.72	0.25	0.005	1	0.4273	0.4808
4.05	0.30	3.03	0.10	0.050	1	1.6522	1.7591
6.69	0.12	2.81	0.25	0.100	3	1.1576	1.1123
3.95	0.95	3.10	0.50	0.100	1	0.9866	1.1325
1.36	0.61	2.61	1.00	0.001	3	0.1257	0.1334
0.73	0.97	2.97	0.25	0.010	5	1.1775	1.1817
4.60	0.33	0.69	1.00	0.005	5	0.1129	0.1687
5.66	0.91	3.80	1.00	0.010	3	0.1872	0.1884
2.13	0.63	2.48	0.25	0.050	3	0.9637	1.0194
0.64	0.67	0.73	1.00	0.005	3	0.3250	0.3331
1.93	0.94	3.14	1.00	0.001	3	0.1050	0.1043
4.74	1.00	3.51	1.00	0.001	3	0.1228	0.1221
5.44	0.07	1.50	1.00	0.001	5	0.0391	0.0401
4.25	0.63	1.98	1.00	0.100	1	0.8758	0.7859
4.90	0.54	3.57	1.00	0.010	3	0.2218	0.2245
5.47	0.73	1.14	0.10	0.010	3	0.3856	0.4460
1.97	0.27	2.77	1.00	0.010	1	0.3601	0.3610
2.47	0.88	2.42	0.25	0.010	3	0.4804	0.5174

P = Average rainfall intensity (in/hr)
A = Relative amplitude of oscillation of rainfall intensity
W = Wavelength of oscillation (min)
T = Sampling interval (min)
S = Size of Tipping-Bucket (in)
CV1 = Coefficient of variation, sampling scheme 1
CV2 = Coefficient of variation, sampling scheme 2

TABLE 8-8
Coefficient of Variation of the Ratio of Measured Rainfall Intensity to Average Intensity at Several Gauges

P	A	W	T	S	N	CV1	CV2
0.46	0.09	3.37	0.50	0.001	3	0.2485	0.2715
0.51	0.98	0.99	0.50	0.100	3	0.0000	0.0000
5.71	0.02	2.32	0.25	0.100	1	1.3016	1.1892
5.51	0.36	3.66	1.00	0.001	5	0.0480	0.0488
4.87	0.47	2.34	0.25	0.100	5	0.5076	0.6934
0.90	0.73	0.69	0.50	0.050	5	0.8300	0.7666
5.27	0.03	1.47	1.00	0.010	5	0.1202	0.1052
0.57	0.53	1.58	0.25	0.001	5	0.2941	0.2004
1.78	0.58	3.25	1.00	0.010	1	0.4926	0.5781
1.71	0.67	2.13	0.10	0.050	1	2.2355	2.4835
1.52	0.38	3.89	0.25	0.010	5	0.4413	0.5117
4.02	0.57	1.91	0.25	0.005	5	0.2438	0.2684
5.86	0.92	3.10	0.25	0.001	1	0.3975	0.4180
5.19	0.00	1.26	0.10	0.005	5	0.2262	0.2961
5.82	0.71	2.33	0.25	0.050	5	0.3904	0.4662
4.09	0.92	1.77	0.10	0.100	1	1.4629	1.4608
5.37	0.53	1.93	0.50	0.001	1	0.1206	0.1236
2.00	0.13	3.23	1.00	0.010	1	0.3053	0.2854
0.73	0.47	2.97	0.50	0.001	5	0.1432	0.1663
2.17	0.20	3.75	0.25	0.050	3	0.7615	0.7530
5.91	0.95	3.37	0.10	0.005	5	0.3210	0.3241
5.25	0.40	2.41	0.10	0.050	5	0.6216	0.6717
1.01	0.68	2.37	0.50	0.001	5	0.1268	0.1153
0.64	0.42	2.51	0.25	0.050	5	1.5455	1.5382
3.67	0.75	0.51	0.50	0.005	5	0.2902	0.2651

P = Average rainfall intensity (in/hr)
A = Relative amplitude of oscillation of rainfall intensity
W = Wavelength of oscillation (min)
T = Sampling interval (min)
S = Size of Tipping-Bucket (in)
CV1 = Coefficient of variation, sampling scheme 1
CV2 = Coefficient of variation, sampling scheme 2

The relationship between the coefficient of variation and the rainfall and decision variables is important because this can be used in design to control the accuracy of the network. Therefore, a regression equation was obtained from the data in Tables 8-7 and 8-8. These equations did not fit the data very well, although relationships similar to equations 20 and 21 were anticipated. The regression equations that were found were:

$$CV1 = .219\frac{P^{.676}S^{.162}A^{.197}W^{.229}N^{.026}}{T^{.393}}$$

36

for sampling scheme 1, and

$$CV2 = .207\frac{P^{.685}S^{.154}A^{.200}W^{.247}N^{.014}}{T^{.437}}$$

37

for sampling scheme 2. The multiple correlation coefficients were .430 and .439, which means that the equations do not account for very much of the variation of C_V.

It was particularly surprising to find that the number of rain gauges did not account for more of the variability than is apparent from these tests. In view of this, and of the surprising average values of CV1 and CV2, these results should not be taken too seriously until an explanation for the observed behavior can be found. It may partly lie in the nature of the assumed random rainfall process that leaves much to be desired anyway,

3.0 RUNOFF

The most successfully used runoff-measuring system for urban areas is a Parshall flume with a stilling well, float and stage-sensing device to measure water levels in the flume. There are two general categories of error in runoff measurements from the system. The first source is amenable to control at the time the system is designed. It occurs because the stage reported by the head-sensing device is not the same as the water level in the flume. The second source is due to random variations of the actual flow rate from the measured flow rate according to the rating curve for the flume.

Each of these categories of error has been investigated separately, and mathematical models for each have been devised. With these models, the extent of error in any proposed system can be anticipated and perhaps controlled.

The final part of this section is concerned with the response of the 47.1-acre Northwood area gauged by the Johns Hopkins Storm Drainage Project. This is perhaps the best-instrumented area of that size that ever has been gauged.

3.1 Water Level Measurement

First, consider the response of the water level changes in the water level in the flume. Let the water level in the flume vary sinusoidally according to the relation

$$H_f(t) = \overline{H}_f(1 + a * \sin(f * t))$$

38

in which

$$f = 2\pi / W$$

39

and W is the wavelength of the oscillation. The relative amplitude of the oscillation is a. Let the diameter of the stilling well be D, and let the diameter of the connection between the stilling well and the flume be d. Assume that friction loss in the connection is negligible. Assume also that the connection behaves as an orifice with an orifice coefficient of 0.5, so the velocity in the connection is:

$$V = 0.5\sqrt{2gh}$$

40

The variable h is defined as

$$h = H_f - H_s$$

41

where H is the stage in the flume, and H is the stage in the stilling well. The velocity is positive when the flow is from the flume into the stilling well. The sign of the square root term is taken to be the same as the sign of h. Therefore,

$$\frac{dH_s}{dt} = \frac{d^2}{D^2}V$$

42

so that

$$\frac{2D^4}{gd^4}\left(\frac{dH_s}{dt}\right) + H_s = H_f$$

43

is a nonlinear differential equation that describes the fluctuation of the water level in the stilling well.

To study the relation between flume and stilling well hydrographs for various combinations of the stilling well ratio d/D and values of $\overline{H}$, a, and W, a computer program was prepared. The results are tabulated in Table 8-9, and typical input/output hydrographs are shown in Figure 8-5. Regression equations for the time lag and for the coefficient of variation of ratios of stilling well/flume hydrograph ordinates, were obtained as a function of the other parameters. The relation for the coefficient of variation is

$$C_v = 0.0152\frac{D_s^{1.33} H^{.41} A^{.70}}{D_C^{1.09} W^{.62}} \qquad 44$$

TABLE 8-9
Lag, Attenuation and Coefficient Of Variation of Stilling Well Hydrograph

H	A	W	DC	DS	TOR	LAG	ATTN	CV
3.33	0.43	17.4	1.70	16.5	8.5	0.0082	1.000	0.0042
4.74	0.02	17.7	2.44	11.9	3.7	0.0000	1.000	0.0024
3.74	0.14	4.2	2.46	14.6	0.8	0.0005	1.000	0.0020
4.41	0.61	21.5	1.71	20.3	0.9	0.0435	0.998	0.0147
3.76	0.60	25.3	1.50	17.6	0.5	0.0249	0.999	0.0073
0.40	0.90	21.2	1.10	7.2	9.7	0.0008	1.000	0.0017
4.82	0.04	16.4	2.37	22.6	8.0	0.0000	1.000	0.0015
3.10	0.52	15.4	2.53	12.9	1.5	0.001	1.000	0.0019
0.07	0.71	4.5	1.70	13.9	4.5	0.0099	1.000	0.0020
0.54	0.63	24.5	2.75	9.6	4.3	0.0000	1.000	0.0022
3.59	0.45	7.7	1.41	13.2	6.1	0.0164	0.995	0.0172
4.18	0.51	16.1	2.40	16.3	0.4	0.0042	1.000	0.0028
4.54	0.13	17.5	2.51	11.8	6.1	0.0000	1.000	0.0620
4.32	0.70	12.7	1.90	9.5	9.4	0.0030	1.000	0.0026
2.90	0.15	15.2	2.10	18.2	4.6	0.0004	1.000	0.0020
1.88	0.13	12.5	2.52	10.2	4.6	0.0000	1.000	0.0023
4.40	0.18	4.4	1.15	9.8	5.7	0.0026	0.995	0.0069
3.60	0.22	25.8	1.16	8.4	2.8	0.0005	1.000	0.0021
4.70	0.14	11.3	1.89	23.2	0.8	0.0031	0.998	0.0037
2.33	0.06	5.1	1.25	13.6	0.4	0.0004	1.000	0.0023
2.61	0.64	5.2	2.21	9.1	0.6	0.0020	1.000	0.0032
4.45	0.84	2.2	2.88	10.4	9.6	0.0298	0.972	0.0826
1.33	0.98	15.7	2.54	10.2	5.3	0.0009	1.000	0.0018
1.26	0.73	4.1	2.68	8.8	6.4	0.0007	1.000	0.0024
1.28	0.80	14.2	2.00	17.2	6.9	0.0094	1.000	0.0049
1.10	0.10	19.3	2.31	22.2	8.5	0.0000	1.000	0.0019
0.96	0.54	15.6	2.38	9.9	1.8	0.0002	1.000	0.0018
4.27	0.51	11.7	2.24	18.1	6.8	0.0113	0.999	0.0077
2.53	0.90	25.0	2.98	22.4	7.3	0.0102	1.000	0.0033
2.45	0.34	18.5	1.69	17.5	3.3	0.0042	1.000	0.0028

H = Average stage (ft)
A = Relative amplitude of oscillation of stage
W = Wavelength of oscillation (min)
DC = Diameter of connection to stilling well (in)
DS = Diameter of stilling well (in)
LAG = Time lag between stage in flume and stage in stilling well
ATTN = (range of stilling well hydrograph)/(range of flume hydrograph)
CV = Coefficient of variation of ratio of hydrograph ordinates

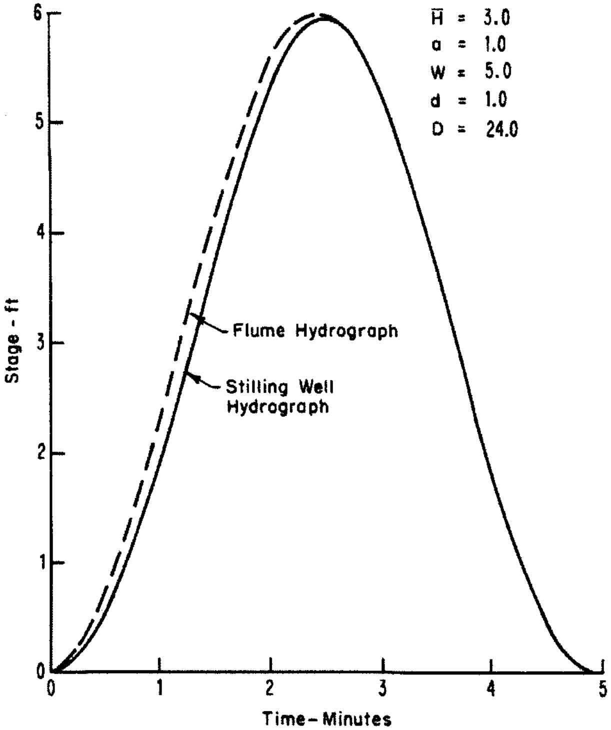


FIGURE 8-5. Water Level Hydrographs Illustrating Effect of Stilling Water

The relation for the time delay is

$$t_\ell = .00041\left(\frac{D_s}{D_c}\right)^{3.44} \frac{A^{2.06} H^{.90}}{W^{.664}} \qquad 45$$

The multiple correlation coefficients are, respectively, 0.84 and 0.995.

The water level reported by the stage-sensing device will lag the water level in the stilling well because of the frictional resistance in the stage-sensing device. Let the torque required to turn the shaft of the stage-sensing device be τ (in.-oz.). Let the radius of the pulley on the shaft of the stage sensing device be R (inches). The force, in ounces, required to move the shaft is

$$F = \frac{\tau}{R}$$

46

To exert the required force, the float must rise out of the water when the stage is falling or sink slightly when rising. The total distance the stage must move to move the float during a change from rising to falling stage is the so-called float lag. This distance, in feet, is

$$L = 0.185F / D_f^{\ 2}$$

47

where D_f is the diameter of the float in inches. The error introduced in the reported stage measurement is shown in Figure 8-6. Small oscillations in the stilling well of amplitude L/2 or smaller are not even detectable.

The stilling well lag and the float lag combine to introduce an error in water stage measurements. This error can be described in several ways, three of which have been considered here. They are: 1) attenuation of flood peaks, 2) lag time between recorded and actual hydrographs, and 3) coefficient of variation of the ratio of recorded/actual hydrograph ordinates for a random selection of times during storms.

A regression relation for the coefficient of variation and time lag has been obtained. These are functions of the amplitude and wavelength of the hydrograph. Included also is the stilling well ratio d/D and the friction torque of the stage sensing device. Other variables have been eliminated by assuming a 6-inch diameter head-sensing pulley and by assuming the float diameter to be 3 inches smaller than the diameter of the stilling well.

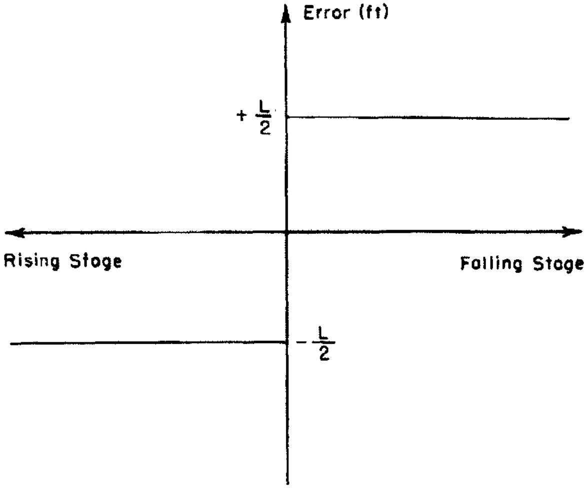


FIGURE 8-6. Float Lag Error in Reported Water Levels.

The regression relation for the time lag is

$$t_\ell = .00036\left(\frac{D_s}{D_c}\right)^{3.40}\frac{A^{2.00}H^{.94}TOR^{.05}}{W^{.64}} \qquad 48$$

and the regression relation for the coefficient of variation is

$$C_v = 0.021\frac{A^{.81}TOR^{.29}D_s^{.46}}{W^{.54}D_c^{1.07}H^{.084}} \qquad 49$$

The multiple correlation coefficients are, respectively, .984 and .79.

The basic data used to develop these relations are tabulated in Table 8-10. They were compiled by a program that generated sinusoidally varying runoff hydrographs and simulated the stilling well and float lag functions.

3.2 The Parshall Flume

The Parshall Flume is an accurate measuring device that is feasible to use for flow gauging in urban areas. A review of the original work by Parshall[6] was made to establish a sound statistical measurement error model.

The flow formula originally proposed by Parshall is the empirical relation

$$Q = 4WH^{1.522W^{0.026}} \tag{50}$$

An analysis was made, as part of this study, of 48 flow measurements reported by Parshall for flumes ranging in width from 1 ft to 8 ft and for flows ranging from 0.3 cfs to 50 cfs. For each test, the ratio of the estimated flow (by equation 50) to the observed flow was computed. The averages for each of the flumes tested are given in Table 8-11. The average ratio for all flumes was 1.037, which means that Parshall's equation overestimates the flow rate by 3.7% on the average. The standard deviation of the ratios was only .023 so that approximately 2/3 of all estimates, after being corrected for the bias in the formula, would be within ± 2.2% of the observed values.

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[6] Parshall, R.L. 1926. "The Improved Venturi Flume." *Transactions, ASCE.* Volume 89, pp. 841.

TABLE 8-10
Lag, Attenuation and Coefficient Of Variation of Recorded Hydrograph

H	A	W	DC	DS	TOR	LAG	ATTN	CV
3.33	0.43	17.4	1.70	10.5	8.5	0.0082	0.999	0.0047
4.74	0.02	17.7	2.44	11.9	3.7	********	0.992	0.0029
3.74	0.14	4.2	2.46	14.6	0.8	0.0005	0.999	0.0031
4.41	0.61	21.5	1.71	20.3	9.9	0.0435	0.999	0.0149
3.76	0.60	25.3	1.50	17.6	9.5	0.0247	0.999	0.0078
0.40	0.90	21.2	1.10	7.2	9.7	0.0000	0.977	0.0739
4.82	0.04	16.4	2.37	22.6	8.0	********	0.998	0.0020
3.10	0.52	15.4	2.53	12.0	1.5	0.0011	1.000	0.0025
0.07	0.71	4.5	1.70	13.9	4.5	0.0009	0.987	0.0100
0.54	0.63	24.5	2.75	9.6	4.3	0.0000	0.996	0.0048
3.59	0.45	7.7	1.41	13.2	8.1	0.0164	0.995	0.0175
4.18	0.51	16.1	2.40	16.3	0.4	0.0043	1.000	0.0032
4.54	0.13	17.5	2.51	11.8	6.1	0.0002	0.998	0.0024
4.32	0.70	12.7	1.90	9.5	9.4	0.0036	0.999	0.0039
2.90	0.15	15.2	2.10	18.2	4.6	0.0003	0.999	0.0020
1.88	0.13	12.5	2.52	10.2	4.6	********	0.995	0.0030
4.40	0.18	4.4	1.15	9.8	5.7	0.0026	0.993	0.0074
3.60	0.22	25.8	1.16	8.4	2.8	0.0000	0.998	0.0024
4.70	0.14	11.3	1.89	23.2	8.8	0.0031	0.998	0.0034
2.33	0.06	5.1	1.25	13.6	9.4	0.0004	0.990	0.0022
2.61	0.64	5.2	2.21	9.1	0.6	0.0020	1.000	0.0039
4.45	0.84	2.2	2.88	19.4	9.6	0.0298	0.972	0.0828
1.33	0.98	15.7	2.54	10.2	5.3	0.0008	0.999	0.0172
1.26	0.73	4.1	2.68	8.8	6.4	0.0007	0.997	0.0054
1.28	0.80	14.2	2.00	17.2	6.9	0.0095	0.999	0.0057
1.10	0.10	19.3	2.31	22.2	8.5	0.0000	0.997	0.0026
0.96	0.54	15.6	2.38	9.9	1.8	0.0001	0.999	0.0028
4.27	0.51	11.7	2.24	18.1	6.8	0.0113	0.999	0.0079
2.53	0.90	25.0	2.98	22.4	7.3	0.0103	1.000	0.0036
2.45	0.34	18.5	1.69	17.5	3.3	0.0043	0.999	0.0033

H = Average stage (ft)
A = Relative amplitude of oscillation of stage
W = Wavelength of oscillation (min)
DC = Diameter of connection to stilling well (in)
DS = Diameter of stilling well (in)
LAG = Time lag between stage in flume and stage in stilling well
ATTN = (range of stilling well hydrograph)/(range of flume hydrograph)
CV = Coefficient of variation of ratio of hydrograph ordinates

TABLE 8-11
Estimated/Observed Flow Ratios for the Parshall Flumes According to Parshall's Formula

Size (ft)	Number of Tests	Average Ratio	Standard Deviation	Coefficient Variation
1	10	1.065	.026	.025
2	10	1.047	.026	.025
3	15	1.024	.018	.018
4	4	1.050	.022	.021
6	6	1.011	.019	.019
8	3	1.019	.016	.016
TOTAL	48	1.037	.023	.022

In a discussion of Parshall's paper, Carter[7] proposed the formula

$$Q = 3.9WH^{1.58} \tag{51}$$

to correct for the bias in the original formula. The results of a similar analysis of Carter's formula are given in Table 8-12. The average ratio is 1.005, which indicates a slight bias (0.5%) toward overestimation. The standard deviation is 0.017 that indicates that 2/3 of all estimates should be within $\pm$ 1.7% of the observed value. The coefficient of variation also is 0.017. Both these formulas are empirical. There is no theoretical basis for either of them, although the exponents and coefficients seem to be reasonable, intuitively.

Stormwater flow rates may be expected to exceed the 50 cfs range of Parshall's tests and wider flumes than 8 ft may be necessary. In later studies of large flumes, Parshall developed the empirical relation

$$Q = (3.6875W + 2.5)H^{1.6} \tag{52}$$

for 10 ft to 50 ft flumes.

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[7] Carter, C.E. 1926. "Discussion of the Improved Venturi Flume by R.L. Parshall." *Transactions, ASCE.* Volume 89, pp.856.

There are two theoretical approaches that could be taken to analyze a Parshall flume. These are based on the energy and momentum principles. The usual approach is to use the energy principle because it gives a simple and clear explanation. Both approaches have been considered here, but the momentum principle proved to be more useful than the energy principle.

To illustrate the analysis, some of the symbols used are shown in Figure 8-7. The depth is measured at location 1. At section 2, which is near the entrance to the throat of the flume, the flow passes through critical depth.

Let the specific energy at the gauging section be

$$E_1 = h_1 + \frac{V_1^2}{2g} = h_1 + \frac{Q^2}{2gh_1^2W_1^2} \qquad 53$$

and at the throat

$$E_2 = h_2 + \frac{V_2^2}{2g} \qquad 54$$

TABLE 8-12
Estimated/Observed Flow Ratios for the Parshall Flumes According to Carter's Formula

Size (ft)	Number of Tests	Average Ratio	Standard Deviation	Coefficient Variation
1	10	1.017	.018	.018
2	10	1.007	.016	.016
3	15	.997	.015	.015
4	4	1.050	.022	.021
6	6	.982	.019	.019
8	3	.988	.013	.013
TOTAL	48	1.005	.017	.017

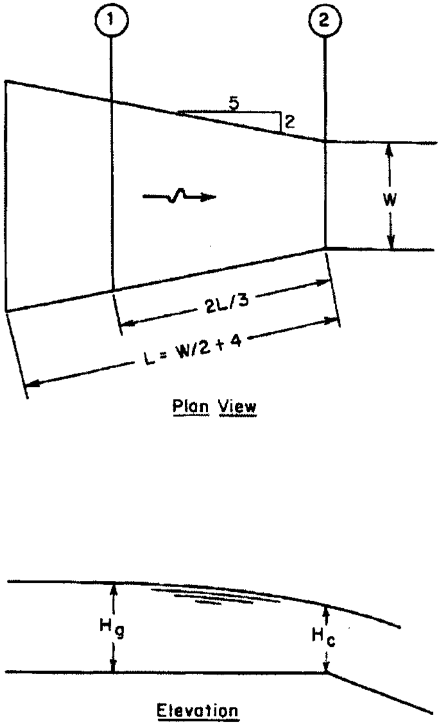


FIGURE 8-7. The Parshall Flume

Assuming that critical depth occurs at section 2,

$$h_2 = h_c = \sqrt[3]{\frac{Q^2}{gW^2}} \qquad 55$$

so that

$$E_2 = 1.5\sqrt[3]{\frac{Q^2}{gW^2}} \qquad 56$$

Using Parshall's data, values of E_1 and E_2 have been computed for each test. The average ratio E_1/E_2 for each flume is given in Table 8-13. These ratios are greater than 1.0, which indicates that there is an internal dissipation of energy due to the convergence of the flow. The convergence can be measured by the ratio W_1/W, which is larger for the smaller flumes. The internal energy loss also is greater for the smaller flumes, as indicated in the Table, by the larger ratios of E_1/E_2 for the smaller flumes. They seem to follow the empirical relation

$$\frac{E_1}{E_2} = \left(\frac{W_1}{W}\right)^{.56} \tag{57}$$

The ratios E_1/E_2 were not constant for all flows in any given flume. They decreased slightly with increasing depth, but the change with depth was much less than the change from flume to flume.

The second theoretical approach is to apply the momentum principle. The control volume is taken as the volume of fluid between the gauging section and the entrance to the throat. The forces acting on this body of water are identified and evaluated. Those acting in the downstream direction are F_1; those upstream, F_2. The ratio F_1/F_2 is equal to 1.0, theoretically. The external force of friction is neglected because it should be much smaller than the other forces.

TABLE 8-13

Ratio E_1/E_2 for Parshall's Data

Size (ft)	W_1/W	Number of Tests	Average E_1/E_2	Standard Deviation of E_1/E_2	Coefficient Variation of E_1/E_2	$(W_1/W)^{.56}$
1	2.20	10	1.558	.028	.018	1.555
2	1.67	15	1.341	.043	.032	1.332
3	1.49	10	1.248	.037	.030	1.250
6	1.31	6	1.148	.012	.010	1.163
8	1.27	3	1.138	.005	.004	1.143

The forces acting on the control volume in the downstream direction, F_1, result from the momentum of the flow passing the gauging section and from the hydrostatic pressure of the water. The forces acting on the control volume in the upstream direction, F_2, result from the momentum of the flow passing the throat section, the hydrostatic pressures at the throat section, and the component of the hydrostatic pressure forces exerted by the sides of the channel in the upstream direction. The forces downstream are

$$F_1 = \frac{Q^2}{gA_1} + \frac{h_1{}^2 W_1}{2}$$

58

The forces upstream are

$$F_2 = \frac{Q^2}{gA_2} + \frac{h_c{}^2 W}{2} + F_p$$

59

where F_p is the total upstream pressure force exerted by the sides of the flume. Assuming that the water surface profile between the throat and the measuring section follows a parabolic curve, the equation for F_p is

$$F_p = \frac{(h_1 - h_c)^2}{4} + \frac{2h_1 h_c}{3} - \frac{h_c{}^2}{6}(W_1 - W)$$

60

In these equations,

$$h_2 = h_c = \sqrt[3]{\frac{Q^2}{gW^2}}$$

61

and

$$A_2 = h_c W$$

62

The equation for the flow profile can be developed by changing the control volume to a body of water extending across the flume with a thickness dx. The difference in the forces across this volume must be exerted by the sides. If

$$F = \frac{Q^2}{gA} + \frac{h^2 W}{2}$$

63

and the force exerted by the sides is

$$dF = h^2 dW$$

64

then

$$\frac{dF}{dx} = h^2 \frac{dW}{dx} = -\frac{Q^2 W}{gA^2}\frac{dh}{dx} - \frac{Q^2 h}{gA^2}\frac{dW}{dx} + hW\frac{dh}{dx} + \frac{h^2}{2}\frac{dW}{dx}$$

65

so that

$$\frac{dh}{dx} = \frac{\left(\frac{h^2}{2} + \frac{Q^2 h}{gA^2}\right)\frac{dW}{dx}}{hW - \frac{Q^2 W}{gA^2}}$$

66

at x = 0, h = h_c, so that

$$\lim_{x \to 0} \frac{dh}{dx} = \infty$$

67

For the Parshall flume

$$\frac{dW}{dx} = 0.2$$

68

so that

$$\frac{dh}{dx}=0.2\left(\frac{W}{h}\right)\frac{\left(\frac{h^2}{2}+\frac{Q^2}{ghW^2}\right)}{\left(h^2-\frac{Q^2}{ghW^2}\right)} \qquad 69$$

Equations 58 and 59 have been used to compute F_1 and F_2 for Parshall's data. The ratio, F_1/F_2, for each test is tabulated in Table 8-14. The average ratio is .958; the standard deviation is .027; and the coefficient of variation is .028. The computed forces in the downstream direction may be less than the computed forces in the upstream direction, because critical depth does not occur exactly at the entrance to the throat. If critical flow should occur downstream from the entrance, the bottom of the flume would exert a small downstream force. This could lead to the computed results found here.

TABLE 8-14
Ratio F_1/F_2 for Parshall's Data

W	H	O	F_1	F_2	F_1/F_2
1.	0.200	0.31	0.0508	0.0509	0.9972
1.	0.398	0.91	0.2036	0.2104	0.9677
1.	0.600	174	0.4672	0.4934	0.9470
1.	0.599	1.74	0.4636	0.4847	0.9564
1.	0.801	2.72	0.8361	0.8908	0.9386
1.	0.802	2.69	0.8349	0.8819	0.9467
1.	1.002	3.86	1.3143	1.4134	0.9299
1.	1.001	3.78	1.3037	1.3841	0.9410
1.	1.202	5.10	1.8947	2.0451	0.9265
1.	1.603	7.73	3.3528	3.5810	0.9363
2.	0.201	0.62	0.0852	0.0844	1.0091
2.	0.201	0.61	0.0846	0.0829	1.0199
2.	0.402	1.87	0.3504	0.3618	0.9686
2.	0.402	1.82	0.3461	0.3513	0.9852
2.	0.598	3.49	0.7858	0.8255	0.9519
2.	0.602	3.38	0.7808	0.7992	0.9770
2.	1.002	7.89	2.2522	2.4255	0.9286
2.	1.000	7.70	2.2191	2.3597	0.9404
2.	1.602	16.51	5.8626	6.4408	0.9102
2.	1.593	15.89	5.7001	6.1620	0.9260
3.	0.100	0.92	0.1180	0.1159	1.0181
3.	0.298	1.71	0.2666	0.2641	1.0092
3.	0.399	2.75	0.4873	0.4940	0.9866
3.	0.503	3.93	0.7785	0.7936	0.9810
3.	0.595	5.13	1.0982	1.1290	0.9727
3.	0.600	5.13	1.1590	1.1318	0.9798
3.	0.699	6.74	1.5431	1.6151	0.9554
3.	0.900	10.09	2.5955	2.7535	0.9426
3.	0.996	11.84	3.1941	3.4032	0.9386
3.	0.999	11.92	3.2178	3.4325	0.9374
3.	1.000	11.81	3.2031	3.3869	0.9429
3.	1.004	11.79	3.2139	3.3940	0.9469
3.	1.200	15.99	4.6974	5.0612	0.9281
3.	2.039	35.23	13.5173	14.5232	0.9307
3.	2.358	45.66	18.5650	20.3717	0.9111
4.	1.014	15.09	4.1243	4.1760	0.9876
4.	1.339	24.55	7.5164	7.0029	0.9511
4.	1.702	35.44	12.2035	12.8793	0.9475
4.	2.008	46.31	17.2128	18.3455	0.9383
6.	0.900	20.11	4.9599	5.1124	0.9702
6.	1.143	30.04	8.2555	8.6846	0.9506
6.	1.171	29.94	8.4156	8.6877	0.9687
6.	1.396	40.20	12.2354	12.8203	0.9544
6.	1.562	49.51	15.7920	16.8459	0.9374
6.	1.617	50.07	16.4051	17.1822	0.9548
8.	1.064	35.08	9.2806	9.5579	0.9710
8.	1.246	45.12	12.8735	13.3484	0.9644
8.	1.270	45.23	13.1088	13.4291	

Average ratio = 0.9575
Standard deviation of ratio = 0.0267
Coefficient of variation = 0.0278

To verify the rating formula that can be deduced by the momentum principle, a computer program was written to solve the momentum equations for the flow rate as a function of the depth at the gauging section. Data for the 1-foot, 3-foot, and 6-foot flumes (Figure 8-8) show excellent agreement between Parshall's data and the theoretical results. In making these computations, critical flow was assumed to occur downstream from the throat of the flume, so that the bottom of the flume exerts a downstream force equal to 5% of the total forces exerted at the gauging sections. The momentum principle was used to estimate the flow rate for each of Parshall's reported tests. The average estimated/observed flow ratio is 1.010; the standard deviation is .048; and the coefficient of variation is .047. These results could be improved upon by more complete consideration of the position of the critical section.

3.3 Data from Northwood

Included among the many areas gauged by the Johns Hopkins Storm Drainage Research Project is a 47.1-acre area in Baltimore City known as Northwood. It encompasses a shopping center that occupies about half of the area, and a residential community that occupies the other half. The imperviousness is 68%. Rainfall is measured with a tipping-bucket rain gauge at the gauging site. Runoff is measured with a 12 ft Parshall flume. These data are recorded on a Stevens Type A35 recorder operating at a chart speed of 0.6 inches per minute. The recorder is actuated by a specially designed timer circuit that turns it on when the .01-inch rain gauge bucket tips and keeps it on until some time (one-half hour) elapses or another tip resets the timer.

The synchronous record allows detailed study of the watershed, although manually plowing through the endless streams of chart paper is a monumental task. Data from two selected storms are presented here in Figures 8-9 and 8-10 so they may serve as examples of the information collected.

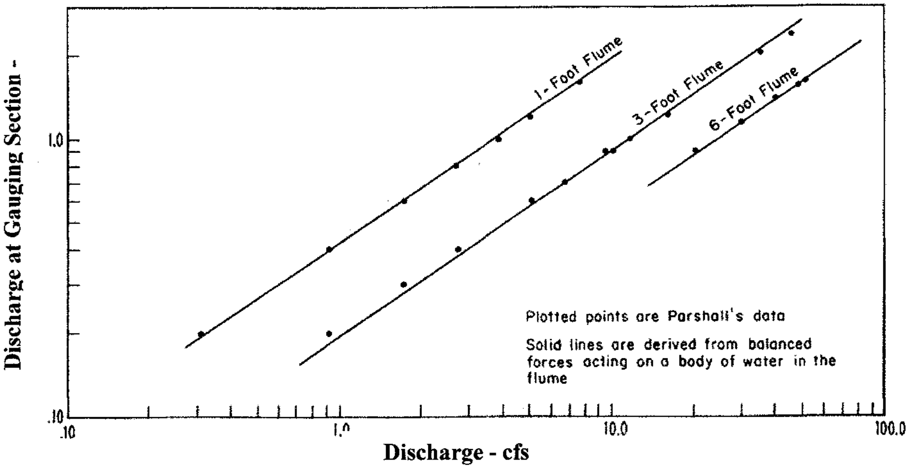


FIGURE 8-8. Verification of Rating Curve Derived From Momentum Equation.

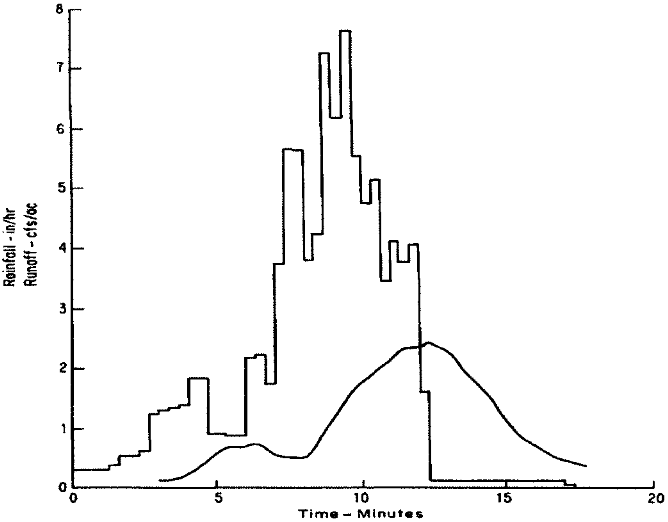


FIGURE 8-9. Storm Hydrograph (1) – Northwood.

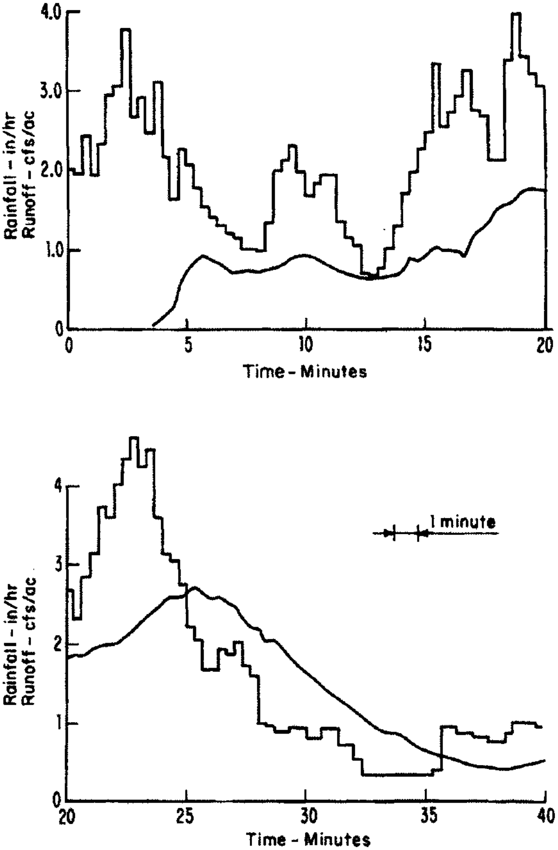
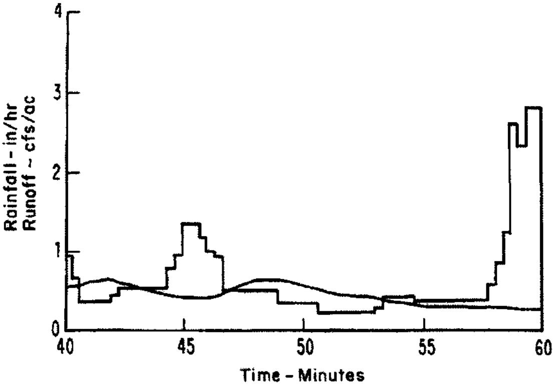


FIGURE 8-10. Storm Hydrograph (2) – Northwood.



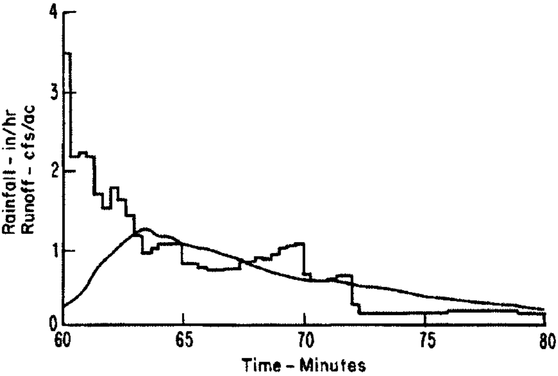


FIGURE 8-10 (continued). Storm Hydrograph (2) – Northwood.

Data in these figures were sampled from the original chart records at 20-second intervals.

Some relevant statistics were tabulated for each storm. For the storm in Figure 8-9, the lag time between the centroid of the observed rainfall hyetograph and the centroid of the runoff hydrograph was only 2.91 minutes. The total volume of rain was 0.55 inches during the 17-minute storm duration. The volume of runoff was exactly one half the volume of rainfall. The average infiltration rate was 0.97 in/hr for 17 minutes.

A spectral analysis was made of the longer storm in Figure 8-10. The total variance of the rainfall about the mean is 1.135 in^2; the total variance of the runoff about its mean is 0.404 $(\text{cfs/ac})^2$. The average rainfall intensity is 1.45 in/hr, and the average runoff rate, 0.94 cfs/ac. (These statistics apply only to the first 70 minutes of the storm.) For both the rainfall and runoff, all of the variance is concentrated at the low frequency end of the spectrum. Rainfall variation for wavelengths shorter than 2 minutes is negligible during this storm. More than one half of the variance is due to frequencies with wavelengths longer than 20 minutes. Runoff variation is also concentrated at these low frequencies because the catchment filters out any high frequency components that may be present. This filtering activity for the Northwood area during this storm is illustrated in Table 8-15. The proportion of the rainfall variance passing through the catchment at various frequencies is tabulated in this table. Less than 5% of the rainfall variability at wavelengths shorter than about 8 minutes could be found in the runoff.

TABLE 8-15

Rainfall Variation Passing Through the Northwood Catchment at Various Frequencies

Wavelength (min)	Percent of Rainfall Variance in Runoff
66.67	.608
33.33	.575
22.22	.320
16.67	.183
13.33	.151
11.11	.158
9.52	.206
8.33	.116
7.41	.044

4.0 WATER QUALITY

Studies of urban stormwater quality usually have attempted to measure general parameters such as average concentrations, maximum concentrations, and total pollutant loads during storms. No significant theory is reported in the literature to account for the mechanisms that affect water quality.

Recently, Weibel, Anderson, and Woodward,[8] reported a PHS study of runoff quality from a catchment in Cincinnati, Ohio. Samples were taken at 10-minute intervals, which also is the lag time of their study area. These data were collected from the limited number of storms during a one-year period between July, 1962, and September, 1963. According to that study, there seems to be a relationship between total storm volume and total pollutant load. There also seems to be a relation between the flow rate and the concentration of suspended solids.

In planning for water quality sampling of urban areas, the response characteristics of alternative sampling schemes are of interest as they relate to specific measurement objectives. It will be assumed here that bottled samples are to be taken for subsequent chemical and biological analysis. The purpose here is to consider water quality sampling schemes only. No consideration is given to how concentrations in the sample are determined in the laboratory. Essentially, it is assumed that any measurement error in the laboratory method for detecting the true sample concentration must be added to the sampling errors for a particular measurement objective for a particular sampling scheme. In addition to the measurement objectives usually considered, an additional objective to account for the mechanisms affecting runoff quality is needed.

One approach to designing a water quality data system is to assume, on the basis of the limited information available, that certain factors affect water quality. These assumptions would be used to construct a water quality "generator" for generating stream flows and quality concentrations that are inputs to a computer simulation model of alternative sampling schemes. The quality concentrations in the simulated samples are recorded, and relevant statistics regarding the performance of the sampling schemes are computed. A very simple model is described below to illustrate the feasibility of the procedure. When the pilot system is designed for a specific area, a similar procedure should be used as part of a total instrumentation system simulation study. At that time, a random component should be added to unknown mechanisms that are not accounted for in the quality model.

4.1 Water Quality Model

Assume that a catchment behaves according to unit hydrograph theory so that the excess rainfall occurring at any time is distributed downstream according to the unit hydrograph. Assume that this excess rainfall picks up, in a solution and in

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[8] Weibel, S.B., R.J. Anderson, R.L. Woodward. November 1964. "Urban Land Runoff as a Factor in Stream Pollution." *Journal Water Pollution Control Federation*. P. 914.

suspension, a pollutant as it passes through the catchment. Assume also that the downstream concentration of this pollutant is distributed as a function of the unit hydrograph. Let h(t) be the instantaneous unit hydrograph. Let w(t) be the concentration of pollutant in the runoff from an instantaneous unit storm. Assume that

$$w(t) = \alpha h(t) \tag{70}$$

The runoff hydrograph from the storm I(t) is

$$Q(t) = \int_0^t h(\tau) I(t-\tau) d\tau \tag{71}$$

The concentration of pollutant from the storm is assumed to be

$$W(t) = \int_0^t w(\tau) I(t-\tau) d\tau \tag{72}$$

The total flow of pollutant would be

$$U(t) = Q(t) \cdot W(t) \tag{73}$$

Let

$$u(t) = \alpha h(t)^{\beta+1} \tag{74}$$

so that

$$U(t) = \int_0^t u(\tau) I(t-\tau) d\tau \tag{75}$$

To illustrate the significance of the original assumptions, we will compare the runoff and water quality hydrograph for a unit storm. First, let the unit hydrograph be:

$$h(t) = \lambda t e^{-\lambda t}$$

76

so the concentration of pollutant is

$$w(t) = \alpha \lambda^{\phi} t^{\beta} e^{-\lambda \beta t}$$

77

The mass flow rate of pollutant is

$$u(t) = h(t) w(t) = \alpha \lambda^{\beta+1} t^{\beta+1} e^{-\lambda(\beta+1)t}$$

78

Both u(t) and h(t) are of the general form

$$y(t) = c t^{a} e^{-bt}$$

79

The centroid of y(t) is at

$$\bar{t} = (a+1)/b$$

80

and the variance of y(t) is

$$\sigma^{2} = (a+1)/b^{2}$$

81

The centroids of h(t) and u(t) are

$${}^{\bar{t}}h = \frac{2}{\lambda}$$

82

and

$$\bar{t}_u = \frac{\beta+2}{\lambda(\beta+1)} \tag{83}$$

so that

$$\bar{t}_u = \frac{\beta+2}{2(\beta+1)}\bar{t}_h \tag{84}$$

which is less than 1.0 if β > 0.

Moreover, the variance of h(t) is:

$$\sigma h^2 = \frac{2}{\lambda^2} \tag{85}$$

and the variance of u(t) is

$$\sigma u^2 = \frac{\beta+2}{\lambda^2(\beta+1)^2} \tag{86}$$

Thus,

$$\sigma u^2 = \frac{\beta+2}{2(\beta+1)^2}\sigma h^2 \tag{87}$$

which is also less than 1.0 if β > 0.

From these results, we have deduced that the mean and variance of the quality "unitgraph" are likely to be less than those for the runoff unit hydrograph. Therefore, the shape of the concentration hydrograph should be somewhere between the shapes of the rainfall hyetograph and the runoff hydrograph. The shape approaches the rainfall pattern for large values of β. It approaches the runoff pattern for small values of β.

4.2 Optimum Sampling Policy

Let $q_i(t)$ be the quality hydrograph for storm i. Consider sampling $q_i(t)$ at discrete instants of time. Let the index j denote the j-th sampling instant. The problem is to decide when to sample if the total number of samples is limited, say, to N. The feasible sampling instants are denoted by j, but the number of feasible instants, M, is greater than the maximum number of samples, N.

Let δ_j be a decision variable that indicates by its possible values (0, 1) whether a sample is to be taken at instant j. The constrain imposed is equivalent to

$$\sum_{j=1}^{M} \delta_j \leq N \tag{88}$$

A sampling policy is needed to decide if δ_j should be 0 or 1 for any particular j. Many policies could be devised, but the relative merit of any of them depends on the need for the data produced.

Assume that the data are needed to measure the total amount of pollutant discharged during a storm. An optimal sampling policy can be defined for an appropriate objective such as to minimize the sampling variance of the estimated pollutant discharged. In information theory, the inverse of the sampling variance is usually called the information content. An equivalent objective is to maximize the information content of the samples actually taken when as many as N samples are permitted.

The optimal policy is not obvious because the duration of the storm is not known until it is over. Then it is too late to take any more samples. On the other hand, the major part of the storm may not come at the beginning, so it seems wise to defer taking some of the samples to hedge against future flows.

Other specifications of information content could be devised for other measurement objectives. For example, the sampling variance of a statistic for estimating the parameters in a water quality model, such as described here, would lead to another measure of information content. Each measurement objective will have its own measure of information content. These will probably lead to different optimum gauging policies. To satisfy multiple measurement objectives, mixed measurement models must be devised, or minimum levels of information content, according to the alternative definitions, must be maintained.

The purpose here has not been to solve the problem, but to raise it as a significant question. It needs to be solved to make the best use of available sampling capability at a particular gauging site. The approach to this problem can also be extended to finding the optimum investment plan for gauging networks where the objective could be to allocate limited sampling capability to a set of gauging sites so that the information content of the data from the network is maximized. Methods of Operations Research and Statistical Decision Theory should be centered on this problem.

Chapter 9

SYSTEMATIC STUDY AND DEVELOPMENT OF LONG RANGE PROGRAMS OF URBAN WATER RESOURCES RESEARCH[1]

October 1968
Urban Water Resources Research Council
Of The American Society Of Civil Engineers

1.0 BACKGROUND

The Project Steering Committee for this effort consisted of:

- William C. Ackermann
- John C. Geyer
- Carl F. Izzard
- Stifel W. Jens (Chairman)
- D. Earl Jones, Jr.

Readers are urged to pay particular attention to the appendices, as they contain then state-of-the-art information on computer modeling, damage, storage, societal issues, and feasibility factors. The report on long-range urban water issues and programs follows.

2.0 GENERAL

The Steering Committee is of the opinion that this report on urban water resources research, supplemented by the work of the USGS-ASCE project on "Analysis of National Basic Information Needs in Urban Hydrology" (scheduled to terminate in April 1969), identifies the major problems and recommends studies and research for approaches to their solutions. It is gratifying that these first comprehensive national efforts have built such a sound, substantial foundation for continuing work to improve the development, use and control of water in the urban environment. The basic problems are all related to the time-space-quality characteristics of water as encountered in our nation's populous areas. Every concern of urban man has been to achieve better understanding and control of these highly complex and interrelated characteristics. Much remains to be known of the effect of water on man and his activities.

3.0 USGS-ASCE PROJECT

The "Analysis of National Basic Information Needs in Urban Hydrology" has focused on: "Data Needs," "Data Devices," and "Data Networks." Primarily aimed at

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[1] Note: Only selected appendices (A, C, D, E, and H) are attached. The full report contains Appendices A-H; however, only selected appendices are attached due to space limitations. Missing appendices can be obtained by contacting EWRI.

improvement in design of storm drainage, intensive study is being made of the data requirements for analyzing the rainfall-runoff-quality relationships, with study of the need for suitable data collection instrumentation, and consideration of the types of networks required for the collection of adequate data. Suitable data collected with properly coordinated instrumentation in networks representing a variety of climatic, topographic and land-use conditions, are virtually non-existent. There are very meager amounts of performance data with which existing or proposed storm drainage facilities can be checked or designed. Transfer of data findings between metropolitan regions is a desirable primary objective. Urban water data acquisition and use would also be invaluable in facilitating the exchange of various quantities and qualities of water from one service to another, such as using stormwater for water supply. There are important implications in the use of data as part of the intelligence required for comprehensive management and operation of urban water services.

4.0 OWRR-ASCE PROJECT

The Office of Water Resources Research project has been aimed at a "Systems Overview," and at identification of research needs and their initiation through three Task Committees on: "Methods of Analysis," "Damage and Storage," and "Non-Hydrologic Aspects."

5.0 SYSTEMS ANALYSIS

"Systems Analysis" can be defined briefly as "a body of techniques that can materially aid the study of technological sub-systems through simulation of many possible alternative responses to specified inputs." (Appendix H, attached) The research objective is to formulate and test a systematic analytical structure and method for guiding the various team members involved in decision making, toward economically optimal, comprehensive urban water planning, investment, design and operation. To choose optimally from a large number of technological and economic alternatives is not a new engineering goal, but the "systems" approach to attain it is armed with relatively new, highly sophisticated "tools" and methods. Each of the four pre-feasibility report (Appendices G, H, I and J) find that comprehensive yet detailed coordinated analyses are practicable and feasible. Data such as are considered under the USGS-ASCE project are essential (as are certain economic and social data) for effective analysis.

6.0 METHODS OF ANALYSIS

The "Methods of Analysis" Task Committee addressed itself to the technical relationships of urban rainfall-runoff-quality and found that there is need "to concentrate efforts on the development of modeling techniques which can be used to extend and interpret the significant implications of short-term records" (Appendix A, attached), and to stimulate and guide the acquisition of adequate historical data on urban rainfall-runoff-quality. Models responsive to change in the physical environment would possess a highly valuable degree of transferability.

Initial emphasis on deterministic approaches appears to offer better prospects for understanding cause and effect relationships. Extensive comparative testing of existing models offers a possibility of near-term advancement in design methodology. Sensitivity analyses of the models available now or in the near future should assist in producing near-term approximations of practical value.

More urban hydrologic data are needed for calibrating and testing mathematical models. There is need to understand better the loss function (interception, depression storage and infiltration) and how to approximate more realistically the equations describing it. "The most glaring data weakness at this time is in the area of water quality," i.e., the time rate of delivery of the water quality constituent for the pertinent runoff period.

Attainment of satisfactory, realistic detailed knowledge of time and space variability of rainstorms is most desirable.

7.0 DAMAGE AND STORAGE

Very little useful documentation exists on damages from flooding in urban areas, despite evidence that national annual losses are in the vicinity of a billion dollars. This Task Committee focused on the research required to assess realistically urban storm drainage costs as they should be and are related to flood damage assessment, and explored alternatives to direct disposal of runoff such as recharge basins or other detention storage schemes.

It suggests that a careful analysis of total investment in urban drainage including costs of operation, maintenance, replacement, repair, etc. might provide an indirect, first estimate of the cost or value of damage from flooding "on the assumption that such investment has been intuitively gauged at a level for which the value of protection is at least as great as the cost of service . . . Only with some indication such as this can an analysis of the cost-effectiveness of urban drainage be started."

The more than minor character of damages from storm sewer flooding is indicated by a field study of a 5-square-mile Chicago suburban area where such costs were found to be $200,000 per year.

The Steering Committee agrees with the emphatic statement of the Task Committee that even a rough first cut at evaluation of the scope and scale of urban flooding damage would be much better than the purely subjective approach all too commonly used today.

A parallel study is desirable to determine who pays for damages and their share of the total, including property owners, communities, states, insurance firms and the federal government. Consideration should be given to indirect controls over the extent of flood damages, by flood-proofing, flood-zoning practices, and the kind of developments permitted in urban areas.

Provision of storage attenuates urban runoff hydrographs, reducing the flood peaks, with an attendant possible reduction in overall costs for a drainage system. Storage can sometimes be used for recreation or for water reuse or recycling for other services. Tradeoffs can often be exploited by diffusing peaking problems among types of services in multi-purpose development.

The smoothing of water variations is the underlying objective in water resources development and management. The incorporation of storage is inextricably tied to the total design of drainage systems, and also to water supply and distribution systems.

8.0 NON-HYDROLOGIC ASPECTS

In alphabetical order, the principal, predominantly non-hydrologic, aspects of urban water are:

- Administration of works;
- Economics of planning and operation;
- Esthetics of urban environment and specific works;
- Financing of works;
- Health and safety of population served;
- Legal latitudes and constraints;
- Planning and land use, metropolitan-wide;
- Political impacts on planning and implementation of works;
- Recreational facilities planning and operation; and
- Sociological inferences between a viable metropolis and development of its water resources.

The above listing is based on the assumption that development of future services, including water, will tend towards a metropolitan region scale (Appendix D, attached).

A problem-directed team approach is an absolute necessity for effective research on urban water resources. An *a priori* condition is the necessity for clear definitions of vocabularies, nomenclature and terminology mutually understandable by urban water engineers, social scientists and other planning team representatives.

More detailed information on costs and benefits of different scales of facilities is needed to arrive at equitable charge schedules and judicious selection among alternatives.

Approaching as closely as possible to an optimum resolution of public needs and desires, a good plan must be responsive to social, economic and political goals.

When regarded in turn at the local, state and national level, these criteria wear different attire. Furthermore, massive, restive, dynamic change is currently evident in public needs, desires and goals.

The preferred institutional framework within which metropolitan water resources planning should take place remains to be identified.

Research is needed on institutions for management of water at various levels, extending from an entire metropolis to the local neighborhood.

With respect to social impacts of urban water resources development, the Task Committee (and the Steering Committee) "feels very strongly that serious study should be given to the effect in the past of technological change on social change, of social change on technological change, and the relationships between them."

The scope of administrative authority for planning public water services and their role needs study. What are the specific responsibilities of the various governments (local, state and federal) in urban water, legally, sociologically, economically, administratively, etc.? What are the proper, best methods for communicating technical facts to the public, such as alternatives including the cost of disaster, enhanced benefits of multiple-use projects, and goals of long-range plans?

Financing practices and restraints should be considered in arriving at improved policies for more efficient investments within the total urban budget. Detailed information is needed on the basis for charging for various water services, particularly with regard to who pays for what part of each service.

What legal encouragements and constraints operate to influence the policies of public agencies to enhance, benefit, direct, constrain, inhibit, etc. urban water resource development? How can the legal profession be induced to enter into research on water law in an interdisciplinary, interfacial way, with some emphasis on national needs as a whole while augmenting current research at the state level? To repeat a finding of the Engineering Foundation Research Conference of August 1965: "Legal considerations need research to develop solutions for the lack of uniformity in existing state (drainage) law, which is divided and often mingled among common-enemy, civil and 'reasonable use' rules." The greatest legal question is the one of liability, which hinges on identification of who is responsible in a given instance.

9.0 COLLABORATION BETWEEN SOCIAL SCIENTISTS AND ENGINEERS

In Appendix E, attached, Professors Duane W. Hill and Charles L. Garrison make an eloquent and convincing plea for highly desirable collaboration of engineers and physical scientists with social scientists. They "conclude that the most probable areas for successful and fruitful collaboration . . . are human organization, field theory and survey research, communications and cybernetics, decision making and gaming, economic analysis that extends beyond traditional theoretical designs and their heavy dependence on rational models, and sociological analysis that moves out from independent variables, like income and education level, to the development of coefficients of change over time (historical ones) that should provide a higher degree of control in experimental testing. In outlining the above, we assumed, of course, that both economists and legal scholars would continue to collaborate, as they have in the past." The Appendix concludes with a challenging list of 15 viable research enterprises.

10.0 SALIENT RECOMMENDATIONS

All aspects of urban water resources research should be prosecuted concurrently with provisions for ample intercommunication and feedback.

1) The results of this first year of nationally funded, centrally directed research on the manifold problems of the heretofore long neglected field of urban water resources argue strongly for the urgent need for a continuation of a coordinated, centrally directed national research program.
2) To implement the foregoing, to resolve the multi-aspect problems, serious effort should be made to create an appropriate entity for stimulating, coordinating, and undertaking urban water resources research. The OWRR-ASCE and USGS-ASCE findings should be topics of that research. Such an entity should be an information repository and clearinghouse and an agency for securing "effective communication between affected fields, particularly for interdisciplinary, interagency, inter-municipality, multi-disciplinary and multiple-service groups."
3) The unsatisfactory status of urban hydrology is due to the absence of a meaningful body of field data. The acquisition of rainfall-runoff-quality data should be started as soon as possible. Starting with the very small amount of data now available, existing mathematical models for simulating the rainfall-runoff-quality process should be continually tested and improved. Research on metropolitan storms should be pursued not only to develop inputs suitable for future use but for contemporary management and operation of works.
4) Future new towns might advantageously be used as field laboratories by incorporating several different drainage schemes in the same town, utilizing different extents of storage and imaginative multi-purpose works.
5) Research is needed on: costs and benefits of various scales of facilities; planning process with socio-system simulation incorporated in the research; preferred institutional framework within which metropolitan water planning and management should take place; ways to measure or evaluate social efficiency compatible with those existing for economic efficiency; the relationship between technological change and social change, and the effects in the past of the one on the other.
6) Studies on needed policy changes to lessen or remove legal, political, jurisdictional, administrative, planning or other non-engineering impediments to more efficient and effective water resources planning.

7) Intensive research is needed on the ways in which public attitudes and ideological formations affect engineering outcomes; and vice versa (collaboration between social scientists and engineers).
8) Development and refinement of measures for quantifying social values, social costs and social benefits to achieve more valid and reliable scales of social values and attitudes for more effective decision making at all levels.
9) System simulation model design should be started immediately, continual cycling of refined versions of simulation and sensitivity analyses should next be undertaken, and an operational stage systems overview capability should be attained as soon as possible. A single approach may not be desirable. More than one level of development might be pursued concurrently, each for a particular type or scale of objective.

11.0 ACKNOWLEDGEMENTS

The high quality work reported upon herein represents the teamwork of more than 100 people, including several organizations. To each of these the Steering Committee expresses its deep appreciation. It is especially grateful to the Project Officers and their associates in the OWRR and the USGS, and to the members of the Ad Hoc Task Committees and Task Groups.

The ASCE Executive Secretary, W.H. Wisely, and Research Director, D.C. Taylor, have given freely of their invaluable advice and wise counsel. M.B. McPherson, Program Director and L.S. Tucker, Deputy Program Director, deserve accolades of the highest kind. The leadership, inspiration and direction of Program Director McPherson are attested to by the extremely high quality of this report. All those now and in the future, who have an interest in urban water resources, will be indebted to him for his part in directing this basic initial comprehensive study.

INTRODUCTION, SUMMARY AND RECOMMENDATIONS
SECTION I

1.0 ABSTRACT

ASCE is assisting in outlining, developing, and initiating a coordinated national program of urban water resources research. Deliberate and systematic study of urban water problems on a comprehensive basis has long been neglected. The objective of the research is to provide guidelines for initiating and expanding a program of long-range studies in urban water problems. This report covers the first year of work.

12.0 INTRODUCTION

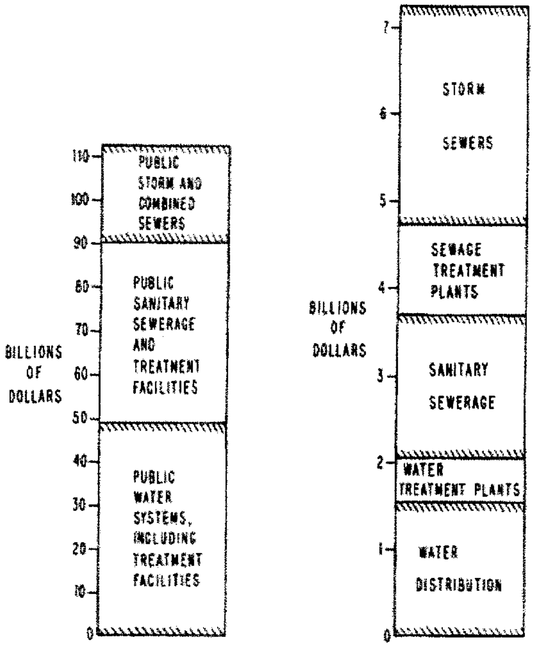
Urban water resources planning and supporting research are at levels well below those for river basins and large regional complexes. Deliberate and systematic study of urban water problems from an overall point of view had long been neglected. This report covers the first year accomplishments of an ASCE project, supported by OWRR, to assist in outlining, developing and initiating a coordinated national program of urban water resources research. The project will also have significant international benefits.

National capital investment and projected annual construction costs for some principal components of urban water service are indicated in Figure 9-1. As will be noted in the next chapter, average total expenditures by local governments on urban water will be on the order of $12 billion or more per year through 1980.

The primary objective of the OWRR project is to provide guidelines and give meaningful direction for initiating and expanding a program of long-range studies of all types of urban water problems by means of a centralized major effort.

General objectives of the project include: studies of needs and justification for research on urban water problems; identification of high priority and key research needs; development of logical methods and means for accomplishing or at least initiating needed research; stimulation of and collaboration in complementary or parallel studies on portions of the overall program by municipalities, research firms and universities; initiation and pursuit of research and development on key and immediate needs; establishment of an efficacious method and appropriate media to facilitate incorporation of research findings into practice; and establishment of a formal or informal information exchange system for use in continuing research on urban water problems.

First year project objectives were as follows: conduct a pre-feasibility study to determine the possible effectiveness, cost, and time requirements for a comprehensive systems engineering analysis of all aspects of urban water; conduct a pre-feasibility study to determine the possible effectiveness, cost, and time requirements for a general economic analysis of costs and pricing parameters of all aspects of urban



REPLACEMENT VALUES OF PUBLIC WATER, WASTEWATER AND DRAINAGE FACILITIES. CONTEMPORARY DOLLARS

ESTIMATED ANNUAL AVERAGE PUBLIC CONSTRUCTION COSTS FOR WATER, WASTEWATER AND DRAINAGE FACILITIES OVER THE NEXT SEVERAL YEARS

FIGURE 9-1. Some components of urban water investment and costs

water; conduct a state-of-the-art study of mathematical models and related simulation methods potentially usable for analyzing urban rainfall-runoff-quality processes; study requirements for assessment of drainage damage and explore alternatives to direct stormwater runoff, such as the utilization of recharge basins or other storage schemes; study political, economic, legal and social problems related to urban water management with recommendations for further work; and prepare appropriate reports of the research results of the project suitable for publication in engineering and professional journals.

Background on the status of urban water resources as it relates to research needs and a rationale of prioritization for research are set forth in Section II, "Urban Water Resources."

13.0 REPORT SUMMARY

Emphasis in the first project year was deliberately on subjects requiring earliest consideration.

Considerable attention was given to several technical aspects of storm drainage. Major goals are: to arrive at general mathematical descriptions of the rainfall-runoff process combined with a basis for quantifying pollution loadings from storm sewerage systems applicable nationwide; to develop adequate technical knowledge for reliable planning of water quality and quantity exchanges between the several urban water service functions for multi-purpose development; and to facilitate and improve the planning, design and management of drainage works.

Because the hydrology of urban drainage is the keystone for improved understanding of the rainfall-runoff-quality process, the highest immediate technical priority is for development of better and more compatible measuring and sampling devices, their perfection in field installations, and analysis of data therefrom. Very little data exists on the rainfall-runoff-quality process. Sewerage systems and canalized drainage works are generally constructed, operated and maintained by local governments. The acquisition of hydrologic and quality data for local drainage has not previously been considered an overall federal responsibility, and this springs at least partly from a question of jurisdiction. Very little data have been obtained by local agencies on their own systems and research of national scope is almost non-existent. Two of the several reasons for this situation are: the absence of a suitable assemblage of measurement devices and a typical lack of research facilities in local agencies charged primarily with design, construction, operation and maintenance of works.

The Geological Survey, U.S. Department of the Interior, is studying requirements for data.

Under the ASCE project for OWRR, an assessment of the potentials, liabilities and available knowledge on the rainfall-runoff-quality process has been made and model requirements for process simulation have been detailed (Appendix A).

Anticipating an eventual melding of adequate process data and process simulation capability, research needs on metropolitan storm occurrences on a probabilistic basis have been explored. Results of storm research would be essential for reaching the major technical goals cited above, and constitute a principal component for national transferability of results.

The absence of meaningful process data and knowledge has frustrated accounting for flood damages, although these are of substantial economic importance. Immediate research needs with regard to damage evaluation have been enumerated together with those for utilization of storage to ameliorate flooding (Appendix C).

Civil engineers are fully aware of the fact that non-technical considerations frequently transcend technical ones, and that some of the most complicated and intractable issues are beyond the usual scope of typical engineering activity. Considerable attention was given to research needs in political, economic, legal, social and related aspects (Appendix D). Potentials in social research for helping solve urban water resource problems have been suggested (Appendix E).

Some neglected research areas in the water works field have been emphasized in the project (Appendix D) and these have been blended with related needs identified elsewhere.

Pre-feasibility studies of engineering systems (Appendices G and H)[1] and economic systems (Appendices I and J)[3] clearly indicate physical feasibility, and the implementation of systems analyses is recommended to serve as an overall reference for a coordinated program of long-term research.

It is safe to say that all original first year project objectives have been reached; indeed, that in several instances they have been exceeded. This has been made possible through a closely coordinated team effort by scores of dedicated and unselfish professionals who are sincerely convinced that serious attention should be given to the largely neglected area of urban water resources research. An appeal is made in this report for a coordinated research attack, simultaneously on several fronts. The work reported can be viewed in principle as a very modest precursor of what can be accomplished when the elements of motivation, professional identity and individual expression are taken into account. It should be carefully noted that project participants concentrated on highly practical problems, even with regard to the social sciences. A problem orientation should be preserved to the greatest extent practicable.

Because research on many aspects of urban water has so long been neglected, there has been a clear evidence of uncertainty among project participants as to where to start. With no intention of redundancy, the place to start is at the beginning. That is, at the beginning of each neglected aspect, <u>simultaneously</u>. Hence, the urgent need for a coordinated, centrally directed national research program. A single aspect research

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[1] Appendices G, I, and J are not included due to space constraints.

approach is totally inadequate and, indeed, is entirely inappropriate, for resolving multi-aspect problems. The former simplistic approach of regarding a unit of water as a fixed entity, such as "stormwater," must be abandoned for that same unit at a different point in time will be categorized as "water supply," "recreation," "aesthetics," etc., perhaps several different times before leaving a given metropolis.

Simultaneous, concerted, synchronized attacks require a scale of effort foreign to many research organizations, particularly those of universities.

In a follow-on contract with OWRR, ASCE intends to extend further the work reported here, exploiting the contagious enthusiasm the first year work has generated among all disciplines involved.

The ASCE Urban Hydrology Research Council, which sponsored the proposal for the OWRR project, conducted an Engineering Foundation Research Conference in August 1968, on "Water and Metropolitan Man," and the conference findings have had material beneficial impacts on this report.[2]

14.0 SCOPE OF RECOMMENDATIONS

The remainder of this section contains the central, or main, recommendations of the first year of the OWRR project. A recital of the multitude of suggestions set forth in the Appendices would not only be duplicative but would divert attention from the major and most urgent issues.

15.0 FUNDAMENTAL REQUIREMENTS

The major research needs to be identified by the "Non-Hydrologic Aspects" Task Committee follow:

1) At this stage, a serious effort should be made to bring about the creation of an appropriate entity for coordinating and undertaking urban water resources research. Items of concern and definitions of what needs to be done that have been provided by the ASCE program should constitute a part of that research. This entity should probably provide a translating function for effective communication between affected fields, particularly for inter-disciplinary, interagency, inter-municipality, multi-disciplinary, and multiple-service communication.
2) More detailed information on costs and benefits of different scales of facilities is needed to arrive at equitable charge schedules and judicious selection among alternatives, particularly for water pollution abatement-water quality control.
3) Research is needed on the planning process with regard to meeting overall community goals; socio-system simulation should be incorporated in the research.

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[2] Note to readers: The proceedings of this conference are provided as Chapter 9 in this book.

4) The preferred institutional framework within which metropolitan water resources planning should take place remains to be identified.
5) With respect to environmental quality, if research is not now in progress on the efficacy and efficiency of mechanisms for measuring public attitudes and opinions, such as public hearings, it should be undertaken promptly; and similarly, on decision-making processes at all other levels.
6) Research is needed on institutions for management of water, at various levels, extending from an entire metropolis to the local neighborhood.
7) Serious study should be given to the effect in the past of technological change on social change, social change on technological change, and the relationship between them.
8) Ways must be found to measure or evaluate social efficiency compatible with those extant for economic efficiency.

16.0 URBAN WATER RESOURCES DEVELOPMENT

Considerable potential synergistic benefits could be obtained in comprehensive, multi-purpose development of urban water, on a scale of involvement that in the aggregate could be much greater than for river basins. Reliable planning of water quantity and quality exchanges between the several urban water service functions is hampered particularly by a poor understanding of the urban rainfall-runoff-quality process. The primitive status of urban hydrology is the consequence of an absence of a meaningful body of field data. Techniques needed directly and indirectly for simulation in water resources planning, development and management are largely latent for a lack of suitable data to test and develop them. The input for the water cycle is precipitation, and little is known about it in the urban context. As opposed to data needed for simulation model development, a form of historical storm data is needed as inputs for planning, development and management.

The following research should be pursued simultaneously:

1) Plans being formulated by the USGS for the acquisition of rainfall-runoff-quality data and its dissemination should be implemented as soon as possible, because model development is dependent on data availability and several years of data may be required for process mastery.
2) Existing mathematical models for simulating the rainfall-runoff-quality process should be under continual testing and improvement, starting with the small amount of data now available. Considerable detail and specification of needs are included in Appendix A, prepared by the "Methods of Analysis" Task Committee.
3) Research on metropolitan storms should be pursued not only to develop inputs suitable for future uses, but for contemporary management and operation of works.

17.0 URBAN DRAINAGE

Because urban flooding is of considerable economic importance, in addition to the research cited in the preceding section, which would have a very strong bearing on drainage, the following studies are recommended by the "Damage and Storage" Task Committee (Appendix C):

1) The amount and place of drainage facilities investment in the expenditures of local governments should be defined.
2) Needed policy revisions should be identified for removal of legal, political, jurisdictional, administrative, planning and other roadblocks to more efficient and effective drainage facilities development.
3) Much more knowledge on urban drainage flood damages is needed if marginal values are to be discerned, particularly on intangibles.
4) Consideration should be given to using future new towns as field laboratories by incorporating several different drainage schemes in the same town, utilizing different extents of storage and imaginative multi-purpose works.
5) The efficacy of selected existing drainage systems should be systematically evaluated and appraised, alternative hypothetical designs should be qualitatively tested and the most promising combinations and most effective practices should be identified.
6) In conjunction with the preceding study, analyses should also be made of the inherent storage capacity of the selected existing drainage systems and the effects of varying degrees of hypothetical storage capacity designed for each catchment.
7) Compared with requirements for process evaluation, relatively simple simulation models could be used for the two studies immediately above, and these should be adapted or developed with the full cognizance, but preferably assistance, of process model researchers.

In addition (from the "Non-Hydrologic Aspects" Task Committee, Appendix D), legal considerations need research to develop solutions for the lack of uniformity in existing state law, which is divided and often mingled among common, civil and "reasonable use" rules. The law can provide means for minimizing conflicts by: a uniform drainage code; a uniform trial procedure; and enabling legislation for the effective organizing, governing, administration and financing of drainage districts. Fundamentally, a model law or uniform code based on drainage and ramifications of drainage is needed. The greatest problem in this area of law is the question of liability, which hinges on identification of who is responsible in a given instance.

18.0 SYSTEMS ANALYSIS

The total water resources of urban areas have not been appraised on a comprehensive scale. Also, technical and economic systems analyses of the full spectrum of water occurrences and interactions should provide an overall reference for future research.

Their economic feasibility is not as evident, because of a lack of precedent for comparison.

A properly structured systems analysis could provide a vital overview role for a national plan of urban water resources research.

System simulation model design should be started immediately, continual cycling of refined versions of simulation and sensitivity analyses should next be undertaken, and an operational stage systems overview capability should be attained as soon as possible. A single approach may not be desirable. More than one level of development might be pursued concurrently, each for a particular type or scale of objective.

URBAN WATER RESOURCES
SECTION II

The nation is confronted with demands for the resolution of new social values and reversal of the trend in worsening environmental quality, and these are interrelated. The social and physical decay of central cities has been accompanied by suburban growth at the expense of adjacent rural areas. Increasing urban populations and technological activity and the associated spread of urbanization have exacerbated environmental pollution. The greatest concern in urban water will increasingly be on quality.

Over 90% of the nation's population is expected to be resident in urban areas by the end of this century,[1] occupying only a few percent of the total national geographic area. "In the next 35 years . . . we must rebuild the entire urban United States."[2] A forecast of the extent of urbanization in the 48 conterminous states a century from now is given in Figure 9-2. The key descriptor for the future is intensification, of people, of activity of all kinds, and of competition and conflict. In addition to its vital roles of supporting human life and providing environmental amenities, the manner in which urban water is managed can be expected to have an ever-greater influence on national economic growth in the decades ahead.

1.0 ASPECTS OF URBAN WATER MANAGEMENT

1) Urban water uses:
    a) Water supply (domestic, commercial, industrial, agricultural, and for fire protection);
    b) Conveyance of wastes (from buildings and industries);
    c) Dilution of combined and storm sewerage system effluents and treatment plant effluents (by receiving bodies of water);
    d) Water-oriented recreation (and fish management);
    e) Aesthetics (such as landscaped creeks and ponds in parks and parkways);
    f) Transportation (commercial and recreational); and
    g) Power generation.
2) Protection of urban areas from flooding:
    a) Removal of surface water at source (by conduit systems or canals);

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[1] *Waste Management and Control,* National Academy of Sciences, National Research Council, Publication 1400, GPO, Washington, DC, 1966.

[2] Shelton, M.J. December 1967. "An Engineer Plans a New City in Megalopolis." *Journal Urban Planning and Development Division,* ASCE, UP4, Proc. Paper 5665.

   - b) Conveyance of upstream surface water through the area (by conduit systems or canals);
   - c) Barricading banks, detaining or expressing flow of natural streams to mitigate spillover in occupied zones of floodplain (by levees, dikes, upstream storage, or canalization); and
   - d) Flood-proofing of structures.

3) Manipulation of urban water:
   - a) Groundwater recharge (using processed water or stored surplus surface water); and
   - b) Recycling of water (reuse of effluents for water supply, recreation, etc.).

FIGURE 9-2. "Anticipated Major Urbanized Areas in the United States in 100 Years"[3]

4) Pollution abatement in urban areas:
   - a) Conveyance of sanitary sewage and industrial wastes in separate sewerage systems;
   - b) Interception and treatment of sanitary sewage and industrial wastes (from separate sanitary sewerage systems or dry weather flow from combined sewerage systems);
   - c) Interception and treatment of storm sewer discharges or combined sewer overflows (by means of detention tanks);
   - d) Reinforcing waste assimilation capacity of receiving waterbodies (by forced aeration, low-flow augmentation or other means, to raise ambient dissolved oxygen content); and
   - e) Treatment of sanitary wastes at point of origin (household and building treatment plants).

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[3] Claire, W.H. September 1967. "Manual on Urban Planning, Chapter X: The Future." *Journal of the Urban Planning and Development Division.* ASCE, UP3, Proc. Paper 5475.

5) Interfacial Public Services:
   a) Snowstorm and rainstorm traffic routing;
   b) Street cleaning scheduling;
   c) Snow removal strategies;
   d) Lawn irrigation conservation; and
   e) Air pollution control.

One form of water pollution arises from overflows from combined sewers[4] and discharges from separate storm drains.[5] Evaluation of these sources has been retarded by the absence of suitable precise measuring devices and a rather primitive status of knowledge on urban storm drainage hydrology. Only a very few catchment areas have been gauged.[6]

19.0 INVESTMENT IN URBAN WATER RESOURCES DEVELOPMENT

Estimated replacement values of public water, wastewater, and drainage facilities are given in Table 9-1. The investment in public water systems, alone, "is greater than the combined total investment in iron, steel, food and kindred products, and is more than twice the investment in gas utilities and considerably more than the investment in railroads."[7]

Projected growths in population served by municipal water and wastewater facilities are given in Table 9-2, and projected construction costs for water, wastewater and drainage facilities are given in Table 9-3, both through 1980. The construction costs for water utilities in Table 9-3 do not include costs for multi-purpose source of supply development or transmission facilities, land requirements, maintenance and repairs, or for company owned industrial water and wastewater facilities.

At the request of the ASCE program, the Water Resources and Engineering Services Division (Mr. K.L. Kollar, Director), Business and Defense Services Administration, U.S. Department of Commerce, made a special study of local government expenditures for water and wastewater. Yearly expenditures from 1958 through 1966 were obtained by the Division on total capital outlay, the construction component of total capital outlay, current operation, and interest on debt. These figures do not include expenditures by private utilities or for source of supply development or transmission facilities, and their bases are reasonably comparable with those for the estimates in Table 9-3.

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[4] *Pollutional Effects of Stormwater and Overflows from Combined Sewer Systems—A Preliminary Appraisal.* 1964. U.S. Public Health Service Publication No. 1246, Washington, DC.

[5] Weibel, S.R., R.J. Anderson and R.L. Woodward. July 1964. "Urban Land Runoff as a Factor in Stream Pollution." *Journal of the Water Pollution Control Federation*, Volume 36, No. 7.

[6] Jens, S.W. and M.B. McPherson. 1964. "Hydrology of Urban Areas." *Handbook of Applied Hydrology.* V.T. Chow, Editor, McGraw-Hill, Inc., New York, New York.

[7] Gustafson, G.V. August 15, 1968. "Long-Range Planning for Distribution System Facilities." *Willing Water.* American Water Works Association.

From the data supplied, the annual expenditure on water distribution and water treatment facilities and services for operation, capital outlays over and above construction and debt interest were uniformly close to 1½ times expenditures for construction over 1958 through 1966; and for sanitary sewerage and sewage disposal facilities and services, uniformly almost half. Assuming these ratios will obtain in 1967-1980, and applying them to the construction cost estimates in Table 9-3, the average annual expenditures for operation, capital outlays over and above construction, and debt interest, will be: about an average of $3 billion per year for water distribution and water treatment facilities and services; and an average of about $1.3 billion per year for sanitary sewerage and sewage disposal facilities and services.

Comparable data on expenditures for other urban water services are not available, such as for storm drainage and water-oriented recreation. However, it is evident that average annual total expenditures by local governments on urban water resources will be on the order of $12 billion or more per year through 1980. While the sum total will be only a relatively small fraction of the Gross National Product, it is important to note that the bulk of these costs will be borne at the local level, via taxes, rentals and fees, and as part of private property costs.

TABLE 9-1

Replacement Values of Public Water, Wastewater, and Drainage Facilities

Public Water Systems, Including Treatment Facilities (at 1966 Construction Cost Levels)[1]	= \$49.1 billion
Public Sanitary Sewerage and Treatment Facilities (at 1966 Construction Cost Levels)[1]	= \$41.3 billion
Public Storm and Combined Sewers (at 1965 Construction Cost Levels)[2]	= \$22.0$^+$ billion

[1] "Regional Construction Requirements for Water and Wastewater Facilities, 1955-1967-1980," U.S. Department of Commerce, Business and Defense Services Administration, October 1967.

[2] "Urban Drainage, Practices, Procedures, and Needs," American Public Works Association, December 1966.

TABLE 9-2

Estimated Population of Conterminous U.S., in Millions[1]

	1955	1960	1965	1970	1975	1980
Total Population	164.3	179.5	191.7	206.2	223.4	243.0
Served by Municipal Water Facilities	120.5	135.0	149.8	166.8	185.4	206.0
Served by Municipal Wastewater Facilities	97.9	110.7	124.3	140.0	157.6	177.9

[1] "Regional Construction Requirements for Water and Wastewater Facilities, 1955-1967-1980," U.S. Department of Commerce, Business and Defense Services Administration, October 1967.

TABLE 9-3

Estimated Utilities Construction Costs for Water, Wastewater and Drainage Facilities

Service	Year for Cost Base	Period for Average	Deficiencies, Obsolescence, Depreciation	Annual Averages (Millions)	
				Growth	Total
Water Distribution[1]	1966	1967-1980	\$758	\$788	\$1,546
Water Treatment Plants[1]	1966	1967-1980	\$253	\$264	\$517
Subtotals			\$1,011	\$1,052	\$2,063
Sanitary Sewerage[1]	1966	1967-1980	\$919	\$726	\$1,645
Sewage Treatment Plants[1]	1966	1967-1980	\$576	\$454	\$1,030
Subtotals			\$1,495	\$1,180	\$2,675
Storm Sewers[2]	1965	1966-1975	\$1,300	\$1,200	\$2,500

[1] "Regional Construction Requirements for Water and Wastewater Facilities, 1955-1967-1980," U.S. Department of Commerce, Business and Defense Services Administration, October 1967.

[2] "Urban Drainage, Practices; Procedures, and Needs," American Public Works Association, December 1966.

A recent estimate of the construction cost for eliminating combined systems of sewerage by separating sanitary sewage and stormwater is \$48 billion.[8] Were universal separation adopted in the near future, the projected construction costs for deficient storm sewers in Table 9-3 would be reduced somewhat because in a number of cases both new sanitary sewers and new storm sewers would be needed to effect separation.

The American Public Works Association observes that "the annual national investment in urban drainage facilities exceeds that of the Federal Flood Control Program" . . and "it is estimated that the average annual losses (from urban drainage flooding) in all parts of the United States in recent years total \$1 billion or more."[9]

According to the 1968 *Report of the National Advisory Commission on Civil Disorders*, ..."education is the single biggest form of expenditure by local governments (including school districts), accounting for about 40% of their outlays." Although figures on the total water resources expenditures by local government are

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[8] *Problems of Combined Sewer Facilities and Overflows*. U.S. Department of the Interior, FWPCA, Water Pollution Control Research Series, WP-20-11, Washington, DC, December 1967.

[9] *Urban Drainage Practices, Procedures, and Needs*. APWA Research Foundation, Special Report No. 31, Chicago, IL, December 1966.

not presently available, the expenditures are considerable; and research to make more efficient use of budgets for water would surely ease the total fiscal burden of local governments.

20.0 URBAN WATER RESOURCES PLANNING AND RESEARCH

Only in recent years has national policy for water resources planning shifted from single-purpose to multiple-purpose, or comprehensive, development. The river basin is the basic geographic entity for planning and development. The specialized missions of the several federal agencies engaged in water resources activities are particularly well adapted to the support of development on a river basin scale. However, urban areas within a river basin are generally regarded as isolatable entities, with only modest attention given to internal urban water problems and their solution.

On the other hand, "urban communities probably have not been as alert as they might be to the potential value of the rivers that serve them."[10]

Each urban political subdivision evaluates and solves its own problems, within the constraints of local, regional, and river basin plans. The departmental structure of many cities and metropolitan areas provides for a division of responsibility, such as for water supply, sewerage, flood protection, and sewage collection and treatment, in separate departments, districts, or authorities. An organization with comprehensive responsibility for water resources for a city or metropolitan region is a rarity.

"There are now 38 federal agencies involved in some aspect of water resources activities. About 18 of these are active in water resources research."[11] Responsibility and interest in studying urban water problems is divided among several federal agencies, several trade and professional associations and other diversified groups. Because of specialized missions or limitations, no one group has the overall interest, capability or authority for reviewing the total problem of supply, distribution, use, collection, disposal, reuse, as well as costs, water rights, aesthetic considerations, etc., pertaining to urban water resources.

As a consequence of these and other contributing factors, it may be fairly stated that urban water resources planning and supporting research are at levels well below those for river basins and large regional complexes, despite substantial financial outlays for facilities at the local government level. Research is needed to improve technology for design, planning and operation, and "progressive management will utilize technology in seeking improvements in the institutional structure."[12]

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[10] Kelnhofer, G.J. December 1967. "River Basin Planning and Urban Development." *Journal of Urban Planning and Development Division.* ASCE, UP4, Proc. Paper 5619.

[11] Renne, R.R. July 1968. "Status of Water Research in the United States." *Journal American Water Works Association,* Volume 59, No. 11.

[12] Smith, R.L. November 1967. "Total Management of Water Resources." *Journal American Water Works Association,* Volume 59, No. 11.

"Ideally, research should be programmed in such a way that problems are anticipated, investigated and somewhat solved or even prevented before they are upon us."[13] Extensive construction of new and replacement urban drainage systems and facilities for reduction of pollution from storm discharges and combined sewer overflows over the remainder of this century will involve significant national expenditures. Existing knowledge is inadequate for efficient and optimal planning and development of such facilities. Better methods of analysis and more realistic design criteria are needed for engineering design of new drainage systems. "More research is needed to develop understanding of the whole stormwater pollution problem. This research should cover the hydrology, the hydraulics, the treatment, the effects on receiving waters, and related factors."[1]

"Urban runoff, now viewed as damaging from both quantity and quality viewpoints, may soon be viewed as a productive source of water for recreational development or even water supply."[12]

"In most urban areas there is concern with inundation by great floods and also by local storms. There is widespread concern with amplifying the opportunities for water-based recreation ready of access to urban residents, and with enhancing the conditions of urban living. A third focus of concern is improvement of systems for metropolitan area water supply and carriage of waterborne wastes. These concerns interact with management of the water resources of a river basin." . . . There is "special need for planning attention to this complex of water resources problems because wholly new means of dealing with them must be devised. This is so because metropolitan areas present conditions that are vastly different from the ones for which water resources management technology had been developed heretofore."[14]

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[13] *A Ten-Year Program of Federal Water Resources Research*, Federal Council for Science and Technology, Committee on Water Resources Research, Office of Science and Technology, GPO, Washington, DC. February 1966.

[14] Easton, E.D. May 16, 1968. "Water Resources Planning Strategy." New England Council of Water Center Directors, Water Resources Planning Conference, Boston, Massachusetts.

Chapter 9, Appendix A

CONSIDERATIONS FOR MODELING URBAN RAINFALL-RUNOFF-QUALITY PROCESSES

September 1968
By The Methods of Analysis
Task Committee, OWRR Project

1.0 PREFACE

The Methods of Analysis Task Committee conducted a state-of-the-art study of mathematical models and related simulation methods potentially usable for analyzing urban rainfall-runoff-quality processes. This Appendix is the Committee report.

The scope of this report was resolved in three Committee meetings:

November 6-7, 1967 at Lawrence, Kansas
March 28, 1968 at Stanford, California, and
September 4-5, 1968 at Cambridge, Massachusetts

Mr. David Dawdy attended the meetings on his own time; and the preparation of this part of the Committee report was supported by his agency, the Geological Survey, U.S. Department of the Interior. A portion of the cost of preparation of the parts by the other four Committee members was borne by the OWRR Project, on an equal share per member basis.

The objective of the research sponsored by OWRR is to provide guidelines for initiating and expanding a program of long-range studies in urban water problems. In the context of rainfall-runoff-quality processes, the Committee objectives were to assess prevailing capabilities for analysis, point out limitations and difficulties that might be encountered in analysis using these capabilities, and suggest what long-term research should be undertaken to fortify current knowledge for progressive development of firmer foundations. The work was somewhat arbitrarily divided, into those sectors requiring major attention, and the report, therefore, does not purport to cover all facets of the subject. For example, while surface runoff has been emphasized, it should be tacitly understood that the role of groundwater eventually should be linked with surface water, including quality considerations. At that time, recharge, interflow, etc., could be accommodated, but in the interim a compatibility should be sought between simulation techniques for groundwater analysis and requirements of surface water models, both for quantity and quality.

Economic questions are not addressed because a position was taken that results from hydrologic modeling should be a source of information for economic analyses. The ultimate goal is to approach as closely as possible a physical simulation of natural occurrences. Much less is known and, hence, much less is definable on quality than quantity. The Committee undertook an awesome and difficult assignment. The subject is of considerable importance in a national program of orderly urban water resources research, being a keystone engineering aspect.

Each chapter was independently written by individual Committee members. No attempt was made to impose a consensus viewpoint on any member, except where demanded by continuity and overall coverage requirements. The summary recommendations were reached in concert.

M.B. McPherson
Program Director

GENERAL OBSERVATIONS RELATING TO ANALYTICAL NEEDS IN URBAN HYDROLOGY

By Robert L. Smith
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1.0 INTRODUCTION

This report is limited to a discussion of potential methods for analyzing urban rainfall-runoff-quality processes in the urban environment. The subject of inquiry is most broad, but in another sense it is quite limited. For example, little consideration will be given herein to methods of evaluating the impact of urban growth on the processes of groundwater recharge.

Some indication of the need to focus inquiry on the rainfall-runoff-quality relationships may be ascertained from consideration of information recently developed by the American Public Works Association.[1] The APWA suggests the annual damage inflicted by urban runoff totals $1 billion or more, and it cites the situation in Chicago, where as of 1965, there were more than 4,000 miles of combined sewers and a comparatively few miles of separate sewers, as illustrative of the need to include quality considerations in any concerted corrective program.

The APWA estimates the current capital investment in storm sewers (replacement value as of mid-1965) approximates $22 billion. It also finds that current average unit construction costs vary widely, but have a weighted average of $1.55 million per square mile sewered. Maintenance costs in most large communities were found to range from $5,000 to $15,000 annually per square mile of sewered area. Finally, the APWA estimates current annual expenditures for storm sewers approximates $360 million and that an additional $240 million is spent for urban drainage associated with street and highway development. These figures are awesome, but there is indication they are inadequate to meet the need. The APWA appraisal of capital requirements for storm sewer facilities in new and expanding urban areas and to overcome existing deficiencies during the decade 1966-75 approximates $25 billion.

The figures cited reflect the educated guess of public works administrators, but they also reflect inherent acceptance of existing codes and design habits. What are the design criteria and design procedures on which these estimates are based? The APWA study provides but limited insight to this question. Table A-1 summarizes design criteria, in terms of rainfall frequency, based on APWA sampling by questionnaire of 597 communities.

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[1] Poertner, H.G., R.L. Anderson and K.W. Wolf. December 1966. *Urban Drainage Practices, Procedures and Needs.* Project 119 APWA Research Foundation.

TABLE A-1

Indicated Design Criteria Now Utilized in Urban Storm Drain Design (% of Communities)

Rainfall Frequency	Local Street Drainage	Sewer Systems	
		D.A. Less than 1-square-mile	D.A. Greater than 1-square-mile
1 Year	2.5		
2 Years	9.2		
5 Years	34.7		
10 Years	46.7		
Under 5 Years		11.7	
5-10 Years		47.6	
Over 10 Years		19.5	
Under 10 Years			33.6
50 Years			28.6
100 Years			4.0
No answer or don't know	6.9	21.2	33.8
TOTAL	100.0	100.0	100.0

As to design procedures, the APWA survey indicated 31.7% of the respondents utilized the "rational formula" for areas of less than 1-square-mile, 28.8% applied the formula to areas in excess of 1-square-mile, and 22.3% used it in both instances. In all probability, these results are low in all categories for there is strong reason to believe that many not claiming use of the formula were, in actuality, using charts based on rational formula concepts. In addition, a high percentage of the "no answers" probably use the rational method approach.

Assuming the "no answer" category in Table A-1 is distributed in proportion to the other replies, it may be concluded that the 10-year criteria is very popular. What is the basis for this selection? Does it properly relate hydrologic probability to damage? Inasmuch as rainfall frequency does not equate to runoff frequency for pervious areas, what flood criteria do the figures in Table A-1 actually represent, and how does this answer vary in response to the urban environment? Is it realistic to seek a design criterion that provides protection against events whose average probability of occurrence is less than 0.1 when the drainage area exceeds 1-square-mile? Table A-1 would suggest this is true. If so, is there reason to be concerned about the effect of urbanization on flood and drainage design? A paper by James[2] suggests that for events having average recurrence intervals in excess of 10 years, the effects of urbanization on peak discharges is negligible.

The foregoing are but a few of the questions that need resolution. One point is clear at the moment. Owing to the magnitude of development expenditures involved, a concentrated effort to provide new insight to the combined hydrologic and economic considerations is warranted.

2.0 THE CASE FOR MODELS

The historic lack of data on urban areas presents a serious obstacle to retrospective analysis. Individually, and collectively, members of the profession have long lamented the lack of adequate urban data programs. Two major constraints have limited program efforts to date. They are: a) the difficulty in instrumenting and servicing the data network (the time response function is most sensitive), and b) the inability to control significant environmental change in the catchment area. Paradoxically, where programs have been installed, the decision to initiate a study has often been motivated by the fact that significant change was taking place. In this light, the choice of a Sag City, Iowa as a potential research area in lieu of a rapidly growing Chicago suburb is foreign to thinking simply because it does not relate directly to the typical picture engendered by "urbanitis." Actually, long-term documentation of the difference in urban and non-urban response functions should probably be limited to areas for which significant change is no longer possible or is most improbable.

The instrumentation limitations can be overcome by adequate investment, but the problem of changing environment is, if anything, becoming more difficult. It seems quite probable that for many geographic locations, the only way to minimize this problem will be by utilization of relatively short-term, three-to-five-year records. The foregoing considerations, and the relative need for near-term refinement in our understanding of the processes, and in the resolution of questions concerning adequacy of design criteria, argue the need for extensive exploration of modeling techniques.

3.0 DESIRABLE CHARACTERISTICS OF MODELS

The model builder invariably seeks the best of all worlds and invariably settles for something short of this target. Basic considerations in appraising the adequacy of models can be classified under the categories of cost, utility, and reliability.

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[2] James, L.D. 1965. "Using a Digital Computer to Estimate the Effects of Urban Development on Flood Peaks." *Water Resources Research.* Volume 1, No. 2, pp. 223-234.

Model costs can be subdivided into developmental costs, input data costs, and operational costs. Of major concern to subsequent chapters in this report is that of input requirements which invariably reflect appraisal of hydrologic or other physical information. Obviously, the more extensive the input requirement, the more costly the model and, usually, the more reliable the product. The balance point calls for resolution of the utility and reliability considerations.

The primary consideration in evaluating the utility of urban models would appear to center on the model's ability to accommodate change in the physical environment. This need is twofold. First, the model to be useful for either planning or operational purposes must be able to evaluate the effects of change within a selected watershed. Second, the general utility of a given model is very much a function of its adaptability to transfer. That is, a general model applicable to a variety of climatic and physiographic considerations is preferable to a solution that requires development on an individual problem basis. The importance of developing a model having a high degree of transferability cannot be stressed too strongly in light of the magnitude of the overall challenge outlined earlier.

Model reliability is, of course, measured by its ability to provide an acceptable solution under a variety of input considerations. The hydrologic processes and the boundary geometry of urban watersheds are sufficiently complex that absolute accuracy must remain an objective, not an accomplishment. One question must always be asked and answered—is the model sufficiently reliable to justify its use in resolution of the engineering investment and operational decisions it is intended to serve? Some models found to be adequate for general planning, or even design purposes, may be inadequate when utilized in an operational capacity.

It is often assumed that the high damage potential and the fast time response of the small basins normally encountered in urban watersheds will require a relatively detailed model to achieve reasonable reliability and utility. Actually, little is known concerning the relative degree of detail required. The degree of detail required should be responsive to the needs to be served. In this regard, definitive exploration of the distribution of the urban damage function with size of tributary area would be most useful in determining model needs, in identifying research priorities and in specifying data requirements.

4.0 TYPES OF MODELS

Urban hydrologic problems can be studied by three different approaches. These include: 1) direct field measurements, 2) scale and analogy models, and 3) analysis and synthesis utilizing electronic computers, both digital and analog.

Field measurements are indispensable in evaluating the results obtained from both scale models and analytical models. However, field studies are both expensive and time consuming, and, as indicated earlier, only limited reliance on field data will be possible for the problems of current concern.

Scale models can be used effectively to study quantitatively a component of the hydrologic and hydraulic system. Their input requirements are often restrictive and their transfer utility may be limited. Moreover, the modeling techniques required have not yet been developed for the entire urban drainage system.

The literature clearly indicates the recent trend in hydrologic research has been toward analysis and synthesis utilizing computers. These efforts, therefore, are emphasized herein. Two basic analytical approaches are involved. The first involves application of the equations of flow to one or more processes in the hydrologic cycle. The second can best be described as systems simulation.

Formulation of the first approach is based on the general equations derived according to the physical laws governing the motion of water, namely, the continuity equation and the equation of motion. The boundary and other initial conditions are based on an appraisal of physical factors. The non-linearity of the governing partial differential equations and the complexities of the boundary conditions encountered make this approach most difficult in many instances. Simplifying assumptions are usually imposed as regards both boundary complexity and mathematical complexity. Solutions are often based on one-dimensional analysis of unsteady flow and involve either differential or finite difference procedures. This approach has been used most successfully in dealing with a component of the hydrologic system where relatively simple boundary conditions can be used. Application of this approach in the study of overland flow and channel routing will be discussed extensively. A major difficulty in seeking total system analysis via this approach lies in establishing accurate linkages between different components of the system. Data requirements for this approach are relatively severe. On the other hand, verified solutions can be considered relatively exact.

System simulation techniques fall into two principal categories: deterministic methods and stochastic methods. The former attempt to develop relationships among the physical parameters and processes involved in the hydrologic cycle based on recorded data. These relationships are then used to generate or to predict non-recorded hydrologic sequences. The principal input function is rainfall and the principal output function is runoff. In the case of quality models, the input requires magnitude and location of system loads and the output is the time distribution of constituent concentration.

Deterministic approaches can be subdivided into parametric system synthesis, transformation methods, and correlation analysis.

Parametric System Synthesis—The basis of synthesis of the hydrologic system is the law of conservation of mass. Thus, during a given time interval, the amount of water entering the system is equal to the amount stored in the system plus that leaving the system. The conceptual structure of the model is based on the knowledge, or presumed knowledge, of the physical processes involved in the cycle. These processes start with the input precipitation and include interception, depression storage, infiltration, percolation, interflow, groundwater recharge and discharge, runoff distribution and evapotranspiration. Mathematical equations are used to represent the component processes and their linkages. Equation constants and basin physical parameters are adjusted so that the output of the model simulates the recorded output. Continuous accounting of moisture transfer is handled by the computer program.

The approach has much to recommend it. Tests have shown that but a few years of data are needed to resolve program constants to where good reproduction of a much longer record can be attained. Presumably, widespread testing on a number of watersheds with different physical and cultural characteristics would provide a basis for parameter selection and, thus, enhance model applicability or transferability. Data requirements are believed to be relatively minimal. Examples of the models involved are those reported by Crawford and Linsley,[3] Dawdy and O'Donnel,[4] and Viessman.[5] The approach is not without its difficulties. As a practical necessity, many of the processes must be represented by linear and lumped functions when in actuality they represent non-linear and distributed functions. Accordingly, successful application of this approach to a selected record period does not guarantee model reliability for other operating conditions.[6] This latter criticism is especially pertinent if the model is not tested on a wide variety of watersheds.

The foregoing discussion assumes the use of a digital computer. It should be noted that a similar procedure can be followed on an analog computer.[7] It would appear, however, that the analog is more conducive to treatment of a specific basin. Whereas major cities may have reasons to "hardware" their individual systems, the digital approach would appear to offer more flexibility for broad professional inquiry.

Transformation Methods—Transformation methods are used to determine a mathematical expression of the system that can reproduce closely the recorded output when it operates on the recorded input. The method is not concerned with the physical processes of the system. For linear models in which the total output of the system is equal to the sum of the output of each component input, the mathematical inversion of the convolution integral is involved. This can be accomplished by using Laplace Transform,[8] Fourier Series,[9] Laguerre Coefficients,[10] and other mathematical

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[3] Crawford, N.H. and P.K. Linsley. July 1966. *Digital Simulation in Hydrology: Stanford Watershed Model IV.* Department of Civil Engineering, Stanford University, Technical Report No. 39.

[4] Dawdy, D.R. and T. O'Donnell. 1965. "Mathematical Models of Catchment Behavior." *Proceedings ASCE, Journal of Hydraulics Division.* Volume 91, HY4, pp. 123-137.

[5] Viessman, W., Jr. 1968. "Runoff Estimation for Very Small Drainage Area." *Water Resources Research.* Volume 4, No. 1. p.87.

[6] Amorocho, J., and W.E. Hart. June 1964. "A Critique of Current Methods in Hydrology Systems Investigation. *Transactions, AGU.* Volume 45, No. 2, pp. 307-321.

[7] Harder, J.A., L. Mockros, and R. Nishizaki. 1960. *Flood Control Analogs.* University of California, Berkeley, Water Resources Center, Contribution 24.

[8] Paynter, H.M. 1952. "Methods and Results from M.I.T. Studies in Unsteady Flow." *Journal of B.S.C.E.* Volume 39, No. 2, pp. 120-165.

[9] O'Donnell, T. 1960. "Instantaneous Unit Hydrograph Derivation by Harmonic Analysis." *Commission of Surface Water Publication.* No. 51, IASH, PP. 546-557.

techniques. The approach is not readily adaptable to the urban problem where emphasis is on change and understanding of the effects of physical change on the several hydrologic components.

Correlation Analysis—In this analysis, availability of recorded input and output functions is required. The input function is operated by a linear normal model of the system whose physical characteristics are represented in the model by a set of chosen parameters based on the knowledge of the watershed. Regression analysis and analysis of variance are then used. The former determines a series of regression coefficients for the model and the latter the error estimates and measures of correlation. These are then used to select the best model. The method of least squares[11] and time series analysis[12] are also used in some instances. Generally speaking, this system requires more input and recorded output than the parametric system described earlier, and provides less insight to sub-system processes.

Stochastic Methods—In contrast to the foregoing deterministic approaches, stochastic processes use statistical measures of hydrologic variables to generate non-recorded sequences to which evaluation of probability levels are attached. Sufficient long-term records of the variables are required to give a true representation of their statistical nature. Often, the stochastic approach is limited to analysis of the output variable with little insight provided to the cause and effect relation within the system. In many instances, the needed long-term records are not available. Sometimes a combined use of both deterministic and stochastic methods is possible. A deterministic method can be used to generate non-recorded variables in an otherwise long-term record. Stochastic techniques are then applied to the output variable to determine the level of probability of the various sequences of the output variable. Conversely, stochastic methods can be applied to a recorded input variable to define the latter's probability distribution. The deterministic method may then be applied to find the probability distribution of the output variable. Mathematical techniques used in the stochastic methods are Monte-Carlo analysis for a system with no memory,[13] Markov Chain analysis for a system with memory,[14] and functional analysis.[15]

5.0 CONCLUSIONS

The following summary observations are warranted:

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[10] Dooge, J.C.I. 1965. "Analysis of Linear Systems by Means of Laguerre Functions." *Journal of SIAM Control.* Series A, Volume 2, No. 3.

[11] Snyder, W.M. 1962. "Some Possibilities for Multivariate Analysis in Hydrologic Studies." *Journal Geophysical Resources.* Volume 67, No. 2, pp. 721-729.

[12] Kisiel, C.C. 1968. "Time Series Analysis in Hydrology." *Advances in Hydroscience.* (Academic Press).

[13] Dyck, S. and M. Schramm. September 6-8, 1967. "Application of Monte-Carlo-Method to Reservoir Design." *Proceedings, The International Hydrology Symposium.* Fort Collins, Colorado, Volume 1, pp. 406-413.

[14] Beard, L.R. September 6-8, 1967. "Simulation of Daily Stream-flow." *Proceedings, The International Hydrology Symposium.* Fort Collins, Colorado, Volume 1, pp. 624-632.

[15] Chiu, C.L. September 6-8, 1967. "Analysis of 'Black Box' Relationship Between Rainfall and Runoff." *Proceedings, The International Hydrology Symposium.* Fort Collins, Colorado, Volume 1, pp. 608-615.

1) Current assessments of urban drainage construction needs and current estimates of urban water damage indicate both need and justification for a significant exploration of urban rainfall-runoff-quality relationships.
2) The absence of extensive historical data on urban rainfall-runoff-quality relationships indicates a need to concentrate efforts on the development of mathematical modeling techniques that can be used to extend and interpret the significant implications of short-term records.
3) Definitive exploration of the urban storm damage function would provide appreciable guidance in determining the analytical detail and data needs necessary to provide an adequate model.
4) Emphasis should be placed on the development of models that are responsive to change in the physical environment, i.e., models possessing a high degree of transferability.
5) In all probability, all of the techniques will prove useful as the profession pursues efforts to improve its knowledge of urban hydrology. Initial emphasis on deterministic approaches to better understand cause and effect relations appears most desirable. In this regard, exploration of the extent to which rigorous analytical solutions can and should be applied must be encouraged.

6.0 ACKNOWLEDGEMENTS

The writer is indebted to Dr. Y.S. Yu, Professor of Civil Engineering, University of Kansas for the preparation of a digest of the appropriate literature. Errors of omission or fact incurred in reduction of the material to the general observations set forth above rest entirely with the author.

APPLICATION OF MODELS IN URBAN HYDROLOGY

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1.0 INTRODUCTION

Earlier in this report the necessity for intensive inquiry into problems of urban hydrology was outlined. The outline emphasized the need to utilize modeling techniques in the search for better understanding of the rainfall-runoff-quality relationships encountered in the urban environment. The arguments presented do not reflect any desire, on the part of the task committee, to insist on over sophistication of what has long been treated as a routine design problem. Nor do they reflect any great desire to engage in these pursuits solely to satisfy intellectual curiosity. They do reflect the need for urgent professional response to the problem described in the APWA report,[16] and for realistic appraisal of the futility of relying solely on the accumulation of long-term records of field investigations.

The Committee believes that a concerted effort to improve modeling techniques will, over the span of three to five years, provide an improved basis for making many of the design and programming decisions that confront the profession. It also believes it most important that this effort have the active attention and participation of all segments of the profession—the design engineer, the public works administrator, the data collector—not just the researcher. Consideration of several possible model uses will serve to illustrate the need for broad professional participation.

2.0 A GUIDE TO RESEARCH AND DATA PROGRAMS

There is a definite need to relate model development requirements to the programming of associated basic data and research activities. The overall objective, of course, is to provide an analytical tool that will meet the needs of the profession. Yet, these needs are largely undefined at this time as they relate to either acceptable criteria for measuring model adequacy, or to the formulation of a supporting basic data program. In short, little is known concerning the relative ease of modeling the desired relationships.

There is no need to make a model more complex than is necessary to perform its intended function at the desired level of accuracy, but care must be taken to assure the desired goals are achieved. To illustrate, the Stanford Watershed Model as initially developed by Crawford and Linsley[17] utilized basic methods in calculating the time distribution of the hydrograph at the basin outlet. The latest version of the model provides for inclusion of the equations of motion in calculating the overland flow to

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[16] Poertner, H.G., R.L. Anderson and K.W. Wolf. December 1966. *Urban Drainage Practices, Procedures and Needs*. Project 119 APWA Research Foundation.

[17] Crawford, N.H. and R.K. Linsley. July 1966. *Digital Simulation in Hydrology: Stanford Watershed Model IV*. Department of Civil Engineering, Stanford University, Technical Report No. 39.

the channel system, and a reach-by-reach routing of the flow in the channel. Are these added refinements necessary in any situation, all situations, or selected situations?

It may be argued that the high percentage of impervious surface in urban areas simplifies model treatment of the loss function. To what extent is this apparent advantage offset by such environmental factors as: a) lawn watering, b) artificial detention imposed by varied landscaping and architectural practices, and c) non-uniform enforcement of construction codes relating to the installation of individual drainage systems?

An obvious concern relates to the sensitivity of the time distribution of the runoff hydrograph, for various size catchments, to areal and time variations in the input rainfall. What density of instrumentation is needed to clarify this matter? A recent theoretical discussion of the problem by Eagleson[18] provides some guidelines, but should be tested.

The foregoing and similar questions need to be answered. Model simulation presents a mechanism for studying the relative sensitivity of the desired output function to variations in selected input data whether the latter be climatic or physical. This use of modeling techniques provides opportunity to examine alternative analytical treatments with a minimum of field expense. Thus, programming of data programs should be responsive to model development needs.

The foregoing suggests an initial need to concentrate on the development and testing of deterministic models. Comparative testing of existing models using the available historic data should be sought and investigators should be urged to report negative as well as positive results. In order to test the transfer capability of the models, a program designed to acquire three to five years of data on a variety of watersheds, sufficient in number to provide an appropriate range of input variables, should be implemented. Most existing models, owing to limitations in available data, utilize limited climatic input; usually this consists of low-density precipitation coverage plus evaporation (or temperature). Similarly, watershed parameters are usually limited to readily identifiable factors such as slope, surface characteristics, and channel conveyance characteristics. Measured output is normally limited to surface flow. Even if initial instrumentation is limited to the minimum information listed above, these new watersheds would serve a very useful purpose. Primarily, they would provide a basis for determining the relative effect on model constants of selected variations in the input parameters, and for measuring the general transfer capability of different models.

The foregoing program will not, however, resolve all questions concerning the relative adequacy or internal consistency of the model. Several watersheds should be of true research quality and include high-density (both space and time) precipitation

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[18] Eagleson, P.S. 1967. "Optimum Density of Rainfall Networks." *Water Resources Research.* Volume 3 No. 4, pp. 1021-1033.

coverage, multiple station flow measurement, necessary quality instrumentation, etc. Analysis of short-term records from these latter basins can then be used as a guide in resolving questions concerning desired refinements in modeling techniques and in programming additional data needs for other watersheds.

Model developers should be encouraged to consider techniques that will be responsive to more than one output function. As noted earlier, development of a model that relates rainfall input to runoff output for a selected period of record on a given watershed does not assure an adequate analytical tool. Subsequent widespread testing of the model on other watersheds, which provides a basis for relating program constants to physical parameters, provides an improved level of confidence. Integration of a quality response function with existing rainfall-runoff models would further enhance model qualifications. Model formulation that provided a basis for the testing of additional output functions, e.g., soil moisture or groundwater levels, would obviously be welcome. Again, basic data programs should be coordinated with, and responsive to, model development needs. This, of course, is the primary argument of this discussion for reliance on modeling techniques, as advocated herein, can only prove successful if there is close coordination between model builders and basic data organizations.

3.0 EVALUATING HYDROLOGIC RISKS

Urban storm drainage design should not be dependent on an arbitrary specification of a rainfall frequency criterion. Instead, it should represent a reasonable optimization of the benefits received for the costs incurred. Methods of assessing urban damage are currently being studied by another task committee and are not of direct concern to this statement. The hydrologic phase of the problem, however, requires adequate definition of the probability of occurrence of floods of varying magnitude. The current absence of extensive long-term flow records for a representative number of urban watersheds makes present evaluation of this problem a most formidable task.

To shortcut the time consuming process of allowing history to resolve this dilemma necessitates consideration of the model approach. Fortunately, long-term climatic records, especially precipitation and temperature, are available in at least limited quantity. A deterministic model that could accept these climatic inputs together with definitive watershed parameters, and then generate with reasonable accuracy the accompanying runoff response for the watershed would do much to clarify the engineering investment decision. Further appraisal of the probability question utilizing stochastic treatment of the generated streamflow characteristics might prove appropriate.

The same general argument can be advanced in the appraisal of the water yield capability of small streams in and near the urban environment. In either case, the importance of making such information available to the design engineer and the public works administrator cannot be minimized.

4.0 ANALYSIS OF THE ADEQUACY OF EXISTING SYSTEMS

Perfection of adequate modeling techniques would be of great assistance to public works administrators in appraising the adequacy of their existing storm drainage systems. Impairment in service that was tolerable several years ago may no longer be acceptable with changing traffic patterns and property values. Increasing urbanization of catchment areas has undoubtedly led to increased occurrences of flood conditions, at least in the lower end of the flood spectrum. The result may be inadequate facilities throughout the system or perhaps inadequacy in only selected portions. Which problem areas deserve priority attention? In the absence of thorough analysis, these trouble spots evolve only with the passage of time and the occurrence of as yet unrealized flood conditions, and at great cost and public inconvenience. Again, the availability of an adequate deterministic model would allow appraisal of the situation and identification of needed remedial works or other disaster control practices, before the event—not after. The savings to city operation and maintenance needs could be most significant.

On the quality side, much concern is being expressed over the possible need to separate existing combined storm and sanitary facilities. In some cases the costs will be astronomical. How serious is the problem? What is the extent and frequency of occurrence of undesirable overflows? What construction and/or operational alternatives are available and to what degree will they minimize, eliminate, or aggravate the pollution problem? Again, a deterministic model which could quickly examine the options over the full range of potential climatic conditions would be of great help in arriving at meaningful decisions. The pioneering work at Minneapolis-St. Paul[19] under the auspices of a Federal Water Pollution Control Demonstration Grant is indicative of the nature of future concerns in this area of inquiry.

5.0 APPRAISAL OF ALTERNATIVE DESIGN ASPECTS

The foregoing comments concerning appraisal of alternative risk criteria and alternative corrective measures imply the utilization of alternative design concepts. The latter phrase, however, demands a still broader interpretation. Heretofore, urban stormwater management has been largely single purpose in its orientation. There is increasing desire and need to dovetail urban drainage solutions with other water management activities ranging from water supply to recreation. Similarly, in planning the extension of existing systems or the development of new systems in areas not now served, but which have obvious growth potential, there is real need to have a basis for analyzing alternative ways of staging construction in response to growth.

In these instances, as in the proceeding section, an adequate deterministic model would prove most useful to both the metropolitan planner and the public works administrator.

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[19] Anderson, J.J. 1967. "Dispatching and Routing of Combined Sewer Storm Flows for Maximum Interceptor Utilization." Paper presented at ASCE annual meeting, New York, New York.

6.0 IMPROVEMENT IN DESIGN METHODOLOGIES

The continued and widespread use of the "rational formula" has led to increasing controversy with each new advancement in applied hydrology. The stakes involved have now become so large that continued delay of authoritative appraisal of the problem can no longer be tolerated. The theoretical shortcomings in this formula are well known,[20] and the inequality of precipitation and runoff frequency for small rural (pervious) areas has been demonstrated.[21] Past attempts to appraise the potential errors involved have been hampered by the lack of adequate data.

Perseverance of the "rational formula" can be attributed solely to its simplicity. A concerted analytical effort in the field of urban hydrology should serve to clarify the several issues involved. Quite likely some modifications in design concepts will be forthcoming, but it does not follow that a modified approach must sacrifice the practicing profession's desire for procedures easy of application. To illustrate, Chow,[22] in a study of small rural watersheds, urged that the rational formula be rephrased as:

$$Q = XZA$$

where X = intensity of rainfall excess in inches/hour

Z = a peak reduction factor due to basin storage effect

A = catchment area in acres

Chow's proposal, which is here taken out of context, is cited solely to illustrate a concept. The point to be made is that pursuance of the modeling approach advocated herein might, in addition to providing a useful tool for major system analysis, serve as the catalyst for improving current techniques on relatively routine problems. It is conceivable, for example, that utilization of an adequate deterministic model in systematic repetitive analysis for a variety of conditions would lead to the development of reasonable design relations for X and Z in terms of identifiable climatic and watershed parameters.

7.0 CONCLUSIONS

No attempt has been made to enumerate the many and varied ways in which hydrologic modeling techniques can and should be applied to the study of urban

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[20] Linsley, R.K., M.A. Kohler, and J.L.H. Paulhus. 1958. *Hydrology for Engineering.* McGraw-Hill, New York, New York.

[21] Smith, R.L., G.E. Clausen, and J.A. Henderson. 1966. "Hydrologic Design for Small Watersheds." *Study 6F Runoff From Bottom Land and Hillside Terrain.* Department of the Army, Civil Works Investigations, Project ES-180.

[22] Chow, V.T. 1962. *Hydrologic Determination of Waterway Areas for the Design of Drainage Structures in Small Drainage Basins.* University of Illinois, Engineering Exp. Sta., Bulletin #462.

problems. It is believed, however, that the concepts advanced present sufficient cause to concentrate on modeling techniques for future advancements in urban hydrology.

Near-term advancements in design methodology should be possible via extensive comparative testing of existing models. Investigators should be urged to report negative as well as positive results.

Efforts should be initiated now to acquire three to five years of data on a variety of watersheds sufficient in number to allow adequate analysis of model response for a broad range of climatic and physical parameters. Several of the watersheds should be the recipient of very detailed instrumentation programs to provide the database necessary for comparative study of model detail.

There must be careful coordination between the model builders and the data collection agencies. The end product will be greatly enhanced if professional involvement and communication extends beyond the researcher and includes public works administrators, municipal planners and design engineers.

MODELING OF WATER QUALITY INPUTS FROM URBANIZED AREAS

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1.0 INTRODUCTION

In spite of an increasing awareness of the multiple nature and scale of water resources problems, little serious consideration has been accorded the water quality aspects of urban runoff. Until about 1950, engineers were more concerned with developing procedures for dealing with, and methods for estimating, the exceedingly large quantities of runoff often produced during intense storms on urbanized areas. As a result of the neglect of water quality considerations, only a few efforts have been made to identify the water quality constituents of urban runoff. No positive effort directly related to water quality models for such flows is evident.

This paper summarizes the kinds of data which have been obtained, the outlook for future data collection, the types of water quality models which have been used on large watercourses and which could be adapted to urban drainage areas, the manner in which the urban drainage system relates to the larger regional drainage complex, the kinds of quality components which would likely need to be considered in a generalized urban runoff water quality model, and a general orientation for developing useful water quality models for urban drainage systems.

2.0 CLASSIFICATION OF WATER QUALITY INPUTS

A prerequisite for the development of adequate water quality models is reliable quantitative data on both the amount and quality of runoff from the types of areas to be modeled. Because the quality of surface flows can be expected to vary considerably from area to area, a well-conceived gauging program will be required. Differences related to land development type, climate, income level of residents, season, antecedent moisture conditions, and other factors will have to be considered in designing effective data collection programs. The nature and frequency of practices such as lawn watering, street cleaning and garbage and rubbish collection should also be considered. Land development can be expected to vary from rather uniform residential tracts to heterogeneous commercial and industrial districts or combinations of any or all of these. Such differences undoubtedly have a pronounced non-uniform impact on the quality of urban runoff.

The concern here is with the need for developing methods for predicting the quality of surface wash. There is no attempt to characterize the specific quality constituents of sewage or industrial wastes except to the extent that they find their way into storm drains through improper or illegal connections. The case of the combined sewer requires, of course, that the surface wash component and the sewage component both be evaluated when treatment is considered.

To seek out a pollutant, one must begin by defining the term pollution. Fair, Geyer, and Okun[23] have stated that pollution occurs whenever a substance or substances are introduced into a body of water such that its usefulness is impaired or it becomes offensive to any of the major senses. Such an encompassing definition permits the classification of almost anything as a pollutant providing it creates an impairment which is obvious or which can be determined by known analytical procedures. Having this gross guideline, a second need is that of identifying those locations where pollutants can be introduced. This requires an understanding of the component parts of an urban drainage system. Although some traits of such a system may be found to vary in character with geographic location, extent of urbanization and economic base of the community, the major elements normally include stormwater inlets, drainage pipes or closed conduits, and open channels of a variety of sizes and geometry. In general, pollutants are introduced into the system through inlets or *open* channel sections. Some infiltration at joint openings may also occur. If the nature of this input is objectionable, joints or breaks in the pipe may also be considered points of entry. The scale of the entry locations has a direct bearing on the nature of the pollutant which can be introduced.

Keeping in mind the foregoing definition of pollution and a knowledge of the locations at which pollutants are most likely to be introduced, the task of classifying the major types of pollutants becomes less difficult.

The grossest pollutants include supermarket carts, abandoned toys, concrete blocks and bricks, logs and sticks and a variety of other refuse. Usually such items, because of their size, can be easily introduced only at an open channel site. Unfortunately, many large items are capable of being transported along channels during major storms. For the most part, such objects do not seriously degrade water quality but they are often unsightly and may plug downstream drainage pipes or create objectionable pools through damming action. The introduction of such materials can best be controlled by properly enforced ordinances and policing of open channel segments. Quantities of tree limbs and other objects finding their way by natural means into the drainage system can be kept minimal through programs of periodic checking and removal.

Stormwater inlets, although offering relatively small openings, often become depositories for lawn clippings, excess newspapers, small sticks and rubble, and other smaller items of refuse. These materials: 1) often tend to fill stormwater inlets causing partial or total bypass of flow; 2) create stagnant pools within the inlet that become ideal breeding places for mosquitoes; 3) if organic in nature, may decompose in the inlet creating a nuisance condition and/or possible health hazard; 4) may become lodged in downstream points in the drainage system thus retarding or stopping flow; and 5) if they become lodged in the system, may decompose over a period of time, thus increasing the pollutional component of the drainage through the

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[23] Fair, G.M., J.C. Geyer, and D.A. Okun. 1966. "Water and Wastewater Engineering—Volume 1." John Wiley and Sons, Inc., New York, New York.

system. Again, it would seem that the effective use of ordinances and periodic policing would be the most fruitful way to combat this form of pollution.

The kinds of materials discussed to this point are by and large introduced into the drainage system by deliberate acts of man, either maliciously or through ignorance of the consequences of such acts. As a result, it would appear that such pollutants could be largely eliminated through the development and enforcement of proper ordinances and education of the public as to the nature of the problems created by thoughtless injection of these materials into storm drainage works. It would seem that gross pollutants can and should be eliminated from urban flows and therefore do not require consideration in urban water quality modeling. On the other hand, useful research could be directed toward making drainage systems more immune to the introduction of such pollutants and toward developing effective procedures for educating the public in this respect.

The remainder of the paper will be directed toward identifying some forms of urban pollution which are not as obvious or easily controlled as the aforementioned ones and discussing various approaches to water quality modeling for urban areas.

Water quality inputs can be classified broadly as conservative or non-conservative in nature, somewhat more narrowly categorized as organic, inorganic, radiological, thermal, and biological, and finally further subdivided into specific forms such as BOD, nitrogen, phosphorous, etc. A useful insight into the classification of pollutants may be had by considering the following excerpt from the State of Maine Statutes on Water Quality Standards.[24] These standards are typical of those developed for most states and, thus, adequately serve the illustrative purpose intended here.

> *Section 363. Standards of Classification of Fresh Waters. The commission shall have 4 standards for the classification of fresh surface waters.*
>
> *Class A shall be the highest classification and shall be of such quality that it can be used for recreational purposes, including bathing, and for public water supplies after disinfection. The dissolved oxygen content of such waters shall not be less than 75% saturation or as naturally occurs, and contain not more than 100 coliform bacteria per 100 milliliters.*
>
> *These waters shall be free from sludge deposits, solid refuse and floating solids such as oils, grease, or scum. There shall be no disposal of any matter or substance in these waters which would impart color, taste, or odor other than that which naturally occurs in said waters, nor shall such matter of substance alter the temperature or hydrogen-ion concentration of these waters or contain chemical constituents which would be harmful or offensive to humans or which would be harmful to animal or aquatic life. No*

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[24] "Water and Air Environmental Improvement Commission Revised Statues of 1964." October 1967. File 38 Ch. 3, Augusta, Maine.

> *radioactive matter or substance shall be permitted in these waters other than that occurring from natural phenomena.*

Since the control of water quality must relate to imposed water standards, the foregoing review can provide guidance in identifying the kinds of substances which should be analyzed for in samples of runoff from urban areas. Among the most important pollutants which might be found in urban surface wash are: fecal organisms (domestic animals, rodents, and wildlife); silt; pesticides; fertilizers; oil and gasoline; hydrocarbons and tars from road surfaces; road salt (primarily cold regions); leaves; and various solid wastes (paper, rags, etc.).

A limited amount of research has been undertaken to identify the water quality constituents of urban runoff. The results[25,26,27,28,29,30,31,32,33] are partially summarized in the following paragraphs. Analyses of water samples of urban runoff have been reported in Oxney, England 1954; Moscow, U.S.S.R. 1936; Leningrad, U.S.S.R. 1948-1950; Stockholm, Sweden 1945-1948; Pretoria, South Africa 1961; Detroit, Michigan 1949; Seattle, Washington 1959-1960; and Cincinnati, Ohio 1967 to the present. Most of these studies included the determination of concentrations of constituents from isolated samples without attempting to relate the pollutional load to the rate of runoff from the study areas. Other more complete results have been reported at Cincinnati, for example, where hydrographs of runoff were compared to the concentrations at corresponding times for the constituent evaluated. The relatively few investigators concerned with urban water quality appear to be unanimous in their agreement that adequate hydrologic data needed for the determination of pollutional loads is absent.

A summary of constituent concentrations found in the runoff from a 27-acre (residential and light commercial) area in Cincinnati, Ohio by the FWPCA, July 1962

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[25] Weibel, S.R., R.J. Anderson, and R.L. Woodward. July 1964. "Urban Land Runoff as a Factor in Stream Pollution." *Journal Water Pollution Control Federation.* Volume 36, No. 7.

[26] Weibel, S.R., R.B. Weidner, A.G. Christianson, and R.J. Anderson. 1966. "Characterization, Treatment, and Disposal of Urban Stormwater." *Proceedings Third International Conference on Water Pollution Research, WPCF.* Washington, DC.

[27] Dawdy, D.R. November 1967. "Knowledge of Sedimentation in Urban Environments." *Proceedings of the American Society of Engineers.* Hydraulics Division, No. Hy6.

[28] Weibel, S.R., R.B. Weidner, J.M. Cohen and A.G. Christianson. August 1966. "Pesticides and other Contaminants in Rainfall and Runoff." *Journal American Water Works Association.*

[29] Wolman, M.G. and A.P. Schick. Second Quarter 1967. "Effects of Construction on Fluvial Sediment, Urban and Suburban Areas of Maryland." *Water Resources Research,* Volume 3, No. 2.

[30] Evans, F.L., and others. May 1968. "Treatment of Urban Stormwater Runoff." *Journal of Water Pollution Control Federation,* Volume 40, No. 5, Part 2.

[31] Calvert, J.T. 1966. Discussion of "Characterization, Treatment, and Disposal of Urban Stormwater." Weibel and others. *Proceedings of Third International Conference on Water Pollution Research, WPCF.* Washington, DC.

[32] Horler, A. 1966. Discussion of "Characterization, Treatment, and Disposal of Urban Stormwater." Weibel and others. *Proceedings of Third International Conference on Water Pollution Research, WPCF.* Washington, DC.

[33] Gomella, C. 1966. Discussion of "Characterization, Treatment, and Disposal of Urban Stormwater." Weibel and others. *Proceedings of Third International Conference on Water Pollution Research, WPCF.* Washington, DC.

to July 1964, is given in Table A-2 to point out the degree of pollution that might be equaled or exceeded in urban surface wash. The authors classify this area as a "relatively clean type of urban land use" and therefore infer that untreated runoff from many urban areas is likely to be unacceptable to receiving water environments.

TABLE A-2

Constituent Concentrations Found in the Runoff From an Urban Area in Cincinnati, Ohio, July 1962 Through July 1964*

Constituent	Range in Values	Mean Storm Value
Turbidity (J.u.)	30-1,000	176
Color (C.u.)	10-460	87
pH	5.3-8.7	7.5
Alkalinity (mg/L)	10-210	59
Total Hardness (as $CaC0_3$) (mg/L)	19-364	81
Chloride (mg/L)	3-428	12
SS (mg/L)	5-1,200	227
VSS (mg/L)	1-290	57
COD (mg/L)	20-610	111
BOD (mg/L)	1-173	17
ΣN ** (mg/L)	0.3-7.5	3.1
Inorganic N (mg/L)	0.1-3.4	1.0
Hydrolysable PO_4 (mg/L)	<0.02-7.3	1.1
Organic Chlorine (μg/L)***	0.38-4.72	1.70
Coliform Organisms Number per 100 mL	2,900-460,000	--

* January and February 1963 not included.

** Arithmetic sum of the four forms of nitrogen.

*** From 11 storms, August 1963 to February 1964.

The data in the table were derived from several samples taken during various storms for the period reported. A conversion of these values into cumulative totals for a complete runoff volume indicates that the magnitude of pollution stemming from urban runoff can be significant. A discussion of the importance of certain of the constituents reported in Table A-2 follows.

Quantities of nutrients found in the Cincinnati data are of particular interest since they are very much greater than the 0.3 mg/L inorganic N and 0.03 mg/L inorganic PO_4 indicated as threshold levels for algal blooms.[34] This finding is important in light of the increasing concern over eutrophication of fresh water bodies. In a study of Green Lake in Seattle, Washington, it was recommended that storm waters should not be permitted to enter the lake because they would accelerate shoaling and would obviate, because of their high nutrient content, any methods for algae control involving limitation of the food supply.[35] Based on the few observations available, it would appear that water quality models for urban runoff should include nitrogen and phosphorous components. The generous use of lawn fertilizers by many homeowners further substantiates this need.

A variety of pesticides and herbicides are also used in residential areas. In many cases, the rate of application per unit area is probably much greater than in rural areas.

Many of these chemicals contain organic chlorine and thus measures of this constituent can be used as an index to the quantities of the chemicals employed. Such an index should be used with caution, however, since all organic chlorine is not derived from pesticides or herbicides. Analyses of samples from the Cincinnati area were not conclusive as to the actual amounts of pesticides contained since identification of specific compounds was not possible. The studies did indicate, however, that levels up to about 3.9 µg/L of pesticides as a group average (25 compounds) could have occurred. Additional research on pesticide loadings is needed.

The Cincinnati studies indicated a wide range in coliform group densities per 100 mL. In 90% of the samples, a density of 2,900/100 mL was found. This considerably exceeds the criterion of 1,000/100 mL commonly used as a maximum for bathing waters in the United States. Both fecal coliforms and fecal streptococci were found. Their presence indicated that pathogenic microorganisms originating in the fecal matter of warm-blooded animals might be expected. The lower density of fecal coliforms when compared to that of fecal streptococci suggested a predominance of animal sources. The implied health risk was considerably strengthened by the finding of Salmonella Thompson in one highly contaminated sample.

Very little in the way of measurement of road salt input to water courses from urban areas has been reported although some studies of salt concentrations in highway

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[34] Sawyer, C.N. 1947. "Fertilization of Lakes by Agricultural and Urban Drainage." *Journal New England Water Works Association, 61.*

[35] Sylvester, R.O., and G.C. Anderson. February 1964. "A Lake's Response to Its Environment." *Proceedings ASCE, Journal of Sanitary Engineering Division.* Volume 90, No. SA1.

drainage have been conducted in Maine and Illinois.[36,37] In the Cincinnati study, it was stated that constituents were not reported in January and February because the runoff was mostly snowmelt, high in chlorides.[1] That considerable quantities of road salt are used in northern regions may be verified by considering that in the state of Maine, for example, the average annual salt application per 2-lane highway mile is about 25 tons. The impact of this chemical on the water quality of urban receiving waters needs quantification.

In Seattle, Washington, appreciable amounts of oil were found in the urban runoff samples analyzed. A median value for 35 observations of 59 mg/L of oil was reported. Other investigators have also pointed this out as a problem. Additional concern over the pollution component of hydrocarbons and tars derived from the washing of urban streets has been indicated. However, little quantitative data related to amounts of oil, hydrocarbons or tars is available. The Seattle study would suggest a definite need for additional research in this area.

A primary pollutant in both urban and natural waters is sediment. It is well known that during periods of construction, large sediment yields from urban lands can result. Wolman and Schick state "the equivalent of many decades of natural or even agricultural erosion may take place during a single year from areas cleared for construction." While only a fraction of the total urban complex is under construction at anyone time, there is an increasing expansion of urban centers and the construction phase for at least part of these lands can be expected to extend well into the future. Rates of erosion from lands already developed also need to be determined. The data available on sediment loads in urban areas are not extensive. This was well illustrated in a 1967 paper by Dawdy. It appears that most of the available information has been summarized by Wolman and Schick. Dawdy has suggested that sediment yield data for non-urban areas might provide an insight into relations to be expected for urban areas. This would appear, at best, to suffice only as a short-term, stopgap measure, however. Observations from an area of residential construction in Kensington, Maryland, indicated mean concentrations of sediment ranging from about 60,000 ppm in 1959 (the approximate period of maximum construction) to about 1,500 ppm during the 1961-1962 period. These data illustrate the significance of the sediment problem and also point out its extreme variability. Although some research projects have produced data on maximum sediment yields during construction, few, if any, have attempted to predict the transient yield by relating it to soil types or construction practices.[5] Studies relating sediment yields from natural backgrounds to those from urbanized backgrounds are also lacking. Research is needed here as it is very likely that the variance between the two is considerable. Quantitative relationships between natural and urban sediment yields would serve a useful purpose in appraising the magnitude of the overall urban sediment problem.

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[36] Hutchinson, F.E. Winter 1968. "Effect of Highway Salting on the Concentration of Sodium and Chloride in Rivers." Research in the Life Sciences, University of Maine, Orono, Maine, pp. 12-14.

[37] Sullivan, R. September 1968. American Public Works Research Foundation, personal communication.

It seems apparent that urban stormwater runoff must be evaluated as a potentially significant waste load input to its receiving body of water. The previously summarized research and a 1968 study by the American Public Works Association[15] have borne this out. Without doubt, considerable reliable quantitative data on various kinds of urban environments is needed. Data obtained from one or two isolated areas can be of only limited usefulness because of the divergence of topographic, climatic, and land use characteristics typical of urban areas across the country.

Based on current information (1968), it would appear that programs for sampling the water quality of urban runoff should provide for measurements of the following parameters:

1) Biochemical oxygen demand (BOD)
2) Chemical oxygen demand (COD)
3) Dissolved oxygen (DO)
4) Sediment
5) pH
6) Alkalinity
7) Hardness
8) Chloride
9) Suspended solids (SS)
10) Volatile suspended solids (VSS)
11) Total nitrogen
12) Inorganic nitrogen
13) Phosphate (total hydrolysable as PO_4)
14) Organic chlorine
15) Fecal coliforms
16) Mineral oils
17) Specific conductance
18) Temperature
19) Other parameters

It is recognized that this list is not exhaustive. Many other pollutants could have been included but little useful purpose would be served. In the final analysis, each urban area will have to be evaluated in terms of its own characteristics. It is quite possible that some important urban pollutants have not yet been identified.

Hydrologic data necessary to compute the pollutional loadings of various constituents are very meager. Such data are essential, however, since knowledge of the frequency and time-distribution of loading may often be far more important than a knowledge of the total loading. This would be particularly true in cases where the objective was to determine the impact of the waste flow on a receiving stream. For example, a short, high-peaked surface runoff hydrograph of suspended matter could be expected to more seriously affect a receiving water than a hydrograph which released the same volume of suspended matter over an extended period of time. On the other hand, when stormwater enters a lake, the annual volume of contaminants also takes on special importance. In general, urban water quality data must be recorded in a continuous manner so that the time rate of delivery of the constituent loading can be determined. If this is not done, only very gross estimates of the impact of urban water quality inputs on receiving waters will be obtained. In summary, it seems fair to conclude that the meaningful management of urban stormwater quality will not be achieved until pollutants are properly identified as to nature and rate of delivery for individual storm events, types of urban areas, seasonal and antecedent factors.

3.0 WATER QUALITY CONSIDERATIONS AT SUCCESSIVE STAGES OF THE URBAN DRAINAGE SYSTEM

Urban drainage systems vary widely in size from a few acres up to many square miles. In general, the larger systems will include extensive segments of natural or artificial open channels. The relationship between water quality constituent input loading and such factors as area size, land-use practices, seasonal characteristics, local hydrology, antecedent conditions and perhaps other parameters will have to be determined. These input loadings must be determinable at various points of entry to the drainage system such as inlets, channels, etc. Once the input to the system is known, it must be routed through the system so that its effect can be determined at any downstream point of interest. For this reason, constituent concentration versus time data must be predictable for all stormwater flows. Flows in closed conduit systems are usually subjected to considerable turbulence at tributary points and therefore the assumption of complete lateral dispersion is probably reasonable in most cases. Longitudinal dispersion can be assumed to conform to some type of mixing model[38] although crude water quality models may even exclude this consideration. Because the time of residence within sewer systems is normally small (compared to that for natural streams and rivers) the assumption that pollutants are conservative, at least until they reach a receiving body of water, is probably justified.

Considering the nature of the typical urban drainage system, it appears that a water quality-modeling scheme should provide for:

1) The determination of constituent concentrations versus time at all points of entry into the drainage system.
2) The synthesis of individual constituent inputs into total time-distributed outputs at the outfall or at a point of interest.

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[38] Rich, L.G. 1961. "Unit Operations of Sanitary Engineering." J. Wiley and Sons, Inc., New York, New York.

3) The treatment of drainage discharges along a receiving watercourse in a manner readily permitting them to be incorporated in a useful water quality model for the receiving body. Non-conservative water quality models will likely be necessary for some constituents at this point.

4.0 WATER QUALITY MODELS

The development of useful water quality models for urban areas will require: first, a well-conceived sampling network to provide data representative of the varied conditions of topography, climate, and urban land use practices found in the United States; and second, the development of satisfactory relationships between water quality constituents and key hydrologic, topographic, land use, seasonal, and/or other parameters found to be important.

There is no evidence in the literature to indicate that any water quality models for urban areas have been developed. Nevertheless, a serious effort in water quality modeling for rivers and streams is in progress. Some of the models proposed for that use are either directly applicable for developing adequate urban water quality models or could serve as guidelines.

In theory, two approaches or their combination can be used in predicting pollutional loads imposed on receiving water by an urban drainage system. These are the stochastic approach and the deterministic approach. The first of these is based upon determining the likelihood of a particular output quality response by statistical means. This approach may be classified as "black box" in nature in that the response to the hydrologic input is determined without any attempt to evaluate the many interactions that occur in producing the water quality output. The serious shortage of urban water quality and runoff data essentially rules out the use of this approach at present.

A deterministic approach requires that some type of model be developed to relate the water quality loading to the known or assumed hydrologic input. Such a model may vary from an empirical concentration-discharge relationship to a soundly based physical equation representing the hydrochemical cycle. The ultimate modeling technique, of course, is that which best defines the actual mechanism triggering the water quality response. The cause of a given state of pollution can then be specifically identified.

It would appear, however, that some form of crude deterministic modeling will have to serve as an initial approach since even the data requirements for gross modeling are largely unsatisfied at this time (1968). With the advent of a comprehensive data collection effort, more sophisticated models such as that proposed by Woods can be developed and tested.[39]

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[39] Woods, P.C. June 1967. "Management of Hydrologic Systems for Water Quality Control." Contribution No. 121, Water Resource Center, University of California, Berkeley, California.

5.0 NON-CONSERVATIVE WATER QUALITY

Various unstable pollutants such as radioactive wastes and heat that have a time-dependent decay can be considered non-conservative. The outstanding example of our concept of such a pollutant is probably biochemical oxygen demand.

For the relatively short residence times normally encountered in storm sewers it is unlikely that many constituents would have to be considered non-conservative. On the other hand, that portion of an urban drainage system that is in reality a natural or man-made open channel might have a long enough flow time to require the employment of a non-conservative model.

Numerous investigators have proposed non-conservative models for rivers in the United States.[40,41,42,43,44,45]

As an indication of modeling techniques, a brief discussion of BOD models is presented here. The models used are based principally on the classical Streeter and Phelps oxygen sag equation. This relationship considers that the net rate of change in the oxygen deficit of a stream is equal to the difference in rates of oxygenation and de-oxygenation occurring in the stream. Simply stated, these are the oxygen removed by the BOD exerted on the stream and the oxygen introduced into the stream by atmospheric reaeration. The rate at which oxygen is removed is approximated by a first-order kinetic reaction equation. Reaeration is considered to be proportional to the net oxygen deficit. A general equation relating these reactions is of the form:

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[40] Davidson, B.R. October 1967. "Analog Computer Simulation of Stream Pollution Dispersion Models with Chemical Reactions." and

Davidson, B.R., and R.W. Bradshaw. October 1967. "Thermal Pollution of Water Systems." New Jersey WRRI, Rutgers—The State University, New Jersey.

[41] Smith, R.L., W.J. O'Brien, A.R. LeFeuvre, and E.C. Pogge. January 1967. "Development and Evaluation of a Mathematical Model of the Lower Reaches of the Kansas River Drainage System." Civil Engineering Department, University of Kansas, Lawrence, Kansas.

[42] Dobbins, W.E. June 1964. "BOD and Oxygen Relationships in Streams." *Proceedings ASCE Journal of Sanitary Engineering Division.* Volume 90, No. SA3.

[43] Thayer, R.P., and R.G. Krutchkoff. August 1966. "A Stochastic Model for Pollution and Dissolved Oxygen in Streams." *Water Resources Research Center Publication.* Virginia Polytechnic Institute, Blacksbur, Virginia.

[44] Loucks, D.P., and others. December 1967. "Linear Programming Models for Water Pollution Control." *Management Science.* Volume 14, No. 4.

[45] Revelle, C.S., and others. February 1968. "Linear Programming Applied to Water Quality Management." *Water Resources Research.* Volume 4, No. 1.

$$\frac{dD}{dt} = K_1L - K_2D$$

where dD/dt = rate of change of oxygen deficit D with time

K_1 = de-oxygenation rate coefficient (base e)

L = ultimate BOD

K_2 = reaeration rate coefficient (base e)

Because the original equation did not account for such occurrences as the removal of BOD by sedimentation, the addition of BOD by bottom scour, the addition of oxygen by photosynthetic action and other occurrences, various modifications accounting for such factors have been proposed. Smith et al. have summarized many of these.

An inherent difficulty in the use of the oxygen sag equation is the determination of values for K_1 and K_2. The term K_1 reflects the rate at which bacteria utilize oxygen and is determined from BOD laboratory tests. The reaeration constant K_2 presents the greatest difficulty of evaluation. Many theories of mass transport from the gas to the liquid phase have been proposed. All of these use the basic mass transport equation but the postulated mechanism of absorption varies among the theories. Smith, et al. in a study of several current approaches proposed for evaluating K_2 found variations between the methods as large as 300% at a flow of 1,000 cfs. Thus, the choice of a model for transfer appears to be somewhat unique for a given stream, and a final choice must rest on agreement with actual data from the stream in question rather than from any purely theoretical consideration.

The application of non-conservative BOD water quality models to actual or hypothetical river systems is clearly illustrated by Smith, et al., Thayer and Krutchkoff, Loucks, et al., and Davidson. It is considered that these approaches incorporating a modified form of the Streeter Phelps equation could be used in their present form or could be altered to apply to urban streams as well. At the very least, they can serve as useful guidelines for developing urban water quality models.

In general, the development of non-conservative water quality models must take into consideration the mixing characteristics of the stream in question as well as the reaction kinetics involved. Often the mixing characteristics of streams defy exact mathematical description. As a result, engineers have resorted to the use of various kinds of generalize mixing models. Several of these are discussed by Rich.

It is expected that adaptations of such mixing models could be made to urban drainage systems. Likely some compromise between a plug flow model and a completely mixed model would describe the conditions in the closed conduit and open channel portions of the system.

In real flow systems a deviation from idealized plug flow occurs because of velocity variations that are encountered in the flow regimes of both streamline flow and turbulent flow.

In streamline flow there is an absence of lateral mixing and a well-defined velocity profile that result in a variation of residence times across the channel section. As a result, conversions will differ accordingly and in general will differ from conversions predicted using a residence time based on the average bulk velocity. Examples of such differences for flow in circular pipes for first and second order reactions in isothermal and adiabatic reactors are available in tabular form.[46]

In turbulent flow, the velocity profile is relatively flat but velocity fluctuations occur across nearly all of the profile due to the turbulent action of the fluid. An impulse input to the system will thus tend to be reshaped into the form of a normal curve at a downstream location. The variance of the curve is a function of the dimensionless group D/uL (a function of Reynolds number) where D is the axial dispersion, u is the average velocity, and L is the reactor length. The extremes range from D equal zero for plug flow to D equal infinity for total mixing.

Levenspiel[47] gives the formulation of the so-called dispersion model as follows:

$$(D/uL)\frac{d^2C}{dz^2}-\frac{dC}{dz}-\mathrm{k}\tau\mathrm{C}^{\mathrm{n}}=0$$

> where z is the fractional length of the reactor measured from the entrance; τ is L/u, the residence time; k is the reaction constant; n is the reaction order; and C is the constituent concentration.

At small values of D/uL the condition of plug flow is approximated. As D/uL increases, the effects of back mixing become more pronounced.

Development of useful non-conservative models for urban drainage systems may be said, in summary, to require the following: 1) reliable input data related to the storm runoff; 2) a representative mixing model; and 3) knowledge of the reaction kinetics. Results from studies of larger stream systems can probably be used to provide considerable insight into the urban water quality problem.

6.0 CONSERVATIVE WATER QUALITY

Many inorganic pollutants can be considered conservative in nature. In addition, some pollutants normally considered to be non-conservative such as BOD might be treated as conservative at least for the smaller portions of an urban drainage system.

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[46] Bobalek, E.G. July 1967. "Chemical Reactor Theory Applied to Modeling the Dynamics of a Control System for Water Quality of a River." *Project Completion Report.* Water Resources Center, University of Maine, Orono, Maine.

[47] Levenspiel, O. 1962. "Chemical Reaction Engineering." J. Wiley and Sons, Inc., New York, New York.

The essential features of a conservative water quality model include: 1) the determination of the input to the system by relating it to the study area hydrology and other factors (if necessary); and 2) the determination of a mixing model to be employed.[48,49,50,51] Steele gives a very good summary of previous studies designed to relate water quality to various parameters.

Models to represent the transport of solutes from an urban watershed to its drainage system, may be developed according to known physical laws or may be largely empirical in nature.

The strict physical approach attempts to evaluate the change in the water's chemical character as it moves through the phases of the hydrochemical cycle. For each phase, the quantity of runoff involved, the nature of the chemical reactions governing the changes, and the character of any natural constraints must be known or simulated. The hydrologic simulation or modeling can be carried out according to various schemes. The Stanford Watershed Model (Crawford and Linsley) is one example.

The problem of the chemical change is difficult to resolve. Few reliable studies of the chemical phenomena of dissolution and ion exchange under natural conditions are available. Physical conditions affecting chemical reactions occurring in the field are not subject to the simple control of the laboratory. In addition, the chemical, physical, and hydraulic character of the land is itself highly variable. The complexities of the total hydrochemical system of a watershed defy an exact determination of the contribution to total solute load by individual phases of the cycle. Steele has stated that "although the hydrochemical cycle *is* easy to describe in qualitative terms, it is much more difficult to resolve quantitatively."

Fortunately, it has been demonstrated that the complexities of natural water quality systems can sometimes be circumvented. Several studies have shown that such systems frequently exhibit consistent chemical behavior that can be explained rationally in terms of adequate field data. An empirical approach can thus be used to identify the more important factors controlling water quality changes on a watershed and to develop useful working relationships for predicting water quality inputs.

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[48] Steele, T.D. May 1968. "Seasonal Variations in Chemical Quality of Surface Water in the Pescadero Creek Watershed." San Mateo County, California Ph.D. Dissertation, Stanford University, Stanford, California.

[49] Ledbetter, J.O., and E.F. Gloyna. February 1964. "Predictive Techniques for Water Quality Inorganics." *Proceedings ASCE, Journal of Sanitary Engineering Division.* Volume 90, No. SA1, Part 1.

[50] Wischmeier, W.H. 1960. "Cropping-Management Factor Evaluations for a Universal Soil-Loss Equation." *Proceedings Soil Sciences Society.*

[51] Woods, P.C. June 1967. "Management of Hydrologic Systems for Water Quality Control." Contribution No. 121, Water Resource Center, University of California, Berkeley, California.

Several studies have indicated that chemical quality data can be satisfactorily related to discharge by an equation or *series* of equations of the form:

$$C = KQ^n$$

where C is the constituent concentration, Q is the streamflow, and K and n are regression parameters. Ledbetter and Gloyna hypothesized that the inorganic quality of streamflows could be related to discharge by a hyperbolic function, a logarithmic function with a constant exponent or by a logarithmic function with a variable exponent. Their studies indicated that the logarithmetic equation[4] with a variable exponent was most satisfactory. The parameter n was related to discharge and an antecedent flow index. Steele, in his studies of a 46-square-mile natural basin in California, found what he termed "surprisingly consistent relations" between chemical quality data and level of discharge. Figure A-1 shows a graphical correlation between chloride concentration and discharge reported for the Solomon River in Kansas by Smith, et al. Numerous other examples are to be found in Steele's dissertation. Figure A-2 is one example. Steele also reported that there is an appreciable amount of pickup of solutes from the land surface and that this occurs very rapidly. He also found that solute concentrations varied considerably from one area to another.

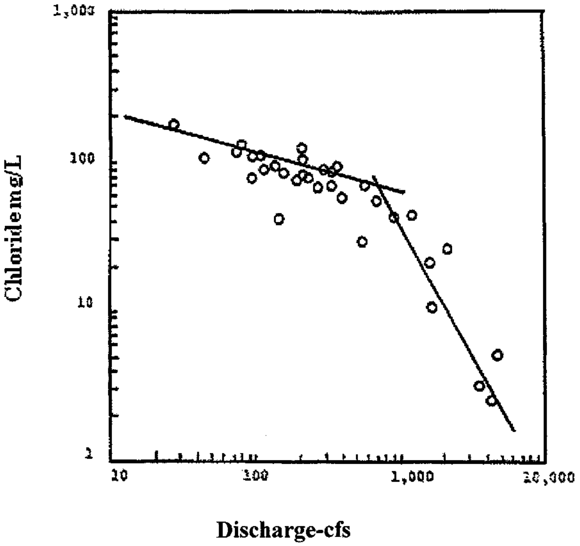

FIGURE A-1. Sample Plot of Chloride Concentration vs Discharge, Abstracted from data by Smith, et al.[19]

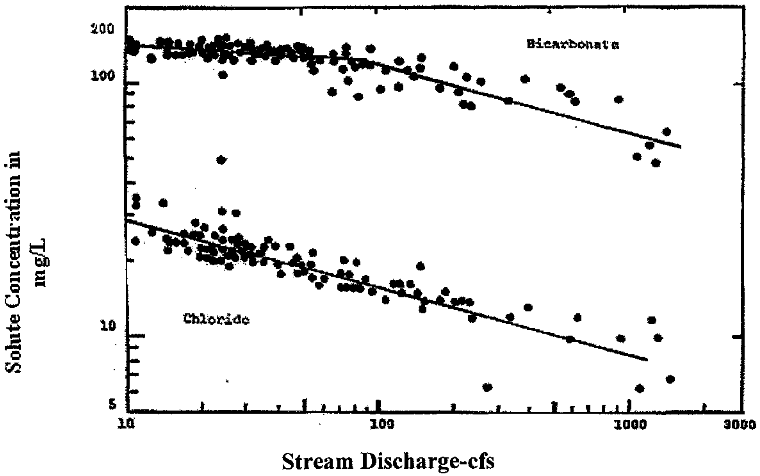

FIGURE A-2. Concentration-discharge relations for selected Solutes—San Lorenzo River at Big Trees, California, after Steele.

Based on somewhat limited data, Weibel, et al. have published graphical correlations between rain (in/storm) and BOD (lb/storm), rainfall (in/storm) and suspended solids (lb/storm), and suspended solids (mg/L) and flow ranges (cfs) for the data on the 27-acre urban area in Cincinnati. Although these relationships are primarily on a total storm basis they do indicate some apparent trends for the area.

No attempt to relate rainfall or runoff to urban water quality constituent concentrations in a generally useful manner has been reported as far as the author is aware (September 1968). As a result of this deficiency, researchers interested in developing conservative water quality models for urban areas will have to be guided by the findings of studies such as those previously summarized. The complexity of urban areas coupled with the seasonal activities of the inhabitants of these areas tend to indicate that urban water quality models may be very sensitive to seasonal and antecedent factors.

6.1 Sediment

Figure A-3 gives a relationship between average annual sediment discharge and cropland in percent. It is presented here to further indicate the potential of correlating urban land use practices and other factors with pollutant loading. Dawdy has theorized that an approach similar to that developed by Wischmeier might prove useful in gaining an insight into the effects which construction and land use practices have on sediment yield. Wischmeier's equation is basically of the form:

$$A = RKCLSP$$

It expresses-the average annual loss in soil in tons per acre A as a function of rainfall pattern, soil, topography, combinations of cropping and management, and conservation practices. Other terms are a rainfall factor R, a soil erodibility factor K, a cropping management factor C, and L, S, and P are factors related to slope length, percent slope, and conservation practices respectively. For urban areas, sediment production might be related to rainfall sequence, area type, topography and other factors, for example. Approaches of this type are probably the logical first step in developing useful sediment production relationships. These relationships will in turn provide the input for some type of sediment transport model.

A problem that is somewhat unique to the transport of sediment in urban drainage systems is the determination of relationships between deposition, scour, and period of retention in the system. Dawdy has discussed this problem as has Wolman. Obviously the tools used in studies of sediment transport will be a necessary adjunct to others employed in appraising the entire urban water quality problem. An interesting sediment transport model has been proposed by Negev.[52]

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[52] Negev, M. "A Sediment Model for A Digital Computer." *Technical Report.* Civil Engineering Department, Stanford University, Stanford, California.

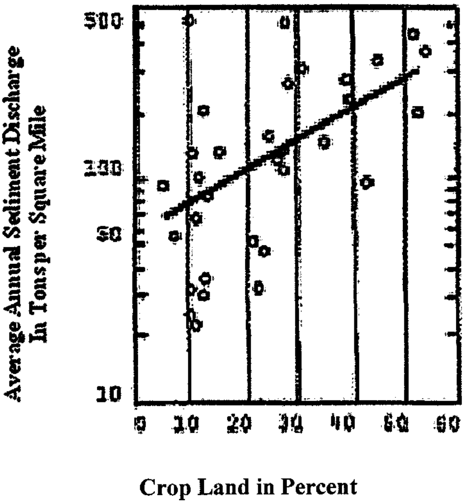

FIGURE A-3. Relation between sediment discharge and cropland in sub-basins in the Potomac River Basin, after Dawdy.5

It is important that the geomorphological effects of sediment movement in urban areas be identified and understood. The principles of quantitative geomorphology should be employed in programs established to provide relationships for prediction of sediment yields from urban areas.

6.2 Summary

That the problem of urban water quality has long been neglected and needs attention is obvious. Reliable basic input data are needed to quantify constituent input loading. Little useful data are available and most of these do not demonstrate the time rate of delivery of the water quality constituent for the runoff period for which they were obtained. Even if such records were available, they would have limited value in model development unless other data on the antecedent conditions at the time of the observation period were known. There is little doubt that striking variations in the kind and quantity of input loadings will result from changes in season and geographical area. Such urban practices as lawn watering may also have a profound effect on the ultimate destination of lawn fertilizers and other chemicals. Research opportunities related to such factors appear to be extensive.

In addition to the water quality data need is a need to improve our understanding of the hydrology of urban areas. This is because reliable hydrologic models must be used if meaningful water quality models are to have any real value. The old saying that water quality and quantity problems cannot be divorced is unquestioned.

When a greater insight into the complex nature of the physical, chemical, and biological processes that affect urban water quality is attained, it should be possible to effectively simulate constituent concentrations or loadings for the actual or simulated hydrology of the area.

Although initial efforts in water quality modeling will probably, of necessity, be largely empirical in nature, it should be the ultimate goal to relate these, as far as is practicable, to the known laws of physics, chemistry, and biology.

General water quality models should incorporate the following features:

1) They should be compatible with dynamic hydrologic models for the area in question;
2) They should incorporate reactions typifying non-conservative processes; and
3) They should consider the mixing and dispersion characteristics of the receiving water.

Woods has also proposed that a general model have the facility for including time-delays in movement of quality constituents that result from the interactions of the constituents and the media over which or through which they pass.

While little research directed toward the development of the needed quantitative models of pollutant input loadings for urban areas has been reported, some studies of this nature for large drainage areas have been made. It is considered that the findings of many of these can have direct utility in the development of similar prediction techniques for urbanized localities. In order of importance, the following recommendations are made:

1) A well-conceived national water quality data collection program for urban areas should be promoted.
2) Support to stimulate research on the development and testing of urban water quality models should be provided.

If this is not done, very little can be accomplished in the way of quantification or control of what has been identified as a very important factor in the overall water quality management of our urban streams.

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CONSIDERATIONS INVOLVED IN EVALUATING MATHEMATICAL MODELING OF URBAN HYDROLOGIC SYSTEMS[53]

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1.0 INTRODUCTION

Quantitative hydrology is modeling. The development of hydrology is the development of mathematical statements about hydrologic processes. Thus flood routing is a mathematical model that describes how a flood wave is translated and attenuated as it travels down a channel. Whenever a past event is reconstructed or a future event is predicted, a model is involved. Interest in the hydrology of the urban environment leads to interest in modeling as soon as knowledge is sufficient so that quantitative answers are desired to hydrologic questions.

The field of mathematical modeling can be subdivided by at least two different methods of classification. First is the stochastic-deterministic classification. A stochastic model relates input to output statistically. This can be by the relation of an input as an independent variable to the output as a dependent variable, as in the case of Carter,[54] who related change in flood peaks as a result of urbanization to change in lag time. It can also be by a statistical simulation of a synthetic streamflow trace. Fiering,[55] for instance, computes the statistics of a given recorded trace, then uses those statistics to generate "equally likely" traces. A deterministic model relates input to output in such a manner that, once the input is known, the output is wholly predictable. The most often used deterministic models in hydrology are those based on the laws of hydraulics, which use equations of continuity and motion. Examples are Liggetts' study of open channel flow[56] and Grace and Eagleson's study of similitude for modeling runoff from small, impervious areas.[57]

A second subdivision of hydrologic modeling is the analytic-synthetic classification. An analytic model usually describes a restricted area of hydrology in which the laws governing the process are fairly well known and accepted. Given these laws, a means of solution is developed, either in closed form or by finite difference approximations. A synthetic model specifies a conceptual relating function, and system parameters are identified through the use of input and output data. The conceptual model may vary between modelers, if the physical laws operating are not well defined or generally agreed upon. Thus, an analytic model usually describes a narrow or restricted sub-

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[53] In press as U.S. Geological Survey Water-Supply Paper 1591-D.

[54] Carter, R.W. 1961. "Magnitude and Frequency of Floods in Suburban Areas." U.S. Geological Survey Professional Paper 424-B, pp. B9-B11.

[55] Fiering, M.B. 1967. "Streamflow Synthesis." Harvard University Press.

[56] Liggett, J.G. 1659. "Unsteady Open Channel Flow with Lateral Inflow." Department of Civil Engineering, Hydrodynamics Laboratory Report No. 2, Stanford University.

[57] Grace, R.G., and P.S. Eagleson. 1965. "Similarity Criteria in the Surface Runoff Process." Department of Civil Engineering, Hydrodynamics Laboratory Report No. 77, M.I.T.

system in hydrology so that the problem is made manageable, whereas a synthetic model can be made as complex and cover as broad an area of hydrology as desired. Grace and Eagleson's study is an example of the analysis approach, and Crawford and Linsley's rainfall-runoff model[58] of the synthesis approach.

During the process of model building the model builder is continually faced with a decision concerning simplicity versus completeness in the model. The simpler the model is the easier it is to understand and use, and probably the cheaper it is to use. However, no important element of the system that bears on the process to be modeled should be omitted from the model. For instance, the use of hydraulic equations is familiar to most hydrologists. The general form of the equations is difficult to solve. Therefore, simplifications are made. Each simplification requires a statement of an assumption such as that steady uniform flow occurs in a trapezoidal channel. Once these assumptions are made the problem is reduced to a manageable size and can be solved, but the solution does not apply if the assumptions are violated.

The same problem faces the synthetic model builder as faces the analytic model builder but the decisions are not so easy nor the assumptions so obvious. First, the process to be modeled must be broken down into sub-systems, each sub-system must be judged as to relative importance, each must be modeled, and all must be tied together in a master program. However, generally accepted concepts such as steady, uniform flow or gradually varied flow do not exist. Rather, the question facing a rainfall-runoff model builder might be the relative importance of interception storage as opposed to detention storage, and how the two can be differentiated if the two are to be modeled separately. Equations for both must be derived from empirical studies and data for neither will be available for most areas of use. Both interception and detention may exist on a basin, yet neither may be necessary in a model that aims at a given level of output accuracy. A desire for completeness in a model tends to lead toward inclusion of all components that intuitively are known to exist. However, the desire for completeness may lead to the inclusion of many parameters that are merely curve fitting factors rather than physical parameters describing the process they supposedly are modeling. The optimal rule to follow would be that of Occam's Razor, which states that if a simple model will suffice, none more complex is necessary. This fails in practice unless "suffice" is better defined.

An important problem that faces both the model builder and the model user is that concerning the transferability of results. This is particularly true in urban hydrology, for little data exists for urban watersheds. Practically no data exists for the modeling of the quality of urban runoff. Therefore, any model developed must be applicable to ungauged areas. Transferability implies that model parameters must be derivable from physical measures of the drainage basin. For instance, Carter's study requires measures of length, slope, drainage area, and degree of urbanization. With these, an estimate of the mean annual flood can be computed. Fiering's streamflow synthesis

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[58] Crawford, N.H., and R.K. Linsley. 1966. "Digital Simulation in Hydrology." Stanford Watershed Model IV, Department of Civil Engineering Technical Report No. 29, Stanford University.

model requires a mean flow, a variance, and a first order serial correlation between adjacent flows. From these a synthetic trace can be generated. Urbanization studies require that the change in parameters as a result of man's influence must be estimated. Carter estimated man's influence by using the percent of impervious area as a parameter. If parameters have true physical significance, such methods may be very effective.

The physical significance of the parameters in a typical rainfall, runoff model and their changes as a result of urbanization are discussed by James[59] in his study of the hydrology of Sacramento, California. The lack of direct physical significance of the statistical parameters in a streamflow synthesis program is a possible shortcoming. Means, variances, and serial correlations must be related to a mapable measure. In addition, the statistical parameters used to establish any relation must be defined on the basis of a measured period of record. Adequate records to define a range for the statistical parameters for the urban environment are inadequate or non-existent. For this reason stochastic models are not considered feasible at this time for modeling urban basins, and therefore, are not discussed further. However, many of the points which are discussed are applicable both to stochastic and deterministic models.

2.0 CRITERIA OF GOODNESS OF FIT AND THE FITTING PROCESS

Any model must be easily usable and must give satisfactory results. A user is not as concerned as a model builder is with how a model is derived. He is more concerned with what is derived and how well it can predict results for his particular problem. "Satisfactory results," however, have no meaning by themselves. There must be some criteria for judging goodness of fit.

A best linear predictor for linear systems can be derived by the least squares criterion if residual errors are independent, normally distributed, and homoscedastic. Even when these conditions are not met, some least squares fitting are often used. Yet, even a least squares criterion requires that deviations from something must be computed in order to have a squared measure to minimize. Eagleson, et al.[60] explained the rationale for the use of a least squares analysis when they stated that the basic problem of linear black box analysis is to solve for a meaningful response function when the input and output are related by a system that is not truly linear. Eagleson obtained an optimal realizable unit hydrograph by using linear programming to solve a Weiner-Hopf formulation for the rainfall-runoff system. A solution of the Weiner-Hopf equations gives the least squares fit or, in Eagleson's language, minimizes the integral square error. The Weiner-Hopf equation, in Eagleson's case, minimized the squared differences between the simulated and observed total streamflow traces. This was an operational decision made in order to obtain a hopefully meaningful response function. Any other squared error term could

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[59] James, L.D. 1965. "Using a Digital Computer to Estimate the Effects of Urban Development on Flood Peaks." *Water Resources Research.* Volume 1, No. 4, pp 223-224.

[60] Eagleson, P.S., R.R. Mejia, and F. March. 1966. "Computation of Optimum Realizable Unit Hydrographs." *Water Resources Research.* Volume 2, No. 4, pp. 755-764.

have been used, would have given a different Weiner-Hopf solution, and would have been equally optimal.

Dawdy and Thompson[61] indicated that the criterion set for optimization influences the resulting set of optimal parameters for a given model. They indicated that the modeling process can be considered as analogous to a linear programming problem. Eagleson transformed the problem to an explicit formulation of a linear programming problem. As Eagleson showed, the goodness of fit criterion in modeling is an objective function, and the model itself, expressed by the Weiner-Hopf convolution equations, is a set of constraints.

Sensitivity analysis in linear programming studies the changes in the optimal solution as any set of coefficients are varied, including those of the objective function. Dawdy and Thompson indicated that in their study three different sets of optimal parameters existed, each corresponding to a different objective function. If all three of their objective functions were combined into one weighted function, their three solutions would be end member solutions with weights of (1,0,0), (0,1,0) and (0,0,1), where the numbers in each set of parenthesis represent the relative weights given to each of the three objective functions. Each weighting produces a different set of optimal parameters. Research into the variation between sets of optimal parameters produced by the use of different objective function weightings would give insight into the modeling process and into the interpretation of a given model. Each optimal set of parameters is optimal only in terms of its objective function. This seeming paradox is familiar to most in simpler cases of an optimal solution not being a unique solution, and is here merely extended to the general field of simulation. Eagleson summarizes the problem when he states, "When each storm is analyzed independently, the several unit hydrographs obtained are thus likely to have widely varying geometrical properties, and the method of averaging them . . . is not clear." A measure of the effect of averaging of parameters on peak estimation is shown in Figure A-4. A rainfall-runoff model was fitted to four different years of record for Arroyo Seco near Pasadena, California. Each year was fitted separately, and the simulated peaks are shown. In addition, the optimum parameters for each year were averaged, and these average parameters used to simulate the same peaks. The scatter increases, particularly for the lower peaks when averaged values are used for simulation.

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[61] Dawdy, D.R., and T.H. Thompson. 1967. "Digital Computer Simulation in Hydrology. *Journal AWRA*. Volume 59, No. 6, pp. 685-688.

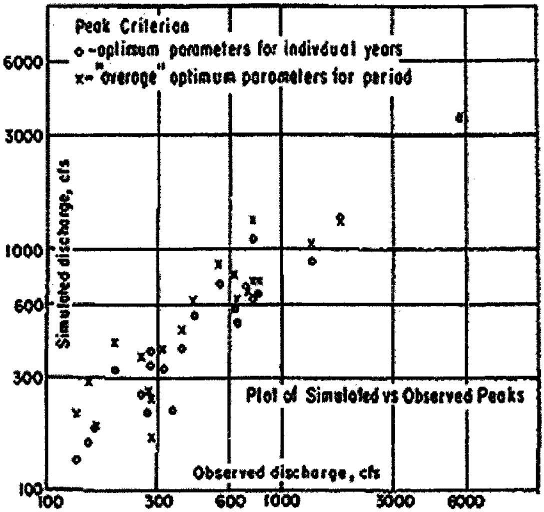

FIGURE A-4. Comparison of simulation results using optimum parameters for each year with results using averaged parameters.

There is a direct interaction between the objective function and the fitted parameters. Therefore, the choice of the objective function itself should be optimal in some sense to the model user. There is no objective method for choosing the objective function, however, so that the choice of the objective function is a very subjective decision. Research in this area is vitally needed.

An example of the effect of the choice of the objective function on the fitting process is shown in Figure A-5. For the same rainfall-runoff model, parameters were fitted to a given control period on the basis of two different objective functions. The first objective function minimized the sum of squared deviations of simulated from observed discharges for peaks, the second for days. The purpose of the fitting was to develop a model for estimating peak discharges. The goodness of fit was tested by estimating peak discharges for a test period on the basis of the two sets of averaged parameters. As can be seen, the scatter is about the same for the two sets of simulated peaks and is on the same order as the scatter during the control period, as shown in Figure A-6 for the average parameters. However, the parameters based on fitting to daily values result in unbiased estimates, whereas fitting to peak values resulted in what seems to be a biased estimate. This effect in terms of a frequency diagram is shown in Figure A-6. Part of the hydrologic model became a curve-fitting function and, therefore, did not retain its supposed physical significance when peaks alone were used for fitting. Although parameters were "reasonable," the water balance was grossly erroneous. This created errors of prediction in the test period.

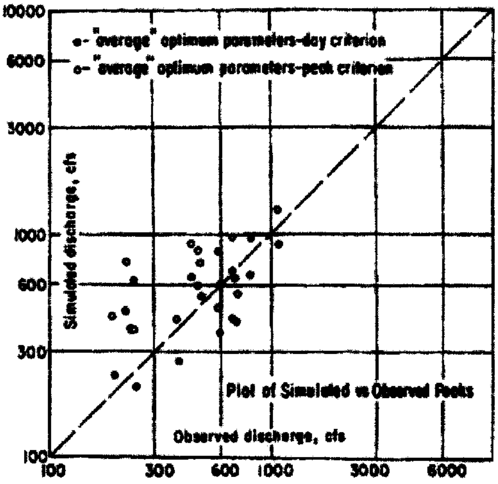

FIGURE A-5. Comparison of prediction using two sets of optimal parameters. Fitting for one is to peak flows, for the other, to daily flows.

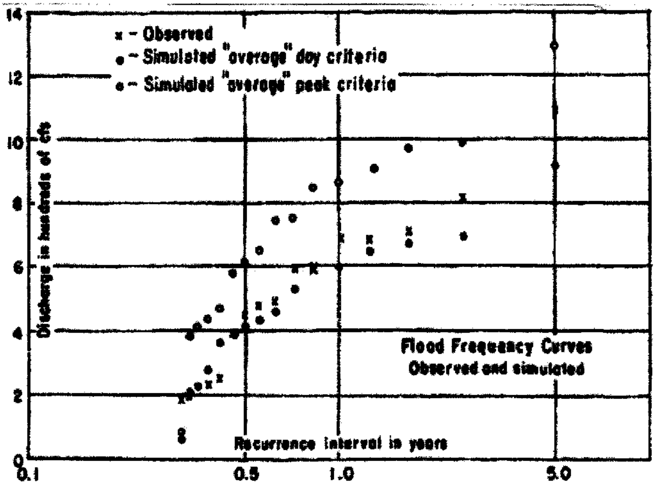

FIGURE A-6. Resulting frequency curves for scatter diagrams of Figure A-5.

Sequential fitting methods can use different criteria for fitting different sub-sets of parameters in a model. A first stage could hold routing constants at some first estimate and fit the parameters for water balance. Then the water balance parameters can be held fixed while routing parameters are fitted to peaks. However, there is much interaction among parameters, and some parameters may influence equally both water balance and peak flows. Research in fitting methods can go hand-in-hand with research in the effects of choice of objective function on the optimal fitted solution.

Interactions cannot be eliminated, for they exist in the physical system. Their importance in the curve fitting process can be minimized, however, by determining as many parameters outside the model as possible. For instance, the Stanford Watershed Model has 20 parameters. Two are based on meteorological data, four on hydrograph separation, five are computed from physical measures, three are estimated from empirical tables, and six are fitted. The six fitted parameters all are involved in the loss function, which includes infiltration, drainage, and evaporation. Crawford and Linsley[6] discuss the interactions among the fitted parameters and suggest a combination of fitting both sequentially and to selected data to define the parameters as independently as possible.

The determination of parameters *a priori* eliminates, to a degree, the use of a simulation model as a black box device, and adds a degree of grayness. It masks a part of the interaction of parameters because some are held constant. However, certain "physical" parameters may be indices rather than measures. Resistance coefficients and slope when applied to basins as "average values" are examples. If fitted parameters are obtained in order to correlate with measured indices the interactions plus errors in data and in the model may result in physically meaningless values for some or all parameters.

To summarize, fruitful areas of research exist in the development of fitting methods, the choice of criteria for judging goodness of fit, and the effect of choice of criteria upon fitted parameters.

3.0 EFFECTS OF ERRORS OF DATA ON MODELING

Errors in data are reflected in errors in the fitted parameters in a simulation model. If perfect input data are routed through a perfect model, the output produced would agree perfectly with an error free output record. If errors are introduced into the input or output record, or both, the output will not be exactly reproduced even by a perfect model. If a fitting process is used, the parameters will deviate from their true values in order to minimize the deviations between the simulated and recorded traces as specified in the objective function. The "optimal" set of parameters will now be in error, and the fitted values of the objective function will be less than its "true" value.

This process is analogous to statistical least squares analysis. The fitted parameters deviate from their population values because of random errors in the data. The standard error of estimate is a measure of error of reproduction of the fitted data. The standard error of prediction, however, is somewhat greater than the standard error of estimate, for it includes both the measure of lack of fit of the data used to fit the model and the measure of error in the fitted parameters. These relationships are shown in Table A-3.

TABLE A-3

Qualitative Comparison of Errors Involved in Hydrologic Modeling With Analogous Errors in Standard Statistical Analysis

Source of Error	"Size of Error"	Statistical Analogy
Data	a	Measurement and sampling error
Comparison of measured to simulated during period used for fitting	a-b	Standard error of estimate
Comparison of measured to simulated during period <u>not</u> used for fitting	a+c	Standard error of prediction

If the assumptions of regression theory hold (a linear model with normally distributed and homoscedastic errors of the dependent variable), the error of prediction can be computed from the standard error, the deviations of the independent variables from their mean, and the error in the coefficients for the independent variables. The assumptions seldom hold, however, so that statisticians often resort to split sample testing. A similar situation holds for hydrologic simulation, except that there is no theory by which to compute the error of prediction. In order to present a measure of utility of a model to the potential user, the data used to test a model should not include any data used to develop the model or its parameters.

Non-linearity of the hydrologic process precludes any theoretical description of the mechanism by which errors in data are transferred to model parameters and then combined with input errors in the test period to produce errors in the simulated streamflow trace. An empirical study for the response for a particular model can be made as in Table A-4. A recorded rainfall trace was assumed error free and routed through an optimized set of parameters, which were assumed correct values, to obtain a "true" streamflow trace. Then a random error with mean zero and standard deviation of 10% was applied to all rainfall values. These "erroneous" rainfall values were then routed through the true model, and the resulting standard error of the simulated streamflow trace computed. An optimization run then was made which adjusted the parameters to minimize the standard error. The "optimized" set of parameters is shown, along with the resulting standard error. The "true" rainfall trace was then routed through the optimized parameters and the standard error computed. Assuming independence of the two sources of error, one in the input data and the other in the model parameters, the error of prediction should be approximately equal to the square root of the sum of the squares of the two separate estimates. Similar results are shown for random errors in the input with 20% standard error, and for random errors in the output of five and ten percent. A further test is then made for each case by applying a new set of random errors to the rainfall and routing this new set of erroneous data through the "optimized" parameters and computing the resulting errors of prediction. These also are shown in Table A-4. Such an analysis gives insight into the fitting process in simulation models, shows how errors in data are transmitted to the model, and gives an empirical measure of the effect of data errors on the accuracy of prediction in simulation.

Data errors have different results depending upon the type of data considered. All transfer part of their errors into the model parameters, but each differently. Each, therefore, must be considered separately.

3.1 Input Errors—Rainfall

The major source of error results from the rainfall input data. Rainfall errors have several sources. There is sampling error, as evidenced by variability between measured catches for different designs of rain gauges, and changes in catch when minor changes are made in the configuration of a given rain gauge. There are changes with time as buildings are built or trees grow nearby, changing wind patterns and thus changing catch. There are changes that occur when the gauge is physically moved, as most long-term records have been at some time. There is spatial variability of rainfall over the basin that a rain gauge does not measure because it measures point rainfall. Finally, any rainfall record must be considered representative of the basin to be simulated, and there may be adjustments made before it is considered representative.

TABLE A-4

The Effect of Errors in Data on the Fitting Process

Parameter[1]	**Parameter Value**				
	True	**Optimized Rainfall Errors**		**Optimized Runoff Errors**	
		10%	**20%**	**5%**	**10%**
SWF (inches)	3.6	3.6	3.8	3.7	3.7
KSAT (inches/hour)	.063	.063	.06	.063	.061
KSW (hour)	1.0	1.04	1.06	.98	.98
EVC	.56	.57	.58	.559	.56
SMSN (inches)	4.0	4.02	3.98	4.04	4.04
RGF	12.0	11.9	11.94	12.12	12.21
RR	.8	.796	.8	.796	.796
DRN (inches/hour)	.020	.018	.017	.020	.019
U, Pd[2]	(given)	.405(13)[3]	1.45(26)	.063(4.9)	.247(9.4)
pd	(fitted)	.261(10.5)	1.06(19.8)	.046(4.0)	.191(8.3)
pD		.105(6.5)	.41(13)	--	--
U, Test[4]	(predicted)	.530(14)	2.4(30)	--	--

[1] The model parameters have the following meaning: SWF is suction at the wetted front; KSAT is saturated soil permeability; KSW is surface storage coefficient; EVC is evaporation pan coefficient; SMSN is a nominal soil moisture storage; RGF is a multiplicative factor controlling the range of effective SWF; RR is a runoff ratio; and DRN is the soil drainage rate.

[2] P = true parameters; p = optimized parameters; D = correct data; d = erroneous data.

[3] N.B. First value is sum of squares of difference of natural logarithms of the 18 peaks plus half the squares of the 18 storm volumes. The second value converts the first value to an equivalent "percent standard error" by SE = antilog, $\sqrt{U/27}$ and averaging plus and minus percentages.

[4] Average of nine separate test runs.

Lumped parameter models of basin response take a rainfall record as representative of a basin or sub-basin, and assume uniform rainfall in space. As distributed parameter components or sub-systems are included in a given model, input data needs are increased. For example, an areally distributed rainfall input becomes necessary in order to be compatible with the model, and, therefore, to realize the benefits of the detailed description of the model. The problems of rainfall network density for both lumped and distributed parameter modeling have been discussed by Amorocho, et al.[62] and Eagleson.[63] Both agree that one or two gauges are sufficient to estimate mean convective storm precipitation for drainage areas of size similar to those encountered in urban areas. In addition, Amorocho and Brandstetter[64] have studied the problem of rainfall input for a distributed system in space. Their conclusions state,

> *The investigation of questions regarding catchment damping is also essential in order to determine the range of departures from the real input field which these systems can accept without significant change in the output.*

The study of the relation of rainfall network density to accuracy still is a fruitful area for research. However, errors in data *per se* are not of as much importance as the effect of those errors on the decision-making process. The next logical step, as Amorocho and Brandstetter imply, is to study the effect of rainfall errors on simulated runoff records. In particular, the use of distributed parameter models requires a study of the effect upon simulated streamflow traces of an assumption of uniformity of rainfall in space as opposed to the use of a spatially varied rainfall.

3.2 Input Errors—Evapotranspiration

Evapotranspiration (ET) in the main is a depletion from soil moisture. An Antecedent Precipitation Index (API) is a simplified model of the persistence of the effect of past rainfall on future events. ET is used in simulation models to construct a better index, or perhaps a "time variable API." ET determines soil moisture depletion, and, therefore, affects the rate of losses of rainfall to soil moisture through infiltration. If any concept of a limiting value of minimum infiltration is included in a simulation model, major storms of long duration and high intensity approach that limiting value, and the effect of initial soil moisture conditions is decreased. Thus streamflow traces for periods of major storms, years of high precipitation, and regions of high precipitation should generally be fitted more accurately than traces where initial soil moisture conditions are critical.

Measured values of soil moisture are seldom available for testing depletions from soil moisture through the process of drainage and ET. If models of ET loss are to be constructed and then tested by means of rainfall-runoff data, the ET model can best be tested in regions where soil moisture conditions are critical but variable, or else only selected periods of record should be used for model construction. Such periods should be selected so as to cover the range of effects of soil moisture on storm runoff and peak flows. With such a test, the problems of errors in ET can be studied in the

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[62] Amorocho, J, A. Brandstetter, and D. Morgan. 1967. "The Effects of Density of Recording Rain Gauge Networks on the Description of Precipitation Patterns." IASH Publication.

[63] Eagleson, P.S. 1967. "Optimum Density of Rainfall Networks." *Water Resources Research.* Volume 3, No. 4, pp. 1021-1034.

[64] Amorocho, J., and A. Brandstetter. February 1966. "Characterization of Gauge Level Precipitation Patterns." Presented at the *Pacific Southwest Regional Meeting, American Geophysical Union.* San Jose, California.

manner suggested by Amorocho and Brandstetter for the study of rainfall errors by measuring the impact of errors on simulated outputs.

True measures of ET seldom exist. Rather, pan evaporation data is collected, pan coefficients are determined, which supposedly adjust pan data to a potential evapotranspiration (PET), and then the calculated PET is used as input. Alternatively some PET can be calculated by an empirical (i.e., Blaney-Criddle) or semi-empirical (Thornthwaite or Penman) equation. Penman[65] presents various theories of "availability of water for evapotranspiration". These different theories represent different modulating functions that convert PET to ET. Errors of output result from errors in PET, errors in the conceptual modulating function, and errors of the parameters in the modulating function. Studies of the relative importance of these components of error would be worthwhile. In addition, the impact of various levels of urbanization on the simulation results will give some measure of the degree of attention to these data necessary for hydrologic simulation in the urban environment.

3.3 Output Errors—Streamflow

A discussion of errors of recorded output data centers around errors in stream gauging. The output of interest will depend upon the objective function, whether that objective function is stated explicitly or is an implicit "let's see how it looks." However, generally the only output data available are recorded streamflow. Streamflow data are much more accurate than rainfall data because they measure an integrated runoff from the total basin. Most streamflow records are rated as "good" by the USGS, which can be interpreted as meaning that daily mean discharges have a standard error of 5% (95% are within 10%).[66] Peak discharges are somewhat more inaccurate. Peaks fairly well defined by discharge measurements would have a standard error of about 5%. By "fairly well defined" it is meant that the peak flow is no more than twice the highest current meter measurement. When not so defined, peak flows may be computed by means such as slope area measurements or other indirect methods, and the standard error then may be of the order of 10%.

Errors in the output are transferred to the parameters through the fitting process just as are the errors in input. As indicated by the results of Table A-3, however, output errors are not as critical as input errors. Errors in input are magnified because a residual of excess precipitation is used in the routing, and any absolute error in input becomes an absolute error in the residual before routing. The generally minor amount of surface storage in urban basins does not greatly attenuate input errors. The objective function usually is stated in terms of some measure of streamflow, so that proportional errors in measured streamflow are transferred as proportional errors in output.

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[65] Penman, H.L. 1963. "Vegetation and Hydrology." *Technical Communication No. 53, Commonwealth Bureau of Soils, Commonwealth Agricultural Bureau.* Farnhan Royal, Bucks, England, pp. 124.

[66] U.S. Geological Survey. 1966. "Water Resources Data for California, Part 1." *Surface Water Records.*

Errors in input and output can create errors in different parts of the model. Random, unbiased errors in input usually are compensated for by adjustments in the parameters of the loss function, which includes infiltration, drainage, interception, and detention. Similar errors in output usually are compensated for in the routing function. Biased errors in either may act differently. For rainfall, the bias may be an error in the adjustment of the recorded record to "average basin precipitation." For streamflow the bias may vary with discharge, with little error for the smallest peak and a maximum error for the largest peak, if the general slope of the stage-discharge relation used is computed in error.

3.4 Interpretation of Errors

The impact of errors upon the simulation process depends in part upon whether the error is a random error of a quantity that is measured or whether the error is in an index that is used as an approximation to something that cannot be measured. Data and parameters cover the spectrum from streamflow discharge, which is truly measurable, through point rainfall, which is measurable but is used as an index to basin rainfall, to PET, which is not directly measurable and is used as an index to relate water demand and water availability, to basin slope, which is some index of a varying quantity,[67] the effect of which can only be estimated in itself. Even grosser indices might be "basin roughness" or "trans-basin groundwater seepage" measures.

Errors of indices are errors of approximation in the model. Even a "best" value of an index may lead to serious errors in simulation when the data are outside the range for which the particular approximation applies. Therefore, errors in indices generate both random errors and errors of approximation. It is errors in indices which usually cause outliers, for gross errors usually are the results of poor approximations in a case where the approximation is the controlling factor in simulation of a given event.

Errors of approximation become masked in the fitting process. The sub-systems within the hydrologic cycle are highly interrelated, with many interactions among parameters taking place even in the simplest models. Therefore, the handling of outliers introduces an important area of subjectivity in any modeling process. If methods of systems identification and systems specification can be developed to separate the modeling process into a series of relatively independent problems, a major step forward will result. Sequential fitting methods, as mentioned earlier, are a first step in this direction, but much more could be done to advance the model development phase if the independent factors could somehow be treated as is done in statistical hydrology through eigen vector analysis and synthesis.

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[67] Benson, M.A. 1959. "Channel-Slope Factor in Flood-Frequency Analysis." American Society of Civil Engineers, Separate No. 1494, pp. 9.

4.0 SENSITIVITY ANALYSIS

Sensitivity analysis studies the effect on the optimal solution of changes in the input-output coefficients and in the objective function. According to Dantzig:[68]

> *In many applications, the information thus obtained (through sensitivity analysis) is as valuable as the specification of the optimum solution itself.*
>
> *Sensitivity analysis is important for several reasons:*
>
> *(a) Stability of the optimum solution under changes of parameters may be critical. For example, using the old optimum solution point, a slight variation of a parameter in one direction may result in a large unfavorable difference in the objective function relative to the new minimum, while a large variation in the parameter in another direction may result in only a small difference. . . . . it may be desirable to move away from the optimum solution in order to achieve a solution less likely to require essential modification.*
>
> *Values of the input-output coefficients. Objective function coefficients, and/or constraint constants may be to some extent controllable, and in this case we want to know the effects that would result from changing these values.*
>
> *Even though the input-output and objective function coefficients and constraint constants are <u>not</u> controllable, the estimates for their values may be only approximate, making it important to know for what ranges of these values the solution is still optimum. If it turns out that the optimum solution is extremely sensitive to their values, it may become necessary to obtain better estimates.*

Sensitivity analysis in hydrologic simulation is not as straightforward as in linear programming. The same principles apply, however. Discussions of sensitivity of results to parameter variability are given by Crawford and Linsley and Dawdy and O'Donnell.[69] In addition, sensitivity to changes in the objective function was mentioned earlier. A conclusion drawn by Dawdy and O'Donnell was that

> *Any further development of automatic parameter optimization techniques must use some criterion of response sensitivity (or its equivalent) in selecting what can be considered adequately optimized parameters. Indeed, the minimization of differences from recorded data cannot be the sole criterion in interpreting the fit of any model.*

Although all the parameters discussed by Dawdy and O'Donnell apparently were equally sensitive to either positive or negative changes in parameter values, this does

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[68] Dantzig, G.B. 1963. "Linear Programming and Extensions." The Rand Corporation, and the University of California, Berkeley, pp 632.

[69] Dawdy, D.R., and T. O'Donnell. 1965. "Mathematic Models of Catchment Behavior." American Society of Civil Engineers. *Procedures Paper 4410*, HY4, pp. 123-137.

not necessarily occur in all cases. A case in point is shown in Figure A-7. A parameter representing the rate of drainage of moisture from the soil through percolation to the groundwater table was estimated for a simulation run. The sensitivity of the objective function to changes in rate of drainage shows that the rate is critical until a certain value is reached. Beyond that critical drainage rate, the objective function is quite insensitive to increases in the drainage rate. Therefore, for this model and data it is far better to overestimate than to underestimate drainage.

Sensitivity plots for both optimized and computed parameters give insight into the modeling process and the interpretation of the physical meaning of the parameters in a given model. Sensitivity plots cannot be plotted unless the objective function is stated in objective terms. Yet, as Dawdy and Thompson indicated, the formulation of the objective function itself influences the results. At the present time, the use and interpretation of sensitivity analyses is quite subjective. A valid field of research that is, to date, almost untouched is the methodology of developing and using sensitivity analyses for comparing different models and for comparing results for a given model under differing conditions, as touched upon by Crawford and Linsley and Dawdy and O'Donnell.

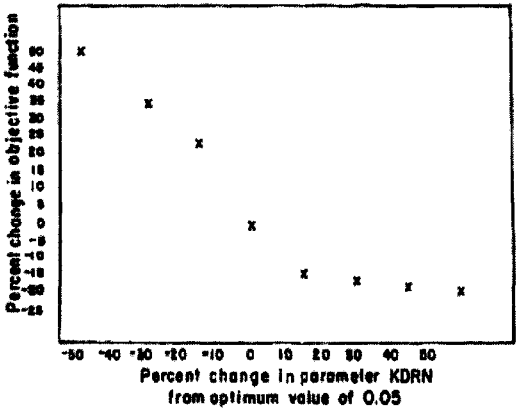

FIGURE A-7. Changes in goodness of fit as soil moisture drainage rates are varied, illustrating non-symmetry of response.

4.1 Optimization vs. Sub-Optimization

Up to this point, all discussion has centered upon hydrology as a system. Optimization has been in terms of errors in simulation of hydrologic data. In fact, hydrology is a sub-system, and the discussion has concerned sub-optimization. Hydrology is one input into a decision process concerning resources development. In optimization of the decision process accuracy of hydrologic simulation both costs and benefits with reference to the ultimate development scheme. Costs are incurred in gathering data of a certain type at a certain level of accuracy for a given length of time. Benefits result from better decisions that result from better data. Discussion of this broader systems study of the marginal cost and marginal worth of hydrologic data is beyond the scope of the present study. Nevertheless, the broader problem must be considered. It also presents many areas where research is needed.

5.0 CONCLUSIONS

Hydrologic simulation today is half science and half art. It is an art to the extent that subjective decisions enter into the modeling process and its assessment. Research is needed to move the field closer toward the area of science by developing and using the necessary systems analysis techniques. Measures of error are necessary in order to judge between models, whether on the part of the model builder or the model user. In addition, measures of error must usually be stated in some form of an objective function in order to use the tools of systems analysis.

Fruitful areas of research are:

1) The construction of meaningful objective functions and the interplay between objective function and simulation results. The use of objective functions for the disparate aims of fitting and predicting within a single model and of comparing between models should be considered.
2) The development of tools for sensitivity analyses, and of methods for using sensitivity analyses for developing and using simulation models.
3) The development of mathematical tools for fitting the independent portions of hydrology. This would be in the sense of eigen vector analysis, in which the independent factors are not necessarily the original variables nor sub-sets of the original variables, but some combination of the original variables.
4) The determination of guidelines for error analysis in hydrologic simulations. Hydrologic models are non-linear, so that standard linear theory does not apply. Errors in data are transferred into fitted parameters. These errors in fitted parameters, plus those in computed parameters, are transferred into predictions. Little is understood of the mechanism.
5) The comparative effects of input errors and output errors on simulation results so that some concept of network density can be applied to urban hydrologic data in terms of the end results of hydrologic prediction through simulation.

CONCLUSIONS REGARDING MODELING

1.0 INTRODUCTION

Current assessments of urban drainage construction requirements and current estimates of damage from urban stormwater indicate a need and justification for a significant research effort on urban rainfall-runoff-quality relationships. Mathematical modeling seems to be the method most useful for defining such relationships. Standard statistical techniques for analyzing data are inadequate at this time because of the lack of a database. The present state-of-the-art can give answers to questions concerning the urban environment, but the level or accuracy of those answers can be improved considerably. Better accuracy can be translated into direct dollar benefits, and the ideas presented earlier discuss areas where it is felt research dollars invested will reap the maximum in benefits.

Throughout all the previous chapters a common thread has run. There is a considerable body of knowledge available for analyzing the hydrology of natural basins. Much of this knowledge should be transferable to the urban environment. However, certain conditions must be met before this transfer to the urban environment can be made. First, more urban hydrologic data are needed. Second, that data that exists now or that will be collected in the near future should be made readily available for calibrating and testing mathematical models of urban hydrology. Finally, research must be conducted both in the fields of model building and model assessment, and in the ways in which urban conditions affect the natural hydrologic environment. Each of the three will be discussed in turn and an attempt will be made to bring together the suggestions of the various chapters along these lines.

2.0 DATA NEEDS

The present database for urban hydrology is inadequate. Mathematical modeling requires data for calibration of the models developed. Therefore, a network of rainfall-runoff-quality data should be collected on a variety of urban watersheds. The network should be large enough, say 15 to 20, to provide a reasonable range of climatic and watershed parameters. Such a database collected over the next three to five years should be adequate for widespread testing of various modeling techniques in order that the transferability of the various models can be explored.

These rainfall-runoff-quality data should include several fully instrumented urban watersheds where data of research quality are collected. These research data should be on a time and space grid at least as small as that recommended in Chapter 4 for the finite difference scheme. These data, for research purposes, should be collected over a long enough period to provide for standard split-sample testing of existing and future models. Thus, one part of the record can be used for model calibration and the remainder for model testing. The research quality data should be collected on basins in both arid and humid climates.

The most glaring data weakness at this time is in the area of water quality. The fully instrumented research watersheds should include an extensive quality-sampling program designed to provide insight into the time and space variation in quality loadings for the various constituents discussed in Chapter 5. Of special concern is the variability of quality loadings in response to the urban environment and to climatic variations, including season, and the effects of flushing by antecedent rainfall.

Finally, data are needed on the distribution of the urban damage function, both in time (frequency) and space (by size of drainage area). There is a need to avoid "overkill" in the formulation of the data collection and research efforts. Of particular concern are the instrumentation and potential analytical requirements associated with the rapid time response of flows from small urban basins. For instance, determination of a minimum size of drainage basin on which damage occurs may set a minimum time response of interest in modeling, and, therefore, may be the determining factor in setting the frequency for data collection. A systems analysis study of the tradeoffs between data costs and damage abated should provide insight into the necessary frequency of data collection. Data needs should not be based solely on theoretical hydrologic grounds.

3.0 DATA REDUCTION

A central clearinghouse for the available data on urban watersheds should be established. This should include rainfall, streamflow, water quality measurements, and watershed characteristics. Perhaps this clearinghouse could include all data for basins below a certain maximum size where the basins are considered adequate for calibrating models for natural conditions. The data should be stored in a form which is suitable for data processing by digital computers, so that it can be made readily available to various model users and model developers.

More seemingly anomalous events occur on urban than on natural watersheds as a result of the variety of activities and man-made changes. Therefore, a reasonably comprehensive analysis of the data from urban catchments should be undertaken, and anomalous data noted and explained where possible. For data being collected to add to the data bank, this screening of data should take place as the data is collected, if possible, or else as soon after as is possible. This step will improve the usefulness of the data for model development and testing.

Each hydrologist who obtains data from the bank for modeling purposes should be requested to furnish, in return, a description of his model, the fitted parameters used, the criteria used for fitting, and some measure of accuracy of fitting. These results can then be furnished to later users of the same data for comparison of results and of fitted parameter values. Thus, the data bank will include a set of data on the models themselves. As experience in using the data grows the clearinghouse can develop a recommended standard minimum set of data for model development and testing, and can describe a set of standard criteria being used by researchers for testing model response.

An index should be published of all data in the data bank. At the present time a listing should be prepared of all gauged catchments below some maximum size, a major part (major to be defined) of whose catchment area is contained within a SMSA, or some more meaningful measure of urban areas. This listing should include a description of data available and where it is available for each record. Also a statement of available rainfall and evaporation records should be included.

3.1 Long-term Research Needs

Perhaps the greatest need for research is the modeling of the quality of water in the urban environment. The least work has been done in this area, and, therefore, most work is needed.

Mathematical modeling is chosen as a tool of maximum utility, because it can be developed with a minimum of data. However, research is needed in the area of the stochastic simulation of input, for both rainfall and pollution loads so that mathematical models may be used to extend the database in time.

Little is known of the loss function in the urban environment. Complete quantification of this aspect of urban hydrology is impractical, because of both the inability to control man's artificial management of moisture conditions and the difficulty of modeling the soil moisture system in sufficient detail to maintain exact equivalence to the physical system. There is a need to understand better how to approximate the equations describing the loss function and yet maintain some degree of equivalence. There is need to define more accurately the magnitude and impact of interception and depression storage in the urban environment, and the relative importance of infiltration, e.g., under what condition can the fact that flow is from a pervious surface be disregarded in appraising peak flow requirements? Long-term field measurements may be required before detailed quantitative solutions are possible, but near-term approximations of practical value should result from sensitivity analyses of the various models available now or in the near future.

There is a need for research in the development of the objective functions to be served by urban models, and in the standardizing of criteria for testing model adequacy. Thus, there should be research both in the criteria used for fitting parameter values for a model, and in the methodology of fitting. In addition, the fitting of "effective" parameter values to a given model and the relation of such "effective" parameters to the physical parameters that they model should be investigated.

In a quantitative sense, more progress has been made in the area of hydrograph development and translation than in any other area of model development. For that very reason, however, the quantitative solutions are demanding as regards both data requirements and complexity of their analytic treatment. Research directed toward definition of the point of diminishing return per increment of analytic accuracy and toward more efficient computational processes is needed.

In closing, the standardization discussed herein does not imply that models will be standardized, but rather that means of judging models will be. An investigation of the utility of special purpose models for solution of particular problems would be helpful. More models might result, but the area of maximum utility for each model would be better known.

Chapter 9, Appendix C

FINDINGS OF THE DAMAGE AND STORAGE TASK COMMITTEE

September 1968
By The Methods of Analysis
Task Committee, OWRR Project

1.0 PREFACE

The OWRR project Task Committee on Damage and Storage studied requirements for assessment of storm drainage flood damage and explored alternatives to direct disposal of stormwater runoff, such as the utilization of recharge basins or other storage schemes.

The findings set forth herein were reached at two Committee meetings: January 15-16, 1968 at Lexington, Kentucky, and June 18, 1968 at Chapel Hill, North Carolina. Members attended either on their own time, or through the courtesy of their organization.

The objective of the research sponsored by OWRR is to provide guidelines for initiating and expanding a program of long-range studies in urban water problems. The objective of this Committee was to recommend research needed on urban storm drainage with regard to flood damage assessment and utilization of storage. Findings reported are confined to that objective, it being tacitly understood that closely related and dependent aspects of urban drainage research needs are being given necessary attention by others in the ASCE program.

Committee members' participation in the two meetings was under their own names. Although their involvement was possible only through the cooperation of their organizations, particular care was exercised to insure that meeting findings were community, or "sense of the meeting," views and that no direct quotation of policy of the members' organizations could be inferred. This was accomplished by the Program Director writing the meeting findings, under his name and mostly using his own wording, interpreting a majority consensus.

M.B. McPherson
Program Director

FINDINGS OF THE DAMAGE AND STORAGE TASK COMMITTEE

1.0 DEFINITIONS AND SCOPE

"Assessment of damages" in an urbanized region connotes evaluation of disbenefits from flooding.

All of the basic engineering methods for in-stream flood regulation are functionally applicable to urban flood mitigation works, but generally on a much smaller scale: acceleration of flows by canalization through threatened reaches with consequent reduction in stages; isolation of contiguous land in threatened reaches from flows by means of embankments, such as levees and flood walls; and 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 socio-administrative-legal controls to preclude some degree of damages, such as floodplain zoning, flood-proofing of structures, flood insurance and related schemes.

The storms which result in floods are universally conceded to be random events, the occurrence of which on natural streams are usually analyzed by application of statistical theories for extreme-value distributions. The resulting floods, their magnitude and frequency, are also evaluated in terms of levels of probability; although given a storm and the other conditions existent on a given watershed, the resulting flood can be determined in principle. Because damage is primarily related to the flood, damages are likewise evaluated with a sense of probability of occurrence. In establishing frequencies for different degrees of flooding, considerable weight is normally placed on the past history, or record, of floods at the location of concern. The contemporary absence of a satisfactory body of hydrologic and economic field data on urban storm drainage system floods constitutes a liability of monumental proportions in the assessment of those floods and their associated damages.

Federal flood mitigation programs relate to "navigable" streams and tributaries thereof; and municipal or other local government drainage areas, are designated "interior drainage" and not a responsibility of federal programs except that accommodation is provided for transmission of flows originating from interior drainage through or over federal works into natural streams. Federal flood protection works are located on public land or on public easements.

Urban drainage works are also built on public land, but much of the surface runoff carried by these works originates on private property; and there are direct or indirect connections between private and public drains. While serious attention is given to present and projected land-use in development of federal flood protection projects, the intimacy of private and public property and the very high density of private holdings make the consideration of land-use factors a more critical matter in planning and development of urban drainage facilities. Additionally, intangible damages such as inconvenience and nuisance are much more extensive than for stream flooding and generally recur more frequently.

Federal flood protection works are commonly associated with natural watercourses, whereas urban drainage systems predominantly comprise artificial channels and man-made storage chambers. Urban systems are also much more compact and contain a very dense multiplicity of components or units. These units are relatively on a micro-scale with the smallest entity being the drainage system for an individual building, e.g., serving a family residence.

Justification for flood protection works is usually based on a maximization of the value of damages obviated in terms of the cost of protection, over different levels of values versus costs, with each level associated with a given mean recurrence interval or frequency of flooding stage. Assessment of the value of urban damages foregone has always been frustrated by the presence of formidable intangibles, such as public inconvenience and interruption of commerce. Further, while claims may legitimately be made for reduction of loss of life by river protection works, this leverage would very rarely be appropriate in the justification of urban drainage facilities. Protection of the public health can be claimed for any flood protection afforded, but this, too, is an intangible difficult to evaluate.

The only basis for appraising urban drainage flooding damages commonly extant is indirectly via an analysis of recorded complaints made to responsible local government officials by residents and the operators of commercial and industrial enterprises. Complaint records have been found to be of tenuous validity because not all persons affected make a complaint and representativeness of the sample is usually suspect; and the records themselves are seldom structured in a manner appropriate for damage evaluation. Perhaps more important, local flooding is not infrequently the immediate result of a service inadequacy in the neighborhood, such as an improperly operating street inlet or a street inlet of inadequate capacity.

The only reliable way to assess the source of damage is to make on-site observations during the occurrence of flooding, but this is rarely feasible. Compounding this difficulty is the fact that basement flooding is mostly on private property and completely hidden from outside view. Flooding of yards is also on private property but often visible from public ways. Street and open stream flooding are the only types that can be readily observed, as opposed to access to private property or underground facilities, but here again the observations must be made at the time of flooding to be meaningful. Observation of flooding could be made by means of signals from field sensors, with telemetering of water stages to local or remote recorders. This is an objective sought by the ASCE program. Few systems of this type exist, and those that do sense stages in main sewers and not in the smaller drains where most water enters from the ground surface.

It appears safe to make the generalization that most urban flooding occurs at low or sag locales in the topography. This commonality forms one basis of need for coordinated study of water use and land-use in urban planning. The close relationship between land and water is probably more evident for drainage than for any other urban water service.

The Committee assumed that prevailing value-standards are points of reference, with a desirable national goal being economic growth. Undue flooding damages sustained would tend to inhibit economic growth. With economic growth will come greater intensification of urbanization, with sharpened conflicts over water use and development, and aggravation of flooding problems. In planning for optimum economic growth considerable information must be at hand for a definition of what is going on at present and as a basis for projections into the future; and in planning for development one mix of various levels of services would be balanced against another. All urban community functions and services are in need of re-examination, including urban drainage. While accepting the theme from the Engineering Foundation Research Conference of 1965 on 'Urban Hydrology" that stormwater is a "resource out of place," the investment in storm drainage facilities and other water services as compared with the total budget of an urban area is not at all evident. Before considering the possibilities for multiple-use of stormwater, a better understanding of its single-purpose importance should be attained.

Some idea of the scale of the problem may be appreciated from an example. Within the 200-square-miles of the City of Chicago (excluding O'Hare Airport) there are 3,600 miles of combined sewers. Practically all of the City population of 3,550,000 people is served by combined systems. Of the 3,600-mile total length, about 82% is equal to or less than 24-inch diameter, and about 90% is equal to or less than a 48-inch size.[1] Figure C-1 is a greatly reduced map showing all sewers in Chicago 24-inch and larger, a size range that includes only a small portion of the total.

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[1] (From "Study of Approximate Lengths and Sizes of Combined Sewers in Major Metropolitan Centers," ASCE Combined Sewer Separation Project Technical Memorandum No. 4, New York City, May 1, 1967).

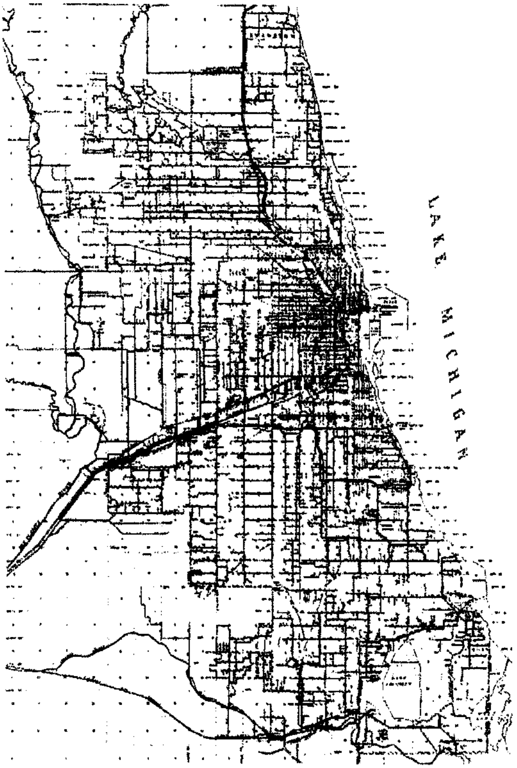

FIGURE C-1. City of Chicago, storm sewers 24-inch and larger.

2.0 BACKGROUND FOR RECOMMENDATIONS ON NEEDED RESEARCH

Because of the autonomous operation of urban governments and the absence of an overall national cohesive communication between these governments with regard to planning and development, very few useful generalizations can be made on virtually all facets of urban drainage. In order to invest more efficiently in drainage on a national scale, it is essential that some research be undertaken soon to determine contemporary practices and policies as a framework for seeking new and improved approaches. To some extent this will require documentation of the obvious, from the viewpoints of several specialties.

Compared with river flood mitigation, intangibles are more prominent in urban drainage. Costs of urban drainage are highly interrelated with those of other drainage facilities, such as for highways. Because of the very large number of properties involved, and their small individual size, evaluation of intangibles becomes a formidable difficulty; and there are a significant number of intangibles involved, e.g., the value of stormwater pollution abatement. More knowledge is needed if marginal values are to be discerned. Essentially, at present it is not possible to compare alternative plans for drainage on a reliable basis. Drainage facilities cannot be conveniently isolated for planning and analysis, as for water distribution systems, because the outflow from a catchment becomes an inflow of another. Flow from suburban areas often passed through sewers and open drainage canals of contiguous major cities. Land-use simply cannot be divorced from drainage considerations.

The philosophy in financing urban drainage facilities is for the community at large to share in the disproportionate cost of protection for low and sag areas within the community. This is fundamentally a subsidization in ameliorating the risks of a minority of cost-sharers, and commonly results in public displeasure on drainage referenda. No viable arrangement presently exists for a more palatable and/or more equitable basis for financing facilities.

Storm drainage damages are the consequence of storing water in the wrong places, e.g., in basements of homes rather than public watercourses. Hence, greater use of designed storage in lieu of undesirable storage depends for justification on relative protection from flooding afforded by different system plans at corresponding differences in cost. That is, trade-offs between benefits and disbenefits should be resolved by choosing among various mixes of flow-acceleration and storage components. Evaluation of damages is an appraisal of disbenefits and incorporation of storage induced benefits; and determination of damages infers the value of storage. For these reasons the Committee regarded damage and storage as dependent issues, and generally felt that, of the two, the question of damages was of more immediate concern that concepts of damage prevention provide the rationale for utilization of storage, and that in the immediate future relatively more will be gained by emphasis on damages rather than on storage.

The damage issue can be described in terms of simple questions (that have no simple answers at present):

1) What constitutes damage?
    a) Physical—what amount and kind is suffered at residences, by public utilities, by commerce and industry?
    b) Economic—who loses what?
    c) Health—how damaged, or is it simply a potential threat?
2) With respect to the community where a flood occurs and a community where the flood is routed, in terms of the following categories who benefits from damage control?
    a) Individuals.

   b) Commerce.
   c) Industry.
3) Of the following, what is their share of the cost for benefits?
   a) Those damaged.
   b) Whole community.
   c) Groups of communities.
4) What costs are associated with what levels of damage; and is damage protection worth the cost?

3.0 EVALUATION OF DAMAGES

Research is needed to determine: 1) damages sustained in sectors of existing communities; 2) increase in damages in existing communities as a consequence of growth of surrounding and other upstream communities; 3) the value of land appropriated in new areas for drainage; and 4) the nature of damages and ranges of damages on individual properties representing various categories of land use. That such damages are not inconsequential is evident from a field study made in a Chicago suburb where drainage flooding costs were found to be about $200,000 per year for the 5-square-mile study area. The magnitude of drainage costs for specific local cases should be appraised with a view towards ultimately arriving at an overall evaluation on a national scale, which in the ideal would be comparable to the scope covered by the Corps of Engineers on river flood damages.

Techniques set forth in the following reports could be adapted for damage surveys: *Choice of Adjustment to Floods*, by Gilbert F. White, Research Paper No. 93, 1964; and *Industrial Flood Losses: Damage Evaluation in the Lehigh Valley,* by Robert W. Kates, Research Paper No. 98, 1965; both issued under the name of the Department of Geography, University of Chicago. Here, as in most past studies oriented towards river flood damages, the rank of emphasis was firstly on industrial, secondly on commercial and lastly on residential damages; and this is the relative rank of estimate precision. This order of priority would have to be inverted for urban drainage application.

The Committee emphasized that even a rough first cut at evaluation of the scope and scale of urban drainage flooding damage would be much better than the purely subjective approach all too commonly used today.

Reduction of findings in terms of some simple parameters would be very desirable, such as damage cost versus volume of water from a storm stored on the catchment versus a land-use characterization versus some measure of frequency of occurrence. However, designation of a frequency of damages commonly requires particularized special definitions, and the urban case is at least as complicated as river flood damage evaluation. It is expected that each metropolitan area might have relatively unique characteristic "curves." Special thought should be given to methodologies, criteria and techniques for evaluating intangibles. Accounting should allow for biases introduced by public opinions and habits; for example, residents who keep all belongings off their basement floor are practicing an informal voluntary form of partial flood-proofing.

4.0 THE ROLE OF DRAINAGE IN LOCAL GOVERNMENT EXPENDITURES

The extent of capital investment in this country in storm drainage facilities has been suggested in *Urban Drainage Practices, Procedures, and Needs.*[2] Needed now is an evaluation of the role of storm drainage in the expenditures of urban governments. Initially, pilot or sample communities should be used, principally ones where suitable fiscal practices and records are maintained, and where adequate cooperation of public officials could be anticipated, but particularly where management people in charge are outstanding and sympathetic to the objectives sought. The underlying question is: just how important is the cost of urban drainage facilities in the overall urban budget? That is, does urban drainage compete intensively with allocations for schools, streets and other public services, or is the proportionate investment so small that, say, doubling the investment in drainage would hardly be felt?

Because proper accounting would include the costs for operation, maintenance, replacement, repair, etc., in some form of capitalization, a time scale of at least one decade of past records of expenditures should probably be used.

The analysis of investment proposed immediately above is addressed to urban drainage; however, it is intended that all services would be likewise isolated, particularly other water services such as water distribution, water treatment, sanitary sewerage, sewage treatment, etc.

Stated in a different way, the question is: where does consideration of the cost-benefit of urban drainage fit into an economic mode for a metropolitan area?

On the assumption that investment in urban drainage facilities has been intuitively gauged at a level for which the value of protection is at least as great as the cost of service, the study proposed would provide an indirect, first estimate of the cost or value of damage from flooding. Only with some indication such as this can an analysis of the cost-effectiveness of urban drainage be started; and justification for upgrading design, more elaborate planning and more intensive supportive research also hangs in the balance. At a lesser level, results of the study proposed would indicate to what extent a detailed evaluation of damage costs could be justified.

The group for undertaking the study proposed above should be an interdisciplinary team. A public works administrator is recommended as team captain, with the team including at least an economist, a sociologist, and engineer and a cost-accountant.

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[2] Poertner, H.G. December 1966. APWA Research Foundation, Special Report No. 31 (See Chapter II, *Investment in Urban Water Resources Development).*

The Committee realized fully that investment in urban drainage facilities was often partly in response to community tastes and desires, and protection and service has often been pragmatically assumed by public officials to be about right for their community.

The proposed study would help determine the relative validity of this position and set it in perspective.

As did other committees, an attempt was made to put the subject of discussion in its place among national, regional and local values and goals. It was recognized that many sociologic and psychologic factors need examination. For example, the public at large has an almost superstitious view of automobile accidents, feeling that an accident-free record is a probabilistic guarantee of immunity; and this attitude prevails to some extent with regard to floods. Of course, technical people have failed to differentiate between a mean recurrence interval or frequency, the one usually cited, and a specific recurrence interval or frequency, the one of more relevant concern.

Attitudes and values are deeply involved in some of the questions raised. What is the minimum in drainage facilities that could be lived with? Is there truly a potential for disaster with regard to public safety or public health? The central question is: what is justified in the way of drainage works; and as a corollary, what is needed to design effective, economically justified systems?

The proposed investment study would serve as a background for investigating these questions, but an additional background study should be pursued at about the same time to determine who pays for damages and their share of the total, including property owners, communities, states, insurance firms and the federal government. Recurring and non-recurring and direct and indirect damages should be considered.

Additionally, some indication should be revealed of the extent of flood-proofing and flood zoning practices and the kind of developments permitted in areas served by urban drainage systems.

A more specific delineation is needed of responsibilities of the property owner vis-a-vis the community and others for drainage flood protection. Concurrently, a better understanding is needed of what share of the costs is borne by the property owner, the community and possibly others. Out of these studies should come suggestions for alternative arrangements suitable for the future. It is recognized that division of payment may be dictated by local policies, physical conditions, economic situations and other peculiarities. Doubts have been repeatedly expressed about the suitability of local government departments in exclusive urban water management, rather than drainage districts or metropolitan planning and utilities entities. However, there is always the danger of a central authority setting standards and priorities that would be in the best regional interests but disastrous for small elements of the community.

One of the most valuable uses for data of the type sought in the USGS project is in the verification of analyses; and it is suggested that the least sophisticated types of devices proposed by the Data Devices Task Group of the USGS project be intensively used in future gauged areas where there is an outstanding potential for damage.

5.0 LEGAL, POLITICAL, JURISDICTIONAL AND PLANNING IMPLICATIONS

What are the legal, political, and jurisdictional roadblocks to effective action in environmental management? In many instances the operating base is already stratified, partly as a consequence of inadequate land-use planning. Federal support for local facilities is hinging more and more on satisfactory compatibility of new construction with a comprehensive regional plan. (For example, see "HUD Grants Hinge on Regional Planning," Engineering News-Record, July 18, 1968.) If storm drainage is not presently considered as a part of metropolitan plan requirements for federal support of facilities construction, the Committee feels it should be.

Research is needed to identify policy revisions that must be made to overcome legal, political and jurisdictional roadblocks. First, by means of on-site documentations of specific cases, the national scope and scale of policy decisions should be determined; but not so much to seek a "typical" case as to resolve ranges from sampling, say, about a dozen problem areas from each of about a dozen metropolitan areas. A knowledge of what is being done should lead to identification or development of improved procedures.

There are questions on the size and characteristics of a unit or module that can be efficiently used or should be used for planning; and for regulation, if different. The desirable unit size and/or type would also have implications in defining the preferred territorial extent of separable jurisdictions. However, as a concomitant, new bases for equity in cost-sharing should be developed or old bases refined, and these may influence selection of efficient jurisdictional service area size. Definitions on a regional scale may be possible, and at worst the range of differences from metropolis-to-metropolis would be defined. The question of efficient size of jurisdictional territory has profound implications in the justification of fragmented jurisdictional units versus special purpose jurisdictions versus the super-government approach. Simultaneously, the inverse question might be answered on the effect of drainage considerations in structuring overall legal and jurisdictional needs for metropolitan areas.

6.0 SOME ADDITIONAL PLANNING CONSIDERATIONS

In established urban areas it is very difficult to hold urban land out of development, such as for floodplain zoning. However, desirable types of land use should be promoted in planning or existing desirable land use types should be preserved in planning.

Existing systems, such as in core cities, are certainly important, but the needs for virgin systems in new developments are also of concern to the planner.

The objectives in development of private property under a profit motivation may conflict with interests and fiscal resources of the community. Acknowledging that the private developer is attempting to optimize his own profit, the use of land and the intensity and type of development can be planned in harmony with community hopes and goals. The objective of the urban land planner is to optimize the overall community investment.

Approximate relationships suitable for planning purposes have been obtained between sanitary sewage loads and housing density. While it is commonly stated that the extent of drainage facilities is not related to density of development, the proportion of land devoted to streets and other impervious surfaces has a bearing on the magnitude of flood peaks (see the expected variations in peak flows for different land-use classifications per *The Hydrology of Urban Runoff*, by A.L. Tholin and C.J. Keifer, Transactions, ASCE, Vol. 125, 1960), and some degree of dependence between extent of facilities and density of development may exist. Planners are beginning to ask such questions as: at what density of land-use are drainage facilities mandatory, and at what higher densities are more elaborate systems required or desirable?

Although the public is surely entitled to a choice among alternatives, private development is often privately financed and bankers should also be given some choice among alternatives. It is important that relationships be established between avoidance of various types of flooding damage and property value.

7.0 FIELD STUDY NEW TOWNS

Much more knowledge is needed on the implications of alternative drainage patterns on land use, as part of metropolitan planning capability. The challenge is particularly acute in new towns. Some sort of benefit-cost approach is needed to test alternatives, but damages are the vaguest component, as compared with a much greater existing capability with regard to damages in river floodplains.

There is simply not enough in-depth planning of the interior drainage in metropolitan areas.

The Committee recommends that drainage of existing new cities, such as Columbia and Reston, be studied as a field data source for use in the next generation of new towns. As an essential part of these field studies, the effects of pollution and the aging of lakes should be analyzed because of their importance in water recreation and other services emphasized in the design of new cities.

The Committee suggests that future new towns could serve as field laboratories by incorporating several different drainage designs, each incorporating different amounts of in-system storage. The alternative designs should be planned around the main drainage channel.

Nationally, local building regulations currently control the approaches that are used, and field laboratories are, therefore, generally best in new towns.

Builders would be vitally concerned with the results for drainage-planning research. Because housing developments usually grow in stages of around a hundred homes each, implications on stage development in new cities and on urban sprawl should be considered.

For field observation of flooding, it might be possible to take advantage of civil defense facilities, particularly those operated by universities and city departments.

8.0 EFFICACY OF EXISTING DRAINAGE FACILITIES

The Committee recommends that the best available rainfall-runoff mathematical models be applied to long-term history rainfall records and that estimated peak flows for specific catchments be computed, therefrom. The objective would be to gauge the effect of past storms on existing facilities, for the purposes of exploring mean and extreme occurrences of flooding and to see what modifications in drainage facilities design would be required for incremental improvements in flooding protection. Combined with estimated costs for losses from flooding (Section 3 above), such a study would lead to a first cut view of benefit-cost relations. Additionally, syndication would be obtained of the maximum damage that might have occurred had the existing facilities been subjected to the full history of recorded storms.

In the planning context, fully developed and raw or underdeveloped land should be regarded as totally different cases. As a reference or basing point, peak runoff rates for the history of storms should also be computed for pre-occupance, undisturbed conditions.

The recommended studies could be extended to include mixes of old catchment systems and various stages of new developments where the latter drain into the former and vice versa.

Of course, the studies would benefit from availability of a long record of data from a metropolitan rain gauge network, although there are only a few of these of any size in existence.

As the USGS initiates a program of field urban rainfall-runoff-quality research stations the data therefrom can be effectively used to upgrade and verify the findings from the proposed studies; but there is no excuse for postponing studies until potential verification is at hand.

A supplemental benefit from the proposed studies and the damage studies (Section 3) would accrue if examples were documented of applications where good planning has resulted in an efficient drainage plan.

9.0 ADDITIONAL ENGINEERING QUESTIONS

What happens when the design capacity of drainage works is exceeded? Is the result serious, only an annoyance, or what? What is the relation between damage incurred and loading of works beyond their design capacity?

The "little engineering" aspects of storm drainage works have a very important and vital place in flood prevention: street inlets; curb and street configurations; house, roof and yard drainage; etc. Although the ASCE program is giving considerable attention to urban drainage sewer systems, should not considerable research on these relatively mundane aspects be receiving serious research attention?

Should not a serious and intensive effort be made soon to assess the effectiveness of existing facilities, to spot probable causes of troubles or deficiencies?

Designs for drainage works are often detailed by technicians. Is use of a higher competence justified? More broadly, would there be obvious nationwide benefits in upgrading designers?

Should not transportation and accumulation of silt be taken into greater consideration, particularly with regard to maintenance?

10.0 STORAGE AS AN ALTERNATIVE

As noted earlier, urban drainage is designed to accommodate flow peaks, i.e., extremes rather than common occurrences. Artificial channels (sewers and drains) accelerate flows and reduce stages of water on the ground surface to levels less than would occur without these underground facilities. Provision of storage is an alternative to acceleration, and is a pertinent economic factor when its use would result in a reduction in cost for a drainage system.

Care must be taken in the assessment of storage as an alternative or supplement to artificial channels because the frequency effect of flooding can be modified by storage, and this is particularly important where the storage is used for recreation or stored water is reused or recycled for other services. The disturbance of the balance of natural storage above and below ground by new construction causes a completely new regime of flood frequencies.

Considering the many facets of the Chicago Tunnel Plan,[3] just about all aspects of urban life seem to be involved whenever multiple-purpose or multiple-use projects

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[3] Deep Tunnel Plan Proposed For the Chicago Area," *Civil Engineering*, ASCE, New York City, July, 1966, p. 87.

are undertaken. There are many possibilities for exploiting trade-offs by transferring peaking problems from one type of service to another in multipurpose development. After all, the smoothing of water variations is the underlying objective in water resources development and management. The incorporation of storage is inextricably tied to the total design of drainage systems.

11.0 POTENTIALS FOR STORAGE EXPLOITATION

The studies recommended in this part would complement and supplement the studies recommended in Sections 3 and 8, and to some degree those in Section 5. In fact. it would be very desirable to pursue these simultaneously, in a coordinated manner. Storage potentials of rivers for development of water supplies have been estimated by several organizations.[4] The acquisition of analagous estimates for sewered urban catchments is recommended by the Committee.

For the analyses, say, 5-minute interval data for the local USWB station over a 20-year period might be used. From mass curves of accumulated precipitation and accumulated runoff the maximum basin storage could be identified. Surface storage could be segregated by deducting assumed infiltration volumes. Analyses should be run for both pre-development and current development. It is hoped that some empirical relationships would thus be obtained between maximum available system storage and the cost of drainage (installed cost and present worth).

As a part of the above storage study it would be desirable to make a series of designs of alternative storage capacities for each catchment. The cost-storage relationships obtained would be extremely useful, when tied with the earlier recommended damage-storage study results, for planning and development of drainage facilities.

Concomitantly, an analysis should be made of existing practice in providing localized storage, such as on roofs, in streets, at intersections, etc. This analysis should lead to a better understanding of the potential value of such storage.

The recommendations thus far in this part are oriented only towards single-purpose use. A reliable basis for multiple-purpose planning must be developed; however, until the recommended studies are undertaken from the single-use standpoint there will be no intelligent reference for multiple-purpose analysis. It is generally conceded that inferred marginal investments in the optimization of multiple-purpose water resources projects can be largely a reflection of imprecisions in costs and physical quantities, unless reasonable care is exercised in arriving at the estimates for each service component.

Because of concern for the quality of our environment, at some early stage in the proposed research an attempt should be made to relate stormwater quality to cost-damage-storage.

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[4] "Storage Requirements for Water in the U.S.," by Lőf and Hardison, *Water Resources Research*, Volume 2. No. 3, 1966; or RFF reprint No. 61.

The work recommended in this part, as well as Sections 4 and 8, require the use of the best available mathematical models of rainfall-runoff. Although elaborate or sophisticated models are considered necessary for evaluation of rainfall-runoff-quality processes (Methods of Analysis Task Committee objective), the studies recommended here could use much simpler models; and the exercise of these simpler models might well lead to insights for simplifying more complex models. Simplicity in modeling for the present purpose appears appropriate because dollar values of damage and storage would at best be only approximate.

As in Sections 3 and 8, the initiation of USGS rainfall-runoff-quality field stations in sewered catchments would provide data for verification and refinement of the results from the studies here recommended.

One approach to urban flood control is to zone catchments on the basis of land use to a certain defined maximum flow rate per acre. This approach implies the use of storage whenever conventional drainage would have excessive outlet capacity flow rates. However, reliable application of this concept on a metropolitan basis would require first undertaking the type of research proposed above.

Study areas could be categorized on the basis of rainfall intensities and topography, the two major factors affecting the volume of water that can be effectively stored.

12.0 CONCLUDING REMARKS

APWA Special Report No. 31 of December, 1966, page 4, notes that "the annual national investment in urban drainage facilities exceeds that of the Federal Flood Control Program." This would seem to suggest that at least as much investment on practical urban drainage research is justified as that on flood control by the Corps of Engineers. The almost complete neglect of urban drainage research in the past makes the commencement of a program particularly difficult. It should be noted, however, that the studies recommended in this Appendix are of very modest scale in terms of their potential value.

Chapter 9, Appendix D

FINDINGS OF THE NON-HYDROLOGIC ASPECTS TASK COMMITTEE

September 1968

PREFACE

The OWRK project Task Committee on Non-Hydrologic Aspects studied political, economic, legal and social problems related to urban water management.

The findings set forth herein were reached at three committee meetings: December 7-8, 1967 in Washington, DC, February 29, 1968 in San Antonio, Texas; and August 14, 1968 in Andover, New Hampshire. Members attended either on their own time or through the courtesy of their organization.

The objective of the research sponsored by OWRB, is to provide guidelines for initiating and expanding a program of long-range studies in urban water problems. The objective of this committee was to recommend research needed beyond the purview of usual engineering practice related to management. However, some overlap with engineering was inevitable because of the broad subject scope. In alphabetical order of surname, the members' professional disciplines are; sociology, law, civil engineering, political science, economics, civil engineering, psychology and civil engineering.

Participation by Committee members in the three meetings was under their own names. Although their involvement was possible only through the cooperation of their organizations, particular care was exercised to insure that meeting findings were community, or "sense of the meeting," views and that no direct quotation of policy of the members' organizations could be inferred. This was accomplished by the program director writing the meeting findings under his name and mostly using his own wording, interpreting a majority consensus.

Appendix E, on potentials for collaboration between engineers and social scientists on water resources research, should be regarded as an extension to the findings in this appendix.

M.B. McPherson
Program Director

FINDINGS OF THE NON-HYDROLOGIC ASPECTS TASK COMMITTEE

1.0 DEFINITIONS AND SCOPE

In alphabetical order, the principal, predominantly non-hydrologic aspects of urban water are:

- Administration of works;
- Economics of planning and operation;
- Esthetics of urban environment and specific works;
- Financing of works
- Health and Safety of population served;
- Legal latitudes and constraints;
- Planning and Land Use, metropolitan-wide;
- Political impacts on planning and implementation of works;
- Recreational facilities planning and operation; and
- Sociological inferences between a viable metropolis and development of its water resources.

The above listing is based on the assumption that development of future services, including water, will tend towards a metropolitan region scale.

Committee deliberations were largely confined to long-term research needs, consistent with the objective of the OWRR project. However, it is evident that many immediate, critical social problems must have a high priority for attention. A section of this Appendix is devoted to immediate needs, as defined at the 1968 Engineering Foundation Research Conference on *Water and Metropolitan Man*, melded with Committee recommendations. Similarly, Conference positions on institutions for management of water are co-joined with Committee views. There is a natural blending in both instances because Committee members were heavily involved in the entire Conference proceedings.

Specialized questions, considerations and recommendations are grouped in separate sections towards the back of this Appendix, to minimize distraction in the presentations on major themes.

2.0 BACKGROUND FOR RECOMMENDATIONS ON NEEDED RESEARCH

Common practice in designing urban water control structures is to commence with a quantitative mastery of prevailing field conditions, defining and evaluating these in detail, projecting public needs for up to 50 years hence, and then proceeding with planning and design of auxiliary, modified and new structures on a progressive program of construction in which time-varying needs can be accommodated in an orderly, economical schedule. In principle, one progresses from that about which most is known, the present, to that about which least is known, the future. A good design will permit tactical flexibility via choices among alternatives for future stages of construction, while optimizing public services within prevailing and projected project constraints. Imaginative staging of public works provides flexibility to take advantage of technological advances as they develop even when economies of scale might indicate otherwise.

Essentially the same principles used in design are used by urban water use and land-use planners, except that their scale of reference is generally much larger and specifics are considered in much less detail. From this point of view, research needs of urban planners, urban regional economists, design engineers and public works administrators differ mostly with regard to depth or detail but should differ little in terms of overall scope or comprehensiveness.

The Committee was of the unanimous opinion that immediate needs for facilitating research are not so much for more analytical tools but for more effective use of those available. Rather, the greatest needs are for: a) more information and data of a basic nature; and b) better interdisciplinary communication of analytical developments in the several aspects of urban water resources and interfacial subjects.

3.0 COORDINATED RESEARCH MANAGEMENT

For effective research on urban water resources a problem-directed team approach is an absolute necessity. However, there is an underlying dilemma: comprehensive or integrated research, planning or design (or a systems approach, if you prefer) requires a bigness in effort and cost which greatly complicates the staffing, financing and functioning of such teams as compared with requirements for single-purpose, limited-scope projects traditional in the past.

Social scientists are arriving at capabilities for quantifiable evaluations. Much is being done or being developed in the social sciences in projecting future expectations which should be translated or converted for use in economic and engineering analyses and planning. "Continuing education" programs could contribute here, including exposures of newer developments in Interdisciplinary seminars. An *a priori* condition is the necessity for clear definitions of vocabularies, nomenclature and terminology mutually understandable by urban water engineers, social scientists and other planning team representatives. It would be extremely helpful if civil engineers took the initiative and prepared texts or primers on urban water resources as references for non-engineers.

Several axiomatic truisms must be cited to further set the stage for the first major recommendations of the Committee. In planning, one must make allowance for continual, dynamic, changing conditions. Problems and crises force changes. Man continues to try to control his environment lest he be at the mercy of his environment. One must anticipate effects of regulatory change, such as revisions in standards for water quality and developments in technology, on specifications for analyses. Advances in technology more often result as the consequence of declarations of national goals, although the reverse may have been the case in the past. The validity of current social values and how they will change or be changed is a subject of current public debate. The attitudes of individuals and communities have a great bearing on what is being done, what should be done and what can be done. One of the major contributions in this regard can be the body of research methods in sociology, rather than sociologic theory, such as opinion research. Public action programs are geared to exploit or enhance public attitudes and opinions. The linkage or dependency or interaction of public services of all types with our social structure is not well understood; e.g., the role of urban water in reviving a declining community or neighborhood is not clear. There even appears to be a lack of definition of what goals urban area planning should have.

What will be the mutual effect of the physical structure and growth potential of the cities of the future? How do you adapt a plan for meeting the future to insure flexibility and room for individual tastes and choices? How does the engineer insure that he is providing proper inputs for overall planning? Should decisions on implementing plans be based on a cost-benefit approach entirely, only in part, or what?

In consideration of the preceding views and observations, the Committee recommends that at this stage the federal establishment should initiate a move to bring about the creation of an appropriate entity for urban water resources research; items of concern and definitions of what needs to be done that have been provided by this Committee should constitute a part of that research; and this entity should probably provide a translating function for effective communication between affected fields, particularly for interdisciplinary, multidisciplinary and multiple-service communication.

4.0 EVALUATION OF BENEFITS

As noted in the preceding sections, sooner or later bases must be established for presenting the public with alternatives among scales of services and their mixes, in terms of levels of required investment. An enigma arises because substantial capital facilities often must be constructed and paid for much in advance of full realization of service benefits, even in a well-planned staged development.

In recognition of the above, and in consideration of the availability of analytical tools that go begging for lack of suitable detailed inputs and boundary condition definitions, the Committee recommends that: more detailed information on costs and benefits of different scales of facilities is needed to arrive at equitable charge schedules and judicious selection among alternatives.

In his presentation of the overall summary on August 16, 1968 of the Engineering Foundation Research Conference on *Water and Metropolitan Man*, General William Whipple, Jr. (Rutgers University) stated:

> *In all of the emphasis on methodologies of systems analysis there was relatively little discussion of economic principles to guide such analysis. Dr. Charles Howe (Resources for the Future) spoke of the desirability of further increasing our ability to measure what formerly were considered intangibles. In particular, he stated that it is desirable to measure the economic benefits of water quality.*
>
> *This point is of enormous importance. Water quality is undoubtedly the primary problem of most metropolitan areas, and unless its benefits are measured, the systems analysis must incorporate an assumption rather than an objectively determined value, and the entire optimization process will be largely deprived of its validity.*

5.0 METROPOLITAN PLANNING

This section is centered around regional planning, with emphasis on institutional frameworks within which water resources planning takes place and the need for adequate provision to take public desires and needs properly into account in the planning process.

Most states are setting up state water planning boards. The board in Texas has divided that state into 26 metropolitan areas, which are not necessarily compatible with river basin boundaries. Conflict between plans of local and regional jurisdictions complicate metropolitan planning. Such boards work from the state level, not from a grass-roots level. Directions and goals will be influenced by the then incumbent governor and legislature, and there may not be a continuity in policy from one administration to another.

Implementation of state planning boards is seemingly in response to new federal philosophies requiring that federally supported intrastate facilities be in harmony with a regional plan. While there has been a general accommodation to this philosophy, there appear to be weak or missing linkages with other state planning groups. The reorientation in planning is affecting and changing the structure of state government.

A good plan must be responsive to social, economic and political goals, approaching as closely as possible to an optimum resolution of public needs and desires. These criteria bear different attire when regarded in turn at the national, state and local level. Further, public goals, needs and desires are currently undergoing massive, restive, dynamic change. Present planning arrangements do not appear to be satisfactorily responsive to dynamic processes and changing social values. Definitions of the "public interest" seem to be nebulous and intractable.

Fundamental problems in planning involve an assurance of the preservation or upgrading of environmental quality at the local level, in a scheme of implementation compatible with the responsibilities and means of the several levels of government. While the systems approach is being rather universally adopted, the scope or framework of a system is defined by its designer. To reiterate, the structure of a local water resource system would differ in significant detail from that of a national system.

Dissatisfaction by the public at large with its role in the planning process is becoming increasingly more evident. "We must begin to think in entirely new terms of citizen involvement and decision-making." While this remark was made in connection with civil rights, it could be applied equally well to any facet of the planning process. [1]

Hard-core resistance and strong support groups of people sometimes are not recognized by public administrators. Administrators appear to have a proclivity to "tell" rather than "sell" ideas and plans. Yet, power structures, social action programs, diffusion processes and related factors have been studied and can be reasonably well defined. Some types of public behavior or reactions can be reasonably well anticipated. There are methods for assessing receptivity to change. Research is needed on communication methodology. Individual disciplines tend to work from their own criteria and these must be balanced out in any interdisciplinary approach, for example economic efficiency versus social values.

The benefit-cost approach seems to contain some liabilities in accounting for all relevant factors in a systems orientation in planning. In proceeding through the echelons of administration, such as through the Corps of Engineers, a modulation or distortion often occurs in communication and public response. The engineer should always be given the benefit of public needs and desires prior to the design of a project.

Patterns of development in some instances seem to be steered to some extent by the course of water resource developments. Such cases should be overtly, not covertly, recognized. Rather, it would seem that services should be consistent with and guided by an overall community plan. However, there will always be competition between water resource development and other community services, such as schools. It is also recognized that because initiation of projects for change commonly arises from prevailing or impending crises or conflicts, maintenance of equity may be extremely difficult. Decisions on engineering works are often made in response to the vocal public sector, which is not necessarily representative; and while this sector used to represent largely the well-to-do, the other end of the income scale is now in the forefront.

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[1] Hamilton, Charles V. April 14, 1968. *An Advocate of Black Power Defines It*, by, The New York Times Magazine.

The scope in "VI. Water Resources Planning" of "A Ten-Year Program of Federal Water Resources Research," Office of Science and Technology, February, 1966, is too confining and should be broadened.

Facilities for utilizing water resources are usually fixed, permanent installations, paid for by commitment of future revenue. Water resource development is all too commonly an irreversible process, with opportunities foregone being irretrievable. With the growing competition for water and the exacerbating complexity of providing public services, the needs for research become more critical. Water is increasingly a greater component of political power, which should enhance interest in the support of research.

Research is needed on the planning process with regard to meeting overall community goals; and socio-system simulation should be incorporated in the research.

Ways must be found for the basic levels of government to do better planning, including all factors, in proceeding from goal to goal, while avoiding a frozen or fixed plan too early in the planning process, with due attention being given to politics, infrastructure and related matters. The preferred institutional framework within which metropolitan water resources planning should take place remains to be identified.

With respect to environmental quality (e.g., preservation of historic landmarks, irreplaceable trees, scenic sites, noise abatement, etc.), if research is not now in progress on the efficacy and efficiency of mechanisms for measuring public attitudes and opinions, such as public hearings, it should be undertaken promptly. Additionally, augmentation of methods for getting ideas and goals to the public by mechanisms exercisable in advance of planning, and supplementary mechanisms, should be studied.

It is suggested that efficacy and efficiency of decision-making processes, at all levels, be analyzed. Start with a classification system to insure thorough coverage. Include consideration of ways in which different levels of government relate in the decision-making process. Propose new or revised organizations and managerial procedures for better planning and management. Develop new measures of relative value for the intangibles of environmental quality. Urban water planning must be highly coordinated with all other aspects of urban planning. It is important to ascertain how metropolitan planning fits in with other levels of planning. Research on the scale needed would be beyond the capacity of a single university.

The above recommendations relate to only part of the larger problem of optimum organizational structure for carrying out the mission of overall urban planning, including institutional frameworks of urban government and financing.

6.0 INSTITUTIONS FOR RESOURCE MANAGEMENT

In his presentation of the overall summary of the Engineering Foundation Research Conference on *Water and Metropolitan Man* on August 16, 1968, General William Whipple, Jr. (Rutgers University), noted:

> *The metropolitan area outside of a large city (including the watersheds that supply that city) is normally constituted of numerous towns and townships (or corresponding subdivisions) within which the responsibility for water supply, storm drainage, water quality control and land use planning is fragmented to a very large degree. In the disposition of wastes, optimum plans of disposal seldom can be adopted either because an optimum layout of works should extend across subdivision boundaries, or because the favorable effects of a high standard of treatment accrues to subdivisions other than those required to pay for them. Water supply is similarly hampered. As Dr. Leonard Dworsky (Office of Science and Technology) said at this conference, within the last generation, school districts have been consolidated across the country, in order to allow organization of more effective central schools. Perhaps a similar process will occur for certain aspects of water supply and waste disposal. An engineering forum is certainly not the best place to recommend the fundaments reorganization of our system of government. But it seems entirely suitable to recommend research into the present institutional handicaps to proper water resource development in metropolitan areas, and to study how, within larger concepts of government, progress can be made to remove these handicaps.*

The Conference "Legal and Institutional" Work Group[2] recommended four background research projects: "the identification of water uses or programs that might be within the prerogative of a model water resources institution and the identification of legitimate interests and responsibilities of other agencies; a critical analysis by case study of existing water resources institutions, seeking an understanding of their strengths and weaknesses; a critical analysis by case studies of existing law as it relates to water resources institutions; and a critical review of the financing of facilities and operations as related to water uses and programs."

The Non-Hydrologic Aspects Task Committee makes the following recommendation: research is needed on institutions for management of water, at various levels, extending from an entire metropolis to the local neighborhood. However, in areas of impending urbanization the scale of involvement extends to river basins and even larger regions, where conflicting and overlapping jurisdictions are even more numerous.

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[2] E.M. Laursen—University of Arizona; G.F. Mangan—OWRR; J. Shoemaker—Attorney at Law; and J.B. Stall—Illinois State Water Survey.

7.0 CRITICAL SOCIAL PROBLEMS

In exploring linkages between newer poverty problems and water resources development, the "Social Impact" Work Group[3] of the 1968 Engineering Foundation Research Conference on *Water and Metropolitan Man* perceived the following social sub-groups to have problems of highest priority: "inner-city poverty groups, occupational groups of high unemployment that may be particularly susceptible to water related employment, and educational levels which might benefit particularly from the employment and training effects of water projects. "

The Work Group recommended studies which were felt would have a potential for a high return in a relatively short time. The objective of the studies proposed by the Work Group would be to evaluate quantitatively and qualitatively the impacts that might be attained from modification and exploitation of the stated concepts, not the concepts themselves:

1) Physical engineering undertakings:
    a) Ways of utilizing established water distribution systems more fully to provide recreational and other social benefits, e.g., attaching sprinklers to fire hydrants;
    b) Storm drainage layout criteria for recreation, open space and esthetics;
    c) Street cleaning and sanitation;
    d) Public multi-purpose water use, e.g., street pools;
    e) Esthetic improvement of present channels, streams, etc.; and
    f) Reuse of urban waters, including treatment of stormwater for recreation.
2) Institutional-Organizational changes:
    a) Possibility of broadening engineering criteria and outlook to incorporate social objectives, which should include restudy of arbitrary engineering design standards;
    b) Pricing of water, in terms of both cost related to use and ability to pay, e.g., a different price structure in ghetto areas;
    c) Housing codes and public controls over private activity;
    d) Participation in decision-making process by power structures and under-involved groups;
    e) Weather modification, intentional and inadvertent; and
    f) Translation of new engineering ideas into the planning process, e.g., through case studies and demonstration projects.

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[3] P. Bock, Chairman—Travelers Research Center; C.W. Howe, Reporter—Resources for the Future; W.J. Schneider—USGS; D. Van Sickle—Consulting Engineer; J.H. Woods, Jr.—TRW Systems; and W. Viessman, Jr.—University of Maine.

The Work Group recommended: that the above studies be undertaken soon as case studies and via demonstration projects, "with a sense of urgency to meet expressed national goals of improving the urban environment; that the program be on-going with provision for follow-up on the initial projects; and that these procedures seem justified in terms of the high potential returns and low cost relative to existing programs of technical research."

Supplementary views of the Non-Hydrologic Aspects Task Committee occupy the remainder of this section.

With respect to the impoverished, not much is known in the social sciences on how they communicate or how to communicate with them. Public services are not necessarily distributed on a per capita basis and in total the poor may be grossly neglected. Results of research on changes in rural areas may be useful as guides in research on urban poverty.

The Committee feels very strongly that serious study should be given to the effect in the past of technological change on social change, social change on technological change, and the relationships between them.

Lastly, ways must be found to measure or evaluate social efficiency (social values). Present methodologies are crude, but they must be extended and improved upon. The policy or views of the public manager on social values is a very important factor.

8.0 SOME CONSIDERATIONS IN ADMINISTRATION AND FINANCING

The scope of administrative authority for planning public services should be studied, particularly for water or the role of water. What is the most efficient size for such an administrative body, and what should be its scope of responsibility?

What are the specific responsibilities of local and state governments and the federal government in urban water, legally, sociologically, economically, administratively, etc.?

In the planning for development of urban water resources, when should the public be brought in, and to what extent?

What are the proper, best methods for communicating technical facts to the public, such as alternatives including the cost of disaster, enhanced benefits of multiple-use developments and goals of long-range plans? Essentially, what is required to have an "informed public"? How much information should be presented to the public for their response to be adequate, both for guiding the general direction of planning and evaluating summary findings from that planning? Intuitively, the public exhibits a mood of greater concern and involvement when it is confronted with alternative choices than when the choice is between a single mode of action and the apathy of complete inaction.

A number of factors result in a conflict between investment in urban capital programs versus deferral of capital improvements by using larger operating budgets, which affects the staging of projects and assertion of economic values, and the choice is influenced by political considerations. Financing restraints and practices should be carefully studied with a view towards arriving at improved policies for more efficient investments within the total of urban budgets.

Detailed information is needed on the bases for charging for various water services, particularly with regard to who pays for what part of each service, for the purpose of determining what should be done. In new developments, the vehicle for charges is generally the requirements placed on developers.

9.0 SOME LEGAL QUESTIONS

What are the legal implications of water re-use (such as for water supply) and consumptive water use (such as for irrigation)?

In what ways do national public laws (such as on low-flow augmentation storage and water supply storage in federal impoundments) and the policies or public agencies (such as on federal-state cooperative data acquisition programs) constrain, direct, enhance, inhibit, benefit, etc., urban water resource development by local governments?

What legal constraints exist in urban law on development of future cities? What conflicts might arise between existing law and plans for the future? How might existing law inhibit such planning? What are advantages of fee-title versus easements for public services? What legal constraints on construction result from prevailing bonding practices and procedures?

On the premise that legal research emphasizes case law rather than the development of law, how can the legal profession be induced to enter into research on water law, in an interdisciplinary, interfacial way, with some emphasis on national needs as a whole while augmenting current research at the state level?

10.0 SOME PLANNING CONSIDERATIONS

To what extent does enforcement of state and federal standards, such as on water quality, affect urban regional planning, or how much should effects of enforcement be considered in planning?

The tendency in urban planning seems to be towards consolidation of commercial, industrial, residential and other land-use types in segregated enclaves of urban centers. Large industries currently tend to disperse and diversify their activities. What are the advantages and disadvantages of consolidation and dispersion and what are the mutual effects on urban water resource development?

Are civil defense needs being incorporated in urban planning schemes, and if so, to what extent are public water and water services involved?

Perhaps present development control via the relative permanence of zoning should be at least partly replaced by patterns for growth taking into account noise, odors, occupancy density, concentration of wastes, etc. This would require development of performance criteria for the man-environment interface.

11.0 SOME TECHNOLOGICAL ASPECTS

A 'blue-sky' study should be made of prospects for technological changes and trends, such as in desalination, internal recycling of individual building water supplies, "tertiary" waste treatment, augmentation of water supplies using storm drainage runoff, etc.

Assuming new technology in urban hydrology will evolve in the next several years, what will be its socio-economic impact?

Total water use studies are needed. Too little is known compared with the scale of development on water demands, storm runoff and sanitary sewage (separately, groundwater flow and surface water illegally or inadvertently added to sanitary sewage).

It appears that a great deal of work could fruitfully be done to study the total waste loads of urban areas and the possible tradeoffs between solid, liquid borne and gaseous wastes. It would seem that design of waterborne waste conveyance and disposal systems should depend upon the costs of other modes and the possibilities for transforming waste into other forms (e.g., solid into gaseous, gaseous into waterborne, solid into waterborne by grinding, etc.).

When acquiring data of any kind, consideration should be given to the use of the data in various types of sensitivity analyses. Additionally, one should always strive for an adequate national sampling, if findings of research are to have an adequate potential transferability.

Exchange of engineering design tools (such as computer programs) should be facilitated by augmenting extant formal presentations at technical meetings or in technical journals.

12.0 DRAINAGE

Prevailing relative costs of drainage, land, rights-of-way, etc., should be sampled sufficiently to determine present division of investment in drainage; and this assessment should be extended to other water services.

What are the legal potentials and liabilities of rights-of-way for urban drainage? What are the legal aspects in selection of acceptable drainage outlets or outfalls? What new legislation is needed for good planning and economic evaluation?

From the *Report of the Engineering Foundation Research Conference* on "Urban Hydrology," Andover, New Hampshire, August 9-13, 1965: "Legal considerations need research to develop solutions for the lack of uniformity in existing state law, which is divided and often mingled among common, civil and 'reasonable use' rules. The law can provide means for minimizing conflicts by: a uniform drainage code; a uniform trial procedure; and enabling legislation for effective organizing, governing, administration and financing of drainage districts." Fundamentally, a model law or uniform code based on drainage and ramifications of drainage is needed. The greatest problem in this area of law is the question of liability, which hinges on identification of who is responsible in a given instance.

What are the legal and economic considerations in choosing between drainage by canalization or closed conduits and between a surface or groundwater supply?

What is the relation between side slope steepness of artificial open channels and danger of an attractive hazard from the standpoint of public safety?

The measurement of damages from flooding and their correlation with runoff characteristics is quite important and can perhaps most economically be done in conjunction with programs of instrumentation on future study areas.

Urban Drainage Practices, Procedures, and Needs, by H.G. Poertner, R.L. Anderson, and R.W. Wolf, APWA Research Foundation Project 119, Special Report No. 31, Chicago, Illinois, December, 1966, contained a listing of some of the needs for studies and research in urban drainage recognized by the American Public Works Association. The list, less statements of amplification on some items therein, is reproduced below because it substantiates and augments ASCE program recommendations:

1) A program of research into the area of benefits and cost sharing between:
    a) State and local governments;
    b) Urban and suburban communities; and
    c) Private land developers and local public agencies.
2) A study of the desirability of and means by which the activities of the public agencies responsible for urban drainage, particularly in large metropolitan areas, can be better coordinated with other utility operations on a regional basis.
3) A study of the feasibility and means by which the many public agencies responsible for urban drainage in large metropolitan areas could coordinate

their activities on a regional basis to discharge their responsibilities in an integrated and perhaps more efficient and economical manner.

4) A comprehensive program of research into the area of eliminating or reducing pollution from overflows of combined sewers.
5) A study of the comparative costs and benefits of developing multi-purpose impoundments to control runoff, versus the loss of land to floodplains when no such impoundments are provided.
6) Studies conducted by local public agencies and drainage districts to develop long-range master plans for urban drainage control.
7) A program of research into the design and construction of storm drainage systems to assist in controlling groundwater tables.
8) A research into the value of storm drainage systems to control erosion and to stabilize land against slippage.
9) A study of alternative methods of financing storm drainage improvements.
10) An analysis and summary of federal aid financing programs.
11) An analysis and summary of the advantages of accumulating statistics on rainfall patterns and the hydrology of streams.
12) Studies of the multiple use of floodplains for recreation and other purposes.
13) A summary of state-enabling legislation establishing drainage districts and authorities.
14) An analysis of master plans prepared and adopted in many communities for drainage control with a summary of the major considerations which must be included in the preparation of a good master plan.
15) A compilation and analysis of typical drainage control ordinances of counties, cities, and towns including the preparation of model ordinances which may be applicable to the needs of various levels of governments and agencies.
16) A compilation and analysis of erosion and siltation control ordinances including the preparation of model ordinances.
17) A compilation and analysis of typical zoning ordinances as they relate to floodplains and drainage. This would be followed by recommendations of factors to be considered in the preparation of such ordinances.
18) A study of methods used in metropolitan areas to coordinate plans and programs for storm drainage between the various public agencies of the metropolitan area.
19) A study of the safety hazards of open drains and structures conveying runoff into closed drainage systems.

13.0 WATER WORKS

Water works research needs are considered in Appendix F, and are complemented and supplemented by suggestions of the Non-Hydrologic Aspects Task Committee in this section. No attempt has been made to assign priorities among items.

The cost of water shortages of various frequencies in urban areas needs research (the recent drought in the Northeast, although an example, was studied on a regional basis). The economics of allocations during a water shortage deserve close scrutiny. Who should absorb the costs for emergency or remedial measures for meeting shortages?

In anticipation of increased costs for water treatment, the cost attractiveness of existing dual water supply systems should be evaluated and research should be undertaken on the distribution of multiple qualities of water within a common service district or system.

Effects on demand resulting from a citywide, short-term conversion from a non-metered to a metered status in Denver are being investigated at the University of Colorado. A related economic study was undertaken by Resources for the Future in 1966. Research on the effect of metering on water rate (rent) levels and water demand levels should be continued and expanded.

Variations in flow/demand at major drawoffs in arterial water distribution service-district networks have never been synchronously determined. Field research could contribute towards more reliable design and would facilitate implementation of automatic operational control in a pioneer installation.

A state-of-the-art study is needed on what is known about water demands for the purpose of improving design by taking demand patterns more into account. Different kinds of data are used for different purposes: peak rates over short durations govern sizing of distribution systems, whereas longer-term swings are critical for sizing treatment plants and major storage facilities. There are acknowledged gaps in available information. Investment in water storage and distribution could be reduced if these facilities were subjected to lesser peak load. Operational or other changes and/or economic controls could be exercised to even out extremes. Particularly, present fire service criteria should be revaluated and lawn-sprinkling practices should be categorized.

Vitally needed research on water demands was initiated at The Johns Hopkins University, with emphasis on residential use.[4] However, there appears to be a lack of adequate data on commercial and industrial demands, needed for projecting development of commercial and industrial areas and as a consideration in rate setting.

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[4] Linaweaver, F.P. Jr., J.C. Geyer and J.B. Wolff. March 1967. "Summary Report on the Residential Water Use Research Project," by, *Journal AWWA*, Volume 59, No.3.

The "Census of Manufacturers" contains some good, useful data on water use by industries that have not been fully exploited.

The Johns Hopkins Residential Water Use Project served greatly to improve criteria for sizing system components but contained the basic weakness of ignoring the frequency with which extreme demands occur. Perhaps work currently under way on simulating and projecting urban water demand patterns will lead to more appropriate design criteria.[5] Utilization of harmonic analysis,[6] multivariate analysis and analysis of power spectra could lead to transferable indices of demand variations, at least on a regional basis.

Independent concurrence in some of the preceding recommendations is indicated in the following paragraph, from the annual report of the AWWA Research Committee on Distribution Systems[7] of June 12, 1967:

> *Of paramount concern to this committee is the upgrading of design, analysis, planning and operation of water distribution systems. The general consensus is that our most urgent need is for more information on loadings: variations at a point with time; variations of multiple points within a system with time; variations of demands for different classes of users; applicability of findings from one system when used for another (transferability within a particular climatic sector of the country); evaluation of levels of probability of occurrence for extremes in isolatable categories of demand and total demand; and related considerations.*

Lastly, a rational structure of water rates needs to be developed. Decisions underlying the establishment of rate structures are not necessarily responsive to the welfare and budget of the individual user. As a reference for decisions, there should be available an indication of costs, reliability, penalties and related information for various levels of services in terms of several alternative physical plans. For example, it is not clear to what extent metering currently constrains demand in response to price.

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[5] "Main I, A System of Computerized Models for Calculating and Evaluating Municipal Water Requirements," by Hittman Associates, Inc., Columbia, Maryland, June 1968.

[6] "Analysis of Distribution Demand Variations", by Gordon Gracie, *Journal AWWA*, Volume 58. No. 1. January 1966.

[7] E.E. Bolls, Jr.; D.A. Brock; E.B. Cobb; H.A. Cornell; J.E. Flack; F. Holden; F.P. Linaweaver, Jr.; M.B. McPherson, Chairman; R.C. McWhinnie; J.C. Neill; and R.U. Olson.

Chapter 9, Appendix E

THE WATER RESOURCES ENGINEER AND SOCIAL RESEARCH: POTENITALS FOR COLLABORATION

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1.0 PREFACE

It is indeed a sobering experience for practicing social scientists to assess methods, models, instruments, and yields from social research for their utility to scientists in other fields. Such assessments should be made because interdisciplinary collaboration requires it. Moreover, we are also convinced that collaborative efforts are required for scientific and technological progress. It is in this spirit that we seek possible avenues for collaboration between social scientists and hydrological engineers.

First, however, we want to say how impressed we have been with the ASCE Non-Hydrologic Committee's determination and effort to find ways for collecting, ordering, and testing data in a more uniform manner that makes broader comparisons and transferability more possible. We have also been impressed with the committee's expressed desire to relate data and research models and results from engineering and the physical sciences to those from the social sciences. In this regard, probably one of the most crucial factors is what engineers and physical scientists can expect from the social scientists. Our experience would indicate that "nonsocial" scientists tend to expect far too much or nothing at all from the social scientists. Either disillusion or excessive optimism are likely to be misleading and detrimental to the goals of interdisciplinary collaboration and scientific progress. Hopefully, there are more modest grounds for basing our expectations, and hopefully, too, more modest expectations of the social sciences will make it less likely that other scientists will despair of assistance from them. For this reason, we believe that expectations should be generated, ordered, and projected in light of the problems faced by social scientists. Let us turn, then, to some of these problems.

Collectively, the social sciences are at a far lower stage of development when compared to the physical and biological sciences. Engineers and physical scientists should recognize this and be prepared to cope with it. It should also be noted that individually and collectively economists and psychologists seem to be working at a higher stage of development than are scientists in other fields of the social sciences.

Over the past two decades sociology, political science, and anthropology have taken some rather long strides in the direction of systematic theory and application of the scientific method. However, as one might expect, the outcomes have been uneven, and very often, terribly disappointing. It should be appreciated, that social scientists, like many other Americans, live under production pressures and the stresses of status aspiration. The result has often been too little care in the development of conceptualization, theory, and techniques. Moreover, there has been a tendency to be satisfied with rather shoddy efforts at data collection, and most disappointingly, testing procedures frequently sacrifice rigor and/or full use of controls in order to get publishable results which may look good or plausible but lack validity and reliability. For example, one of our most respected professional colleagues speaks of a frequent need for "quick-dirty" research.

This does not mean that development is completely lacking. There has been quite a bit of it. In our field, political science, for example, some of the most notable theoretical advances have been made in the yields from voting studies. The behavior studied here, of course, goes far beyond the voting action. Likewise, economists have developed some rather viable theory on correlative phenomena; namely, decisional choice. We suspect, however, if there has been anything approximating a breakthrough, it has been learning theory developed by the psychologists.

A great deal of this so-called disciplinary underdevelopment can be attributed directly to the nature of much of the social science data as well as to a failure to fashion sophisticated tools for managing and analyzing that data. When one begins to survey the data of social science, he discovers that a great many social scientists have never fully appreciated the true value of or need for rigorous classification systems that are the bedrock of any science. A closer examination of the character of the data reveals that large quantities of it are recursive or interactive. This is to say that two types of phenomena seemingly induce changes in each other at the same time. For example, perceptions of conditions or circumstances seemingly affect or cause changes in attitudes while attitudes are seemingly causing changes in the perceptions. The upshot is that we cannot isolate causal elements or causal chains. The data frequently yield a baffling set of generalizations for the investigator that is generally washed over with the phrase "reciprocal causation" or something similar in tone.

Doubtlessly reciprocal causation exists in social phenomena, just as it does in chemical, biological, and medical phenomena. With some assistance from other disciplines, imaginative social scientists will begin (indeed, have begun) to develop mathematical, statistical, and other means for determining the extent and type of change two or more variables induce in each other at the same moment or along a chain. The challenge is squarely before the social investigators and they need all the interdisciplinary assistance others are willing to provide. This challenge will doubtlessly be met in the very near future. Mathematical and statistical devices that we now possess were devised largely as tools for the "harder" sciences. Scholars in many fields now need to be challenged to develop new and imaginative tools for handling interactive data. When employing tools developed thus far, the social scientist is constantly faced with the misfortune of having one or more unknowns than he has equations to solve for them. He is, in short, under-identified in his equations. Certainly the human imagination is capable of coming to grips with this problem and doing something about it. We would like to put this challenge squarely before mathematicians, statisticians, physical scientists, and engineers, as well as social scientists.

The inadequacy of our existing analytical tools frequently leads us to employ them, despite their deficiencies, and as a result our analyses and generalizations are faulty. Even the most casual perusal of social science yields betrays a tendency for the scientist to employ measures and methods that are only seemingly appropriate. Inspection of the literature over the past 30 or 40 years shows that a commendable eagerness for solutions and answers has led social scientists into a disappointing habit of applying highly precise measures that were devised for other purposes. Almost invariably these precise measures are applied to gross objects, and as a result, we often look ridiculous.

The failures to develop adequate tools and meet the requirements for rigorous classification systems have yet a more serious consequence. The social sciences have not developed, cumulatively. That is, they are largely non-cumulative. Whereas engineers and physical scientists tend to be trained in disciplines that have a broad base of information and theory that builds block-by-block wherein each block is interdependent with other blocks, such conditions hardly exist in the social sciences. Only in economics is there a strong tendency to be cumulative. This deficiency apparently makes it difficult for social scientists and engineers to communicate with each other. Consequently, they tend to talk past each other on many occasions.

Although we have already drawn attention to the great amount of instability that characterizes social data by comparison with physical data, we should emphasize a related problem of great magnitude. Apparently some of the most trying problems arise from the high learning capability of human beings. Each new item of information or each newly learned factor has the potential to interrupt stable patterns and regularities in behavior that upset the best of predictive and explanatory theories. Therefore, a settled and widely employed theory of high validity nay be suddenly rendered suspect.

Another related deficiency arises from the one immediately above. Social scientists have always been given to concentration on what Paul Lazarsfeld has called the surprising irregularities rather than the regularities.[1] They have expended considerable energy over the decades and centuries searching out the surprising and unique, and have tended to neglect the regularities and the systemic relationships among them. Evidently, the surprising interruptions have been far more challenging and fascinating than the regularities. This still appears to be the case for many workers in the discipline.

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[1] Baron, A.H. and P. Lazarsfeld. 1955. "Some functions of Qualitative Analysis in Social Research." *Frankfurter Beitrage Zur Soziolotgie*. Pp. 321-361.

The above may have resulted, generally, in notably few and weak efforts to be rigorous and systematic, especially until very recent times. A great many of the attempts to be systematic have had disappointing results. The best attempts often tend to shatter and fall apart. This, in itself, may cause the avid empiricist to lose heart and retreat to more traditional modes and ventures, a direction in which many empiricists now seem to be moving.

Also coupled to the tendency to be enamored with the unexpected events and behavior, is a failure of social scientists to derive much hard empirical data. Non-social scientists can expect data to be far softer than what we find today in the physical sciences, if for no other reason than that man's learning ability intrudes on the regularities of behavior. But despite this, or maybe because of it, there is an apparent tendency for social scientists to show less concern than physical scientists about developing hard data.

Consequently, the highly pitched ardor that has carried the empiricist to the cutting edges of social science during the past two decades may be waning. The engineer should be acutely aware of a trend at present for social science to move into a thermidor. A thermidor will mean disenchantment with most types of empirical research that has typified much of the imaginative endeavor over the past half century.

Should this thermidor occur, and indeed we think it may already be transpiring, then it is probable that collaborators from other disciplines will find social scientists retreating to the development of powerful deductive theories that are weakly informed by empirical theories, if at all, but do select strategically the events from the empirical world that give credence to the deductive theories.

What we are saying is that it has always been easier to manipulate in the abstract without encountering the empirical aspects that challenge the heuristic framework. This has been, and will continue to be, a weakness in social science research and theory, and it has a high probability of becoming more pronounced during a themidor.

We, therefore, would guess that there will be greater and greater investment of effort in "rational" models by model builders and, that these models will be related to the empirical world through careful selection of events that seemingly fulfill them. It will mean, very probably, that models will be more artifacts of deductive than inductive theories.

We would also hypothesize that lapsing into a thermidor will mean a stepping up of efforts to be prescriptive rather than explanatory. There was always heavy pressure on social scientists, indeed, from engineers and physical scientists, to provide prescriptions for future action. Social scientists have usually responded as best they could, despite the deficiencies in the information and theory that have plagued them since before Caesar.

Since the pressure is still on social scientists, we cannot but guess that they will do otherwise than prescribe when there is disenchantment with induction. A good deal of the efforts to prescribe are likely to be based on impressionistic, un-systematic, and common sense methods and theory. This we can almost certainly expect. When the theory advanced is not prescriptive and normative, there is every indication that it will be descriptive rather than explanatory. That is, it will be explanatory only insofar as descriptive theory seemingly explains.

One more extremely serious difficulty in the social sciences stems from its heavy dependence on survey research. Donald T. Campbell, an outstanding social theorist, laments this condition.

> *Today, some 90% of social science research is based upon interviews and questionnaires. We lament this over dependence upon a single, fallible method. Interviews and questionnaires intrude as a foreign element into the social setting they would describe, they create as well as measure attitudes, they elicit atypical roles and responses, they are limited to those who are accessible and will cooperate, and, the responses obtained are produced in part by dimensions of Individual differences irrelevant to the topic at hand.*[2]

Finally, we would be remiss if we did not acknowledge that the many weaknesses which plague social theory leave broad questions on the horizon about such things as: a) the extent to which scientific methods are applicable in social research; b) the extent to which behaviors can be quantified and the extent to which human behavior is probabilistic (of course, this is an empirical question itself which needs to be tested); c) the validity and reliability of instruments that rely on respondents' memories, the way in which they perceive symbols used on them by researchers, and the validity and reliability of oral and written reporting of respondents; and d) what motivates human behavior. No matter what the psychologists say about that last one, they are still searching and really without much in the way of answers on motivation as yet.

Now, hopefully everything said thus far would encourage collaboration. It is our considered opinion that much is to be gained by everyone through interdisciplinary efforts and that even more can be gained if the non-social scientist appreciates the problems and orientations of his collaborators. For example, recent expressions of dissatisfaction with cost-benefit ratios and the models that are used to derive them provide evidence of need for greater appreciation of the social scientists' problems.

We are convinced that social scientists and their interdisciplinary collaborators must face the problems outlined above forthrightly and squarely. There is, in fact, a great frontier for discovery and with the mechanical and electronic facilities we now possess, discoveries are almost certain to be forthcoming, if the imaginative will acquire skills and apply them. However, we also believe that Kurt Lewin's

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[2] Webb, E.J., D.T. Campbell, R.D. Schwartz , and L. Sechrest. 1966. *Unobtrusive Measures: Non-reactive Research in the Social Sciences*. Rand McNally and Company, Chicago. p1.

admonitions need to be followed. In his words, we must make a determined "attempt to go beyond what is regarded scientifically accessible at any specific time. To proceed beyond the given level of knowledge, the researcher, as a rule, has to break down methodological taboos which condemn as 'unscientific' or 'illogical' the very methods or concepts which later on prove to be basic for the next major progress."[3]

Turning now to the questions about collaborations between social scientists and hydrological engineers, the question becomes one of with whom and on what is collaboration the most possible and feasible. Until now by far the greatest efforts to collaborate have been between engineers and economists. Doubtless, the tendency for economics to be more cumulative than the other social sciences has been a factor in facilitating the attempts to collaborate. Members of the two fields have a fuller understanding of what each other attempts to do than is characteristic of the relationships between engineers and the other social scientists. We probably can expect economists to take the lead for several years to come, despite the expressions of disappointment from time to time with results of particular collaborations, especially the disenchantment of many engineers with benefit-cost and other rational models. In this regard, engineers can help economists to find means by which rational models can be informed through use of empirical models.

Of considerable importance for future collaboration is the growing number of social scientists who are attempting systems approaches to the social sciences. Efforts in this direction have already begun to make the social sciences more cumulative and open greater possibilities for collaboration between the social disciplines and the "hard" sciences. Special heed should be paid to the rapidly developing fields of communications theory and cybernetics. By finding out how people communicate with one another, engineers can determine some of the reasons why good programs get "shot down" in the decision-making arena.

Another developing theoretical area that appears to hold a good deal of promise in the immediate future is organizational theory. Here again, the engineers and physical scientists have had some effect because they have prodded and pushed to obtain answers to vexing problems that they face.

Still another area that seems to be producing many reliable and valid yields is field theory. Field theory is really perceptual psychology which uses human perceptions as a basis for developing predictive and explanatory theory, the latter of which is really more descriptive than explanatory.

Probably the most fascinating but difficult area in which to assess yields is decision-making. No doubt failures to coordinate more fully the many activities in several disciplines has exacted its toll in this area of research. Economists, political scientists, sociologists, and psychologists have been investing considerable effort, some of which has been interdisciplinary and some of which has not. A good deal of

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[3] Lewin, K. 1949. "Cassirer's Philosophy of Science and the Social Sciences." *The Philosophy of Ernst Cassier.* P. Schilipp (editor), Evanston: Library of Living Philosophers, p. 275.

attention has been devoted to game theory and simulation. Again, many of the ventures have experienced uneven and disappointing results, but there have been encouraging results too, and occasionally some very significant by-products have developed from them.

We should also mention another fascinating development that curiously, and for reasons we cannot discern too fully (possibly because the field is fascinating and attracts a number of good thinkers) researchers in international relations have made some rather striking theoretical strides. However, care needs to be taken here because quite a number of those who work in this area are almost totally innocent of the promising theoretical developments in their own field.

Of all the areas for potential collaboration between engineering and social research, none stand in sharper relief than simulation. Simulating actual social relationships enables us to measure and test scientifically without disrupting the social milieu. If we can simulate prizefights between foes who never met and tell who the winner would be (e.g. Dempsey vs. Louis), certainly we can simulate decision-making situations. For example, we could do more than just think about simulating this or that. We could begin now by simulating such things as model administrative methods functioning under model water laws. Simulation does not require us to wait until the events naturally transpire in the real world. Given the huge capacity of present equipment, we can order masses of data, feed the machine, and jiggle one variable after another to see the difference it makes in others, if any, when you jiggle certain ones.

No doubt, successful simulation at the present time will require engineers to think and work more in terms of non-parametric or stochastic models than parametric ones if for no other reason than that social researchers are forced to work largely without knowing the parameters to their systems.

Engineers who collaborate or desire to work with social researchers should be aware of a great many diverse pockets of development upon which they can draw usefully. We have already mentioned the field of Economics and its rich resources. A closely related one is Economic Geography. Another is a sister field called Human Geography.

One of the developing areas right now that appears to hold considerable promise, especially in political science and social psychology, is communications and cybernetics. Somewhat important from the Non-Hydrologic Committee's standpoint, is that some of the impetus for development here has derived from engineers, physical scientists, and those psychologists who perceive of themselves as biological scientists.

The impact of cybernetics or the "science of communication" on the social sciences has been more profound than a cursory reading of the literature in the fields would indicate. In many cases the impacts have been indirect and transmuted. Direct influences, however, appear in those social science research efforts that employ input-output models that stress the interactions of the political system with its environment. Concepts such as "channel," "network," "feedback," and "communications net" are becoming a stock and trade of political scientists who discuss the function and nature of the political system. The models, operations and instrumentation employed by the social researcher closely approximate many of the systems models used by the engineering researcher. Communications research, therefore, provides a common ground of understanding between social scientists and engineers. Moreover, the engineer will be able to employ communications models to estimate and predict more fully what will become of his programs in the decision-making arenas. He can see how his program is treated from step to step in a sequential process of decision-making from the formulation and recommending stages to the application and terminating stages. We would encourage engineers to investigate communications models and the research undertakings of social scientists in this area because it appears that this particular area is especially suited for interdisciplinary collaborations.

Then there are those legal scholars who can, on rare occasions, be coaxed into a true social science context. Contrary to what was inferred at one Non-Hydrologic meeting by a member, there are some legalists who have a rather decidedly strong urge to do behavioral research in the law. The contributions of legal scholarship to the water field are well known and important. They will doubtlessly continue to be. But the committee is correct, of course, in its assessment to the effect that the contributions by legalists have been in narrow channels. We do not know the answer to the committee's question, namely, how to broaden pinched horizons and raise visibility so that lawyers will involve themselves in the broader social questions.

Sociologists, especially demographers, have a wealth of hard and soft data that they wade around in and never get very full use of. A considerable amount of what they possess could be profitably plugged into highly sophisticated designs for a number of purposes. For example, much of what sociologists have put together has a high potential for use as control variables in experimental designs, when and if the stuff is combined in various ways and tested for such use.

On the basis of these somewhat crude and broad generalizations, we would conclude that the most probable areas for successful and fruitful collaboration of the engineers and physical scientists with social scientists are human organization, field theory and survey research, communications and cybernetics, decision-making and gaming, economic analysis that extends beyond traditional theoretical designs and their heavy dependence on rational models, and sociological analysis that moves out from the more or less exclusive dependence on gross indicators or independent variables, like income and education level, to the development of coefficients of change over time (historical ones) which should provide a higher degree of control in experimental testing. In outlining the above we assumed, of course, that both economists and legal scholars will continue to collaborate as they have in the past. Successful collaboration will depend heavily upon engineers' and physical scientists' patience and appreciation for the problems that beset the social sciences.

Moving now to the question about viable research enterprises on which collaboration should be sought, let us suggest just a few in hopes that some appetites will be whetted.

1) Intensive research is needed on the ways in which public attitudes and ideological formations affect engineering outcomes.
2) We also need research on the ways in which engineering outcomes feed back to the social system to affect social change.
3) We need to develop valid and reliable scales of social values and attitudes. Hopefully, such scales will enable us to obtain more valid and reliable information for the decision-makers.
4) Man-land relationships are among the most important areas of study. Here, both physical and human geographers can make contributions of great value. Despite all of the research conducted thus far, we still are woefully deficient in information on how man relates to the land, physical resources, and the environment.
5) Becoming a little more specific, we need research projects that investigate planning and its conduct. Research should be undertaken at once to determine how we might:
    a) Consolidate and obtain more efficient uses of planning enterprises;
    b) Define public and private realms of planning;
    c) Consolidate and obtain greater efficiency in fiscal management of hydrologic developments;
    d) Examine conflicts in existing and projected plans and programs;
    e) Discover the effects of legal structures on plans and programs, to name just a few research needs in this realm.
6) In regard to the latter, extensive legal research is desperately needed. For example, far too little is known about the ways in which laws relate to water institutions.
7) Furthermore, studies should be inaugurated to seek the opportunities which the law gives us as well as the restrictions. The question is not only what cannot be done, but what is open to us.
8) As we stated previously, we need projects that test the ways in which model water institutions function under model water laws. Simulation is a viable tool for these purposes.

9) Further, programs should be undertaken which seek to develop model water laws, model water institutions, zoning criteria, and a host of other necessary features.

10) We especially need criteria for zoning which seeks to maintain safety and aesthetic values.

11) A most promising area for helpful research yields at this time is conflict research. We especially need more knowledge about measures that mitigate, as well as those that generate human conflict.

12) Another question that constantly arises is the one about human coalitions. How are they generated and what are the consequences of the forms a coalition may take? How do they communicate and what are the consequences of communications between the varying forms of coalitions with each other and with the establishments?

13) When it comes to the questions of communications, we certainly need more information on the following:

    a) How publics communicate with each other;

    b) How varying publics communicate with agendas and decision-makers;

    c) How agencies and decision-makers communicate among themselves; and

    d) How publics and decision-makers communicate with the various disciplines of learning such as engineering and the social sciences.

14) Most important, however, is greater knowledge of ourselves and the impacts we have. Engineering and technological changes need to be measured in light of their effects on social change. Moreover, the reverse operations are also needed. We should know how social change affects technological change. In brief, we should determine how technological and social change have advanced together. To put it another way, Items No. 1 and No. 2 on this brief list should be put in historical perspective. Here, needless to say, we must call on historians to work with us.

15) Of critical importance to engineers are decision-making procedures such as voting mechanisms and rules governing the behavior of decision-makers. At the present time, we can only guess what consequences particular voting procedures have for decisional outcomes. For example, there are no valid and reliable answers to the following questions:

    a) What is the meaning of a two-thirds voting requirement? Would differing policy outcomes result from a majority vote? Sixty percent? Eighty percent? Is there a specific function performed by a two-thirds vote?

    b) What does the collective vote mean? Is it an endorsement of one alternative or a rejection of another? What is the level of acceptability of the electoral outcome in the system?

c) What is the meaning of the size of the majority? Do close voting divisions differ from others in their meaning?

d) What does the voting outcome mean for a losing minority? Does the minority reject the system? Does it work for achievement of their goal at the next vote? Or does it attempt to alter the rules so that their goals become the probable outcome? All of these are empirically researchable questions. Valid and reliable answers to them could very well improve the chances for the success of many good engineering programs that must win in the decision-making arena before they become a reality.

The listing above outlines but a few viable areas in which empirical research is both needed and possible. The key to successful research collaboration lies in the efforts expended to put the questions in a systematic and researchable model to which engineering designs and models can be related. Doing this will require engineers to relate themselves and their skills systematically to the efforts and skills of social scientists from several disciplines. It will take more than a systems approach. We will need systematic organization of the research activities, along the lines suggested, by John Woods of TRW, Inc. at the Engineering Research Foundation Conference, Andover, New Hampshire, August 1968. What this means is "team research" on a highly organized basis in which the team includes persons from a number of disciplines and in which all persons are carrying out research which fulfills the requirements of their assigned portion of the model and in which the yields of each portion can be related successfully to the yields in other portions of the design. We stress again the point that the challenge is there. All we need to do is pick it up.

Chapter 9, Appendix H

COMPREHENSIVE SYSTEM ENGINEERING ANALYSIS OF ALL ASPECTS OF URBAN WATER, A PRE-FEASIBILITY STUDY

Water Resources Engineers, Inc.
Walnut Creek, California
July 1968

1.0 BACKGROUND

On October 1, 1967, the American Society of Civil Engineers (ASCE) entered a contract with the Office of Water Resources Research, US Department of the Interior, relating to "Systematic Study and Development of Long-Range Programs of Urban Water Resources Research." One task to be completed under this contract was

> *To conduct a pre-feasibility study and determine the possible effectiveness, cost, and time requirements for a comprehensive systems engineering analysis of all aspects of urban water.*

In an agreement dated January 30, 1968, Water Resources Engineers, Inc. (WRE) of Walnut Creek, California, contracted with ASCE to conduct this task. This report represents WRE's response to that charge. It is hoped that the proposed research effort to apply systems engineering analysis to urban water problems will find prompt and vigorous support. It is WRE's belief that the techniques of the systems approach offer unique opportunities to improve, enhance, and protect the urban water environment.

2.0 SCOPE AND OBJECTIVES

The research and development work proposed in this report, and indeed the report itself, has been derived and scoped to address the needs of people of widely varying interest who operate on or within the urban, metropolitan water environment. These individuals range from urban planners, through designers of storage and conveyance systems, to those concerned with maintenance and operation of constructed facilities. It would seem incredible that a single analysis methodology could emerge that would serve to lessen the day to day work burdens of all these people, much less "optimize" their concerted efforts. Nonetheless, WRE has been faced with identifying such a methodology and determining feasibility for its development and application to the urban water resource system.

It follows that the scope of this report and of the systems analysis methodologies it discusses is, of necessity, extremely broad. ASCE asked that the pre-feasibility study cover "all aspects of urban water," and WRE has attempted to provide such coverage. It may be, because systems analysis in its modern computerized form is still rather new and only slightly explored, that we hardly believe its capability to treat a "system" so all-encompassing. We may have some difficulty in comprehending how it would be possible, much less practical, to keep track of so many technical, legal, economic, social, and political interplays that somehow influence the safe and dependable delivery of water to the proper places and the removal and treatment of associated wastes without the public's second thought. Still we realize that this has been happening for years, without systems analysis as it is now defined and used. Water planners, designers, and operators have performed their functions adequately or very nearly adequately for perhaps a century in this country. Perhaps the major obstacle preventing their efforts from closely approximating the optimal is that they did not or could not communicate or could not budget their functions compatibly. It cannot be forgotten, though, that their achievements have been very substantial despite the institutional barriers that may have separated them.

Hence, the trust of the analysis methods outlined herein is toward even more comprehensive, coordinated public and private management of urban water. This, in turn, should lead to the lessening of costs and the improvement of service in the years ahead. The approach should continuously strive for the "optimum system performance wherein all factors—technological, economic, social, and political—exert their proportionate influences. The major emphasis in WRE's report is on improved engineering technology to aid the attainment of the overall objective of a truly superior urban water environment, certainly a better one than could be obtained without the systems approach.

A prime objective of this work has been to demonstrate the feasibility and effectiveness of systems analysis in its modern context to represent and solve problems of the total urban water resources system and to optimize its planning, design, and operation. This could not be done without first exploring the states of both engineering technology and analytical techniques necessary to represent and operate on the various components of the entire urban water system. Thus, WRE has defined the total system, defined the components it must include, and indicated, for several levels of detail, the research and development effort needed to realize a total analytical capability for the urban water resources system.

Corollary objectives of the pre-feasibility study were:

1) To indicate interfaces that must exist between the proposed research in the more purely engineering, technological areas and concurrent research to be conducted in the field of economics,
2) To indicate the necessary data management activities and systems analysis techniques for data management that should be applied to the total urban water management program, and
3) To indicate areas requiring immediate or auxiliary research effort.

Finally, WRE was to outline probable time, manpower, and cost estimates for conducting the necessary research to arrive at a functional and functioning systems representation and analysis package for the urban water environment.

3.0 ORGANIZATION OF THE STUDY

This work was conducted by the staff of Water Resources Engineers, Inc., under the direct supervision of Dr. Gerald T. Orlob, President. The project was coordinated on a day-to-day basis by Dr. Michael B. Sonnen and received primary inputs from Dr. Robert P. Shubinski, Messrs. Donald E. Evenson and Lawrence S. Costello, and other members of the HRE staff who contributed information in areas where they had special knowledge. Dr. Sonnen was responsible for topics related to water quality, Mr. Evenson developed the information concerning overall system modeling and the necessary systems analysis techniques for doing so, Dr. Shubinski and Mr. Costello were concerned with data management and research scheduling and allocation of manpower and costs.

For assistance in initial scoping of this project, as well as for review of the final product and for some intermediate input in special areas, WRE was fortunate to have the services of Mr. Gene E. Willeke, who acted as Technical Consultant. Mr. Willeke is currently completing his Ph.D. requirements in civil engineering at Stanford University and has considerable experience in urban water resources management in general and urban hydrology in particular. His helpful suggestions and guidance were invaluable.

4.0 ACKNOWLEDGMENTS

WRE, during the course of preparing its report has drawn from many sources, its clients, academic associates, and professional contemporaries. Always, it was accorded helpful and constructive assistance. WRE is particularly appreciative of the generous counsel it received from Mr. M. B. McPherson, Project Director, Urban Water Resources Research Program, and ASCE.

5.0 THE URBAN WATER RESOURCES SYSTEM

5.1 System Definition

To decide what specific analysis problems need to be researched for the urban water environment, it is necessary that WRE establish the ground rules for its ensuing discussions by first defining the "urban water resources system." Although regarded as an essential step, this definition may have the disadvantage of implying that the system can only be defined one way, which is certainly not true. However, some framework is necessary. Furthermore, this framework must be general enough to include urban areas of any and all sizes and characters. We may not have been totally successful here, but it is to be hoped that our definition is sufficiently comprehensive without sacrificing necessary detail.

The "system" chosen is represented in Figure H-1. It depicts the urban water resources environment as an intricate network characterized by some relatively specific "locations" for water in the urban setting and by several varieties of "transfer" elements that connect these locations to one another. These location and transfer "subsystems" receive water from outside, hold and distribute it throughout the entire system, and then discharge it to a receiving environment or lose it to some lost-water reservoirs through several possible loss mechanisms. WRE will propose from here forward, then, that research be instituted to study and to define analytically the system's components and to combine these into an overall functional representation for the purpose of comprehensive treatment of the total system.

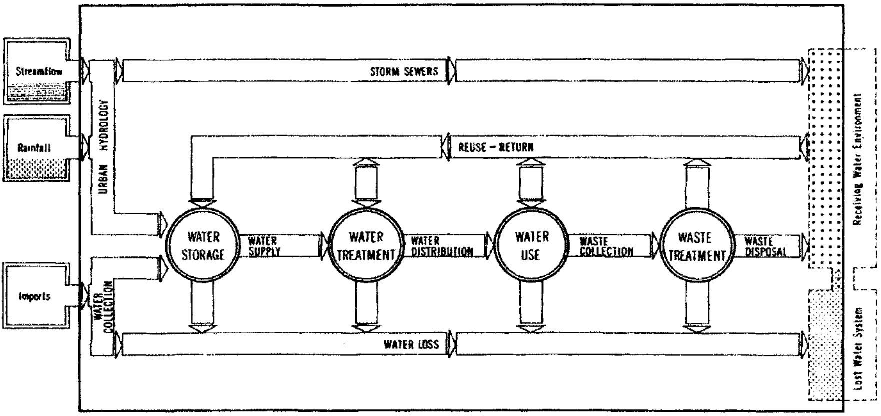

FIGURE H-1.

It is readily acceded that all knowledgeable persons will not separately and at all points in time care to define the urban water system in exactly these same terms. Indeed they should not, since only too obviously there is no one urban water system; each area is unique and should be studied in relation to its own boundary conditions and components. Milwaukee, Wisconsin and Hosheim, Tennessee—to cite two examples—are entirely different urban organisms.

There is some justification, then, for defining the urban system also within definite geographical boundaries rather than with unit operations and functional relationships alone. For an analysis of a specific metropolitan area this will obviously be necessary. In this report, however, it must be assumed that parameters that define the system's geometry are data inputs. The transfer or location subsystems or the water they contain will necessarily have to be identified in each real case by number, length, volume, mass and other pertinent dimensions. These, of course, may be in themselves variable over time as the subsystems are developed and operated as individual entities or integrated parts of the total urban system.

5.2 Subsystem Definitions

Within the Urban Water Resources System (UWRS) illustrated in Figure H-1, four major groupings of subsystems are identified:

WATER LOCATION
WATER TRANSFER
RECEIVING WATER
LOST WATER

Each of these embraces systems of subordinate rank and finer detail. In the ensuing discussion, WRE will endeavor to give some explicit definition to the most important of these and to show how they are related within the total UWRS.

5.3 Water Location Subsystems

"Location" was chosen as an appellation for this category of subsystems to imply that water may be detained at certain places in the urban environment where it may be operated on by man—either held, literally for later use, changed in quality, or used. The four location subsystems defined here are *water storage, water treatment, water use, and waste treatment.* These are, even as system components, rather large and comprehensive groupings. It is true that overall system representation is improved by defining the smallest practical subsystem components. For the general purposes of the present study, this degree of compartmentalization appeared to be the most reasonable, further detailing is left for the expanded research program. Accordingly, it is appropriate, in order to avoid misconception as to what is embraced by each, to describe all four subsystems.

Water Storage—*A water storage subsystem includes all facilities that are constructed or used in a planned way to hold water either for subsequent use or for flood control within the urban environment.* It includes, then, storage tanks and reservoirs within the urban geographic boundary, as well as reservoirs that may be needed and constructed in remote areas for storage of water to be transported to and used within these boundaries.

Right from the beginning, it should also be noted that there are likely to be interrelationships other than simple connections between the subsystems defined here. For example, there is considerable storage volume within a pipe distribution network. However, if account is to be taken of this storage capability, it should be allocated within the "storage" subsystem rather than either: *1) being totally ignored, or 2) being added as an afterthought in the design of the distribution system.* Such multiple purpose ramifications are likely to exist for all urban water system components. However, rather than letting them confound us at this juncture, we should realize that it is precisely these complex interactions that makes the methodology of systems analysis so attractive, not only for keeping them sorted out, but also for allowing us to analyze them all simultaneously.

Water Treatment—*A water treatment subsystem includes many possible components: aerators, flocculators, sedimentation, tanks, filters, ion exchange units, and associated equipment and appurtenances.* It can be municipal or private, large or small, one or many units. It includes, in short, all the water treatment devices necessary at various points throughout a city.

The functional representation of the water treatment system will depend on the stage of development and degree of detail that pertains to that stage. For example, in planning, the concern is simply providing a certain amount of water of prescribed quality to specified locations within the system. If the supply of water available is less than adequate for the prescribed services, then "treatment" will simply mean the removal of an appropriate amount of certain substances. If, on the other hand, the system is already in operation, "treatment" may imply optimizing certain unit operations such as aeration, flocculation, and pumping, usually to some criterion of cost, so as to maintain a certain quality constituent at a given level. In short, the degree of detail required will Influence the objective of the analysis, as well as the necessary data. Conversely, changes in treatment objectives (quality or cost criteria) will require different data detail and different analysis techniques. The spectrum of details and objectives of treatment, though, must be included in the water treatment system package.

Water Use—*A water use subsystem is one that deals with the application of water for any beneficial use in an urban environment.* It may appear at first thought that "water use" is just too big a topic to be embraced by a single subsystem. It is truly large, particularly when one considers the myriad ways water can be used in the urban environment. Yet, again it is a matter of detail needed for our present purposes.

For planning objectives, we wish to keep track of annual flows demanded by a number of users, each having his own quality needs. A simple inventory, then, of Q_i's (flows for each user, i) and C_{ij}'s (quality constituent concentrations for each user, i and constituent, j) may be all that is necessary. Designing for water use is a different kind of problem—one of inventory. How much piping and pumping is needed at specific locations to meet particular demands? Operation of facilities to meet user demands can be aided by routines to optimize distribution of water among users from day-to-day and from hour-to-hour.

Admittedly, water use cannot be written off quite as simply as we have suggested in these brief statements. The system implied here deals with hourly, daily, monthly, and annual demands for water by many different classes of urban users, each requiring different quantities of water of specified quality. Systems analysis techniques are available to assist in making the necessary inventories and statistical analyses, and for defining more accurately, many of the interaction connections. There will, of course, still be some connecting relationships that are not yet well enough known or understood to permit a complete analytical description. Some basic fieldwork and additional research will be required. On the other hand, many water demands are known well enough to allow us to begin modeling the grosser system configurations.

Waste Treatment—*The waste treatment subsystem* is, *like the water treatment subsystem, a mixture of components designed to improve quality.* Only in this case, we must consider a new "supply"—the water left over after it has been applied to some beneficial use. This supply, or a part of, it must be treated to satisfy a new set of users having a different set of criteria. Some of the components, such as secondary treatment processes, will be new and radically different from water treatment components. Yet, they can be analyzed in a like manner as system elements whose "operators" describe the removal of waste substances.

The waste treatment system is often more complicated because it can be either public or private and large or small, while comprised of many discrete components. It can include industrial waste renovation plants that treat wastes to reclaim a saleable product and hence, profit from the wastes. Or it can include a municipal plant that treats waste to protect subsequent water users in other than its own municipality, hence, requiring investment for someone else's profit. The economics of such a system is clearly extremely complex and, indeed, is both politically and technically controversial.

We have just identified here an area likely to be fraught with constraints on our engineering approach. That does not mean that systems analysis is expected to perform poorly, quite to the contrary. Rather, it means that the constraints, once formulated, can be added to the puzzle like any other piece and that the systems approach can assist us in dealing objectively and rationally with these constraints. This is not generally true of today's engineering practice; normally we leave such constraints completely to the judgment of the politicians, thus removing them from the slide-rule before the analysis of choices is complete. This may seem correct in the abstract, but the fact is that it need not be so. Systems analysis allows us to carry objectivity longer and farther into the study. The resulting products should be superior alternatives for presentation to the ultimate decision process.

5.4 Water Transfer Subsystems

"Transfer" was chosen to identify those physical entities, either natural or man-made, which are concerned with moving water from one location to another within the UWRS. Eight subsystems are considered as providing the necessary linkages between location subsystems or accounting for water bypassing either on purpose or unavoidably. Some of the subsystems are physical works; others are merely environmental mechanisms. All of them move water. They are described as follows.

Urban Hydrology—Water is delivered directly by precipitation to the surface of the urban environment and then is conveyed by gravity across the ground surface to other locations. In this process, it is usually concentrated, sometimes dispersed, and generally subject to loss. In and around cities, this is the essence of urban hydrology. To describe these processes, we need equations to describe rainfall distributions in time and space and the mechanics of overland flow. The required set of equations will describe the urban hydrology subsystem.

In planning, we will need to know how much water will fall, when it will be received, how much of it will flow to other locations, and where it will go. In design, we will want to know what size structures are needed to accommodate the flow or store it temporarily. In operation, we need to know how and when to maintain the ditches, culverts, or spillways we provided to accommodate these volumes and flows. It should be noticed that the subsystem as defined here does not include storm sewers or aqueducts, these are included in other subsystems. The Urban Hydrology Subsystem includes only those elements that are intended to prevent damage during periods of natural drainage.

Water Collection—Infiltration galleries, wells, protected watersheds, or long-distance aqueducts are all to be considered, if needed or appropriate, within this subsystem. Furthermore, any pumps or other equipment necessary to move water through these facilities are likewise included.

Provision for and location of such facilities in any water resources plan will likely form the backbone of the technological alternatives for urban water development. A crucial and early decision must be made about the source of water to be tapped, essentially a planning activity.

Storm Sewers—Any system of connected pipes, tunnels, or other conveyance facilities primarily intended to carry excess runoff is a part of this subsystem. Because these systems are quite expensive normally, and several alternatives exist for the fate of the water they carry, planning, design, and operation of component physical facilities for stormwater conveyance are very important urban water functions. Fortunately, a great deal of work has been done to characterize, analytically, their hydraulic behavior. There is, however, still much room for improvement in analyzing and evaluating alternative storm sewer plans and in the process of deciding which alternative plan is superior. Major benefits may accrue to urban water users through optimal planning of wastewater (storm and municipal) systems.

Water Supply and Water Distribution—Any system of connected pipes or other facilities designed to distribute water from points of storage or supply to a water user is included in this subsystem. Water distribution networks, of all "systems" of the urban water environment, have probably received the greatest attention of analysts. The classic techniques of Hardy Cross have been extended and adapted to modern computational equipment with considerable success. Much remains to be done, particularly in optimizing design and maintenance procedures and in characterizing the "dynamic" behavior of such systems. Because of the large capital and operation and maintenance costs and the necessity for high guarantees of reliable service, this represents a prime opportunity for the systems methodology. If water treatment is required and used, the water supply subsystem will include the aqueducts or pipelines to transport water from the water storage subsystem to the water treatment subsystem. The water distribution subsystem will be the pipe network to deliver the water from the treatment plant to the users. If treatment is not included, the water supply system will include all the conduits necessary to deliver water from the storage point directly to the users.

The present "state-of-the-art" of distribution system analysis provides some assurance that this subsystem can be well represented within the overall urban water model. As a separate subsystem, it can still be much improved.

Waste Collection—This subsystem consists of any transfer facilities used to take wasted water (except storm runoff) from a point of use to a point of disposal. Sanitary sewers are the obvious example. These may be characterized by either pressure or free-surface flow relationships. Necessary pumping facilities are included as a part of the subsystem. It includes a discharge facility only if it is simply the terminal conduit in the collection system. If treatment is provided, the waste collection system ends at the treatment plant. If a long outfall sewer is to be provided, the collection system ends at the point where it enters the outfall, the outfall being considered as a separate subsystem.

Waste Disposal—Physically, this subsystem would normally be comprised of one or more outfalls or transmission aqueducts to carry wastes away from an urban area, if it exists at all. In many cities there will be no disposal system as such, since frequently treated wastes are discharged through a single, rather short, effluent pipe directly to the receiving system. An exception might be a recharge pit requiring some special analysis and which would be used strictly for waste disposal, without any particular intention of providing recharge to a local water supply.

Reuse and Return—This subsystem is complicated simply because it has so many possible components. It includes all the necessary facilities and mechanisms by which there is feedback of water within the overall urban system. It covers the gamut of possibilities from return piping within an industry to intentional groundwater recharge to replenish a supply. The functional relationships necessary to describe this subsystem will likely all be developed within other transfer subsystem analyses. The important considerations are to recognize that reuse and feedback are possible, that they can and should be optimized, and that, obviously, provision should be made for including this important transfer function in the overall systems analysis.

Water Loss—This subsystem consists of transfer mechanisms by which water gets away from us before we can use it. Evaporation, Infiltration, consumptive processes, and leakage are obvious examples. The "water loss" subsystem differs from the "lost water" subsystem in that the former is an action system whereas the latter is concerned only with where water goes when it is lost. Some of the processes that belong in this system have been studied rather well, and their descriptions are available to an overall model. Evaporation and infiltration are probably the best known. Consumptive losses during some water uses and water leakage from pipe systems are not as well understood.

Receiving Water System—*A receiving water subsystem is any natural water body, at rest; or in motion, which can accept spent water from the urban environment.* Physically, the "receiving water environment" can include the oceans, streams, estuaries, lakes and reservoirs, or the ground and even the atmosphere. It signifies the many places where water may be returned to the hydrologic cycle after we are through with it in the urban setting. It could, then, even include an aqueduct or "drain" that transports water from one urban area to another. It must be characterized in the systems methodology, therefore, by relationships that describe hydrodynamics and water quality interactions within or among any of these possible portions of the environment. Many of these relationships are not well known. Few have been well represented mathematically.

It is probably this kind of location subsystem for which systems analysis techniques have been must successfully developed to date. The classic Streeter-Phelps equations have been solved, modified, and resolved in countless versions. Yet dissolved oxygen resources and organic loadings represented in these relations are likely to assume a lesser importance in future studies, as concern shifts to the more sensitive indicators of quality changes in the environment. Toxication, bio-stimulation, and subtle changes in lesser-known quality parameters will command the attention of future systems analysts concerned with receiving water subsystems. More attention should be directed also to the hydraulics and hydrodynamics of environmental systems that govern the disposition of pollutants after discharge. It still remains to represent complex hydraulic behavior and coupled water quality reactions within one, generally applicable model. A general model to describe the intricate interactions that govern the rates of eutrophication in natural and artificial water bodies may serve as a good example of what will undoubtedly be needed to deal with receiving water problems of the immediate future.

Lost Water Systems—*A lost water subsystem, may be characterized in a general sense as a "sink" for water removed physically from the urban water environment.* It includes water used consumptively or not otherwise accounted for in a total water balance of the system.

A substantial part of the water transported toward an urban area for use can be lost *enroute*. Some may be lost during distribution, yet it could enter another subsystem. Still more might be lost during use, which means that less water would be available for reuse.

This water could go physically to the land, water, or air portions of the total environment. In this sense, it is only "lost" insofar as the urban system, *per se,* is concerned. In addition, there is the special sort of consumptive loss whereby water is shipped away from one urban area to another in cans of beans or bottles of pop. Consequently, the *Lost Water System* can be physically the same as the *Receiving Water System,* and many of the relationships to represent both systems will be the same. The difference between them is merely a matter of consideration of how and why the water is disposed. If we send water to a stream purposely, the stream is the *Receiving Water System.* If the water is lost to the stream inadvertently before we have a chance to use it, the stream becomes a part of the *Lost Water System.* A conceptual objective of an overall systems analysis model, then, will obviously be to find ways to minimize water loss in the urban areas.

Boundary Effects—WRE has chosen to define rather arbitrarily, the boundaries of the Urban water Resources System. These have been chosen as may be seen in Figure H-1 so as not to include certain elements of the Total Water Resources System, which in essence includes all components that participate in the hydrologic cycle. Thus, stream systems, upland storage reservoirs, certain groundwater reservoirs, and some parts of receiving waters might not be considered as parts of the discrete system with which we have chosen to deal. Actually, it may be a matter of choice in each specific instance as to how one chooses to delimite the system configuration.

However the system may be truncated, it will be necessary to rather explicitly describe the conditions that prevail at the boundaries, real or imagined. These "boundary conditions" must indicate the properties of the water, in quantity and quality terms, that is transferred across. In a few instances, these properties will be those defined by particular transfer subsystems, such as *Water Collection* or *Urban Hydrology.* In the former case, the specific inputs may be diversions programmed from a distant reservoir; in the latter case inputs may be streamflow and natural precipitation on the urban area. Inputs may be deterministic, as for controlled deliveries, or stochastic as for most hydrologic quantities. It will be noted that the structure of certain sub-models of the urban system may be either deterministic or probabilistic depending on the kinds of input information provided and the type of response function desired.

The ultimate outputs of the system may also occur at arbitrarily specified boundaries. WRE has chosen to indicate that the receiving water system, for example, may be only partially embraced by the Urban Water Resources System. This is generally the case of a river system that passes through a community or of an estuary that is gradually relieved of waste discharges by hydrologic and tidal purging.

In contrast, one may consider the cases of a groundwater basin or a lake that may not be so easily considered independent of the urban complex. It may be much more realistic in such instances to embrace the receiving water subsystem totally within the larger entity, thus avoiding some of the particularly complex problems of defining boundary conditions for such difficult to simulate systems. In these instances, it is also generally true that there is an interdependence of the receiving water body and some other subsystem, since the latter can often be a primary source of supply. If such is the case, a systems approach would dictate that the water body be totally included in the system.

6.0 SUMMARY OF SYSTEM DESCRIPTION

Four location subsystems and nine transfer systems comprise the Urban Water Resources System defined by WRE. These, and two additional systems that are concerned with where water goes when it leaves the urban environment, are listed as follows:

- Water Location Subsystems
    - Water Storage
    - Water Treatment
    - Water Use

    - Waste Treatment
- Water *Transfer Subsystems*
    - Urban Hydrologic Cycle
    - Water Collection
    - Storm Sewers
    - Water Supply
    - Water Distribution
    - Waste Collection
    - Waste Disposal
    - Reuse and Return
    - Water Loss
- Receiving *Water Systems*
- Lost *Water Systems*

In descriptions of each of the individual subsystems, particular problems of analysis were highlighted. It was noted that the constraints of a non-engineering sort are generally most prevalent for location subsystems. This is probably because man is rather intimately involved with these systems, not only in their conception but in their day-to-day operation. It was observed, however, that these constraints may well be considered by means of systems analysis in its most up-to-date methodology.

Certain subsystems were highlighted as having been better described analytically than others, notably water distribution and collection systems and certain special systems of the receiving water. The present state of development of these should facilitate their integration into a comprehensive urban water resources model.

Attention was given briefly to some obvious attractive possibilities in applying the systems methodology to describe subsystems. This was noted to be especially attractive where the capital investments are likely to be large and operation and maintenance costs high. These possibilities will be expanded on later in this report as the program for a comprehensive systems engineering approach is developed in detail.

Finally, note was made of the problem of accurately defining "boundaries". No hard and fast rules can be made as to where to circumscribe the specific subsystems comprising the total. Rather, each particular case must be considered for its unique properties at the boundaries.

The definition offered here is assuredly not the "best" of all possible definitions of the Urban Water Resources System. It is merely a reasonable point of beginning. It is an interesting advantage of systems analysis, however, that an initial trial solution is likely to lead to a succession of better definitions. Having started, then, we should be well on our way to the ultimate objective, a systems methodology for the Urban Water Resources Environment. The means of achieving this objective will be presented in succeeding chapters.

7.0 SUMMARY AND CONCLUSIONS

7.1 Summary

Most urban water systems have already been planned and designed and are now in operation. Also, most of them have been planned and designed in pieces or by subsystems and hence, are likely not operating or being operated optimally. As a consequence, it is a worthy cause now undertaken by ASCE to determine if the operation, design, and planning of these large and complicated systems can be improved through the application of advanced analysis techniques. It seems apparent *a priori,* that they can be so improved, but the feasibility of doing so immediately, and at a reasonable cost, and for all urban areas simultaneously is not so apparent. The study reported here was to determine if such a comprehensive systems engineering analysis is indeed feasible.

To begin this task, MRE found it necessary to adopt an arbitrary definition of the Urban Water Resources System, which was shown in Figure H-1. This was done to allow distinctions to be made about the work that had been done and the research remaining to be done to characterize all the subsystems involved through systems analysis techniques. As might have been expected, it was found that certain subsystems had received more attention than others. It is interesting that the Receiving Water Environment has been studied through systems techniques more than any other location subsystem. The Water Use subsystem, which forms the basis for having all the other subsystems, has been given the least systems analysis attention. Of the transfer subsystems identified here, the Water Distribution subsystem has probably been aided most by systems analysis, whereas the Reuse-Return subsystem has been virtually ignored by systems engineers. Consequently, time will be required to bring some of the subsystems up-to-date in terms of relative completeness of systems analysis description.

Another point that mitigates against immediate solution of current urban water problems is that most of those problems are encountered in day-to-day operation, which means they require very minute analysis at a very high level of temporal and spatial detail. Paradoxically, it is usually necessary to structure computer programs and techniques first on a gross basis, say for annual-value, planning problems, and then to improve these tools to accommodate problems of more precise detail. So, again, the feasibility of immediate systems engineering solutions to the most pressing urban water problems is remote, because some lag time will be necessary to structure more gross models on which to improve.

It appears, on the other hand, to be highly feasible to describe, in fact, all the urban water subsystems simultaneously and to study their combined responses to specified input conditions on any one subsystem. This appears to be a very desirable and plausible objective of developing such capability, and such interrelationships of subsystems should be determinable through sensitivity analysis, WRE's experience with the Santa Ana River Basin study gives us every confidence that a model for an entire Urban Water Resources System can be developed. What should not be inferred from this, however, is that one program will operate for all cities; it will not. We have made the distinction between "program" and "model" in this report, because we know from experience that a program and the data that accompany it for San Francisco Bay form a model of that Bay, but that the same program will not describe Port Phillip Bay at Melbourne, Australia, even if it is given similar data. A model is a program that operates on data, first to reproduce an historical response, and only then to predict future responses. Techniques are likely to be vastly different in two programs that try to do this with data from two different areas.

Based on the foregoing philosophies and a review of the current status of systems engineering techniques, then, WRE has structured its suggested research program. This six-year effort should produce as output demonstrated system methodologies that will:

1) Solve operation and planning problems for a pilot city;
2) Conduct design analyses for example cases of all the urban water subsystems; and
3) Perform all the necessary operations to provide data for the modeling activities and other general data uses.

It must also coordinate continuously with a concurrent research program in the field of Urban Water Economics, whose feasibility is likewise under current investigation by ASCE.

The required level of effort to complete this research is also relevant to the overall feasibility of such an undertaking. WRE has estimated preliminarily that such research would require at least six years, 465 man-years of effort and $15,900,000. That appears to be a rather modest outlay of time, money, men, and materials, considering what the national urban water budget must be, much less the budget for just one city.

Although it should be clearly understood that computer programs developed in the research effort suggested here will not solve all urban problems for all times, it is WRE's opinion that the methodologies uncovered and the insight gained to urban water subsystem interactions during this research would clearly justify the expense and would improve urban engineering remarkably. It should also be emphasized that, in WRE's view, systems analysis should have as its ultimate goal, the improvement of decision-making capability. This implies that simulation alone, although a worthy task, has little purpose if the results of one simulation are not compared to results of another to choose the one that is best, according to some specified and presumably formulated objective. In short, the feasibility of a systems engineering analysis of all aspects of urban water is supported most heavily by the ability it would provide to choose optimally from among a large group of technological alternatives. This goal is as ancient as engineering itself, but the ability to attain it is virtually brand new and growing.

7.2 Conclusions

WRE's major findings and conclusions are summarized as follows:

1) It is possible to describe a given urban region by its boundaries and boundary conditions, to develop systems analysis techniques to describe this region's subsystems, and to model this system's operation under specified "loads" or inputs described by pertinent data.
2) It is not possible and likely will not become possible to write one program that will describe any urban area's complete water situation. A core or executive program can probably be prepared that, with relatively minor refinements plus necessary additions and subtractions of subprograms, will operate satisfactorily on data describing several or many urban areas. The crucial point is that to use the program successfully for another city than the one for which it is originally developed, modifications will be necessary.
3) "Systems analysis" is a body of techniques that can materially aid the study of technological subsystems through simulation of many possible alternative responses to specified inputs. Its ultimate goal, however, should be to support the decision-making process through optimization analyses that indicate which alternative project is "best," and hence, the one to adopt.

Chapter 10

THE NATURE OF CHANGES IN URBAN WATERSHEDS AND THEIR IMPORTANCE IN THE DECADES AHEAD

December 1968
ASCE Urban Water Resources Research Program
Technical Memorandum No. 5
New York, New York

1.0 FOREWORD

The ASCE Urban Water Resources Research Program was developed and initiated by the ASCE Urban Hydrology Research Council (predecessor of the UWRRC). Its basic purpose is to help establish coordinated long-range research in urban water resources on a national scale. The program currently consists of two major projects: "Research and Analysis of National Basic Information Needs in Urban Hydrology," sponsored by the Geological Survey; and a "Systematic Study and Development of Long-Range Programs of Urban Water Resources Research," sponsored by the Office of Water Resources Research. Both agencies are in the U.S. Department of Interior. Work on the projects began in the fall of 1967.

This technical memorandum contains a paper presented at the Conference on *The Effects* of *Watershed Changes on Streamflow* at the University of Texas at Austin on October 29, 1968. The subject of the paper, "The Nature of Changes in Urban Watersheds and Their Importance in the Decades Ahead," was suggested by the conference coordinator, Dr. Walter L. Moore of the University of Texas at Austin.

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2.0 ASCE RESEARCH PROGRAM IN URBAN WATER RESOURCES

Several facets of research on urban water problems have long been neglected. This is true in spite of the fact that, for urban drainage, works alone seldom cost less than $100 per capita, and in the United States not less than $25 billion in direct capital costs will be spent on drainage works in the next 40 years. This does not include the cost of damage to property and life from flooding. Compared to this staggering figure, funds being spent to research urban water problems are minor and almost non-existent.

To bring attention to these problems, and to begin to fill the gaps in present knowledge, a program to study and report on research and data needs for urban water problems has been developed and initiated by the ASCE Urban Hydrology Research Council. The purpose of the ASCE project is to establish a coordinated long-range national program of research in urban water resources. The first year of the project has the objectives of developing guidelines and documenting the need for greatly increased support of research and research-support facilities.

The program currently consists of two major projects, each sponsored by an agency of the U.S. Department of the Interior. They are: 1) "Research and Analysis of National Basic Information Needs in Urban Hydrology," sponsored by the U.S. Geological Survey, and 2) a "Systematic Study and Development of Long-Range Programs of Urban Water Resources Research," sponsored by the Office of Water Resources Research.

Work on the projects began in the fall of 1967, and all phases of the projects are now in full operation. Current commitments for the first phases of the projects are for one year, but it is expected that each will be extended for at least one additional year. The combined first year cost for both projects is estimated at $140,000. The program is directed and coordinated for the ASCE Research Council by M.B. McPherson and L. Scott Tucker, located at Harvard University.

1) Under the USGS-sponsored portion of the program, an intensive study is being made of the types of data needed for improved design of storm drainage facilities, including both quantity and quality of drainage flow needs for data collecting instrumentation; and for appraisal of types of networks necessary to collect adequate data. The ultimate objective is to facilitate transfer of data findings between metropolitan regions. Continuation of the project is expected to lead to the development of a metropolitan water intelligence system.

   The study is being pursued with Task Groups on three subjects: Data Needs, Data Devices, and Data Network. The project technical liaison representative for the U.S. Geological Survey is William J. Schneider.

2) The objectives of the OWRR sponsored portion of the program are to provide guidelines for initiating and expanding a program of long-range studies on urban water problems. This includes:

   a) Conduct a pre-feasibility study to determine the possible effectiveness, cost, and time requirements for a comprehensive systems-engineering analysis of all aspects of urban water;

   b) Conduct a pre-feasibility study to determine the possible effectiveness, cost and time requirements for a general economic analysis of costs and pricing parameters of all aspects of urban water;

   c) Conduct a state-of-the-art study of mathematical models and related simulation methods potentially usable for analyzing urban rainfall-runoff-quality processes;

   d) Study of requirements for the assessment of drainage damage and exploration of alternatives to direct stormwater runoff, such as the utilization of recharge basins or other storage schemes;

e) Study of political, economic, legal and social problems related to urban water management with recommendations for further work; and

f) Preparation of appropriate reports of the research results of the project for publication in engineering and professional journals.

In carrying out points a. and b., above, two sets of studies are being made to determine the possible effectiveness, cost, and time required for comprehensive systems analysis of all aspects of urban water management. Two studies—one by the Economics Systems Corporation of Washington, D.C., a subsidiary of AVCO Corporation, and the other by Water Resources Engineers, Inc., of Walnut Creek, California—will concentrate on the engineering system. Two other organizations—Arthur D. Little, Inc., of Cambridge, Massachusetts, and Autonetics, a Division of the North American Rockwell Corporation of Anaheim, California—will each conduct studies, concentrating on the economic system.

A major part of the OWRR study is being pursued by Task Committees on the three subjects of Methods of Analysis, Damage and Storage, and Non-Hydrologic Aspects. The project technical liaison representative for the OWRR is George F. Mangan, Jr.

Later phases of the OWRR projects would be expected to include a continuing review and revaluation of research needs and studies of projected metropolitan water management needs. They would also stimulate conduct and support of needed research, initiate and recommend ways of developing effective communication among those working on urban water problems, and design a comprehensive model simulating the engineering economic urban water system.

The two current projects of the urban water resources research program are being given general direction by the Control Group of the Urban Hydrology Research Council, which serves as the Project Steering Committee. S.W. Jens (chairman), D.E. Jones, Jr., J.C. Geyer, C.F. Izzard, and W.C. Ackermann are the members of the Steering Committee. Administrative support is provided by Research Manager Donald C. Taylor at ASCE Headquarters in New York. Reports on the first year of work are due in the fall of 1968.

Another project in progress under the direction of the Urban Hydrology Research Council is the ASCE Combined Sewer Separation Project, reported in *Civil Engineering* in the December 1967 issue.

THE NATURE OF CHANGES IN URBAN WATERSHEDS AND THEIR IMPORTANCE IN THE DECADES AHEAD

December 1968
ASCE Urban Water Resources Research Program
Technical Memorandum No. 5
New York, New York

> *Prognostics do not always prove prophecies, at least the wisest prophets make sure of the event first.*
>
> (Horace Walpole 1785)

> *"Prophecy is the most gratuitous form of error. "*
>
> (George Eliot in *Middlemarch*)

The dynamics of urban change are largely economic and social in character. Extensive use is made in the paper of quotations describing views from disciplines other than engineering.

1.0 INFLUENCES OF THE PAST

> *In the latter half of the nineteenth century the railroad facilitated the clustering of urban population in narrowly confined areas. During the past fifty years, and especially since the end of World War II, electric power and the internal-combustion engine have made it possible for urban land-uses and urban ways of life to expand widely into the countryside. New industries have contributed substantially to this expansion.*
>
> (Meier 1966)

> *If any single invention can be credited with shaping the growth of the metropolitan area it is the streetcar, which was unchallenged as the premier vehicle of mass public transit until the late Twenties. Just about then, but for the first of two historic upheavals, the city would have felt the great impact of the automobile. Instead, the impact was postponed, first by the great depression, and then by World War II.*
>
> (Editors of *Fortune* 1958)

> *During this century, the United States has been transformed from a predominantly rural society into a predominantly urban one. As a result, we are becoming, even more than in the past, a land-surplus nation. More than half the counties in the United States have lost population during the last twenty years.*

(Revelle 1967)

Since 1945 our prosperity, increased birth rate, and advancing technology have greatly accelerated the process of urban growth. Our urban expansion has resulted in a sprawling metropolitan region made possible by the private automobile, the extension of power and telephone connections, all-weather roads, the lowly septic tank, and other technological changes.

(Fiser 1962)

When scarcely any but the well-to-do lived in suburbia, a home there was a desirable goal; now (1958) it is becoming a social imperative.

(Editors of *Fortune* 1958)

The principal effects on the process and character of urban growth have been (Dickinson, 1966):

1) The Highway-development along highways radiating from cities, satellite sub-centers beyond the city, and agglomerations of sub-centers about villages and towns, in a centrifugal trend.
2) The "suburbanization of industry."
3) The growth of employment in services.
4) Obsolescence of buildings located around the central commercial districts of central cities.
5) The rapid acceleration in housing and the spread of urban areas outward along the main lines of communication.
6) The fantastic increase in tourism.
7) Ever increasing daily transportation of urban populations.

One problem is 'sprawl,' the scattered pattern which makes utilities, roads and public services either inadequate or very expensive, and leads to chaotic local government.

(Wurster 1960)

Already (1958) huge patches of once green countryside have been turned into vast, smog-filled deserts that are neither city, suburb, nor country, and each day—at a rate of some 3,000 acres a day—more countryside is being bulldozed under.

... Of 465 million acres of cropland in the U.S., only 72 million are Class I land—and over half of this highly fertile Class I acreage is in urban areas. ... The problem is the pattern of growth—or rather, the lack of one. ...There is a

surprising amount of empty land even within the limits of most cities because of the leapfrog nature of urban growth. ...Sprawl also means low-volume utility operation for the amount of installation involved.

(Editors of *Fortune* 1958).

We are reminded that there are over 1,000 governmental units in the New York Metropolitan Area and almost as many in Cook County, Illinois.

(Fiser 1962)

3.0 ASPECTS OF THE PRESENT SITUATION

.. .metropolitan problems are created by exactly the same nation-wide forces that have produced our rising national income. They are the price of national growth and progress in the most direct and literal sense, quite as much so as the problem of rural relief and over-production. "

(Wurster 1960)

In 1960 something like $25 billion was spent on passenger transport in urban and metropolitan areas.

(Wurster 1960)

The fact is that it will never be possible to provide parking space in the largest cities for all the motorists who want to come to them... If all of New York's transit riders drove in by automobile, for example, all of Manhattan below Fiftieth Street would have to be converted to multiple-deck parking garages.

(Editors of *Fortune* 1958)

About half of the manufacturing employment in the larger metropolitan areas is already (1960) outside the central cities, and this trend is likely to continue.

(Wurster 1960)

While difficult to document, it appears that the worker population has increased little if any in the downtown business districts of most big cities. The new jobs for the suburbanites are jobs within—and often beyond—the suburbs.

(Editors of *Fortune* 1958).

> *...in almost every major metropolitan city the slums envelop and squeeze the core of the city like a Spanish boot. ...The reason is that these core areas are the oldest areas; the housing around them is likely to be fifty or even a hundred years old and, therefore, prone to slum formation.*

(Editors of *Fortune* 1958)

Many cities have been suffering not only steady cost increases but also a slowly growing or declining tax base in the face of difficulties in increasing services.

> *For example, education is the single biggest form of expenditure by local governments (including school districts), accounting for about 40% of their outlays. ...To offset even half the growth estimated for the ghetto by 1975 would call for the out-migration from central cities of 217,000 persons a year.*

(National Advisory Commission on Civil Disorders 1968)

Some major consequences of urban expansion are (Dickinson, 1966):

1) An impact of urban land uses on agricultural land uses.
2) A continuing increase in water withdrawals.
3) Urbanization often moves faster than supporting utilities; and pollution of water is aggravated.
4) Available open space steadily decreases.
5) A need for drastic revision of boundaries of municipalities.
6) Transport inadequacies at all levels.
7) A need for a new geographic base for taxation.
8) "Lastly, the central cities, confronted with the flight of people and their money to the suburbs, have suffered for several decades from the depreciation of land and property value and thus of tax revenue; from the steady deterioration of buildings; and the virtual stoppage of building in the Depression years and then during the war."

> *Pollution of the environment is a product of our age, resulting from increase of human population, from technological activity, and from the linked phenomenon of urbanization. At the moment we are losing the battle for man's sense of well-being.*

(Darling, 1967)

> *There is, in general, an appalling lack of information on the causes and consequences of environmental deterioration.*

(Wildavsky 1967)

4.0 IMPLICATIONS FOR THE FUTURE

The greatest task associated with the doubling in population of the urban areas (by the turn of the century), and a multiplication by three or four of the space they occupy, is the orderly distribution of metropolitan services despite the political fragmentation of the metropolitan regions.

(Meier 1966)

In cold figures, the 25 million acres or so occupied by the urban population at this time is less than 1.5% of the country's surface. Highways, railroads, and airports take up perhaps 27 million acres, for a grand total of built-up terrain, of say, 50 million acres, not quite 3% of the face of America. By the end of this century, this might grow by 50%, to 75 million acres, due overwhelmingly to expansion of urban land use.

(Landsberg 1967)

Economic development is clearly reflected in shifts in occupational structure. ...The continuing tempo of economic growth will mean the transfer of more people from agriculture to industry and service and thus, the continued growth of the urban population. In most western countries there is every indication that service occupations, as opposed to agricultural, and manufacturing and mining, ...have constituted the most rapidly increasing employment structure during this century. ...These changes are rooted in technological developments.

(Meier 1966)

Water and climate will impose the only environmental restrictions on location of urban centers throughout the temperate portions of the continent. ... To achieve an adequate supply and distribution of urban open space, and to maintain that open space in a sound natural condition, it will be necessary to place limitations on the size and density of urban centers. ...the health of (man's) environment is promoted by sustenance of variety.

(Strong 1966)

Diversity, be it ever so little, has a value in relieving stress.

(Darling 1967)

Natural environments represent assets of appreciating future value.

(Krutilla 1967)

> *Automation and prosperity will tend to reduce the proportion of industrial jobs, increase employment in the urban professions and services, and shorten the hours of work. ...The demand for outdoor recreation is likely to increase ten-fold in 50 years, simply in terms of the expected growth in population, income, leisure and mobility.*

(Wurster 1960)

> *The people in a big urban region need open space for many different purposes: to conserve water and other natural resources; as a reserve for future needs, often unpredictable; to maintain special types of agriculture that must be near cities; to prevent building in undesirable locations in order to avoid flood hazard or a wasteful extension of services; to provide a rural environment for people who want to live that way; for pleasant views from urban areas; for a sense of urban identity; for buffers against noise and other nuisance; but above all, for recreation, which can be combined with many of the other uses.*

(Wurster 1960)

> *...the explosive demand for outdoor recreation in the United States suggests that there will be considerable pressure to maintain clean water supplies at all times. ...In maintaining water quality, the fundamental problem grows out of the prospect (by the year 2000) of a quadrupling of gross national product, a near doubling of population, and the consequent potential demand for use of streams as waste disposers: In view of this, how clean and sightly should watercourses be kept?*

(Landsberg 1964)

The current national goal appears to be "a minimum of secondary treatment of all domestic, commercial, and industrial wastes discharged to fresh water and for most wastes discharged to salt water," (Moore, Jr., 1968) "and, as effluent quality improves, there will be increasing need and pressure to reduce pollution from agricultural land drainage," (Henderson 1962) combined sewer overflows, storm drain discharges and other sources not currently and commonly intercepted and treated before entering watercourses.

> *Future land-use will be greatly affected by expected enlargement of urbanization. In this process, the landscape of the country will be transformed. Farms will largely vanish from the scene and will be localized in the most fertile agricultural regions. The landscape will return to a semblance of its 18th Century appearance. Instead of being urban or rural as we have known, it will be urban or forest. The forest lands ...will be held for rural non-farm or recreation residential use. Agriculture will be transformed ...to resemble more closely industrial manufacturing processes. There will be a*

new order of conflict at the urban-agricultural interface. The larger urban centers will become the locations of larger and larger percentages of economic activity and, therefore, attract increasing populations. ...An even more striking change will occur concerning the occupancy of the cities. The number of housing units in the Country, and particularly in the urban centers, will increase more rapidly than the population. Continuation of the long-term ...decline in persons per housing unit will be accomplished largely through the individual ownership of multiple housing units. It is already a relatively common phenomenon to find single households owning more than one housing unit for their own purposes. This tendency will grow. Investment in urban plant will grow more rapidly than a proportionate increase in population, as a consequence of the growing use of dual domiciles. While the population will be more concentrated for residential purposes, in terms of its actual location at any instant in time, it is likely to be even more dispersed than it is at present. ...Urban services will be underutilized. At the same time, large segments of the population will be far removed from the urban services at any moment with consequent possible deleterious effects.

(Jones 1965)

5.0 THE NEED FOR COMPREHENSIVE PLANNING

The United States is preponderantly urban, and there is intense urban-based public concern on water resource problems. In most urban areas there is concern with inundation by great floods and also by local storms. There is widespread concern with amplifying the opportunities for water based recreation ready of access to urban residents, and with enhancing the conditions of urban living. A third focus of concern is improvement of systems for metropolitan area water supply and carriage of waterborne wastes. These concerns interact with management of the water resources of a river basin. Often, in fact in most river basins, metropolitan areas may affect and be affected by water resources management at considerable distances. ...metropolitan areas present conditions that are vastly different from the areas for which water resources management technology had been developed heretofore.

(Eaton 1968)

Although the problem of metropolitan regions today is national, indeed international, in scope, we have hesitated to raise the whole question in one forum. We attack only the aspect of water, or of sewage, or of the city slum. ...Comprehensive planning must include both a broad geographic area and a broad spectrum of human values. Mastery of the metropolis cannot be gained with less.

(Fiser 1962)

> However, we are cautioned on the limits of optimality: *No species encounters in any given habitat the optimum conditions for all of its functions.*
>
> (Dansereau 1966)

"Stormwater should be regarded as a 'resource out of place'." *(Conference on Urban Hydrology Research* 1965). In this or a systems context, drainage water and water supply are inextricably linked.

> *It helps to realize initially that in many of its uses, water is either a free or nearly free good and that incentives for economizing are the exception rather than the rule. Thus, projections of future consumption are based more on what people have been led to take for granted as 'needed' than on what they would be willing to buy at prices that more nearly reflect costs. ...As our political and administrative approaches to water management, as well as our costing and pricing mechanisms, receive attention and review, channeling water into the highest-yielding alternatives will assume increasing importance.*
>
> (Landsberg 1967)

6.0 EXPENDITURES FOR STORM DRAINAGE FACILITIES

> *...the annual national investment in urban drainage facilities exceeds that of the Federal Flood Control Program...*
>
> (APWA Research Foundation 1966)

National present estimated values of water facilities are given in Table 10-1, population data in Table 10-2 and projected investments in Table 10-3 (APWA Research Foundation, 1966; U.S. Department of Commerce, Business and Defense Services Administration, 1967). To the estimates of annual average projected investments in water utilities would have to be added the cost of supporting facilities (e.g., stream impoundments), maintenance and operation; and the total direct costs could very conceivably approach $10 billion per year, with no allowance for debt service. While such a sum is only a small fraction of the gross national product, it must be carefully noted that the great bulk of these costs will be defrayed by taxes, fees and rentals levied by local governments. Projected investments in storm sewers are certainly not inconsequential.

About three-fourths of the new storm sewers for growth in Table 10-3 would be constructed by developers, so that fraction would presumably be directly passed on to the new homebuyer as part of the property purchase price. Also, a number of water systems are privately owned and operated, and those water rents are not involved directly in local government taxation-fee structures.

The cost for construction of new sanitary sewerage to effect separation of combined systems of sewerage has recently been estimated at $48 billion. (U.S. Department of the Interior, FWPCA, 1967). Some overlap in costs with those for storm sewers in Table 10-3 would be probable were universal separation to be adopted.

TABLE 10-1

Replacement Values of Public Water, Wastewater and Drainage Facilities

Public Water Systems, Including Treatment Facilities (at 1966 Construction Cost Levels)[1]	= $49.1 billion
Public Sanitary Sewerage and Treatment Facilities (at 1966 Construction Cost Levels)[1]	= $41.3 billion
Public Storm and Combined Sewers (at 1965 Construction Cost Levels)[2]	= 22.0^+$ billion

[1] U.S. Department of Commerce, Business and Defense Services Administration, "Regional Construction Requirements for Water and Wastewater Facilities, 1955-1967-1980," October 1967.

[2] American Public Works Association, "Urban Drainage, Practices, Procedures, and Needs," December 1966.

TABLE 10-2

Estimated Population of Conterminous U.S. in Millions[1]

	1955	**1960**	**1965**	**1970**	**1975**	**1980**
Total Population	164.3	179.5	191.7	206.2	223.4	243.0
Served by Municipal Water Facilities	120.5	135.0	149.8	166.8	185.4	206.0
Served by Municipal Wastewater Facilities	97.9	110.7	124.3	140.0	157.6	177.9

[1] U.S. Department of Commerce, Business and Defense Services Administration, "Regional Construction Requirements for Water and Wastewater Facilities, 1955-1967-1980," October 1967.

TABLE 10-3

Estimated Utilities Construction Costs
For Water, Wastewater and Drainage Facilities

Service	Year for Cost Base	Period for Average	Annual Averages, Millions		
			Deficiencies, Obsolescence, Depreciation	Growth	Total
Water Distribution[1]	1966	1967-1980	$ 758	$ 758	$1,546
Water Treatment Plants[1]	1966	1967-1980	$ 253	$ 264	$ 517
Subtotals			$1,011	$1,052	$2,063

Sanitary Sewage[1]	1966	1967-1980	$ 919	$ 726	$1,645
Sewage Treatment Plants[1]	1966	1967-1980	$ 576	$ 454	$1,030
Subtotals			$1,495	$1,180	$2,675

Storm Sewers[2]	1965	1966-1975	$1,300	$1,200	$2,500

[1] U.S. Department of Commerce, Business and Defense Services Administration, "Regional Construction Requirements for Water and Wastewater Facilities, 1955-1967-1980," October 1967.

[2] American Public Works Association, "Urban Drainage, Practices, Procedures, and Needs," December 1966.

7.0 CONCLUDING REMARKS

The United States is confronted with demands for the resolution of new social values and reversal of the trend in worsening environmental quality; and these are interrelated. The social and physical decay of central cities has proceeded concurrently with the growth of suburbia at the expense of outlying rural areas. Environmental pollution has been exacerbated by increased urban populations and technological activity, and from the associated spread of urbanization. Over 90% of the nation's population is expected to be resident in urban areas by the end of the century. (National Academy of Sciences, National Research Council, 1966). Conversely, continued migration of rural populations to urban areas is a matter of serious concern. (National Manpower Conference for the United States Senate, Committee on Government Operations, 1968).

The greatest concern in urban water will increasingly be on quality; and the contemporary concept of water as a free or nearly free good will necessarily give way to more systematic and equitable delineations of costs and benefits. As costs escalate, ensuing economic pressures will require intensification of reuse and multiple-service development. Increased competition and conflict will be resolved only by truly comprehensive planning. In resolving optimum plans, there will be a struggle for supremacy between achievement of economic efficiency and attainment of social goals including aesthetic considerations.

Compared with river basins, metropolitan complexes are subject to more dynamic changes, yet urban areas constitute only a few percent of the total national geographic area. The key descriptor is intensification: of people, of activity of all kinds, and of competition and conflict. Much improvement in existing water resources planning and analysis tools and methods will be required.

8.0 ACKNOWLEDGMENT

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Chapter 11

SOME NOTES ON THE RATIONAL METHOD OF STORM DRAIN DESIGN

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ASCE Urban Water Resources Research Program
Technical Memorandum No. 6
New York, New York

1.0 INTRODUCTION AND SUMMARY

Sewerage systems and canalized drainage works 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. The principal 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."[1]

The procedure used for design of storm sewers in the United States is almost exclusively the "rational method." This method has substantial liabilities and concerned engineers, such as the members of the ASCE Urban Hydrology Research Council, have long been seeking improved design procedures founded on field observations of the rainfall-runoff process. Very few catchment areas in the U.S. have been gauged.[1] The U.S. Geological Survey is currently assessing information needs, and the purpose of this technical memorandum is to illustrate limitations of the rational method in an effort to help substantiate the urgent need for field data.

Flow in storm 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. The basic catchment area, about one to several acres in size, is that tributary to an inlet. "For most smaller areas in the upper reaches of an urban drainage system the time required to reach peak runoff after the beginning of a storm is a matter of minutes. Hence, high-intensity, short-duration rainfall is normally the main, if not sole, type of precipitation contributing to critical runoff rates. This type of rainfall is usually associated with thunderstorms."[1]

Studies under way by the ASCE Urban Water Resources Research Program, to be reported in a subsequent technical memorandum, indicate that in some principal cities the median size of storm sewerage drainage system catchments is less than perhaps 200-acres in size; but about half the total sewer area of large cities is represented by catchments about 1,000-acres and smaller. To give an indication of relative flow

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[1] Jens, Stifel W., and M.B. McPherson. 1964. "Hydrology of Urban Areas," Section 20 in *Handbook of Applied Hydrology*, edited by Ven Te Chow, McGraw-Hill Book Company, New York, New York.

rates accommodated by storm sewers, "roughly, capacities for storm sewers have been approximately 100 times the capacities provided in sanitary sewers."[2]

Storm sewers have one-directional open-channel flow actuated by gravity, and seldom form closed-loop networks except in a limited sense where catchments are interconnected. A main drain not only transmits upper reach flow to a receiving watercourse, the usual sole function of a relief sewer, but also serves as a collector of surface runoff all along its route. Relief for overloaded storm sewers can often be achieved only by diverting flow from the upper reaches of the catchment in an 'express' outfall. A relief sewer is a very expensive alternative to the provision in the first place of main drains of adequate size to meet future flow increases. The cost of an error in design, as in water resources development generally, is thus, deferred, and the cost of rectification is almost universally greater than would have been the cost of adequate facilities at the outset.

The rational method applies to a very unique set of assumed conditions. Once a system is designed using the fixed features inherent in the method there is no logical way to analyze modifications, such as provisions for relief, reusing the method. Also, because there is no direct way to verify the method in the field, even the adequacy of the original design cannot be checked.

Three factors affect the magnitude of a design flow in using the rational method: C-value, inlet time and the frequency for the rainfall intensity-duration curve used. Computed flows are larger as the C-value is raised, as the inlet time is shortened and as a curve for a rarer rainfall frequency is used. Intelligent, though arbitrary, selection of values for these three variables has been found to give ostensibly "satisfactory" results in a number of cities. There are a number of factors which may contribute towards conservatively "safe" designs, such as the usual practice of designing sewers to accommodate at least the design flow rate at a flowing-full condition, whereas some degree of surcharge might be sustained without flooding. The probability of all design assumptions being satisfied simultaneously is less than the probability of occurrence of the rainfall rate used in the design, contributing in effect to a "safe" design. "Sewer design practice data from a number of municipal sewer designers show extensive differences in the methods and design factors used for storm sewer capacity design computations."[2]

"There is an obligation to the public, insofar as it is practicable, to equate the cost of a given design to the probable protection and service which it will afford. Otherwise, an adequate criterion for the 'most economical design' does not exist."[1] The tenuous linkage between the design rainfall frequency of the rational method and frequency of flooding have precluded evaluation of economic criteria.

In later sections of this memorandum an attempt will be made to indicate that the range of variability in the rarer peak flows of design significance might be much less

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[2] Stanley, William E., and W.J. Kaufman. October 1953. "Sewer Capacity Design Practice." *Journal*, Boston Society of Civil Engineers, Volume 40, No. 4.

than for natural catchments, suggesting that a high degree of precision must be achieved in gauging peak flows and associated rainfall if subtle differences occasioned by presently unidentified variables or characteristics are to be separated and quantified.

The essence of the immediately preceding arguments is that there is wide latitude in interpreting the concept and in the application of the rational method; there appears to be considerable regularity in the damping of rainfall input variability by the urban system; and the apparent "success" of the method may well spring from a combination of a rather limited range of possible error in applying any method and conservatism arising from cumulative favorable factors, albeit inadvertent, inherent in the rational method. Under prevailing circumstances a debate on questions of under design or over design would be completely academic.

Attempts have been made to develop alternative improved methods for flow determinations based on hydrologic fundamentals, but these have, thus far, been usable only in the locality for which they were developed or their veracity has not been adequately confirmed by means of field gauging verification.

To state the situation very simply, development of improved design methods has been stymied for decades because of a lack of a suitable national field gauging rainfall-runoff program. Mathematical models exist that could quite likely lead to vastly improved design methods were the field data available for their calibration and refinement. There is a real need to account for storm time and space variability in urban water developments, including storm drainage, but the impetus for research on metropolitan storms is inhibited so long as no tangible progress is made in the collection and analysis of rainfall-runoff data.

Essentially, an impasse exists. The foregoing criticisms of the rational method are not intended as a recommendation that its use be abandoned. Unless, and until, improved methods are developed, it is about as satisfactory as any other over-simplified, empirical approach.

More refined and extensive hydrologic information is required to develop adequate technical knowledge for reliable planning of water quality and quantity exchanges between the several urban water service functions for multi-purpose development; and for quantifying pollution loadings from combined and storm sewerage systems applicable nationwide. The rational method is of no use whatever for these purposes. However, if adequate information can be secured on rainfall-runoff-quality processes, the collective needs for storm sewer design, water exchange developments and evaluation of pollution loadings can be met simultaneously. It appears that a high degree of field gauging precision will be needed to achieve any one or all three of these objectives.

The value to the public of improved knowledge on stormwater pollution and methods of analyzing exchanges between service functions might substantially exceed the value from improved storm sewer design. As will be detailed in the final report for the USGS project, average annual expenditures over the next several years in the United States for urban storm drainage facilities is estimated at about $3.5 billion per year: $2.5 billion by local governments and $1 billion by developers. If only 5% of these projected costs could be saved by means of improved design criteria, the average annual savings would be almost $200 million per year.

2.0 BACKGROUND

The origins of the "rational method" of storm sewer design in the United States are generally traced[3] to Kuichling.[4]* "More than 90% of the engineering offices throughout the United States that replied to a questionnaire on storm sewer design practice indicated use of the ... method" and it "must be considered current practice."[3] Practice has changed little since 1960. The rational method has been the predominant approach used for sizing storm sewers (separate and combined) over the last half of the 80-years since Kuichling's 1889 paper was published.

Kuichling deplored the conduit sizing methods employed by his contemporaries, which were wholly empirical and founded on weak logic. If design peak flow rate was related to rainfall at all, the basis was a 1-hour or greater duration rain regardless of catchment area size. The central theme of Kuichling's paper was a call for recognition of the variability of rainfall so that storm sewers could be sized more realistically and reliably.

> *The writer was long since impressed with the fact that during showers the volume of water discharged at the mouths of several large outlet sewers in the City of Rochester, New York, appeared to increase and diminish directly with the intensity of the rain at different stages, but that a certain length of time was required in each case after the termination of a brief and heavy downpour before the corresponding flood showed itself at the outfall; these floods, moreover, seemed to last about as long as the said showers themselves, and the conclusion was, therefore, reached that there must be some definite relation between these fluctuations of discharge and the intensity of the rain, also between the magnitude of the drainage area and the time required for the floods to appear and subside. The conclusion is accordingly irresistible that the rates of rainfall adopted in computing the dimensions of a main sewer must correspond to the time required for the concentration of the drainage waters from the whole tributary area when small, or from so much thereof as will produce an absolute maximum discharge when the area is very large.*

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[3] *Design and Construction of Sanitary and Storm Sewers*, ASCE Manual of Engineering Practice No. 37 (or WPCF Manual of Practice No. 9), 1960, revised in 1961 printing. (Second edition planned for publication in 1969).

[4] Kuichling, Emil. 1889. "The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts." *Transactions*, ASCE, Volume 20, pp. 1-56.

* A.K. Biswas states that the "rational formula originated in Ireland around the 1840s." *Civil Engineering*, November 1967, p. 77.

> *In justification of the prevalent practice, however, it is fair to say that the customary methods of observing the rainfall have hitherto prevented the development of the process just indicated, and have forced engineers to the use of average rates of precipitation deduced from periods of time scarcely ever less than one hour in duration.*[4]

Interpretations of data from Providence, Rhode Island:

> *have since been widely copied by many distinguished writers in proof that great intensities of rainfall are exceedingly rare occurrences in our climate. Such inferences are, however, certainly unwarranted by the records derived from automatic rain gauges, which indicate that heavy rates for short periods of time are very common occurrences.*[4]

> ... *it is now urgently advised to make use of a large number of automatic rain gauges scattered about in the various separate drainage basins and located not more than about 1,500 feet apart, if only a single season be allowed for the collection of statistics... Measures should also be taken to cause every Signal Service station in the cities of our country to be equipped with the best automatic gauges, and to have the results thus obtained carefully tabulated, in order that communities may be spared the great outlay of soon reconstructing sewers whose dimensions were based upon the deceptive records of rainfall as heretofore kept.*[4]

(The U.S. Army had responsibility for weather measurements at that time).

> *Numerous other instances of severe rainfall might be cited; but the foregoing will doubtless suffice to exhibit not only the exceedingly variable character of the rate and duration of heavy storms, but also the necessity of carefully studying the data afforded by each particular locality.*[4]

Kuichling was among the first to recognize the relationship between intensity and duration.

> ... *the maximum uniform intensity of the rainfall diminishes rapidly as its duration increases from a few minutes to one hour, and for rains of uniform intensity lasting more than one hour the rate of diminution is comparatively slow.*[4]

Regarding both U.S. and European practice, the

> *relation between the magnitude of a drainage area, the surface discharge, and the time required for the concentration of such discharge, has long been recognized in a general way, but does not appear to have been very definitely expressed. Several attempts have been made to express the general principles in mathematical terms, but without much success from a scientific standpoint.*[4]

Kuichling made rainfall and outlet-flow peak-stage measurements on five Rochester catchments varying between 25 and 357 acres in size, reporting on his findings in some detail.

> *The results obtained from the gauges used in the work are susceptible of much criticism, yet it must be remembered that not only were there no precedents for the undertaking available, but also that the appliances and methods of observation adopted were, on the whole, much more trustworthy than in the case of similar published experiments elsewhere.*

On the basis of his own data he argued that ...

> *it has been seen that the flood-volume stands in direct proportion to the magnitude of the impervious surface on the drainage area, and to the intensity and duration of the rain; also that such flood-volume reaches practically a maximum when the precipitation continues uniformly for a sufficient length of time to secure the concentration of the stormwaters from all portions of the area. The element of time, therefore, enters twice into the determination of the flood volume, and from the relation between duration and maximum intensity of the rainfall in this locality heretofore established we may accordingly find the duration of that particular rainfall for which the sewer-discharge will become an absolute maximum.*[4]

Obviously, the concept of extreme value probabilities was foreign to Kuichling and his contemporaries and remains the only recognizable major modification since his time in the method he advanced. However, he deduced that peak runoff was directly proportional to the relative amount of catchment area impervious surface and succeeding practice used generally lesser fractions that only partially reflected total imperviousness. Kuichling called for a recognition of antecedent soil moisture conditions in application of his findings, but this consideration was rather universally ignored, probably because designers in the first part of this century became predominantly enamored of great elaborations in intensity-duration-frequency curve fitting procedures and arguments over pet runoff coefficients.

Kuichling's concluding paragraph ends with the following:

> *The above investigations, moreover, show larger quantities of stormwater runoff from urban surfaces than is commonly supposed, and hence it is obvious that a more rational method of sewer computation is urgently demanded. Much room for improvement in this direction is still left, and it is sincerely hoped that the efforts of the writer will be amply supplemented by many valuable suggestions and experimental data that other members of the Society may generously contribute.*[4]

In fairness to Kuichling's memory, the "rational method" should be called the "more rational method of the 19th Century." His expectation of further development has not truly been met to this day. However, there have been a few, isolated exceptions to be cited later.

3.0 PRINCIPLE OF THE RATIONAL METHOD

Kuichling stated that "in drainage areas of moderate size, the heaviest discharge always occurs when the rain lasts long enough at its maximum intensity to enable all portions of the area to contribute to the flow."[4] He referred to the time required "for the full concentration of the stormwater at some point of discharge," and this has since been generally termed the "time of concentration," defined as the "time required for runoff from the remotest part of the drainage area to reach the point under design.[3]

The time of concentration for sewered areas is regarded as having two components, an "inlet time" and a "time of travel" through underground sewers. The inlet time is "the time required for rain falling on the most remote point of the tributary area to flow across the ground surface, along pavement gutters to the street inlet, and through the inlet into the sewer"; and the time of travel relates to residence "within the sewer from the uppermost inlet to the concentration point under consideration."[5]

Rainfall data used are in the form of "intensity-duration-frequency" curves; and these are discussed in some detail in the next section. Suffice it to say at this juncture that for a given drainage area, a set of mean recurrence interval (or return period, or frequency, engineers seldom distinguishing their semantic differences) maximum average rainfall intensities for various durations are used. To compute the design flow at a given point, the maximum average rainfall intensity, for a duration equal to the time of concentration, is entered in an equation of the form:

$$Q = CiA$$

where Q is the rate of flow at the design point, i is the rainfall intensity, A is the size of drainage area tributary to the design point and, C is a coefficient. Because 1-cubic-foot-per-second of flow (cfs) is a rate very nearly the same as for 1-inch/hour of rainfall on an acre of area (1 in./hr. = 1.008 cfs/acre), the coefficient C is essentially non-dimensional when Q is in cfs, i is in in./hr. and A is in acres, all being convenient and common forms of units.

The basic procedure will be illustrated using a hypothetical example, Figure 11-1. Although discussion of procedures for determining time of travel must be deferred until later on, for the purpose of illustration a constant flow velocity of 5-ft./sec. will be employed, or a time of travel of 1-minute per 300-feet of drain. Further, let us use an inlet time of 5-minutes at all inlets. Inlets are shown schematically in Figure 11-1, at the corner of each sub-area. Design rainfall in in./hr. will be represented by i = 90/(t+10), where t is in minutes.

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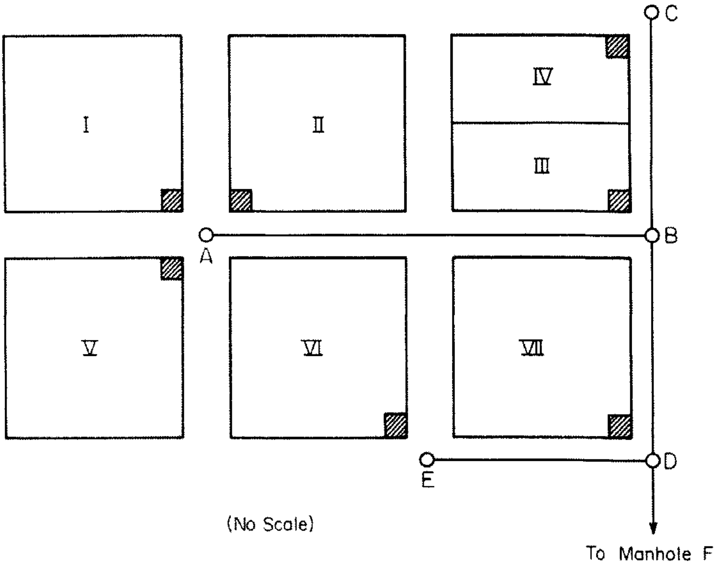
[5] Greeley, Samuel A., and William E. Stanley. 1952. "Sewerage." *Handbook of Applied Hydraulics*, edited by Calvin V. Davis. McGraw-Hill Book Company, New York.



Lengths, feet:

A-B = 1,200
C-B = 600
B-D = 750
E-D = 450
D-F = 900

Area	Acres	C
I	0.4	0.70
II	0.3	0.60
III	0.2	0.65
IV	0.2	0.80
V	0.4	0.75
VI	0.6	0.80
VII	0.8	0.80

FIGURE 11-1. Hypothetical Example

Design rate of flow:

Manhole C:

Time of concentration = 5-min.; via sub-area IV;
Therefore, i = 90/15 = 6.0 in./hr.; and
Q = 6.0 in./hr. (0.80 x 0.6 acres) = 0.96 cfs, the design rate for drain C-B.

Manhole E:

Time of concentration = 5-min.; via sub-area VI;
Therefore, i = 90/15 = 6.0 in./hr.; and
Q = 6.0 in./hr. (0.80 x 0.6) = 2.9 cfs, the design rate for drain E-D.

Manhole A:

Time of concentration = 5-min.; via sub-areas I, II or V;
Therefore, i = 90/15 = 6.0 in./hr.; and
Q = 6.0 in./hr. (0.70 x 0.4 + 0.60 x 0.3 + 0.75 x 0.4) = 4.5 cfs, the design rate for drain A-B.

Manhole B:

Time of concentration = 5-min. inlet time to manhole A plus 4-min. travel time in drain A-B (the time via drain C-B would be shorter, i.e., 7-min., and from sub-area III would be only 5-min.);
Therefore, i = 90/19 = 4.7 in./hr.; and
Q = 4.7 (0.70 x 0.4 + 0.6 x 0.3 + 0.65 x 0.2 + 0.80 x 0.2 + 0.75 x 0.4) = 5.0 cfs, the design rate for drain B-D, serving sub-areas I through V.

Manhole D:

Time of concentration = 5-min. inlet time to manhole A plus 6.5 min. via A-B-D (the time via C-B-D would be 5 + 4.5 = 9.5 min.; via E-D would be 5 + 1.5 = 6.5 min.; and from sub-area VII would be 5-min.);
Therefore, i = 90/21.5 = 4.2 in./hr.; and
Q = 4.2 (0.70 x 0.4 + 0.60 x 0.3 + 0.65 x 0.2 + 0.80 x 0.2 + 0.75 x 0.4 + 0.80 x 0.6 +0.80 x 0.8) = 6.1 cfs, the design rate for drain D-F, serving sub-areas I through VII.

4.0 INTENSITY-DURATION-FREQUENCY RELATIONS

In application of the rational method for a given design location, a rainfall intensity for a duration equal to the time of concentration for that location is used. Extremes such as the maximum-recorded intensity are not used, because provision for near-complete protection from storm sewer flooding can very rarely be justified. Instead, some intermediate level of protection is provided, e.g., by using 5-year frequency intensities. Adoption of specified frequency levels requires the availability of a historical record of some sort. The almost universal data source is the U.S. Weather Bureau, from its first-order stations in principal cities. It should be carefully noted that each station represents a single geographic point, or spatial sample.

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 USWB 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."[6] 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 gauge. 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 hereafter 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. The formula employed for plotting position, or data mean recurrence interval, depends on whether an annual series or exceedance series is arrayed. Regardless, a mathematical fit is made to the array of depths or intensities for each duration (the USWB has applied a "frequency analysis by method of extreme values, after Gumbel") 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. The result is exemplified in Figure 11-2, in the next section.

The attention of the reader is directed to Table 11-1, where a partial array for a 35-year record period at the Chicago USWB gauge[7] is presented. First, note that depths for a given data mean recurrence interval (equivalent to a given frequency curve)

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[6] "Excessive Precipitation. As defined by the U.S. Weather Bureau, *excessive precipitation* is any precipitation that falls at a rate equaling or exceeding that indicated by the following formula for durations of 5-, 10-, 15-, 20-, 30-, 45-, 60-, 80-, 100-, 120-, 150- and 180-minutes: $P = t + 20$ where P is the precipitation in hundredths of an inch, and t is the time in minutes."

"Prior to 1936, the published data generally present the accumulated precipitation amounts during storms in which the rate equaled or exceeded $P = t + 20$, the tabulation beginning with the 5-minute period where the rate of 0.25 in. in 5-minute began and continuing by 5-, 10-, or 20-minute intervals for as long as the excessive rate prevailed. However, the tabulations for 1933, 1934, and 1935 were continued for a total period of 120-minutes, even though the excessive rate terminated sooner. It can be seen that the amount listed for a given duration under the method of tabulation used until 1936 is not necessarily the maximum for that duration."

"Beginning with 1936, tabulations of excessive precipitation give maximum precipitation for periods from 5- to 180-minutes, the amounts being for the periods in which the fall was actually the greatest for the particular duration. Thus, for example, the amount listed for a particular duration, say 5-minutes, is the greatest amount for any 5-minute interval of the 180-minute period. From 1936 to 1948, the lower limit of excessive precipitation for the states of North Carolina, South Carolina, Georgia, Florida, Alabama, Mississippi, Tennessee, Arkansas, Louisiana, Texas and Oklahoma was defined by the formula $P = 2t + 30$, but the use of the formula $P = t + 20$ for the entire United States was resumed in 1949."

"Summaries of excessive precipitation data have appeared in the Annual Report of the Chief of the Weather Bureau from 1895 to 1934, in the United States Meteorological Yearbook from 1935 to 1949, and since that time in the annual issues of *Climatological Data, National Summary*. The techniques used throughout the years are described in detail in *Key to Meteorological Records Documentation 3081* prepared by the Weather Bureau and available from the U.S. Superintendent of Documents."

(From Section 9, "Rainfall," by Charles S. Gilman, in *Handbook of Applied Hydrology*, V.T. Chow, editor, McGraw-Hill, Inc., New York, 1964.)

[7] Chow, Ven Te. 1953. *Frequency Analysis of Hydrologic Data with Special Application to Rainfall Intensities*. University of Illinois Engineering Experiment Station Bulletin No. 414, Urbana.

include depths that are not necessarily from the same storm, e.g., see rank No. 5. Second, note the accumulated depths versus maximum depths example at the foot of the table, where their order is identical for the June 26, 1932 storm but grossly mixed for the August 11, 1931 storm. Third, note the entry of an "extended duration value" in the upper part of the table. Referring to the foot of the table, for the storm of June 26, 1932, no rainfall occurred after the 10-15-minute interval. In order to preserve the homogeneity of sample size for statistical analysis, a dummy value is inserted for all durations beyond the cessation of excessive precipitation for a given storm, viz., the depth for the longest actual duration. Thus, for the June 26,1932 storm an amount of 1.16-in. would be entered for durations of 20-, 30-, etc. minutes through 180-minutes. Hence, the 25-minute maximum depth of 1.39-in. for rank No. 6 never really occurred. Some storms with excessive precipitation last as short a time as five or ten minutes. The approximate typical duration of intense rainfall in a thunderstorm is about 30-minutes, and a duration greater than an hour is rather unusual. Longer periods are usually associated with cyclonic storms.

TABLE 11-1

Intensity-Duration-Frequency Data

For Chicago USWB Station,
1913-1947 (n = 35 years),
Partial Listing.

Return Period, Years			Maximum Depths in Inches and Data for Stated Duration				
Rank	Annual	Exceedance	5-minute	10-minute	15-minute	20-minute	25-minute
1	36	35	.61 September 13, '36	1.1 September 13, '36	1.29 September 13, '36	1.45 September 13, '36	1.64 September 13 '36
2	18	17.5	.58 July 6, '43	0.96 August 11, '31	1.22 August 11, '31	1.39 August 11, '31	1.58 June 20, '28
3	12	11.7	.55 June 26, '31	0.94 June 26, '32	1.16 June 26, '32	1.39 August 11, '23	1.49 August 11, '23
4	9	8.8	.53 June 29, '20	0.92 July 6, '43	1.16 June 20, '28	1.38 June 20, '28	1.45 July 7, '21
5	7.2	7.0	.51 August 11, '31	0.88 June 20, '28	1.15 July 6, '43	1.35 July 7, '21	1.45 July 6, '43
6	6.0	5.8	.50 June 20, '28	0.80 August 11, '23	1.12 August 11, '23	1.32 July 6, '43	1.39* August 11, '31

* Extended duration value

Date	Duration of Excessive Rate	Accumulated Depths					Maximum Depths				
		5-Min	10-Min	15-Min	20-Min	25-Min	5-Min	10-Min	15-Min	20-Min	25-Min
June 26, '32	15-Min	0.55	0.94	1.16	---	---	0.55	0.94	1.16	1.16**	1.16**
August 11, '31	20-Min	0.26	0.71	1.22	1.39	---	0.51	0.96	1.22	1.39	1.39**

Clock Time of occurrence, June 26, 1932, Min:	0-5	5-10	10-15	None	None
Clock Time of occurrence, August 11, 1931, Min:	10-15	5-15	0-15	0-20	None

** Extended duration values. These values would be used for all larger durations to and including 180-minutes; and would affect I-D-F curve fit in accordance with their respective rank for the individual durations.

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 5-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 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, perhaps separated by several years. Hence, the presentation of water surface profiles or hydraulic gradients from upstream extremities to the outlet for "design conditions" can be greatly misleading, the concurrent occurrence of design rainfalls for all catchment components being more unlikely the greater the size of the area. The inverse of this argument is sometimes demonstrated by the occurrence of flooding at different parts of a drainage area from one storm to another, regardless of inlet capacity inadequacies or other mitigating factors. Because high intensities do not necessarily commence simultaneously for all durations, a small part of a catchment may receive a critical intensity before, during, or subsequent to the intensity critical to a larger portion of the catchment. The indulgence of the reader is requested in a later presentation, where the writer has violated the preceding logic and presented hydraulic gradients for system design flows to illustrate the possible effect of doubling a design rainfall frequency.

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.

Some notes on I-D-F data are included as Appendix A.

It should be noted that values taken from I-D-F curves are truly for a *mean* recurrence interval or frequency. For example, a fitted 5-year value may be expected to recur once in 5-years, *on the average*. Thom[8] argues that the probability of recurrence of a given rate being equaled or exceeded within the same year may be more important than a mean recurrence which requires a number of years for its average to apply.

> *It would appear. . . that in certain design problems where the most serious criticism arises from the too close spacing of failures, such as exceeding the*

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[8] Thom, H.C.S. July 1959. "A Time Interval Distribution for Excessive Rainfall." *Journal of the Hydraulics Division*. ASCE Proc., Paper No. 2083, HY7.

capacity of storm sewers, the only concern might best be for the probability of a specific recurrence interval rather than for a mean recurrence interval.

Dr. J.C. Neill (member, USGS Project Task Group on Data Networks) computed the values in the following table using Thorn's equation:

Mean Recurrence Interval, Years	**Percent probability that a given mean recurrence interval value will be equaled or exceeded during a period of successive years:**			
	Any 1-Year	**Any 2-Years**	**Any 5-Years**	**Any 10-Years**
1	100	100	100	
2	39	63		
5	18	33	63	
10	10	18	39	63
50	2	4	10	18

The above table indicates, for example, that there are roughly two chances in ten (18%) that a 5-year mean recurrence value will be equaled or exceeded in anyone year. The odds of a 5-year mean recurrence interval value being equaled or exceeded in any period of two successive years, are about one out of three, and in any period of five successive years are about two out of three.

A record of perhaps at least 15- to 20-years is needed for adequate derivation of suitable I-D-F relations. There is some evidence that roughly the same period of record may be required to obtain essentially the same I-D-F relations for two separated points in a large metropolitan area. In most cities the I-D-F relations used in design of storm sewers are based on the USWB station record, and there is, therefore, no practicable way to correlate flooding from thunderstorms at a point remote from the USWB station with the record at the USWB station, except that flooding occurrences over a number of years be observed, a completely unacceptable procedure for the public works administrator faced with an irate public immediately after a flooding occurrence.

The preceding discussion in this chapter has been on the use of data, not on its acquisition. There are inherent errors in the data from rainfall measurements, due to imprecise indications of catch, biasing of catch by location or placement, and similar factors. Because of limitations in availability of gauge sites in metropolitan areas, these errors can be significant. Intensity-duration-frequency curves are too often regarded as being absolute, when the true standard error at any duration for any curve might conceivably be on the order of 10% or more.

Lastly, there can be a decided difference between intense point rainfall and mean catchment area rainfall, particularly for thunderstorms, and definitely for drainage areas larger than about 300-acres. As will be noted in a subsequent Program

Technical Memorandum, the median size of storm sewerage drainage system catchments in Washington, DC, is only 65-acres; however, the five largest (2,200- to 6,200-acres) of the 93 catchments collectively represent slightly more than half of the total sewered drainage area in Washington. In the absence of a synchronized network of rain gauges, the areal attenuation of rainfall in a metropolitan area can only be inferentially surmised.

5.0 RATIONAL METHOD OF DESIGN

The most authoritative source of information on this subject is a manual[3] prepared by a joint committee of ASCE and the Water Pollution Control Federation. Current storm sewer design practice was surveyed by the committee through "an extensive questionnaire submitted to 380 public and private engineering organizations throughout the United States," and the remarks that follow are "based, in large measure, on 71 returns." "More than 90%—that replied—indicated use of the rational method."

> *Reported inlet times used for design vary from 5- to 30-minutes, with 5- to 15-minutes most commonly used. In densely developed areas, where impervious surfaces shed their water directly to storm sewers through closely spaced inlets, an inlet time of 5-minutes is often reported. In well-developed districts with relatively flat slopes, an inlet time of 10- to 15-minutes is common. In flat residential districts with widely spaced street inlets, inlet times of 20- to 30-minutes are customary.*

"In practice, average flowing-full velocity expected at the prevailing sewer slopes is used"—to compute travel time.

“The range of frequency reported to be used in engineering offices is as follows:

1) For storm sewers in residential areas, 2- to 10-years, with 5-years most commonly reported.
2) For storm sewers in commercial and high-value districts, 10- to 50-years, depending upon economic justification.
3) For flood protection works, 50-years or more.”

Factors reported that may affect choice of design frequency are cited, including: use of a rarer frequency for combined sewers, sectors not amenable to future relief, and special structures such as expressway drainage pumping stations.

The range of coefficients [c-values], classified with respect to the general character of the tributary area reported in use, is:

Business		
Downtown areas	-	0.70 to 0.95
Neighborhood areas	-	0.50 to 0.70
Residential		
Single-family areas	-	0.30 to 0.50
Multi-units, detached	-	0.40 to 0.60
Multi-units, attached	-	0.60 to 0.75
Residential (suburban)	-	0.25 to 0.40
Apartment dwelling areas	-	0.50 to 0.70
Industrial		
Light areas	-	0.50 to 0.80
Heavy areas	-	0.60 to 0.90
Parks, cemeteries	-	0.10 to 0.25
Playgrounds	-	0.20 to 0.35
Railroad yard areas	-	0.20 to 0.40
Unimproved areas	-	0.10 to 0.30"

A design example in the ASCE-WPCF manual is reproduced herein.

"A typical plan for design of a small storm-sewer project is presented" in Figure 11-3. Table 11-2 "is a tabular summary of computations illustrating the application of the rational method to the storm drainage system shown" in Figure 11-3. "The example is based on the following conditions:

1) Runoff Coefficients—(1) Residential area, 0.3. (2) Business area, 0.6. (3) Average coefficient weighted according to amount of each type of area tributary to a given inlet.
2) 5-yr. frequency rainfall for Davenport as shown in Figure 11-2.
3) Twenty-minute inlet time assumed.
    a) Sewer capacity by Manning formula, n = 0.013.
    b) Outlet un-submerged with free outfall.
    c) A drop of 0.1 ft. across each manhole where pipe size occurs. When change in pipe size occurs, set elevation of 0.8 of pipe depth points equal and provide corresponding fall in manhole invert.

        (Note: in larger systems, a more rigorous analysis of hydraulic losses through manholes, at transitions, and at changes in directions must be made for adequate hydraulic design.)"

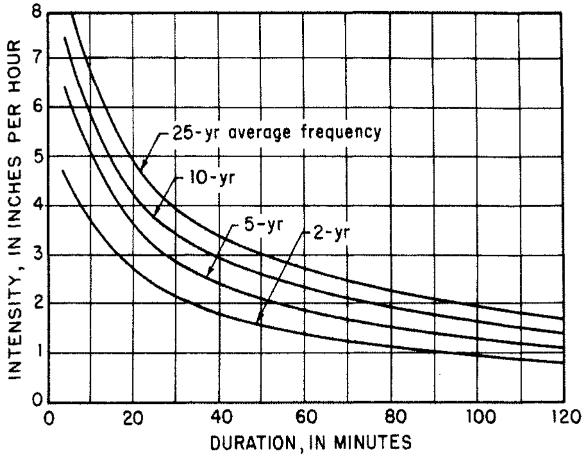

FIGURE 11-2. Intensity-Duration Rainfall curves for Davenport, Iowa (ASCE Manual No. 37, Figure 9, p. 44)

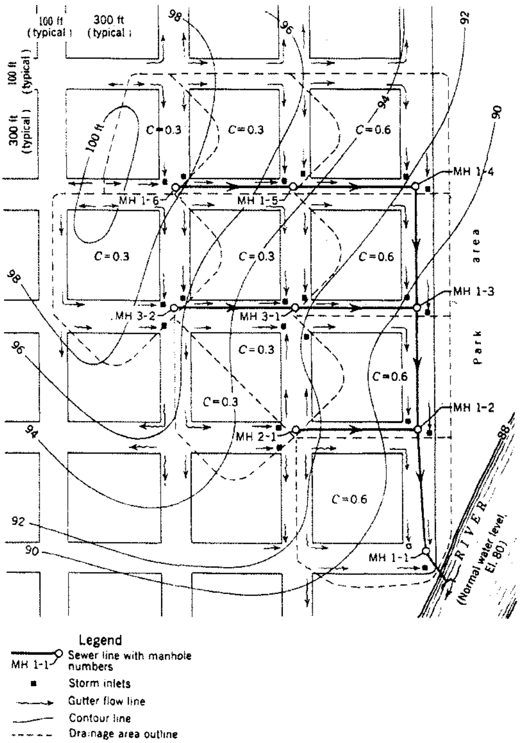

FIGURE 11-3. Typical Storm Sewer Design Plan (ASCE Manual No. 37, Figure 11-12, p. 51)

TABLE 11-2

Typical Storm Sewer Computations for Rational Method for System in Figure 11-3.

(ASCE Manual No. 37, p. 52)

	Manhole No.			Area Acres		Flow Time, Min.										Design Flow							Sewer Invert		Ground Elevation	
Line (1)	From (2)	To (3)	Length, ft. (4)	Increment (5)	Total (6)	To upper end (7)	In selection (8)	Average runoff coefficient (9)	Rainfall, in./hr. (10)	Runoff, cfs per acre (11)	Total runoff, cfs (12)	Slope of Sewer, % (13)	Diameter, in. (14)	Capacity, full (15)	Velocity, full (16)	Velocity, fps (17)	Velocity head, ft. (18)	Depth of flow, in. (19)	Total energy, ft. (20)	Manhole losses, ft. (21)	Manhole invert drop, ft. (22)	Fall in sewer, ft. (23)	Upper end (24)	Lower end (25)	Upper end (26)	Lower end (27)
6	1-6	1-5	400	2.65	2.64	20	1.4	0.3	3.7	1.11	2.93	0.85	12	3.3	40.	4.6		9			---	3.40	93.00	89.60	98.4	94.9
1	1-5	1-4	400	3.61	6.25	21.4	1.2	0.3	3.6	1.08	6.75	0.75	18	9.2	5.1	5.6		11			0.40	3.00	89.20	86.20	94.9	91.8
1	1-4	1-3	400	3.88	10.13	22.6	1.2	0.42	3.4	1.43	14.5	0.45	24	15.2	4.8	5.6		18			0.40	1.80	85.80	84.00	91.8	89.7
3	3-2	3-1	400	5.55	5.55	20	1.1	0.3	3.7	1.11	6.16	1.00	15	6.4	5.1	5.9		12			---	4.00	91.00	87.00	96.2	92.3
3	3-1	1-3	400	6.43	11.98	21.1	1.1	0.3	3.6	1.08	12.92	0.60	24	17.5	5.5	6.1		15			0.60	2.40	86.40	84.00	92.3	89.7
1	1-3	1-2	400	3.92	26.03	23.8	1.1	0.39	3.3	1.29	33.6	0.30	36	37.0	5.1	5.9		26			0.80	1.20	83.520	82.00	89.7	89.5
2	2-1	1-2	400	2.52	2.52	20	1.4	0.3	3.7	1.11	2.80	0.90	12	3.2	4.1	4.7		9			---	3.60	87.50	83.90	92.7	89.5
1	1-2	1-1	400	3.86	32.41	24.9	1.1	0.41	3.2	1.31	42.5	0.24	42	50.0	5.2	5.9		29			0.40	0.96	81.60	80.64	89.5	88.5
1	1-1	Outfall	125	5.44	37.85	26.0	---	0.44	3.2	1.41	53.2	0.30	42	56.0	5.7	6.6		33			0.10	0.38	80.54	80.16	88.5	---

As noted in Section 3 on "Principle of the Rational Method," the design flow at a given point can be calculated as soon as the time of concentration to that given point is determined; and this flow rate determines the required capacity for the next downstream reach of storm sewer. For example, the time of concentration at Manhole 1-5 is 21.4- minutes (20-minutes inlet time to MH 1-6 plus 1.4-minutes travel time between MH 1.6 and MH 1-5), which is the duration used to obtain the rainfall intensity of 3.6-in./hr. The area tributary to MH 1-5 is 6.25-acres, the weighted C-value is 0.3; and, hence, the design flow for the reach of sewer between MH 1-5 and MH 1-4 is (0.3)(3.6)(6.25) = 6.75 cfs.

A minimum flowing-full velocity is selected by the designer. "This practice is based on the assumption that the (resulting) minimum slopes will produce self-cleansing velocities" (p. 98). For factory-made circular conduits, only certain nominal sizes are commonly available, and a minimum diameter (such as 15-in.) is normally adopted.[9] Design sewer slopes are also constrained by prevailing ground surface slopes, minimum elevation requirements for access of inflows, minimum cover for adequate structural capacity, competition for space with other, utilities and other factors, including the overcoming of hydraulic complications such as supercritical flow.

For whatever reasons, the reach under discussion is designed for an 18-in. sewer on a slope of 0.75%. Under the design discharge of 6.75 cfs the depth of flow will be only 11-inches, or about 6/10 full. It should be noted that the resulting flowing-full capacity is 9.2 cfs, an amount more than a third larger than the design rate, which can be carried under incipient surcharging conditions. Hence, the incidence of a flow rate much in excess of the design rate would not necessarily cause flooding, and perhaps not even surcharged flow. This point is raised to indicate the difficulty encountered in attempting to establish a direct linkage between design rates and rates at which flooding will occur. Further consideration will be given to this dilemma in Section 8 on "Flooding in Sewerage Systems."

> *The design of sewers includes three major phases: 1) the capacity design, i.e., the determination of the flow rates ...for which capacity in sewers should be provided; 2) the hydraulic design, i.e., the application of hydraulic science to the determination of the proper size, slope and other characteristics of sewer pipes and structures to provide the capacity to handle the ...flow rates computed in the first phase; and 3) structural design, i.e., providing proper materials, thickness, and structural strength to sewer conduits and structures.*[2]
>
> *The sewer designer's task involves forecasts of the magnitude of three intangible factors which greatly influence the proper capacity design of sewers: 1) he must anticipate the territorial development for upwards of 40-years in the future and be prepared to reasonably evaluate zoning restrictions which quite likely will be modified during the anticipated capacity life of his sewers; 2) he must foresee future sewer operational practices which affect*

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[9] Circular conduits are the predominant type. At least two-thirds of all existing storm sewers are 24-in. diameter or smaller.

> *probable sewage flows; and 3) he must anticipate the probable future desires of sewer users as to adequacy of service. None of these can be ignored—none of these are subject to exact determination.* [2]

6.0 VARIATIONS IN INTERPRETATIONS AND METHODOLOGY

> *Runoff coefficient values appear to be selected on a variety of bases, with no apparent coordination between sewer designers of various cities . . . . . . there appears to be very little, if any, fundamental principle involved in current design practice in selecting the runoff coefficient.* [2]
>
> *Inlet time required for stormwater to reach and pass through the stormwater inlet depends on many factors, none of which can be accurately computed. ...Accordingly, the inlet time period selected by storm sewer designers is only an arbitrary estimate.* [2]
>
> *The time of travel in sewers, or "flow time," computed from the length of sewer divided by the average velocity of flow, will be longer for flat slopes than for steep slopes. . . . there is considerable uncertainty as to both sewage flow rates and internal sewer channel characteristics, i.e., resistance to flow, hence, there is uncertainty as to velocity of flow for the design period of 30 to 40 years in the future.* [2]
>
> *Some evidence indicates that sometimes storm sewer designers adopt a rainfall intensity, in inches per hour, which is used as a constant for all sizes of drainage areas.* [2]
>
> *It is important to note that the rational method uses only average rainfall intensities prevailing over the time of concentration, said average intensities having no relation to actual rainfall pattern during the storm; none of the basic data commonly used in applying the rational method are related to the storm pattern. Failure to recognize this point has led to frequently erroneous usage of the rational method, especially when attempting to evaluate the effect of antecedent precipitation.* [3]

A study was undertaken in 1967 of storm drainage design practices, policies and procedures in 32 Wisconsin cities with a 1960 population greater than 6,000, which collectively represents about two-thirds of the state's urban population.[10] As a part of the study, each city was asked to design an extension of storm sewers for a hypothetical development area of 15 acres. The respondents were to assume the extension was in their city, "i.e., with soils common to the city." The hypothetical development area was two blocks by three blocks in extent, with storm sewers existing along about three blocks of its border. A total of 23 cities prepared designs. "Practically every city reports the use of the 'rational' method in storm drainage

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[10] Ardis, C.V., K.J. Dueker and A.T. Lenz. January 1969. "Storm Drainage Practices of Thirty-Two Cities." *Journal of the Hydraulics Division*, ASCE Proc., Volume 95, No. HY 1, Paper 6365.

design; however, only six cities use it correctly." "Sixteen survey cities do not consider (rainfall intensity) as a variable; it is assumed constant throughout the design." "Within the same soil group, two survey cities using the same frequency and located within 100-miles of each other, design for rainfall intensities that differ by a factor of almost three. The difference in time of concentration was 15-minutes, yet each considered the same tributary area"! Among the 23 designs "no two designs were the same." "The total number of inlets used by each of the cities varied considerably."

The range in estimates of total cost of storm sewers for the hypothetical drainage area, ranked from lowest to highest for each of the 23 survey city respondents is summarized in Figure 11-4.[9]

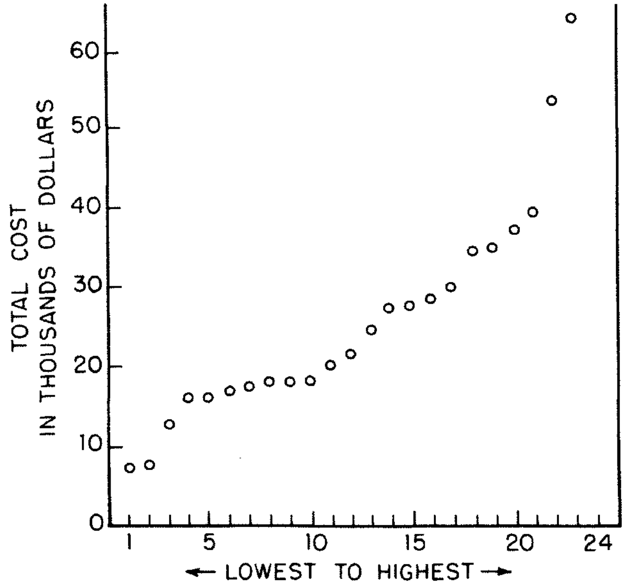

FIGURE 11-4. Eight-Fold Range of Costs, Wisconsin Study.

The major influence on the total variation in cost is a direct result of decisions on how the storm sewer network was laid out. ...It is believed, however, that the wide variation in cost of sewerage for (the hypothetical 15-acre area) is not justified.

The Johns Hopkins University Storm Drainage Research Project was charged by its sponsors to review the rational method in the light of findings from Project field measurements. Findings reported in 1963 are summarized in a published paper.[11] A copy of the paper is included in this technical memorandum as Appendix B. In conjunction with the review, "rational method estimates of the 5-yr. runoff for each of the six areas (two impervious inlet areas, and four larger composite areas) were made using C and (time of concentration) selected by five storm drain designers in accordance with their usual design procedures" (p. 368 of Appendix B). These estimates are compared in Table 11-5, p. 368 of Appendix B, with 5-year peak flows as obtained from frequency arrays of gauged discharges for the six catchments. More pertinent here is the spread of the estimates. The largest differences, relative to the mean of the estimates for each composite area, were:

Name of Catchment	**Area of Catchment**	**Variation About Mean of Estimates**	
		Largest	**Smallest**
Gray Haven	23-acres	+37%	-24%
Northwood	47-acres	+22%	-23%
Swansea	47-acres	+28%	-19%
Yorkwood South	10-acres	+13%	-12%

These figures are a further indication of the rather extensive variability inherent in the application of the rational method. In this instance, all the estimators were senior designers with some background in hydrology, the rational method concept was precisely followed, and the resulting differences in results were almost entirely the consequences of differences between subjective interpretations of numerical values.

Over the more than a decade the Storm Drainage Research Project of The Johns Hopkins University was in operation, flows at outlets of several larger composite catchment areas were gauged. Dr. M. Joseph Willis (a member of the USGS Project Task Group on Data Devices) as part of his consulting work for the Philadelphia Water Department computed some lengths and volume indices for five of these areas in Baltimore and for four Department-gauged catchments, shown here in Table 11-3. First, it must be stated that these catchments were originally selected primarily on the basis of their amenability for a flow gauging installation. They are not necessarily typical for either city. Except for Yorkwood, the cumulative length of storm drains per square mile of drainage basin is less for the Baltimore cases than for all four Philadelphia gauged areas. The internal volumetric capacity of storm drains, expressed in cubic feet per square mile of drainage basin, is fairly constant for all five Baltimore cases; and volumes for the five Baltimore cases are all much less than for the four Philadelphia areas. One contributing factor might be the difference in means

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[11] Schaake, J.C., J.C. Geyer and J.W. Knapp. November 1967. "Experimental Examination of the Rational Method." *Journal of the Hydraulics Division*, ASCE, Volume 93, No. HY6, Paper 5607.

of entry of roof drainage to the street sewer: in Baltimore, roofs drain to street gutters and thence to inlets; whereas in Philadelphia, roofs drain directly to the street sewer. Nevertheless, there appears to have been a tangible difference between design criteria or layout practices used by the two cities. Variations might be somewhat normalized by taking into account land-use characteristics of each catchment.

Urban planners continually seek realistic rules-of-thumb for classifying and allocating various utilities. The limited sample of storm drainage physical characteristics given here suggest that there may be considerable differences from one catchment to another, and that still larger differences may exist from community to community, where variations in design criteria, layout practices and local customs and habits would probably be the major contributing influences. Local peculiarities might also arise from prevailing land values, topography, soils, maintenance practice and problems, history of community development, or from many other factors. However, there is a strong intuitive opinion among drainage specialists that variations in design criteria may be dominant.

TABLE 11-3

Comparison of Physical Characteristics)

(Courtesy, Philadelphia Water Department)

City/Catchment	Area, acres	Impervious-ness, percent	Cumulative Length of Storm Drains, miles per square mile	Volume of Storm Drains, cubic feet per acre
Baltimore				
Northwood	47	68	16.9	368
Swansea	47	44	8.6	340
Uplands	30	52	13.4	300
Gray Haven	23	52	6.5	308
Yorkwood	10	41	54.7	370
Philadelphia				
Bingham	288	67	24.2	1,540
Tustin	138	60	25.0	830
Rowland-Welsh	49	48	19.6	680
Algon	30	61	32.0	1,260

7.0 COMPARISONS OF SOME GAUGED PEAK FLOWS

Flows at four catchment area outlets have been indirectly gauged by the Philadelphia Water Department since 1959. Recorded depth of flow in the outfall near the outlet is converted to an estimated flow by means of an assumed friction coefficient and presumption of near uniform flow. In 1959, weekly stage recorder-chart drives were employed, and these were replaced by daily recorder-chart drives in 1960. For the 1959 records, only the peak stage can be read, and for succeeding years not much more of the stage record can be reasonably well interpreted.

In Figure 11-5, all estimated peak flows near or above design levels that occurred from 1959 through 1963 are plotted in cfs/acre against recorded maximum average rainfalls for the respective times of concentration, from rain gauges on or near the catchments. The times of concentration are: Bingham, 13.2-minutes; Tustin, 10.6-minutes; Rowland-Welsh, 6.1-minutes; and Algon, 9.9-minutes. The slope of an imaginary straight line originating at the $q = 0$, $i = 0$ intercept is a C-value. It may be noted that the C-values for Bingham would generally be lowest and those for Tustin generally highest.

There is no obvious way to account for the variation in C with physical characteristics of the gauged areas. If all 35 data points are regarded as typical or representative of the four gauged areas as a whole, using C = 0.77 as the median gives a fairly symmetrical spread for the plotted points, and all plotted points are within ±26% of C = 0.77, with the C for 8/10 of the plotted points falling within ± 20% of C = 0.77.

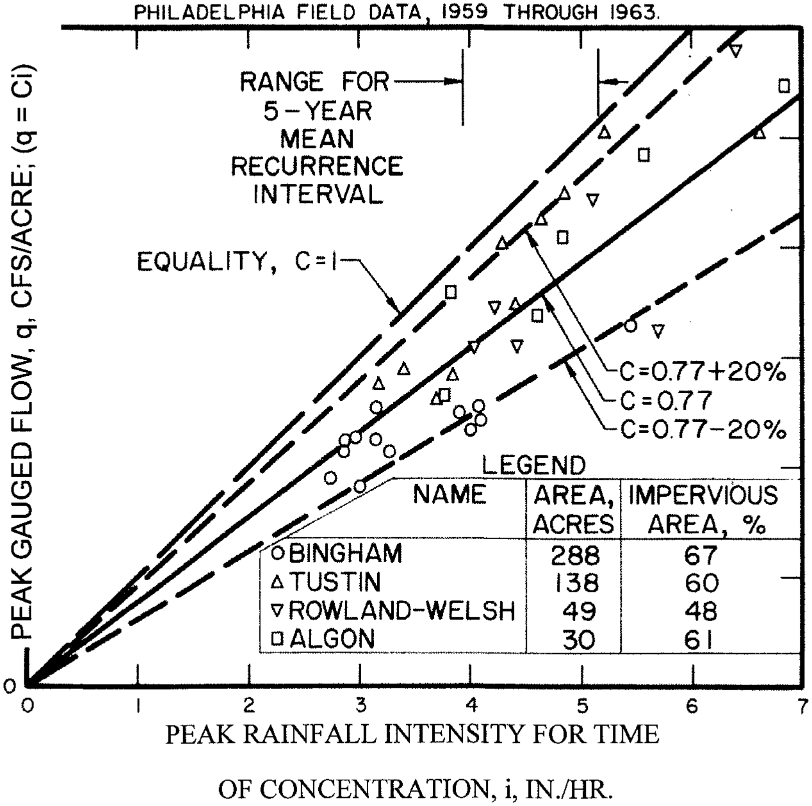


FIGURE 11-5. Rational Method Approximation with Philadelphia Field Data. (Courtesy, Philadelphia Water Department)

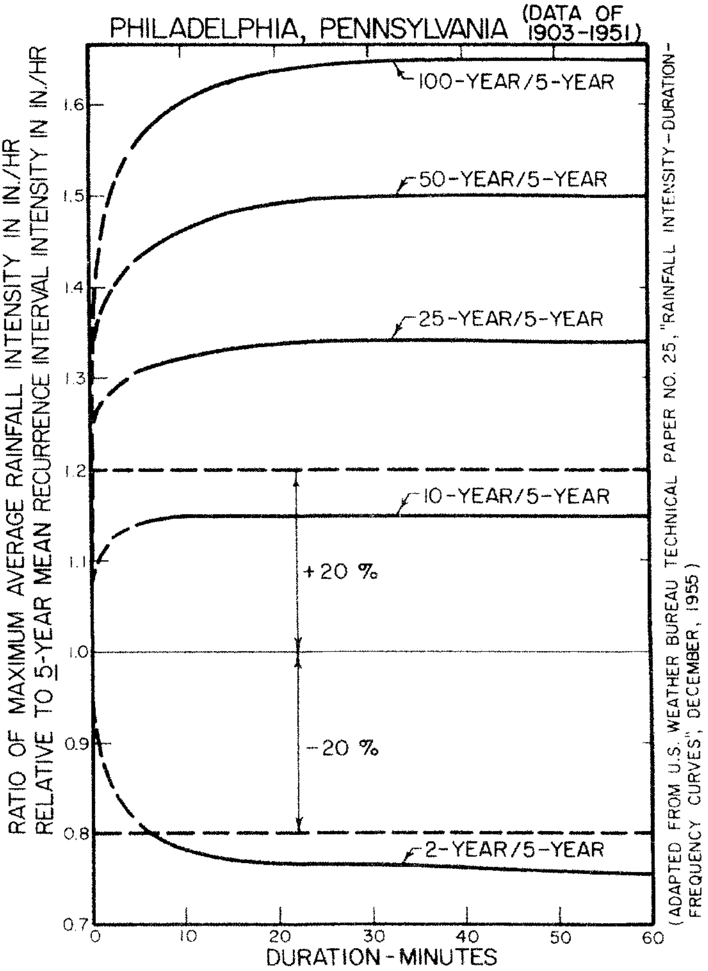
The authors of Appendix B note that design conditions, including maximum average rainfall intensity and concomitant time of concentration are unique values. That is, the rational method can be checked in principle only, when a recorded rainfall occurs in the field that has the same maximum average rainfall intensity over the time of concentration as was used in design. Except by using completely arbitrary procedures of apportionment, there is no reliable way to compare a design flow with a gauged flow unless the rainfall happens to have the proper intensity at the proper duration and occurs at a time reasonably coincident with the occurrence of a peak of recorded flow. The Appendix B authors circumvented this obstacle, but in so doing departed somewhat from the usual concept of the rational method.[12]

In Figure 11-5, although the durations employed for reading the maximum average intensities were all held at the respective times of concentration, not all of the intensities were at about the design level, so the apparent C-values are somewhat fictitious. However, the "range for 5-year mean recurrence interval" of intensity in Figure 11-5 corresponds to the design range of the four catchments; and the spread of points within this range is little different from the spread of all plotted points.

The data treatment used to develop Figure 11-5 is obviously spurious, and questionable at best. However, despite these reservations, a particularly interesting feature stands out: the variability in results is not particularly large considering that a number of different storms are involved, and that inherent errors in the estimates of peak discharge from maximum recorded stages and rainfall measurements could conceivably constitute a substantial part of the variation. Assuming for the moment that Figure 11-5 presents a realistic representation of variability for the rational method, would not something like a $\underline{+}20\%$ precision level in gauged peak flows be adequate for arriving at field C-values and for comparing alternatives?

---

[12] All peak recorded runoff rates for each drainage area have been arrayed by rank and an independent fit made to their distribution. A lag time determined as the time difference between the centroid of *observed* rainfall and the centroid of *observed* runoff has been substituted for a time of concentration per the rational method. Recorded maximum rainfall intensities for durations equal to the observed lag times for each associated storm were also arrayed and an independent fit made to their distribution. The degree of parallelism between the runoff array and the lag time intensity array was investigated to determine the constancy of an associated coefficient analogous to the usual rational method C.



PHILADELPHIA, PENNSYLVANIA (DATA OF 1903-1951)

(ADAPTED FROM U.S. WEATHER BUREAU TECHNICAL PAPER NO. 25, "RAINFALL INTENSITY-DURATION-FREQUENCY CURVES", DECEMBER, 1955)

FIGURE 11-6. Various Frequency Rainfall Intensities Relative to 5-Year Values, Philadelphia. (Courtesy, Philadelphia Water Department)

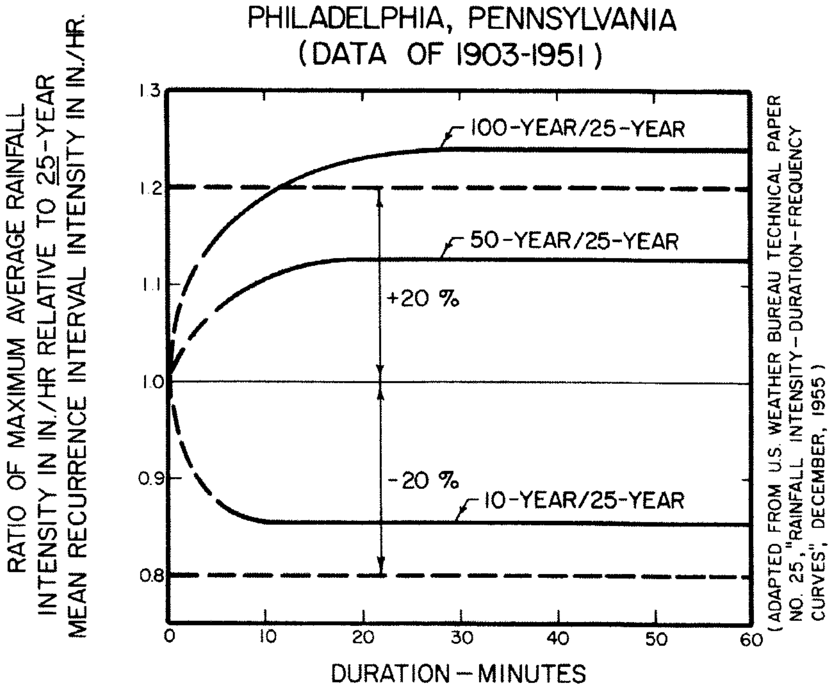


FIGURE 11-7. Various Frequency Rainfall Intensities Relative to 25-Year Values, Philadelphia. (Courtesy, Philadelphia Water Department)

Using Philadelphia rainfall intensities for various durations and frequencies from U.S.W.B. Technical Paper No. 25, and expressing different frequency intensities as ratios to the 5-year intensities at various durations resulted in Figure 11-6. For a given catchment area size and C-value, the rational method design peak flow is directly proportional to the design rainfall intensity. If a total variability of 20% is permitted in the rational method and the entire variability is charged to rainfall intensity it may be seen that a 5-year frequency value could mean anywhere between about a 2-year value and about a 15-year value. This wide a range could make assessment of protection from flooding completely meaningless and surely would greatly obscure the true superiority of one design alternative over another. As a further element of obfuscation, it will be shown in the next section that cost changes may not be very great for a rather large change in design frequency. It seems to this writer that to arrive at reasonable cost-benefit evaluations of alternative drainage system plans will require abandonment of any reference to the rational method and attainment of a very high precision level of field data for verification. This is to say that the range of inherent variation is likely not large and hence particularly good data will be required to discern important but subtle differences.

In closing this section, different Philadelphia frequency intensities are expressed as ratios to the 25-year intensities in Figure 11-1. Here a 20% variation about the 25-year values results in a range between rarer than 50-year and more common than 10-year intensities. Surely this much variation would preclude reasonable flooding protection evaluation. Note that the rarer the intensity event, the smaller the marginal difference in intensity, yet it is in this realm for which drainage investments become most costly.

8.0 FLOODING IN SEWERAGE SYSTEMS

> *Public sewers have been unearthed at Nippur, in ancient Sumeria, dating from 3700 B.C. Scarcely more recent are the ruins of Mohenjo Daro, in what is now Pakistan, where almost every house had a "modern" bathroom. Public sewers were also in use at Tell Asmar, near Bagdad, in 2600 B.C., and at Nineveh and Babylon centuries before the dawn of the Christian era.*
>
> *But the master plumbers of antiquity were the Cretans; before 1500 B.C. their palace of Minos at Knossos had facilities comparable to those of a modern hotel.*
>
> *Rome, of course, was the great political and commercial capital of the ancient world. But Rome was not all temples, arches and forums, any more than New York is all skyscrapers, hotels and theaters. Rome was a vast maze of narrow, winding streets lined with multi-story wooden apartment buildings. Its great sewer, called the Cloaca Maxima, was built about 180 B.C., and continued to serve the Romans down to the present century.*
>
> *Except for the Cretan, none of these disposal systems made any distinction between sanitary sewers and storm sewers. In almost all cases, even into 19th Century Europe, no attempt was made to treat sewage; wastes were merely carried to the nearest body of water and dumped. As cities grew more numerous and more crowded, public calamity became inevitable.*
>
> *In the middle of the last century, for example, recurring epidemics of deadly Asiatic cholera struck the city of London. It became tragically obvious that the great metropolis should not continue to deposit its untreated sewage in the Thames.*
>
> *Similarly, this public awareness was taking place in Continental Europe. Thus, slowly, the era of modern sanitary sewage collection and treatment began.*
>
> *In North America this development took place slightly later than in Europe, for obvious reasons of relative population density. Americans and Canadians*

> *were thus able to profit from the experience of others, almost from the very beginning.*[13]
>
> *Separate sanitary sewers, first used at Croydon, England (about 1850) and in the United States at Pullman, Illinois, and Memphis, Tennessee, were quite generally overloaded due to insufficient capacity. Waring at Memphis, and in many other cities, installed many 4-inch and 6-inch sewers resulting in a great deal of stoppage troubles. His unfavorable aggressive promotional activities and the failure of many of his small sewers due to stoppages (1880-1890) together with a report by Dr. Hering (1886), based on his investigation of European sewerage practices, were important factors that directly influenced the selection of combined sewers by many cities toward the end of the 19th century.*[2]

For the United States, "it is estimated that there are 1,329 jurisdictions, served in whole or in part by combined sewers, having a total population of 54 million." Of these, "it is estimated that 36 million are actually served by combined sewers." ..."The 280 jurisdictions with a population of more than 25,000 had combined sewer systems serving 32 million persons.[14]

Of the 14 largest U.S. cities, ten are partially or wholly served by combined systems of sewerage:[15] New York, Chicago, Philadelphia, Detroit, Cleveland, Washington, St. Louis, Milwaukee, San Francisco and Boston. Of these, the following are served almost exclusively by combined systems: Chicago, Detroit, St. Louis and San Francisco; and more than half of New York, Philadelphia and Cleveland are served by combined systems.

Separate storm sewers usually carry only surface flow, with points of inflow access restricted to street inlets. Under this circumstance, overloading of the sewer system will, at most, choke off flow at inlets and, hence, cause flooding of roadways. Maximum elevation of street sewers is then hydraulically constrained only by the vertical drop needed to provide unhindered flow through inlets at their design capacity. However, some communities additionally require that storm sewers be low enough to accept flow from property footing drains and some keep storm sewers at or below the level of sanitary sewers to minimize entrance of stormwater into sanitary sewers via inadvertent interconnections.

Maximum elevation of street sanitary sewers (and, hence, minimum depths of trench excavation) are dictated by the lowest elevation of building sewers in a given reach of street sewer. Where fixtures are drained in basements and/or floor drains are connected to building sewers, the elevations of basement floors govern depths of building sewers and thereby maximum elevations of street sewers.

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[13] *Sewers and Civilization.* 1959. Portland Cement Association, Chicago.

[14] *Problems of Combined Sewer Facilities and Overflows.* 1967. Federal Water Pollution Control Administration, U.S. Department of the Interior, Report WP-20-11.

[15] Unpublished Findings of ASCE Combined Sewer Separation Project. 1966. Cambridge, Massachusetts.

Because combined sewers receive both surface flow and domestic sewage the more restrictive requirement usually controls the maximum elevation of street sewers: the elevation of basement floors. Unless backflow prevention devices are installed on building sewers, very modest surcharging of combined sewers will cause a mixture of stormwater and sanitary sewage to flow from the street sewer into basements via floor drains and basement fixtures. "Surcharging of combined sewers is more objectionable than surcharging of storm sewers, because of the nuisances and health hazards that result from the flooding of basements and the overflowing of domestic sewage."[5] There are communities where sanitary sewage is collected only from ground level and higher floors, but this situation is rather rare in combined sewer districts.

Because of the more serious nature of combined sewer flooding, hydrologic-hydraulic criteria for their design has been more stringently conservative than for storm sewers.

It should be carefully noted that the intensity of complaints by the public, and a concomitant nuisance level of flooding, are greater for a separate storm sewer system that has inadvertent connections with a separate sanitary sewer system than for a truly independent storm sewer system; and greatest for a combined sewer system.

In Figure 11-8 is graphed the free-surface flow profile for the design example of Section 5, which was for a 5-year frequency of rainfall. The corresponding surcharged flow hydraulic gradient for 10-year frequency rainfall is also shown in Figure 8, based on the assumption that the maximum receiving body of water level at the outlet could be at the level of, but not higher than, the crown of the sewer outlet. It is evident that if this particular system was connected exclusively to street inlets, flooding would occur only at the upper terminals of two of the three branches (if at all) with a doubled design rainfall frequency. On the other hand, were this a combined system, basements in the upper reaches of all three branches would likely be flooded unless backflow devices were installed and operative at each individual building affected. In any event, for this particular example it should be noted that only a portion of the system would be flooded even with still rarer rainfalls. However, inadequate inlet capacity at any point would cause or worsen local roadway flooding, with flow bypassing an inlet aggravating conditions at inlets downstream.

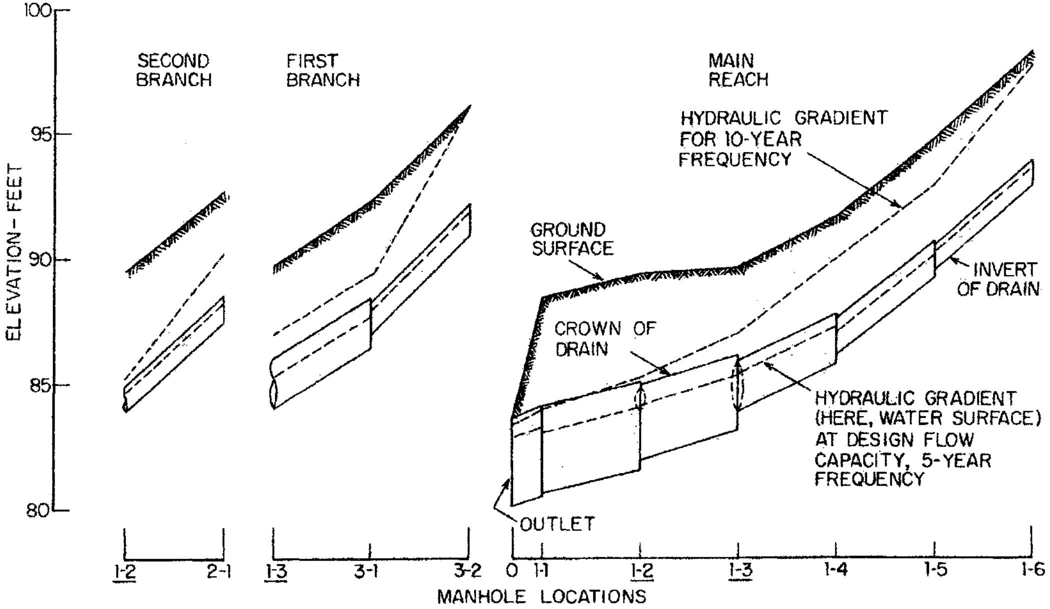

FIGURE 11-8. Example of Additional Flow Capacity with Surcharging

9.0 EFFECT OF VARYING DESIGN FREQUENCY

"It should be recognized that the cost of storm sewers is not directly proportional to design frequency."[3] Rousculp[16] analyzed a "typical" drainage district in Columbus, Ohio, varying the tributary sub-area size, the design C-value, the design rainfall frequency and the design sewer slope for the following combinations:

1) Drainage Area—5 to 1,500 acres;
2) Sewer Slopes—0.001, 0.003, 0.01 and 0.02;
3) C-value—0.30, 0.40 and 0.50; and
4) Frequency—2-year, 5-year, and 10-year.

Using the rational method, he computed the construction cost per acre for each combination of the above four variables, with the aid of graphical displays of variables.

Rousculp presented interpolated results for 100- to 1,000-acre size drainage areas. The range of costs for various frequencies have been indexed to the 2-year frequency costs in Figure 11-9. The upper curve is for 100-acres and the steeper sewer slopes and the lower curve is for 1,000-acres and the flatter slopes, for the full range of C-values.

Greeley and Stanley[5] presented estimated costs per acre for several frequencies for two drainage districts, one in Decatur, Illinois and the other in Oshkosh, Wisconsin, and these have also been indexed to the 2-year frequency costs in Figure 11-9.

Although construction costs are expressed as ratios in Figure 11-9, the absolute values on which these ratios are based are for 32 to 44 years ago. Were these replaced by current costs the trends might conceivably differ by a comparatively large margin because cost components have not all risen proportionally in the past several decades. Nevertheless, Figure 11-9 illustrates the well-known fact that doubling the design rainfall frequency (1-year or greater) does not result in anywhere near as much as a doubling of construction cost. While not necessarily representative of current conditions, raising the design frequency from 5-years to 10-years would result in less than a 10% increase in costs for the full range of variables investigated by Rousculp at 1936 construction cost schedules.

For the full range of variables investigated by Rousculp, raising the design frequency from 5-years to 10-years would raise the design peak discharge by as much as about 40%, for the above increase in costs of less than 10%.

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[16] Rousculp, John A. 1939. "Relation of Rainfall and Runoff to Cost of Sewers." *Transactions*, ASCE, Volume 104, pp. 1473-1487.

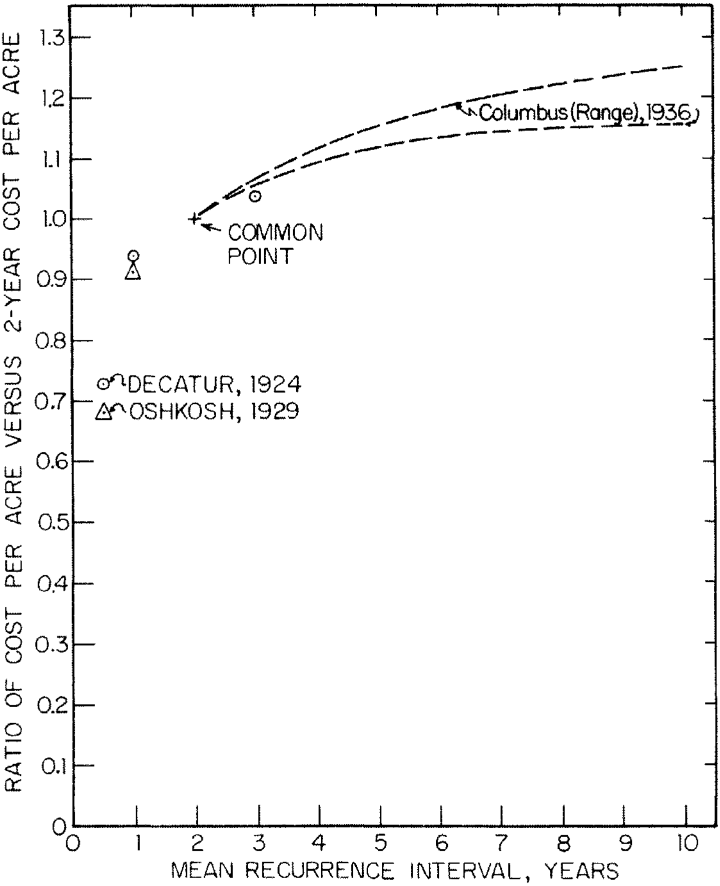

FIGURE 11-9. Trend of Cost and Design Frequency

10.0 SOME LIMITATIONS OF THE RATIONAL METHOD

From the examples given in Section 6, it is evident that there are considerable variations in interpretation and methodology in the use of the rational method. The simplistic scope of the method permits and requires a wide latitude of subjective judgment in its application. The ranges of interpretation and methodology might be narrowed by formalizing national or regional "standards." In that context, why has no group of civil engineers presented detailed "cookbook" instructions for using the rational method? Why are expositions of procedures for general practice couched in obtuse or indirect language?

To codify is to formalize, and formality tends towards rigidity. The writer, for one, would be extremely loathe to be a party to any codification attempt, because the need is for better methods, not improved consensus on an inadequate status quo. He suspects that this view is shared by the majority in the civil engineering profession who are involved in drainage design.

The great majority of drainage areas in U.S. cities are of small size, for which the estimated time of concentration is relatively short. The steepest portion of intensity-duration-frequency curves is for short durations. Design criteria can be seemingly raised by shifting from, say, a 5-year curve to a 10-year curve, but if design inlet time is simultaneously increased the sizes of sewers designed under the "higher standards" may be little different from sizes that would be selected using the original criteria. An equal compensation can be achieved by merely lowering the prevailing schedule of C-values, or by means of a combination of both steps. In confidence, the writer has learned of instances where the C-values of sewer extension programs were adjusted downward until the cost of the planned facilities coincided with available budgets or limits set in bond issue referenda. At any rate, in order to compare the design criteria of one community with another it is necessary to obtain detailed information and sample computations, and even then the task can be exceedingly difficult. Generally speaking, there are no "wrong" storm sewer designs when the rational method is employed.

Improved methods are evolving very slowly because of a dearth of field measurements of rainfall-runoff from which logical, reliable models and quantification of their principal parameters can be developed. No municipality has had the financial and manpower resources to mount a research program suited for national transferability of results, and currently no prospect is in evidence of a consortium of local governments financially prepared to undertake the task.

An outstanding limitation of the rational method is that the product is restricted to a peak flow. By heaping more assumptions on the method a hypothetical hydrograph can be computed, such as by arbitrarily proportioning the contributing area with time, but few designers are willing to elect this option.

The major alternative to expressing collected surface runoff through conduits is to employ various forms of detention storage. Rather than provide the usual drain capacity to remove the increased flows resulting from urbanization, Bauer[17] has suggested that it may be more economical in some instances to provide local detention storage, releasing the flood waters over a larger period of time than the inflow, at a reduced peak and with a consequent reduction in receiving drain capacity. The turf areas separating pavements have been exploited to produce "ponding" of overland flow, with a consequent savings in drain size, at airports,[18] highway

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[17] Bauer, W.J. November 1962. "Economics of Urban Drainage Design." *Journal of the Hydraulics Division*, ASCE Proc., Volume 88, No. HY6, Paper 3321. Discussion in HY4, Paper 3580, July 1963.

[18] Hathaway, G.A. 1945. "Military Airfields: Design of Drainage Facilities." *Transactions*, ASCE, Volume 110, pp. 679-733.

interchanges[19] and for shopping centers.[20] Jones[21] proposed "planned integration of permanent water areas in open spaces, with provision for flood storage." Daily and Associates of Champaign, Illinois, investigated the possibility of using detention basins in the suburban, growing areas adjacent to the City of Memphis, Tennessee. Consideration included use of shallow sodded detention basins near schools and in parks, to provide the dual purpose of flood mitigation and recreation. Effective design of detention storage facilities requires use of complete runoff hydrographs, eliminating the rational method from consideration.

Complete sewer outlet hydrographs are also needed for designing local flood protection works along streams, such as pumping stations for passing local drainage flows over levees and dikes. Because flows from sewer outlets comprise major components of total receiving stream flow at many cities, complete sewer outlet hydrographs are often needed for design of stream and river development works. Here, again, the rational method is of little or no value or utility.

Savini and Kammerer[22] have reported on a review, classification, and preliminary evaluation of the significance of the effects of urbanization on the hydrologic regimen. Quantification of many of these effects is dependent upon the availability of sewer outlet hydrographs, where the rational method is again found wanting, such as for evaluation of the progressive effect of urbanization on receiving stream flood characteristics and the magnitude of volumes of water passed through outlets.

Public concern on urban surface water runoff as a source of pollution is steadily increasing, particularly with regard to overflows from combined sewers[23] and discharges from separate storm drains.[24] However, sanitary engineers have been attempting to indicate the extent of pollution from combined sewer overflows for a number of years.[25] Pollutional loading in units of weight as a function of time must be evaluated as the product of the time-history of contaminants concentration and the runoff hydrograph. The absence of a basis for developing a hydrograph using the rational method precludes its application in pollution loading evaluations; and for the same reasons this applies to sediment transport evaluations as well.

As urban water management problems become increasingly acute, the need for multiple-use of water becomes more evident. In exchanging one use for another, for example using stormwater as a source of water supply, knowledge of the time-history of flows is essential for a reliable design of transfer facilities. It should go without

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[19] Forest, E., and H.G. Aronson. February 1959. "Highway Ramp Areas Become Flood Control Reservoirs." *Civil Engineering.* Volume 29, p. 35.

[20] February 1960. "Storm Water Control for a Shopping Center." *Civil Engineering.* Volume 30, p. 63.

[21] Jones, D. Earl, Jr. August 1967. "Urban Hydrology, A Redirection." *Civil Engineering.* Volume 37, pp. 58-61. Discussion: Volume 37, p. 77, November 1967.

[22] Savini, John, and J.C. Kammerer. 1961. *Urban Growth and the Water Regimen.* U.S. Geological Survey Water-Supply Paper 1591-A.

[23] 1964. *Pollutional Effects of Stormwater and Overflows from Combined Sewers—A Preliminary Appraisal.* U.S. Public Health Service Publication No. 1246.

[24] Weibel, S.R., R.J. Anderson and R.L. Woodward. July 1964. "Urban Land Runoff as a Factor in Stream Pollution." *Journal of the Water Pollution Control Federation.* Volume 36, No. 7.

[25] January 1969. "Selected Urban Stormwater Runoff Abstracts." FWPCA, U.S. Department of the Interior.

saying that knowledge of the time-history of the water quality is also essential. A hydrograph is needed for both aspects, and the rational method is once again inappropriate.

In Section 7, it was noted that in order to check the rational method in a literal sense using field gaugings, it would be necessary that a rainfall of the design intensity for the design time of concentration occur on the gauged catchment. The probability of this occurrence within a period of several years is quite low. The merits of the rational method can be extolled in endless dialogues, but the simple fact that it is almost impossible to verify its true veracity cannot be circumvented. That use of the method tends to result in marginally acceptable designs is not a credit to the rather crude concepts underlying its logic. Instead, the few gauging results available suggest that collection systems tend to modulate flow peaks so that they spread over only a modest range, compared with natural catchments. Further, systems as designed appear to have more inherent "elasticity" than is generally suspected. For example: designs are actually for a no-flooding condition; and the joint probability of simultaneous flooding in all tributary areas of a catchment can be much lower than the probability of flooding in some one part of the catchment.

There is a strong temptation to attack the rationale behind the rational method on hydrologic grounds, but there presently being nothing better to offer this would not be constructive criticism. However, one of its features should be considered further. When the rational method is applied literally, it models a process that can be designated as a linear system. The outputs of a linear system of this type are attenuations and prolongations of its inputs. An identity exists between the statistical properties of the modes (maximum values) of the input and output distributions. For example, the 5-year frequency peak flow would result from the 5-year maximum average rainfall intensity, in the literal use of the rational method. Hence, it is mathematically legitimate to speak of the 5-year peak flow, and this is more useful than referring to rainfall intensity because flow rates are relatable to flooding protection.

Kuichling[4] recognized that the true C-value would be affected by antecedent precipitation and soil moisture conditions. Also, the true C-value could be a function of the rate of rainfall and the rate of runoff, in which case the simple system would be non-linear and a 5-year runoff peak would not necessarily coincide with the occurrence of a 5-year rainfall intensity. All this is to say simply that in the usual application of the rational method the existence of a linear system is implicitly assumed and the ascribing of the input frequency to corresponding outputs is consistent and proper under the circumstances. However, it should be evident that elaborations cannot be added to the rational method without risking further uncertainties about the frequency of flows computed thereby.

Lastly, the pure simplicity of the rational method should be considered. There must be thousands of persons engaged in sewer capacity determinations in the U.S. The majority probably has a very limited knowledge of hydrology, and in many offices routine determinations must necessarily be performed by technicians rather than professional engineers. When better methods become available, there will be a large number of elite designers who will use them simply because they are superior, regardless of their complexity or sophistication. Computer utilization for design is becoming commonplace, particularly in the offices of larger cities and consulting firms.

The ASCE Urban Hydrology Research Council fully realizes that the product of urban storm drainage research must be expressed in simple, direct, unambiguous terms that can be applied across the country by the typical designer. Unfortunately, the typical designer is not fully aware of all of the limitations of the rudimentary rational method he is now using, and much less of the research needs. The following plan should be implemented: a) assess research needs as viewed by elite designers, planners and managers of urban water resources; b) obtain precise field data from a number of representative catchments across the nation on the rainfall-runoff-quality process; c) develop mathematical models using the field data; d) from research on metropolitan storms develop suitable inputs for the models; e) synthesize on a regional basis the model and input-output relations in summary form for the various regions of the country; f) reduce the findings to straightforward and explicit vehicles, such as charts and graphs, for their expeditious use in general practice; and g) promote the use of the more reliable methods for general practice and employment by the elite of the basic, more comprehensive research results.

The ASCE Urban Water Resources Research Program, under the guidance of the ASCE Urban Hydrology Research Council, has undertaken the above item a) with the help and financial support of the USGS, together with a study of instrumentation and data network needs for item b). With the support of a first-year contract with OWRR, research needs on items c) and d) have been defined. Implementation of item b) is absolutely essential, simultaneously with items c) and d), to get to items e), f) and g). Item b) remains the outstanding obstacle to design progress in the urban drainage field.

In closing this chapter, it is appropriate to note that because the rational method is not applicable for catchments smaller than an inlet area, it is not usable in certain urban runoff management, such as the use of roofs and yards for deliberate detention storage, as a part of appropriate land development design.

11.0 ALTERNATIVE DESIGN METHODS

Horner and Jens[26] in 1942 "presented the first application of basic hydrologic hypotheses to urban drainage problems."[2] Hicks[27] in 1943 described the first "major application of the synthetic hydrograph methodology to practical municipal storm sewer design" [2] in the City of Los Angeles. The synthetic hydrograph method, adopted by the City of Chicago, was defined by Tholin and Keifer[28] in 1960. For any

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[26] Horner, W.W., and S.W. Jens. 1942. "Surface Runoff Determination from Rainfall Without Using Coefficients." *Transactions*, ASCE, Volume 107.

[27] Hicks, W.K. 1944. "A Method of Computing Urban Runoff." *Transactions*, ASCE, Volume 109.

[28] Tholin, A.L., and C.J. Keifer. 1960. "The Hydrology of Urban Runoff." *Transactions*, ASCE, Volume 125.

synthetic hydrograph approach, because of a limited knowledge of storm time and space variability, it is necessary to develop synthetic storm patterns from limited field data, such as the technique of Keifer and Chu.[29] Kaltenbach[30] has summarized features of the "inlet method" developed from gaugings of The Johns Hopkins University Storm Drainage Research Project, but the method has not been adopted for design. The principal elements of the preceding methods have been summarized in a handbook.[1]

The initial phase of runoff from rainfall is the street runoff that occurs over pervious and impervious ground surfaces, termed overland flow. Izzard[31] performed the most useful of the earlier overland flow experiments, developing hydrograph characteristics from his laboratory experiments. R.E. Horton, mostly in privately published papers, was the major contributor to the initial research on overland flow and formulated some of the original equations. During World War II and the preceding arms buildup, interest in overland flow was in connection with airport drainage design. The basic principles describing overland flow have been utilized in the design procedures presented by Horner and Jens, Hicks, and Tholin and Keifer.

Only the Los Angeles method has been verified by numerous field gaugings over a long period of time, but the method as developed is applicable only locally. Nationally, only a very few catchment areas have been gauged.[1]

Stanley and Kaufman[2] observe:

> *The most important hydrograph characteristics related to storm sewer design are the peak runoff and the chronological position and general configuration of the peak. These two latter factors are of importance in summing confluent hydrographs in intercepting sewers, ... Without doubt, the hydrograph method can be greatly simplified in application without sacrificing any of its fundamental advantages. These simplifications should take the form of the publishing, in a readily usable manner, of certain basic hydrologic data. If storm patterns and basic infiltration curves were available, a great portion of the problem would be removed from the engineer attempting to apply the synthetic hydrograph. The standardization of the runoff-to-sewer hydrographs for typical city blocks for various slope and precipitation conditions might be accomplished and would solve the problem completely.*[2]

They have addressed the single-purpose application of storm sewer design. In this context their position is well taken. However, more refined and extensive hydrologic information is required to develop adequate technical knowledge for reliable planning

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[29] Keifer, C.J., and H.H. Chu. August 1957. "Synthetic Storm Pattern for Drainage Design." *Journal of the Hydraulics Division*, ASCE Proc., Volume 83, No. HY4, Paper 1332. Discussion by M.B. McPherson in Volume 84, No. HY1, Paper 1558, February 1958.

[30] Kaltenbach, A.B. January 1963. "Storm Sewer Design by the Inlet Method." *Public Works*, Volume 94, No. 1.

[31] Izzard, C.F. 1946. "Hydraulics of Runoff from Developed Surfaces." *Proceedings*, 26th Annual Meeting, Highway Research Board, Volume 26.

of water quality and quantity exchanges between the several urban water service functions for multi-purpose development; and for quantifying pollution loadings from combined and storm sewerage systems applicable nationwide. That is, if adequate information can be secured on rainfall-runoff-quality processes, the collective needs for storm sewer design, water exchange developments and evaluation of pollution loadings can be met simultaneously. Some engineers intuitively feel that the quality of data needed for storm sewer design may be less than for the other two objectives. However, as noted in preceding sections, there is some indication that the variability in rainfall-runoff for sewered areas and/or the marginal differences in costs and benefits may be fairly subtle. In the absence of substantive evidence to the contrary, it appears that to be on the side of wisdom requires a high level of field gauging precision for all three objectives.

> *In larger sewer systems it is desirable to investigate further the timing of the flood wave along the principal trunk sewers. Such a study will usually allow some additional economy, especially where downstream outlet conditions produce a drawdown of the surface profile approaching the outlet.*[3]

Research is under way on the dynamics of free-surface flood waves in circular sewers.[32] It should be obvious that realistic application of findings from research of this kind will require hydrologic information of the highest quality.

Goetsch[33] has outlined needs in community facilities planning. Of the several public works and utilities serving communities it is unfortunate that the least advances in development of improved techniques for analysis have been for storm sewerage, despite the substantial national annual investment in drainage facilities, which are comparable to those in other water services.

It is gratifying that some researchers are interested in urban drainage hydrology, despite the absence of adequate data for truly comprehensive and objective analysis, such as the work of Reich and Hiemstra.[34]

In conclusion, it is necessary to express appreciation to Dr. John C. Geyer, who originated and directed The Johns Hopkins University Storm Drainage Research Project. The project was the rallying point for many engineers concerned about urban drainage, including the writer, until its termination in 1968.

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[32] Barnes, Albert H. September 1967. "Comparison of Computed and Observed Flood Routing in a Circular Cross-Section." *International Hydrology Symposium*. Ft. Collins, Colorado.

[33] Goetsch, Herbert A. 1969. Chapter VI, "Community Facilities Planning." *Urban Planning Guide*, ASCE Manuals and Reports on Engineering Practice, No. 49.

[34] Reich, B.M., and L.A.V. Hiemstra. September 6-8, 1967. "Purpose of Performance of Peak Predictions." *Proceedings of the Internal Hydrology Symposium*, Paper No. 70, Volume 1, Ft. Collins, Colorado. Discussions appear in Volume 2, pages 350, 356-358, and 360-389.

Chapter 11, Appendix A

NOTES ON INTENSITY-DURATION-FREQUENCY DATA

January 22, 1969
Some Notes On The Rational
Method Of Storm Drain Design

1.0 REFERENCES

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Part II– Between 105° W. and 115° W., August 1954.

Yarnell, D.L. 1935. "Rainfall Frequency-Intensity Data." U.S. Department of Agriculture, Miscellaneous Publication 204.

Yarnell studied excessive precipitation data from 211 first-order U.S. Weather Bureau stations. "For a few states, 33-year records were used; for some, 24½-year records; and for the others, the 20-year records covering 1914-1933." He obtained depth-duration-frequency relations for each of the 211 locations. "On outline maps of the U.S. the amounts of precipitation in different periods for different recurrence frequencies. . . were marked in the proper locations. Then isohyetals (lines of equal precipitation) were drawn, the plotted values being weighted according to best judgment, considering the length and character of the records from the different stations." It should be carefully noted that these "isohyetals" 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. Of a total of 56 maps for the continental U.S. that he made, giving 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year depths, 30 were for durations of 5-minute, 10-minute, 15-minute, 30-minute, and 60-minute I-D-F curves for localities where local recording gauge records are not available are sometimes synthesized from the Yarnell maps.

The Corps of Engineers studied the Yarnell maps and found that the depth for any duration was an approximate function of the 1-hour duration for all stations, regardless of recurrence interval. Using the relations between 1-hour and other duration depths, general I-D-F curves were drawn, called "Standard Curves" by the Corps, with the curve designation indexed to the 1-hour intensity value. The "Standard Curves" combined with Yarnell's six 1-hour maps are an approximate substitution for 41 Yarnell maps (the number for durations of up to 4-hour). The Corps curves, in addition to simplifying I-D-F curve construction, are useful in determining overland flow supply rates in one type of airport drainage analysis.

1.1 Intensity-Duration-Frequency Curves; Some Main Features

For data acquired from 1896 to and including 1935 the U.S.W.B. reported "Accumulated Depths during Excessive Rate for Consecutive Periods of Time (minutes)." Data since 1935 (starting with calendar year 1936) are reported by U.S.W.B. as "Maximum Excessive Precipitation" wherein the maximum amounts are taken for the periods in which the fall is greatest. For the latter, the rainfall time pattern is available only from the U.S.W.B. National Records Center, Asheville, N.C. (for a nominal fee).

1) Precipitation data are reported for those rains in which the maximum depths at one or more durations t (5- to 180-minute) equaled or exceeded 0.01t plus 0.20, where t is in minutes and the sum is in inches. Storms fall under the classification of "excessive precipitation" whether or not all durations had depths equal to or exceeding these maxima.
2) For the years 1896 through 1935, the data were reported as sequential cumulative depths starting with whatever first 5-minute depth was equal to or greater than 0.05-inches. While these were reported for durations up to 120-minutes, few storms with significant intensities last longer than an hour, and a 1-hour duration is adequate upper limit for most rational method storm drain

design usage. Because of the arbitrary method employed to commence readings of cumulative depth, the maximum depth for any duration obtained by subtracting the difference in accumulated depths over a 5-minute multiple time interval is not necessarily the true maximum.

Starting with 1936 data, the cumulative rainfall records were searched to find the actual maximum depths for a given duration, irrespective of the occurrence of other durations, i.e., true maxima.

1.2 Excessive Precipitation Occurrences

Because data for only those storms having some rainfalls above a threshold level are reduced and reported by the Weather Bureau, termed Excessive Precipitation, there are a considerable number of rainfalls that go unreported. Such lower intensity rainfalls are of importance for evaluation of pollution from storm sewer discharges and combined sewer overflows.

The limited number of storms that have excessive precipitation can be appreciated from the following tabulation for the Philadelphia U.S.W.B. station for the twenty years from 1935 to 1954 with a total of only 140 such storms:

1935	-11		1940	-7	1945	- 2	1950	- 6
1936	- 3		1941	-5	1946	- 6	1951	- 7
1937	- 4		1942	-7	1947	-18	1952	-10
1938	- 7		1943	-9	1948	- 5	1953	- 8
1939	-10		1944	-3	1949	- 7	1954	- 5

(Starting in 1955 the station was moved from the Custom House to Philadelphia International Airport).

For the above 20-year record, 30 of the top 33 5-minute maximum rainfalls occurred from May to September, inclusive; and the top 15 all occurred from May to September, inclusive. Convectional summer rainstorms are commonly the source of peak intensities in the eastern part of the U.S.

Chapter 11, Appendix B

EXPERIMENTAL EXAMINATION OF THE RATIONAL METHOD (COPY)

January 22, 1969
Some Notes On The Rational
Method Of Storm Drain Design

Schaake, John C., Jr.[1], John C. Geyer[2], and John W. Knapp[3]. November 1967. *Journal of the Hydraulics Division.* ASCE Proceedings, Volume 93, No. HY6, Paper 5607.

(Note: Discussion by S.S. Butler, C.C. Kisiel and L.W. Murphy appeared in Volume 94, No. HY6, Paper 6200, pp. 1583-1590, November 1968; and by J.G. Merkle in Volume 95, No. HY1, Paper 6323, pp. 463-467, January 1969. The closure to discussion, if any, will appear in a 1969 issue.)

Note.—Discussion open until April 1, 1968. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division. Proceedings of the American Society of Civil Engineers, Vol. 9i, No. HY6, November, 1967. Manuscript was submitted for review for possible publication on July 18, 1966.

[1] Asst. Prof. of Environmental and Systems Engrg., Univ. of Florida, Gainesville, Fla.; formerly, Proj. Engr., The Storm Drainage Research Proj., The Johns Hopkins Univ.. Baltimore, Md.

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1.0 ABSTRACT

Rainfall and runoff data collected in Baltimore, Maryland, from 20 gaged urban drainage areas ranging in size up to 150 acres have been used in a study of the Rational Method. The results suggest it may he nearly correct to assume that the frequency of occurrence of the computed design peak runoff is the same as the frequency of occurrence of the rainfall intensity selected by the designer if an appropriate C value has been selected. Presently used values of C are not adequately based on measurements of rainfall and runoff in urban areas, so large errors can occur when the Rational Method is used. In accordance with their usual design procedures, five Storm drainage designers used the Rational Method to estimate 5 yr design peak runoff rates for six gaged drainage areas. These values are compared with runoff values from runoff frequency curves for these gaged areas.

2.0 INTRODUCTION

Since the Rational Method was introduced in the United States by Kuichling[4] in 1889, it has become the most widely used method for estimating peak runoff rates in the design of urban drainage facilities. Although billions of dollars have been spent to construct facilities designed by the Rational Method, it has never been adequately verified by accurate measurements of rainfall and runoff events in urban areas. In 1949 the Storm Drainage Research Project was initiated at the Johns Hopkins University to develop improved methods for the design of urban drainage systems. Thus far the project has, (1) investigated the hydraulic characteristics of storm water inlets, (2) developed (and is now operating) a network of rainfall and runoff gages in the Baltimore area, and (3) is investigating the relationship between rainfall and runoff in urban areas. The present objective of the project is measurement of runoff at storm water inlets and development of a method of predicting inlet hydrographs. The inlet hydrographs will then be routed through the drainage system to obtain the downstream runoff hydrograph. Development of methods for routing predicted inlet hydrographs is under way at Colorado State University tinder the direction of V. M. Yevdjevich.

Because the Rational Method is widely used, and because, as part of the Hopkins Project, inlet and sewer gages have been operated for several years on 20 areas ranging in size from 0.2 acres to 150 acres, the question was raised as to whether or not the rainfall and runoff records from these gages could be used in any way to check the validity of estimates made when using the Rational Method to calculate flows for which storm drains are designed. Study of this question leads to the conclusion that the validity of the Rational Method can be judged from gage records only on a statistical basis. If the record shows that the estimated design peak is reached or exceeded at the frequency assumed when applying the Rational Method, then the method has succeeded.

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[4] Kuichling, E. "Relation Between the Rainfall and the Discharge of Sewers in Populous Districts," Transactions, ASCE, Vol. 20, 1889, p. 1.

A review of the problems encountered when looking into the validity of the Rational Method sheds considerable light on the nature of the assumptions implicit in the method, Such a study indicates little about the nature of the physical factors and phenomena which determine the runoff from a given area during an actual rainstorm. Although the Project is primarily concerned with the latter, it is, nevertheless, believed that a report on the examination made of the Rational Method is worth while on the grounds that it may lead to a better understanding of the method by those who use it. However, the few years of data available are hardly sufficient to judge the Rational Method on a statistical basis, Such analyses as are possible suggest that, although the method can be wide of the mark in any given case, it may on the average he giving reasonable results.

The Storm Drainage Research Project is being carried forward by the Department of Environmental Engineering Science of the Johns Hopkins University through Institute for Cooperative Research contracts with the U.S. Bureau of Public Roads, and with Baltimore City, Baltimore County, and the Maryland State Roads Commission. The Storm Drainage Research Committee, composed of representatives of the sponsoring agencies and of engineering firms in the Baltimore area, reviews and guides the work.[5] [6] [7]

3.0 RATIONAL METHOD

Present engineering practice is to design drainage facilities to carry, without surcharge, the peak runoff rate expected to be equaled or exceeded on the average once in a period of P years. The peak runoff rate estimated by the Rational Method is here referred to as the design peak runoff rate and is found from

$$Q = CIA$$

Equation (1)

in which Q = the design peak runoff rate, hi cubic feet per second, C = a dimensionless runoff coefficient, I = an average rate of rainfall intensity, in inches per hour for a period of time, t_a (here called the rainfall intensity averaging time rather than the rainfall duration), and A = the size of the drainage area, in acres.

Eq. 1 may be rearranged as

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[5] The Design of Storm Water Inlets, report of the Storm Drainage Research Comm., The Storm Drainage Research Project, Dept. of Sanitary Engrg. and Water Resources, The Johns Hopkins Univ., Baltimore, Md., 1956.

[6] Viessman, W., Jr., and Gayer, J. C., "Characteristics of the inlet Hydrograph," Journal of the Hydraulics Division, ASCII, vol. 88, No. HXS, Proc. Paper 3285, Sept,, 1962, pp. 245—268.

[7] Knapp, J. W., Schaake, J. C., Jr., and Viessman, NV., Jr. "Measuring Rainfall and Runoff at Storm Water inlets," Journal of the Hydraulics Division, ASCII, Vol. 89, No. FITS, Proc. Paper 3644, Sept., 1963, pp. 99—115.

$$\frac{Q}{A} = CI$$

Equation (2)

and rewritten

$$q(T) = CI(t_a, T)$$

Equation (3)

in which $q(T)$ = the peak runoff rate per unit area and T = the recurrence interval, in years. The functional notation $q(T)$ and $I(t_a, T)$ has been introduced in Eq. 3 to emphasize an assumption, implicit in the Rational Method as it is now used, that the design peak runoff rate is expected to occur with the same frequency as the rainfall intensity used in the computations. Therefore, in Eq. 3, the value computed for $q(T)$ does not correspond to the peak runoff rate that would be expected from any future storm nor is this the runoff rate that would have occurred from some historic storm.

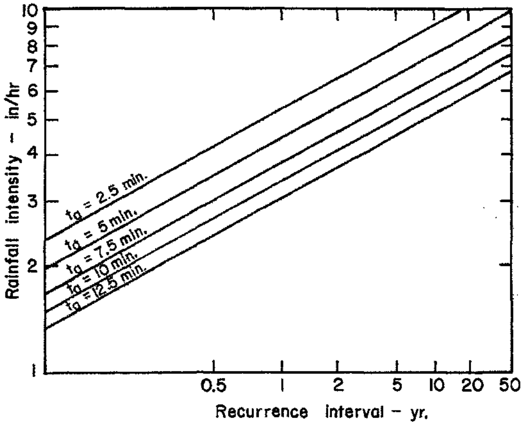


Fig. 1 Rainfall Intensity-Frequency Curves For Various Averaging Times

Eq. 3 is a relationship between the frequency distributions of rainfall and runoff, Therefore, it is not possible actually to measure the values of the terms in this equation during individual storms; to establish the validity of the Rational Method requires the use of statistical methods. However, the few years data available are hardly sufficient to make judgments on a statistical basis. An examination of the Rational Method is believed worth while, nevertheless, and the few years data that are available have been used to illustrate the discussion.

It also is assumed in the Rational Method that the frequency distributions $q(T)$ and $I(t_a, T)$ are log-normal and that these distributions plot as parallel straight lines on logarithmic probability paper. Our investigations indicate that this assumption may be nearly correct for the range of values of T usually used in design practice and for drainage areas as large as 150 acres. (Data for larger areas have not been used in this study.) However, as will be fully described below, the value of C is not independent of the value selected for t_a.

Rainfall Intensity—Duration—Frequency Curves—The average rainfall intensity, I, used in Eq. 1 is ordinarily selected from rainfall time—Intensity— frequency curves similar to those illustrated in Fig. 1, To obtain a value for 1 from these curves, values for t_a and T must he assumed. Commonly used values of 7’ range from 1 yr to 50 yr depending on the potential damage in the area drained. The value of l used in practice is estimated as the time required for runoff to flow from the farthest point timewise in the drainage area. The commonly used value of t_a is the time of concentration and will be indicated in the following by use of the symbol f, There is no practical way to measure I in the field.

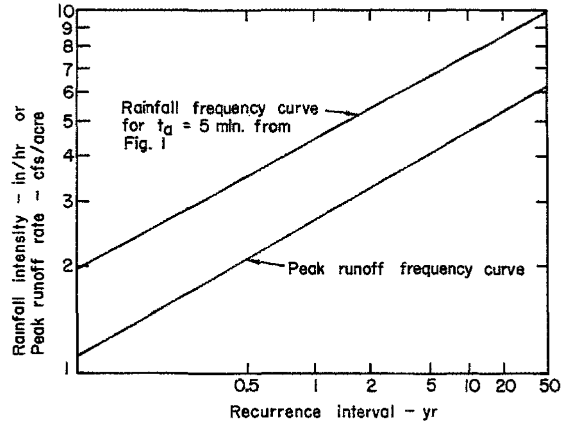


Fig. 2—Rain Fall and Runoff Frequency Curves

Runoff Frequency Curves—If long records of runoff were available for urban drainage areas, frequency curves of peak runoff rates could also be plotted. It would be expected that some relationship between rainfall and runoff frequency curves might exist as is assumed in the Rational Method. To examine whether this is so, any of the rainfall frequency curves in Fig. I could he plotted on the same graph with the runoff frequency curve as shown in Fig. 2. These particular curves are based on an analysis of rainfall and runoff records from a gaged urban drainage area. At first glance these curves appear to be parallel, straight lines: but, in fact, the lines tend to converge with increasing length of recurrence interval. It is implied in the Rational. Method that the runoff frequency curve is parallel to the rainfall frequency curve because the same value of C would be used In Eq. 3 for all recurrence intervals, Judging from the data used in this analysis, rainfall and runoff frequency curves do not appear to be parallel; but the change in *C* over the range of recurrence intervals usually used in practice does not appear to be significant for the cases examined.

If a different value had been selected for t_a (i.e., a different curve selected from the family of rainfall frequency curves in Fig. I), there would result a different value for *C*. Therefore, the area is dependent on the value chosen for t0, the frequency curve may be below the runoff frequency curve in Fig. 2, and the C relating points on these curves would be greater than unity.

TABLE 1.—PHYSICAL FEATURES OF THE GAGED DRAINAGE AREAS

Drainage Area (1)	Abbreviated Identification (2)	Area, *A*, in acres (3)	Imperviousness (4)	Length of Main Channel, *L*, in feet (5)	Slope of Main Channel, *S*, as a percentage (6)
Gray Haven	GRAY	23.3	0.52	1,868	0.91
Hamilton 2	HH2	0.965	0.201	505	0.98
Hills 3	HH3	1.833	0.364	583	0.85
4	HH4	0.216	0.963	583	0.86
5	HH5	1.709	0.318	360	2.10
Midwood 5	MID5	1.276	0.56	360	6.1
Montebello 2	MB2	1.513	0.087	470	1.73
3	MB3	0.451	0.571	153	0.81
4	MB4	0.541	0.648	352	0.79
5	MB5	0.523	0.659	352	0.85
Newark 9	N9	0.636	1.00	575	3.35
12	N12	0.955	1.00	917	0.68
Northwood	NRWD	47.4	0.68	2,264	2.87
South 1	SPL1	0.395	1.00	280	1.71
Parking 2 Lot	SPL2	0.469	1.00	332	2.16
Swansea	SWAN	47.1	0.44	2,000	3.06
Uplands	UPLD	30.1	0.52	2,082	2.56
Walker Avenue	WALK	153.4	0.33	5,620	1.42
Yorkwood	YORK	10.4	0.41	1,041	3.51

4.0 VALUES OF RAINFALL AVERAGING TIMES AND RUNOFF COEFFICIENTS

Data from observed storms on 20 gaged urban drainage areas were used to study rainfall averaging times and runoff coefficients. Some of the physical characteristics of these areas are presented in Table 1. All but three areas are located in Baltimore, Md. Data for two inlet areas in Newark, DeL, have been obtained through the cooperation of the Department of Civil Engineering of the University of Delaware. Data from a 27-acre composite drainage area in Cincinnati. Ohio, were obtained through the cooperation of the U. S. Public Health Service Urban Runoff Project at the Taft Sanitary Engineering Center.

Rainfall Averaging Times—When Kuichling introduced the Rational Method in 1889, there were few records of rainfall intensities during a storm. Only total depths of rain and durations of storms were recorded. Kuichling observed that short duration rainfall intensities often greatly exceeded the average intensities observed for entire storms, He also observed that rates of storm water runoff from small urban areas increased or decreased during storms as rainfall intensities increased or decreased. He concluded that better estimates of peak runoff rates could be obtained if an average rainfall intensity for the period of time, ç., required for runoff to flow from the farthest point timewise in the drainage area were used rather than an average intensity for the entire storm. This was his reason for introducing t_c in his paper.

An assumption of the time required for runoff to flow from the farthest point in the drainage area is presently used In design practice. However, there is no known way to determine this time of flow, either from measurements in the field during storms or from records of rainfall and runoff.

Except for steady state conditions, which rarely, if ever, are reached during a thunderstorm, there is no good reason to believe that the time of flow from the farthest point in a drainage area should necessarily be the best rainfall averaging time to use in the Rational Method. A study, therefore, was made to find for each area the averaging time giving the best correlation between average rainfall intensity and peak runoff rates. Because this correlation was about the same for a range of assumed averaging times, It was difficult to select unique values for t_a for each drainage area from this correlation study.

In order to make some comparison between the statistics of rainfall and those of runoff, as implied in the Rational Method, it is necessary to decide on some consistent method for selecting the rainfall averaging time. Therefore, it was arbitrarily decided to use the average of time lags, I, between the centroids of the rainfall hyetographs and the centroids of corresponding runoff hydrographs, as the rainfall averaging time. The average time lag I; for each drainage area was found by summing the time lags observed during a number of storms and dividing the sum by the number of storms. It was found that the average values of t_j fell in the range of assumed averaging times for which best correlation between peak runoff rates and average rainfall intensities was obtained. Average values of t_l for each of the gaged drainage areas described in Table 1 are presented in Table 2. Typically, the coefficient of variation of t_l was between 0.15 and 0.25.

Runoff Coefficients Obtained From Observations of Rainfall and Runoff— Assuming $t, = t_l$, values of C were calculated for each of the gaged areas. Two methods were used.

Where more than 12 storms were available, frequency distributions of observed average rainfall intensities and peak runoff rates were plotted on log-normal probability paper and values of c were determined from the frequency distributions. Where fewer than 12 storms were available, c was found as the average of values calculated for storms having average 5-mm rainfall intensities greater than 3 in. per hr. Data for essentially all storms producing a measurable rate of storm runoff were used.

Frequency distributions of average rainfall intensities and peak runoff rates are presented in Figs. 3 to 9 for seven of the gaged areas. Straight lines, indicating log-normal frequency distributions, are fitted to the observed data. A test of normality of the data for one of the areas (Swansea, Fig. 7) did not reveal a significant deviation from the log-normal distribution. The remaining data also appear to be adequately represented by the fitted log-normal distributions.

To obtain these frequency distributions, the rainfall data and peak runoff data for each area were separately ranked in decreasing order of magnitude. Therefore, in the sequence of average rainfall intensities, the m^{th} average rainfall intensity may not have occurred during the same storm as the m^{th} peak runoff rate in the sequence of peak runoff rates for the same drainage area. The plotting position for the m^{th} term in the sequence was found from

$$P = \frac{100m}{N+1} \tag{4}$$

in which P (the horizontal axis at the top of Figs, 3 to 9) = the percentage of sample values exceeding in magnitude the value of the mth item, and N = the number of times in the sequence of sample values.

Since more than one storm was observed during each year, the lower horizontal axis in Figs. 3 to 9 has been provided to indicate recurrence intervals estimated from the relation

$$T = \frac{n+1}{m} \tag{5}$$

Table 2.—Average Values Of t_l For The Gaged Drainage Areas

Drainage Area	**Average t_l in minutes**
Gray Haven	8.5
Hamilton Hills 2	8.7
Hamilton Hills 3	7.4
Hamilton Hills 4	4,9
Hamilton Hills 5	4,8
Midwood 5	3.1
Montebello 2	8,0
Montebello 3	4.0
Montebello 4	3.3
Montebello 5.	3.7
Newark 9	3.4
Newark 12	4.9
Northwood	6.5
SPL 1	4.7
SPL 2	6.9
Swansea	4.8
Uplands	7.4
Walker Avenue	11.5
Yorkwood	4.7

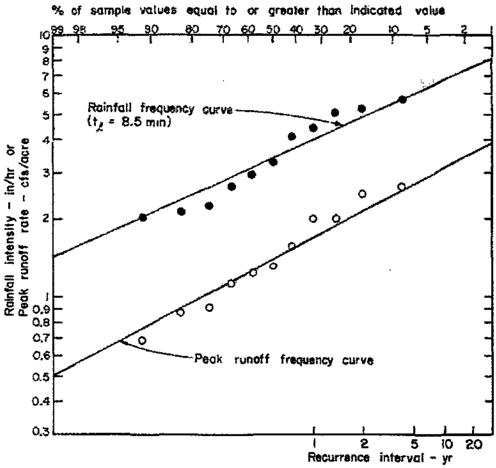
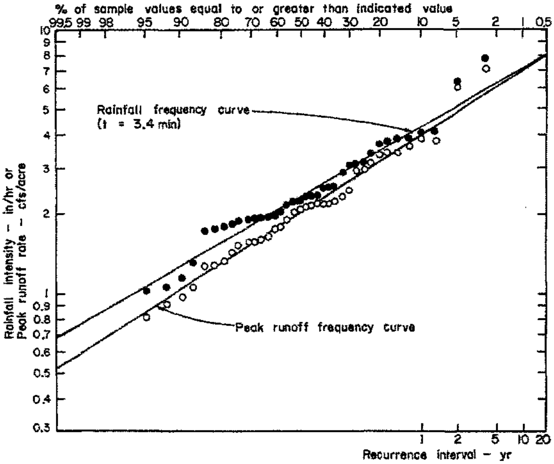


Fig. 3.—Distributions of Rainfall And Runoff (Gray Haven)



Fig, 4.—Distributions of Rainfall And Runoff (9)

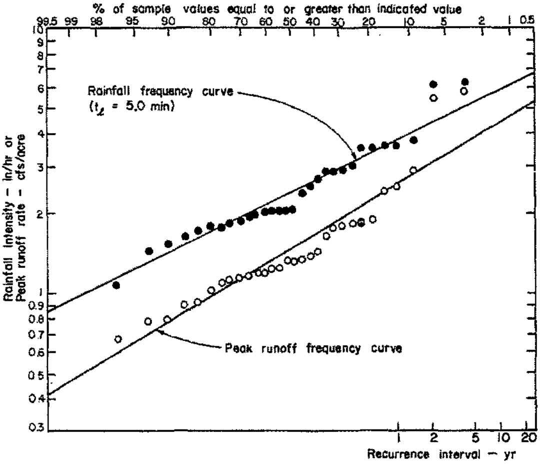


Fig. 5.—Distributions Of Rainfall And Runoff (N12)

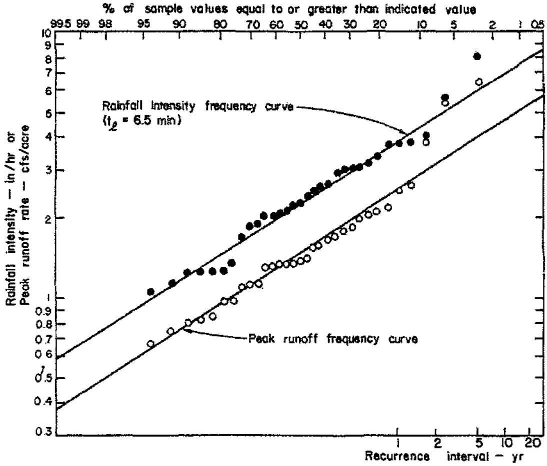


Fig. 6.—Distributions Of Rainfall And Runoff (Northwood)

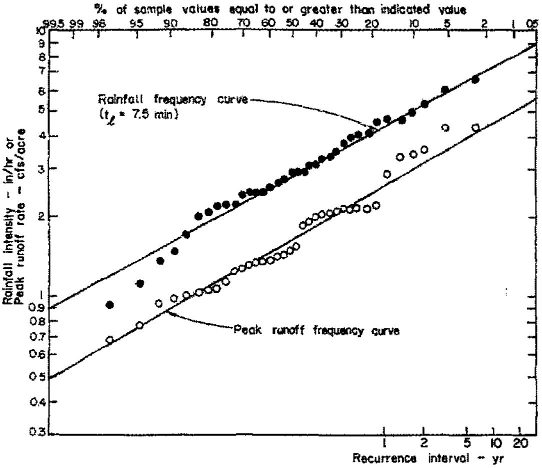


Fig. 7.—Distributions of Rainfall And Runoff (Swansea)

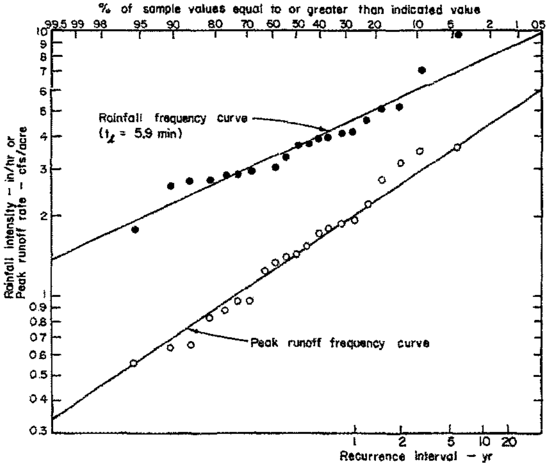


Fig. 8.—Distributions of Rainfall And Runoff (York Wood South)

in which T = the estimated recurrence interval, in years, and a tile lengthof record, in years. The relationship between T and P. therefore, is

$$T = \frac{(n+1)100}{(N+1)P}$$

Equation (6)

The C for each area was obtained from these frequency curves by solving for C in Eq. 3, that is,

$$C = \frac{q(T)}{I(t_a, T)}$$

Equation (7)

If the rainfall and runoff frequency distributions plotted as parallel straight lines in Figs. 3 to 9, the C would have a constant (but different) value for each

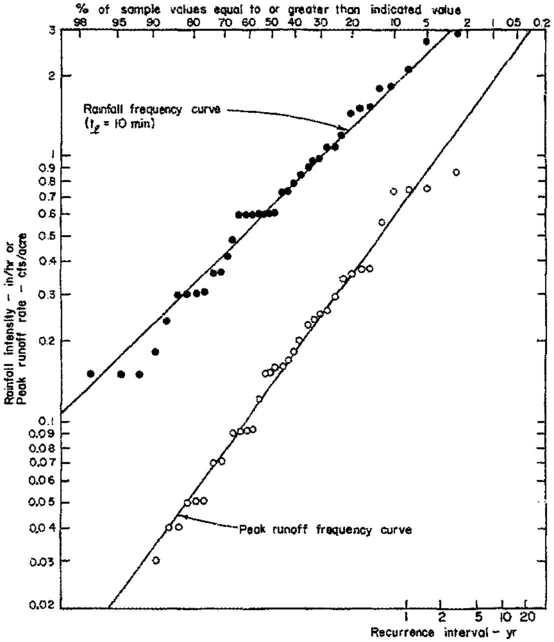


Fig. 9.—Distributions Of Rainfall And Runoff (Cincinnati)

area. However, there appears to be a tendency for the rainfall and runoff distributions to converge for each area. It appears that the C increases with increasing recurrence intervals, i.e., with the more intense, less frequent storms.

From these frequency curves, values of C for recurrence intervals of 1 yr and 10 yr were obtained using Eq. 7. These are presented in Table 3. Since

average increase from C1 to for six of these areas amounts to only 10%, from a practical view it seems reasonable that, pending a better approach to the problem of estimating design runoff rates for urban areas, constant values of C, equivalent to C5 or perhaps C ., would be adequate design use.

One of the major criticisms of the Rational Method arises because the observed C for individual storms usually varies from storm to storm. This variation is sufficiently great that the Rational Method should not be used to estimate the peak runoff rates for a particular storm. A method which would accomplish this would have to involve antecedent conditions and other variables both the drainage area and the rainstorm that caused the C to be different each storm.

A statistical analysis was made of the apparent convergence of rainfall and runoff frequency curves, It was first assumed that both C and I could be

Table 3.—C-Factors Corresponding To 1- And 10-Year Recurrence Intervals[a]

Drainage Area (1)	$C_{1\text{yr}}$ **(2)**	$C_{10\text{yr}}$ **(3)**	**Percentage Increase (4)**
Gray Haven	0.42	0.46	10
Newark 9	0.91	0.99	9
Newark 12	0.69	0.76	10
Northwood	0.66	0.67	2
Swansea	0.58	0.61	5
Yorkwood South	0.44	0.54	23
			Average 10

[a] The assumption that $t_a = t_l$ *was made in determining the values of* C *in this table.*

regarded as random variables. Previously it was found that log I was approximately normally distributed. A study of the data for 36 storms on the Swansea drainage area suggests that log C also may be approximately normal. If both I and log C were normally distributed, and if I and C were statistically independent, then log Q also would be normally distributed. The variance of Q (which determines the slope of the runoff frequency curve) would then be given by the relation

$$Var(\log Q) = Var(\log C) + Var(\log I)$$

Equation (8)

The Swansea data were studied to see if Eq. 8 could predict Var(log Q). The value obtained for Var(log Q) from Eq. 8 was 0.0497, and the value obtained from the runoff data for Var(log Q) was 0.0434. Because there is a slight correlation in the data between C and I, these values are not expected agree exactly.

The average values of C for Storms having 5 mm average rainfall intensities greater than 3 in. per hr are presented in Table 4. For comparison, values of also have been included in Table 4 for those areas where fre quency distributions could be plotted.

It should be noted that the C in Eq. 7 is a function both of f and T. It was observed above that C may be assumed to be nearly constant for a range of values of T. Because C also is a function offl, the values of C presented In Table 2 depend on the assumption that f, = t1. If some other time variable, rather than . had been used to estimate t, different values of C would have been obtained. It is therefore emphasized that the values of C in Table 2 are not equivalent to the values of C usually used in design practice.

Table 4.—Average Observed C-Factors For The Gaged Drainage Areas[a]

Drainage Area	Average *C*-Factor	C_{5yr}
Gray Haven	0.45	0.45
Hamilton Hill 2	0.42	-
Hamilton Hill 3	0.40	-
Hamilton Hill 4	1.02	-
Hamilton Hill 5	0.45	-
Midwood 5	0.51	-
Montebello 2	0.29	-
Montebello 3	0.54	-
Montebello 4	0.58	-
Montebello 5	0.66	-
Newark 9	1.01	0.97
Newark 12	0.76	0.74
Northwood	0.76	0.68
South Parking Lot 1	0.83	-
South Parking Lot 2	0.83	-
Swansea	0.61	0.60
Uplands	0.69	-
Walker Avenue	0.37	-
Yorkwood	0.56	0.51

[a] The assumption that $t_a = t_l$ was made in determining the values of C in this table.

5.0 RELATIONSHIP BETWEEN T AND THE PHYSICAL CHARACTERISTICS OF THE DRAINAGE AREAS

The average time lag between the centroid of the rainfall hyetograph arid the centroid of the runoff hydrograph will vary from area to area, depending on average slopes in the area, length of the main channel, the nature of the surface of the drainage area, and certainly many other factors which have not been considered herein. A relationship to estimate the average value of t for a drainage area is indicated by

$$t_l = f(IMP, L, S)$$

Equation (9)

In which t_l = the average lag time, in minutes, *Imp* = the Imperviousness of the drainage area, L the length of the main drainage channel, and S = the slope of the main channel. For inlet areas, tile length of the main drainage channel was measured from the storm water inlet upstream to the end of the paved gutter. For composite areas the length was measured from the downstream point of interest, proceeding upstream in the direction of the largest drain to an upstream inlet. To this length was added the length of paved gutter contributing to the storm water inlet. The slope, as a percentage, was found by dividing the difference in elevation between the upstream and downstream ends of the main channel by the length of the channel in hundreds of feet.

The imperviousness of the drainage area is included in tile functional relationship for t_l because it represents a measure of the nature of the surface of the drainage area. It appears reasonable to believe that factors which affect the time required for runoff to appear downstream, such as the average length of flow path before reaching a paved surface, average roughness of the various flow paths throughout a drainage area, etc., are correlated with the imperviousness of the drainage area. Hence it would be expected that the Imperviousness of the drainage area would be correlated with t_l.

A multiple regression analysis of the data for 19 gaged urban drainage areas was made to obtain an equation for The Cincinnati area was not included because sufficient data were not available for this area. An equation of the form

$$t_l = KL^a S^b IMP^c$$

Equation (10)

was assumed. The regression analysis was made using this equation rewritten as

$$\log t_l = \log K + a \log L + b \log S + c \log IMP$$

Equation (11)

and values for log K, a, h, and c were obtained from the regression. The equation which resulted was

$$t_l = \frac{1.05L^{0.24}}{S^{0.16}IMP^{0.26}}$$

Equation (12)

A slightly different equation for tl appears in Technical Report No. 1 of the Storm Drainage Research Project[8]. Additional information was used to obtain Eq. 10 given here. The multiple correlation coefficient between the estimated and observed values of t_l was 0.85. About two-thirds of the estimates of t_l from Eq. 12 are expected to be between -22% and +13% of the true value of t_l. The range of values on which the equation for I is based are: (1) Imperviousness of the drainage area greater than 8%; (2) average slope of the main drainage channel greater than 0.5% and not greater than 6%; (3) length of the main drainage channel greater than 150 ft and less than 6,000 ft; (4) gutters draining to storm water inlets were paved; and (5) house roofs were not connected directly to the storm drain.

Eq. 12 should be used only within these limitations. The values thus obtained for t_l are estimates of the average value of t_l that would be obtained from a number of storms. Values observed for individual storms are dependent on the nature of the individual storm and will not necessarily equal the average value.

6.0 RELATIONSHIP BETWEEN ('AND THE PHYSICAL CHARACTERISTICS OF THE DRAINAGE AREAS

The average observed *C*-factors given in Table 4 are highly correlated with the imperviousness of the gaged drainage areas. (The simple correlation coefficient between sample values of *C* and Imp was 0.91.) A linear equation relating these variables was fitted to observations of C and Imp from 18 gaged areas. The resulting equation accounted for 83% (0,912) of the total variation of the *C*-factor, and the standard error of estimate was found to be 0.09. That is, about two-thirds of the estimates of *C* from this equation would be expected to agree with ±0.09 of the true *C* value.

A study of the errors obtained from this initial equation revealed low estimates for areas having steep slopes (slope of the main drainage channel) and conversely for areas having flat slopes. This might have been anticipated since the simple correlation of *S* and *Imp* was only 0.01.

It was apparent that improved accuracy could be obtained by including the slope parameter in the equation for *C A* linear equation

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[8] Schaake, J. C. Jr., Geyer, J. C., and Knapp, J. W., "Runoff Estimates by the Rational Method," Technical Report No. 1, The Storm Drainage Research Project, Dept. of Sanitary Engrg. and Water Resources, The Johns Hopkins Univ., Baltimore, Md., 1964.

$$C = a + b(IMP) + c(S)$$

Equation (13)

was fitted to the observed data. The equation obtained from the regression was

$$C = 0.14 + 0.65(IMP) + 0.05(S)$$

Equation (14)

The multiple correlation coefficient obtained was 0.95, which means that more than 89% of the total variation of *C* has been accounted for by the regression. The standard error of estimate of *C* was found to be 0.074.

The possibility of obtaining improved accuracy by considering terms involving *(Imp)*2, (5)2, and *(Imp)* (5) was investigated, but the multiple correlation coefficient could not be increased above 0.95 nor the standard error reduced below 0.07.

Because it is possible that Eq. 14 is valid only for the type of storm patterns occurring in the Baltimore area, it should be used with caution elsewhere. Also, values of C from Eq. 14 should not be used in the Rational Method without also using Eq. 12 to estimate 0 = 1, The values of *C* estimated by this equation are not the same as values of *C* usually used in design practice, since the assumption that t, = i has been used in obtaining values of C for the gaged areas.

7.0 COMPARISONS WITH DESIGN PEAK RUNOFF RATES ESTIMATED BY THE RATIONAL METHOD

The estimates made using the Rational Method are compared, for six gaged drainage areas, with runoff values from frequency curves. Because only a few years of record was available (3 to 5 yr) the frequency curves have wide confidence limits. However, the rainfall intensities obtained front the rainfall frequency distributions at each gage for a recurrence interval of 5 yr are within ±15% of the values obtained front the 70-yr record of rainfall at the U. S. Weather Bureau gage in Baltimore. Rational Method estimates of the 5 yr runoff for each of the six areas were made using values of *C* and $t_a = t_c$, selected by five storm drain designers in accordance with their usual design procedures. These estimates, referred to ~*q*(5 yr), are presented in Table 5. For each area the 5-yr runoff values from runoff frequency curves are presented in the tabulated data for each area

Table 5.—Comparison of Estimates of (5yr) Obtained By the Rational Method Using Values of C and t_a Reported By Various Agencies For Six Gaged Drainage Areas

Gaged Area (1)	Agency (2)	Value of C (4)	Value of t_a, in minutes (5)	Rainfall Intensity, I(5yr), in inches per hour (5)	Rational Method Estimate, ~q(5yr), in cubic feet per second per acre	Percentage Error (7)
Gray Haven						
A=23.3 acres	a	0.56	13.0	4.87	2.73	+2
Q(5yr)=59.5 cfs	c	0.71	8.1	6.06	4.30	+61
q(5yr)=2.67 cfs per acre	d	0.55	10.3	5.38	2.96	+11
n=3 yr	e	0.52	14.6	4.59	2.39	-10
N=12 storms	f	0.53	7.5	6.21	3.29	+-23
Newark (N9)						
A=0.64 acres	a	0.85	5.0	5.68	4.83	-23
Q(5yr)=4.01 cfs	b	0.90	5.0	5.68	5.11	-19
q(5yr)=6.27 cfs per acre	c	1.00	5.0	5.68	5.68	-9
n=3 yr	d	0.95	6.1	5.15	4.89	-22
N=37 storms	e	0.90	3.5	6.31	5.68	-9
	f	0.97	3.6	6.26	6.07	-3
Newark (N12)						
A=0.96 acres	a	0.85	5.0	5.25	4.46	+11
Q(5yr)=3.87 cfs	b	0.90	5.0	5.25	4.73	+17
q(5yr)=4.03 cfs per acre	c	1.00	5.0	5.25	5.25	+30
n=3 yr	d	0.95	9.9	4.16	3.95	-2
N=32 storms	e	0.90	7.5	4.70	4.23	+5
	f	0.83	5.0	5.25	4.36	+8
Northwood						
A=47.4 acres	a	0.66	14.0	4.53	2.99	-28
Q(5yr)=197 cfs	b	0.80	10.0	5.17	4.14	0
q(5yr)=4.15 cfs per acre	d	0.74	12.2	4.78	3.54	-15
n=4 yr	e	0.71	11.4	4.91	3.49	-16
N=47 storms	f	0.74	6.3	6.20	4.58	+11
Swansea						
A=47.3 acres	0.51	13.0	4.34	2.21	-45	
Q(5yr)=189 cfs	c	0.67	10.9	4.73	3.17	-21
q(5yr)=3.99 cfs per acre	d	0.50	13.0	4.34	2.17	-46
n=5 yr	e	0.54	11.9	4.50	2.43	-39
N=36 storms	f	0.59	7.0	5.79	3.42	-14
Yorkwood South						
A=10.4 acres	a	0.60	6.5	6.23	3.19	-11
Q(5yr)=37.5 cfs	d	0.60	10.1	5.33	3.20	-11
q(5yr)=3.6 cfs per acre	e	0.53	8.1	5.64	2.99	-17
n=5 yr	f	0.60	5.9	6.38	3.83	+6
N=20 storms						

in Col. 1. The letters a, b, c, d. e, f in Col. 2 refer to the various storm drain designers. Rainfall intensities used in the Rational Method to obtain the various estimates of *q*(5 yr) were obtained from rainfall time-intensity-frequency curves from the rainfall record at each gage.

The percentage differences between Rational Method estimates of runoff and corresponding values obtained from runoff frequency curves are given in Table 5.

In Fig. 10 a comparison is presented of runoff estimates from the Rational Method with values of *q*(5 yr) obtained from runoff frequency curves for each agency participating in the study. Considerable variability Is observed in estimates obtained by usual design procedures. More than 2crEb of the designers' estimates differed by 25 or more from values obtained from runoff frequency curves.

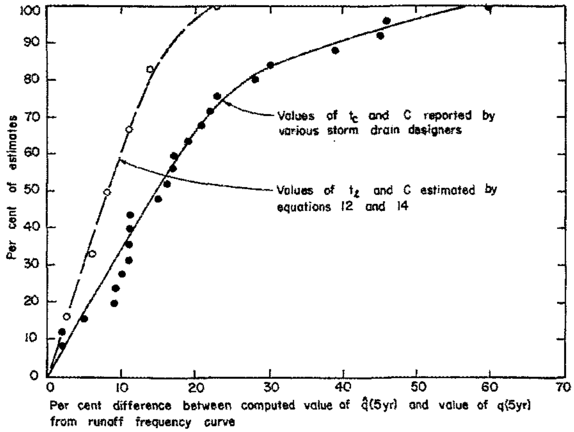

Fig. 10.—Comparison of Values of (5 Yr) Computed By the Rational Method With Values f Q (5 Yr) Obtained From Runoff Frequent Curves

8.0 CONCLUSIONS

Experimental rainfall and runoff data collected In Baltimore, Md., from 20 gaged urban drainage areas ranging in size up to 150 acres have been used in an examination of the Rational Method. Although sufficiently long records were not available to judge the Rational Method on a statistical basis, a report of this examination of the Rational Method is believed worth while on the grounds it may lead to a better understanding of the method by those who use it.

An attempt was made to study the assumption, implicity in the method as it is now used, that the frequency of occurrence of the computed design peak runoff rate is the same as the frequency of the rainfall intensity selected by the designer. The results suggest that this assumption is approximately correct.

Empirical equations have been presented for computing the values of *C* and the "rainfall intensity averaging time" from the physical characteristics of the drainage area. Because the data used to develop these equations have been collected in the Baltimore area, they may not be applicable to other urban areas. The purpose in presenting these equations is to demonstrate the possibility of developing similar equations suitable for design use if sufficient data were collected from many gaged urban areas throughout the United States.

Rational Method estimates of runoff were made by five storm drain designers in accordance with their normal design procedures, and these are compared with runoff values from runoff frequency curves for six of the gaged areas. It was found from this comparison that for every five Rational Method estimates, one of them, on the average, may be in error by more than 259k.

9.0 APPENDIX.—NOTATION

The following symbols have been adopted for use in this paper:

A = drainage area size, in acres:

a = coefficient;

h = coefficient;

C = ratio of peak runoff rate, in cubic feet per second per acre, to the maximum rainfall intensity averaged over the time ta:

c = coefficient;

I = maximum average rainfall intensity, in inches per hour, averaged over the time ta,

Imp = ratio of impervious area to total area:

K = coefficient;

L = length of the main drainage channel, in feet;

m = an integer used to denote the rank of an element in a sequence:

N = number of years of data;

n = number of storms observed during N years of data:

P = percentage of sample values exceeding in magnitude the value of the value in a sequence of observations;

Q = peak runoff rate in cubic feet per second;

q = runoff rate in cubic feet per second per acre ($\sim q$= estimate of q);

S = slope of the main drainage channel, as percentage or feet per 100 feet;

T = return interval, in years;

t_a = rainfall Intensity averaging time, here defined as the length of time during which the instantaneous rainfall intensity is averaged to obtain an average rainfall intensity;

t_c = time of concentration: and

t_l = time lag between the centroid of the rainfall hyetograph and the centroid of the runoff hydrograph.

Chapter 12

A STUDY OF THE EXPENDITURES FOR URBAN WATER SERVICES

February 1969
ASCE Urban Water Resources Research Program
Technical Memorandum No. 7
New York, New York

1.0 SUMMARY

The American Society of Civil Engineers has undertaken for the Office of Water Resources Research, Department of Interior, a contract to assist in outlining and developing a coordinated national program of urban water resources research. Present replacement value of urban facilities for public water, wastewater, and storm drainage totals $110 billion and is increasing at more than $7 billion per year.

This brief study by Travelers Research Corporation (TRC) is in support of the Task Committee on Damage and Storage. The study objectives were to determine the feasibility of and provide guidelines for analyzing all urban water service costs and to determine their relationship to other local government expenditures, such as education, public works, and law enforcement.

The City of Hartford was chosen for the pilot investigation because it satisfied these criteria: 1) the City Manager's fiscal practices were suitable and appropriate records were available; 2) it was ascertained that the cooperation of key city personnel was assured; and 3) TRC is located in Hartford. Total cost of Hartford's water services has decreased from $8.5 million (17.4% of annual City budget) in 1965 to $5.1 million (7.4% of budget) in 1968. The decrease is due to completion of a major storm drainage project. Water sales were nearly constant at about $2.4 million during each of the four years under review. As in most urban areas, Hartford's structure for expenditures for total water services is complex. The City is the major participant in the local metropolitan water district that also embraces many adjacent towns. Hartford has, over the years, been subjected to catastrophic flooding from the Connecticut River. Local efforts and the Corps of Engineers have worked in conjunction to eliminate the flooding problem. However, the City has low-lying areas requiring storm runoff pumping. At present, the sewerage and storm drainage systems are combined, but steps are being taken to separate them. Federal and state funds are mixed with local expenditures to achieve various water service goals.

This study demonstrates that urban water service costs *can* be analyzed and their relationship to other local expenditures determined. For nationwide studies, the recommended approach is as follows:

- Divide the nation into regions that are relatively homogeneous in terms of meteorological storm potential.
- Sub-divide these regions to obtain areas that are relatively homogeneous in terms of water supply potential.

- Select, from within these smaller areas, cities that have well-established water services, but also have various degrees of sewage disposal problems (e.g., some on rivers with good flow all year, some on smaller streams that can be variable flow, and some away from rivers).
- For the cities selected for analysis, coordinate study plans with state and federal agencies for flood and water pollution control to be sure the systems being studied are satisfactory and that anomalies are understood and can be reflected in cost analyses.
- Define the required information to be collected from each of these cities.
- Determine the suitability of the fiscal practices and records maintenance of each city to be sure that the information required for the analysis is in fact available and can be obtained.
- In parallel with the previous operation, ascertain the attitude of the relevant City Managers and the degree of cooperation that can be expected during the analysis for that city.
- Coordinate with and receive concurrence from the study sponsors concerning the cities whose water services systems and expenditures are actually to be analyzed in detail.
- Establish the final details of analysis for each city to be studied, and coordinate plans with the study sponsor to assure all desired goals will be met.
- Conduct the analyses and prepare reports for study sponsor.

Chapter 13

AVAILABILITY OF RAINFALL-RUNOFF DATA FOR SEWERED DRAINAGE CATCHMENTS

(NOTE: ONLY THE INTRODUCTION AND SUMMARY OF THIS DOCUMENT ARE PROVIDED. THIS SUBJECT IS ADDRESSED FOR PARTLY SEWERED CATCHMENTS IN CHAPTER 17).

March 3, 1969
ASCE Urban Water Resources Research Program
Technical Memorandum No. 8
New York, New York

1.0 INTRODUCTION

The USGS Project on "Analysis of National Basic Information Needs in Urban Hydrology" is primarily aimed at improvement in design and management of urban storm drainage with a predominant emphasis on sewered catchments. Data are virtually non-existent that are suitable for the analysis of rainfall-runoff-quality relationships, and that have been collected with properly coordinated instrumentation in networks representing a variety of climatic, topographic and land use conditions. A properly designed information network is essential to secure maximum transferability of analysis results across a given region of the country and from one region to another.

The purpose of this technical memorandum is to make available to researchers, information on the very limited amount of available rainfall-runoff data for sewered drainage catchments. It is hoped that data listed by the ASCE program will meet an immediate need for initial testing of mathematical models developed previously for natural drainage catchments. Although superior, more reliable data are needed for the complex models that must be developed, model development and testing should be in progress in the interim using the limited data available. However, the reader is warned that the precision and reliability of all data cited have been either vaguely defined or are unknown, and "*caveat emptor.*"

Development of mathematical models of urban rainfall-runoff has been almost exclusively devoted to inlet areas, such as the work of Viessman[1] for impervious inlet areas, or for only partially sewered catchments, such as the work of Espey and Winslow.[2] Rainfall-runoff data on a large number of inlet areas representing a range of land use were collected by the Johns Hopkins University[3] until its project was

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[1] Viessman, W. Jr. February 1968. "Runoff Estimation for Very Small Drainage Areas." *Water Resources Research*. Volume 4, No. 1, pp. 87-93.

[2] Espey, W.H. Jr., and D.E. Winslow. September 25, 1968. "The Effects of Urbanization on Unit Hydrographs for Small Watersheds, Houston, Texas, 1964-1967. TRACOR Document No. 68-975U. Austin, Texas (and separate "Data Compilation Appendices" of the same date).

[3] Horn, D.R. and N. Dee. January 1967. "A Description of Drainage Areas Gaged by the Storm Drainage Research Project." Technical Report 5 of the Storm Drainage Research Project. Department of Environmental Engineering Science. The Johns Hopkins University. Baltimore, Maryland.

discontinued in 1968. A description of the instrumentation used has been published.[4] A number of partially sewered areas are being gauged by the USGS.[5] However, this technical memorandum is exclusively limited to sewered drainage catchments because so little analytical work has been done on this type. A major reason for the neglect of sewered catchments by researchers in urban hydrology has been a problem of proper instrumentation for precise data collection.

For most gauged sewered catchments, rate of flow has been determined indirectly via a stage record using an assumed conduit friction coefficient and presuming the near-occurrence of uniform flow. Considerable subjective judgment is required in the reduction of flow data, and there is always a wide latitude of uncertainty in interpreting the probable precision of measurement by anyone who has not been directly involved in the data collection. For these reasons, only installations where rate of flow has been determined by means of flumes or weirs have been considered in detail here. This exclusiveness restricts consideration to flows gauged at sewer system outlets; however, practically all flow gauge installations on sewered drainage catchments have been at outlets. There have been instances where rainfall has not been measured in conjunction with runoff and these cases have been ignored.

Before proceeding with a summary on the availability of data for sewered drainage catchments where rainfall and rate of flow were determined by means of flumes or weirs, features are outlined of two of the better-known installations where stage recorders were employed.

Unit hydrographs for Louisville[6] were used by Eagleson in a published study.[7] He notes:

> *During the years 1945 through 1949, the Louisville District, United States Corps of Engineers, carried on an extensive program of measurement and analysis of hydrographs of storm sewer flow on a number of rather heavily urbanized small drainage areas in Louisville, Kentucky. The aim of the study was to develop unit hydrographs for these areas in order to facilitate the design of specific pumping plants.*
>
> *...the basic data and the numerical computations were not available to the writer.*
>
> *...Float-type water stage recorders were installed in six principal Louisville storm sewers and calibrated by the salt-velocity technique. From these gauges, records of sewer discharge versus time were obtained for numerous*

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[4] Knapp, J.W. and J.C. Schaake, Jr. September 1963. "Measuring Rainfall and Runoff at Storm-Water Inlets." *Journal of the Hydraulics Division*, ASCE Proc. Volume 89, No. HY5, Paper 3644.

[5] Water Data for Metropolitan Areas, compiled by W.J. Schneider, Geological Survey, U.S. Department of the Interior, Water Supply Paper 1871, Washington, DC, 1968.

[6] "Report of Sewer Runoff Investigation for Louisville, Kentucky." Corps of Engineers, Louisville District, 1949.

[7] Eagleson, P.S. March 1962. "Unit Hydrograph Characteristics for Sewered Areas." *Journal of the Hydraulics Division*, ASCE Proc., Volume 88, No. HY2, Paper 3069.

> *storms. Rainfall records were obtained from ten recording rain gauges in Louisville and vicinity during the same storms for which discharge records were obtained.*

The six catchment area sizes ranged from 61 to 4,810 acres, and presumably were all drained by combined sewerage systems.

One of the pioneering urban hydrology studies was by Hicks.[8] He developed a method for computing hydrographs for sewered catchments in the Los Angeles area based in part on local rainfall and runoff measurements. Hicks notes, "much of the complete research, upon which this paper is based, has been placed on file for reference in Engineering Societies Library." The type of device employed for discharge determination for the sewered areas (515 to 1,549 acres) is not evident in his paper.

Rainfall-runoff measurements have also been made by the City of Los Angeles since 1944.[9] There are presently three drainage catchments within Los Angeles for which records are being maintained; and there had been an additional catchment, for which records were discontinued in 1960. The smallest catchment is 19.4 acres and is located in undeveloped Griffith Park. The second catchment is 115 acres and is located in an undeveloped area in the Santa Monica Mountains. The third catchment is located in a developed part of the City, is partially sewered and is 250 acres in size. The largest catchment (422 acres), for which measurements were discontinued in 1960, is located immediately adjacent to the 250-acre catchment.

An automatic rain gauge and stage recorder are used at each of the three drainage catchments. Stage is measured within outfall conduits in the largest two catchments, and in an open stream in the smallest catchment. The clock time is marked on rainfall and stage recording charts when the charts are changed, and the rainfall and stage events are thereby synchronized. The rain gauge chart speed is one revolution per day and the stage recorder chart speed is one revolution per 12 hours.

Reduction of data is restricted to the tabulation of daily rainfall amounts in monthly summaries. The original charts and rainfall tabulations are on file in Bureau of Engineering Offices, City Hall, Los Angeles.

2.0 SUMMARY

Thirteen installations (discontinued and existing) for the collection of rainfall and runoff data from completely sewered urban drainage catchments are discussed in the remaining sections of this technical memorandum. Partially sewered urban drainage catchments are not considered. The installations described are additionally limited to those containing either a flume or weir for measuring runoff.

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[8] Hicks, W.I. 1944. "A Method of Computing Urban Runoff." *Transactions*, ASCE, Volume 71.

[9] Information obtained by personal communication with the City of Los Angeles, Bureau of Engineering, Mr. Lyall A. Pardee, City Engineer.

Discontinued installations, described in Section 3, consist of three catchments instrumented by The Johns Hopkins University Storm Drainage Research Project, a catchment instrumented by the U.S. Public Health Service in Cincinnati, and three catchments instrumented by the City of St. Louis. The Storm Drainage Research Project began operation in 1948 and ended in early 1968.[3] Runoff was measured via a flume on three drainage catchments (excluding other installations on inlet areas). The three catchments are Northwood (47.4 acres), Gray Haven (23.3 acres) and Swansea (47.1 acres), all of which are located in or near the City of Baltimore. Reduced rainfall and runoff data are available for 14 storms for Northwood, 29 storms for Gray Haven, and one storm for Swansea.

Operation of the installation in Cincinnati (27 acres) began in 1962 and ended in 1966. An unusual feature of this installation is that it is one of the very few catchments served entirely by storm sewers where quality data as well as rainfall and runoff data were collected. Results of analysis of the data are summarized in references[10], [11], [12], and [13]. Original data from the study are no longer available, having been discarded when the project terminated.

The installations in St. Louis were pioneering efforts in the collection of runoff data via a flume or weir, simultaneously, with rainfall data. The installations were in operation from 1914 to 1933. The sizes of the catchments were 2.25 acres, 3.27 acres and 4.35 acres. Detailed analysis of the data was made by W.W. Horner and F.L. Flynt and published in 1936.[14] Original records are in storage at the St. Louis Sewer District, but their completeness is not known.

Existing or new installations, described in Section 4, consist of a catchment in Chicago instrumented by the Chicago Department of Public Works, a catchment in Philadelphia instrumented by the Philadelphia Water Department, four catchments in New York City instrumented by the New York City Department of Public Works and a catchment in Washington, DC instrumented by Underwater Storage, Inc. and Silver, Schwartz, Ltd.

Operation of the installation in Chicago, called Oakdale (12.9 acres), began in 1959. Reduced rainfall and runoff data, considered reliable, for 21 storms, are given in ASCE Program Technical Memorandum No. 2.[15]

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[10] Weibel, S.R., R.J. Anderson and R.L. Woodward. July 1964. "Urban Land Runoff as a Factor in Stream Pollution." *Journal Water Pollution Control Federation*, Volume 36, No. 7, p. 14.

[11] Weibel, S.R., R.B. Weidner, J.M. Cohen, and A.G. Christianson. 1966. "Pesticides and Other Contaminants in Rainfall and Runoff." *Journal American Water Works Association*. Volume 58, p. 1075.

[12] Weibel, S.R., R.B. Weidner, A.G. Christianson, and R.J. Anderson. 1966. "Characterization, Treatment, and Disposal of Urban Stormwater." *Advances in Water Pollution Research Proceedings 3rd International Water Pollution Research*. Port City Press, Inc., Baltimore, MD, Volume 1, 329.

[13] Evans, F.L., E.E. Geldreich, S.R. Weibel and G.G. Robech. May 1968. "Treatment of Urban Stormwater Runoff." *Journal Water Pollution Control Federation*. Volume 40, No. 5, Part 2.

[14] Horner, W.W. and F.L. Flynt. 1936. "Relations Between Rainfall and Run-Off from Small Urban Areas." *Transactions*, American Society of Civil Engineers, Volume 101.

[15] Tucker, L.S. August 15, 1968. "Oakdale Gaging Installation, Chicago—Instrumentation and Data." ASCE Urban Water Resources Research Program Technical Memorandum No. 2.

Operation of the installation in Philadelphia, called Wingohocking (5,362 acres), began in 1963. Quality data have been collected in addition to rainfall and runoff data. The catchment is served by combined sewers and the runoff measured at the outfall outlet is thereby a mixture of sanitary and storm sewage that bypasses an interceptor. Runoff records, only, have been reduced for 41 storms that occurred in 1964 and 1965.

The four installations in New York City were completed early in 1969. The catchments are served by combined sewers and are called Spring Creek No. 1 (1,900 acres), Spring Creek No. 2 (1,400 acres), Hendrix Canal (500 acres) and Thurston Basin (2,000 acres). The installations were established in support of an FWPCA research and development contract.

Operation of the installation in Washington DC (30 acres) began in November 1968. Quality data have been collected in addition to rainfall and runoff data. The catchment is served by a combined sewer system. The installation was established in support of an FWPCA research and development contract.

A summary of information on the drainage catchments is given in Table 13-1. The information includes size of catchments, type of data collected, type of flow measuring device used, type of storm sewer system, data available, operator of installation, location and period of operation.

It should be noted in passing that the fieldwork supported by FWPCA at Jamaica Bay and Washington are illustrative examples. Information on other recently initiated installations can be obtained from the Storm and Combined Sewer Pollution Control Branch, Division of Applied Science and Technology, Federal Water Pollution Control Administration, U.S. Department of the Interior, Washington, DC 20242.

TABLE 13-1

Summary Information for Data 1 Collection Installations on 14 Completely Sewered Urban Drainage Catchments

Name of Catchment	Size, Acres	Data Collected	Type of Flow Measuring Device	Type of Storm Sewer System	Data Available		Operation of Installation	Location	Period of Operation	
					Charts	Reduced			From	To
*Northwood	47.4	Rainfall, Runoff	Parshall Flume	Separate	Yes—the completeness of data unknown	14 storms (rainfall & runoff)	Storm Drainage Research Project, Johns Hopkins University	Baltimore	1959	1967
*Gray Haven	23.3	Rainfall, Runoff	Parshall Flume	Separate	Yes—the completeness of data unknown	29 storms (rainfall & runoff)	Storm Drainage Research Project, Johns Hopkins University	7 miles east of Baltimore	1962	1967
*Swansea	47.1	Rainfall, Runoff	Parshall Flume	Separate	Yes—the completeness of data unknown	1 storm (rainfall & runoff)	Storm Drainage Research Project, Johns Hopkins University	Baltimore	1959	1967
*(U.S. PHS)	27	Rainfall, Runoff Quality	Weir	Separate	None	None	U.S. Public Health Service, R.A. Taft Sanitary Engineering Center	Cincinnati	1962	1966
*Station A	2.25	Rainfall, Runoff	Weir	Combined	Some data available, but extent unknown	--	City of St. Louis	St. Louis	1914	1933

Name of Catchment	Size, Acres	Data Collected	Type of Flow Measuring Device	Type of Storm Sewer System	Data Available		Operation of Installation	Location	Period of Operation	
					Charts	Reduced			From	To
*Station B	3.27	Rainfall, Runoff	Weir	Combined	Some data available, but extent unknown	--	City of St. Louis	St. Louis	1914	1933
*Station C	4.35	Rainfall, Runoff	Weir	Combined	Some data available, but extent unknown	--	City of St. Louis	St. Louis	1914	1933
*Oakdale	12.9	Rainfall, Runoff	Parabolic Flume	Combined	Yes	21 storms for which data considered reliable (rainfall & runoff)	City of Chicago	Chicago	1959	Present
*Wingohocking	5,362	Rainfall, Runoff	Weir	Combined	Yes	41 storms (runoff only)	City of Philadelphia	Philadelphia	1963	Present
*Spring Creek No. 1	1,900	Rainfall, Runoff Quality	Weir	Combined	--	--	City of New York	New York	1/1969	Present
*Spring Creek No. 2	1,400	Rainfall, Runoff Quality	Weir	Combined	--	--	City of New York	New York	1/1969	Present
*Hendrix Canal	500	Rainfall, Runoff Quality	Weir	Combined	--	--	City of New York	New York	1/1969	Present
*Thurstin Basin	2,000	Rainfall, Runoff Quality	Weir	Combined	--	--	City of New York	New York	1/1969	Present

Name of Catchment	Size, Acres	Data Collected	Type of Flow Measuring Device	Type of Storm Sewer System	Data Available		Operation of Installation	Location	Period of Operation	
					Charts	Reduced			From	To
FWPCA	30	Rainfall, Runoff Quality	Parshall Flume	Combined	Yes	None as of April 1969	Underwater Storage, Inc. Washington DC	Washington DC	11/1969	Present

* Discontinued Installations

Chapter 14

RAIN GAUGE NETWORKS IN THE LARGEST CITIES

(NOTE: ONLY THE INTRODUCTION AND SUMMARY OF THIS DOCUMENT ARE PROVIDED)

March 17, 1969
ASCE Urban Water Resources Research Program
Technical Memorandum No. 9
New York, New York

1.0 INTRODUCTION

Findings of the first year of the OWRR project on a "Systematic Study and Development of Long-Range Programs of Urban Water Resources Research" have been published by ASCE in *Urban Water Resources Research*, September 1968. The quotations that follow are from pages 3, 4 and 7 of that report.

> *Emphasis in the first project year was deliberately on subjects requiring earliest consideration.*
>
> *Considerable attention was given to several technical aspects of storm drainage. Major goals are: to arrive at general mathematical descriptions of the rainfall-runoff process combined with a basis for quantifying pollution loadings from storm sewerage systems applicable nationwide; to develop adequate technical knowledge for reliable planning of water quality and quantity exchanges between the several urban water service functions for multi-purpose development; and to facilitate and improve the planning, design and management of drainage works.*
>
> *Because the hydrology of urban drainage is the keystone for improved understanding of the rainfall-runoff-quality process, the highest immediate technical priority is for development of better and more compatible measuring and sampling devices, their perfection in field installations, and analysis of data therefrom. Very little data exists on the rainfall-runoff-quality process. Sewerage systems and canalized drainage works are generally constructed, operated and maintained by local governments. The acquisition of hydrologic and quality data for local drainage has not previously been considered an overall federal responsibility, and this springs at least partly from a question of jurisdiction. Very little data have been obtained by local agencies on their own systems and research of national scope is almost non-existent. Two of the several reasons for this situation are: the absence of a suitable assemblage of measurement devices and a typical lack of research facilities in local agencies charged primarily with design, construction, operation and maintenance of works.*
>
> *The Geological Survey, U.S. Department of the Interior, is studying requirements for data...*

> *Considerable potential synergistic benefits could be obtained in comprehensive, multi-purpose development of urban water, on a scale of involvement that in the aggregate could be much greater than for river basins. Reliable planning of water quantity and quality exchanges between the several urban water service functions is hampered particularly by a poor understanding of the urban rainfall-runoff-quality process. The primitive status of urban hydrology is the consequence of an absence of a meaningful body of field data. Techniques needed directly and indirectly for simulation in water resources planning, development and management are largely latent for a lack of suitable data to test and develop them. The input for the water cycle is precipitation, and little is known about it in the urban context. As opposed to data needed for simulation model development, a form of historical storm data is needed as inputs for planning, development and management.*
>
> *The following research should be pursued simultaneously:*
>
> *a. Plans being formulated by the USGS for the acquisition of rainfall-runoff-quality data and its dissemination should be implemented as soon as possible, because model development is dependent on data availability and several years of data may be required for process mastery.*
>
> *b. Existing mathematical models for simulating the rainfall-runoff-quality process should be under continual testing and improvement, starting with the small amount of data now available.*
>
> *c. Research on metropolitan storms should be pursued not only to develop inputs suitable for future uses, but also for contemporary management and operation of works.*

The objectives of this technical memorandum are: to indicate the extent of rain gauge networks in the largest cities; and to document the history of data available. The scope of the study has been limited to the 20 largest U.S. cities. This restriction was imposed by time available for the study. There are rain gauge networks in some smaller cities, but these are few in number.

2.0 SUMMARY

This section is a summary of a survey of the recording rain gauge networks of the 20 largest U.S. cities and their environs. The number of rain gauges in and around these cities varies from one to 162. The cities of Boston, New York, Phoenix, Pittsburgh and San Francisco are served only by first order USWB recording rain gauge stations. Recording rain gauges in addition to the USWB stations are operated and maintained in and around 15 of the 20 largest cities. These cities are Baltimore, Chicago, Cleveland, Dallas, Detroit, Houston, Los Angeles, Milwaukee, New Orleans, Philadelphia, St. Louis, San Antonio, San Diego, Seattle, and Washington DC. Information on the number, type, location and history of the gauges, and on the form, use and availability of rainfall data for these 15 cities is presented in succeeding sections of this memorandum.

The existence and extent of rain gauge networks in each of the 20 largest cities was first determined by telephone. Only recording rain gauges were considered, and if it was determined that a rain gauge network other than USWB stations existed, a visit was made to the city to obtain required details when necessary and if possible.

Major findings from the network survey are as follows:

1) There are more than ten recording rain gauges in the metropolitan areas of 11 of the 20 largest cities.
2) There are eight or more recording rain gauges in the metropolitan areas of 14 of the 20 largest cities.
3) There are five or more recording rain gauges in the metropolitan areas of 15 of the 20 largest cities.
4) The metropolitan area with the largest network of recording rain gauges is Los Angeles where 162 rain gauges are operated. The most extreme distance between rain gauges is in excess of 70 miles.
5) The city with the largest number of rain gauges within city limits is Los Angeles with 42; and the most extreme distance between rain gauges is 20 miles.
6) Seattle has a unique method for recording, reducing and processing rainfall data that was put into operation in early 1965. Rainfall data is recorded on tapes, translated later to computer cards and processed through a computer with a minimum of manual data handling.
7) Chicago has the greatest amount of reduced historical data. There are 79 storms dating from 1932 to 1963 for which rainfall is reduced to 5-minute intervals for a network varying between 12 and 19 gauges.
8) Original rain gauge charts are on file for all city networks for various periods of time. Information on location and availability of charts is given in this technical memorandum.
9) Rainfall data are used in the operation of drainage works, investigation of drainage complaints, preparation and verification of intensity-duration-frequency curves, and research. The most common use is for checking design curves.
10) Rainfall data are centrally coordinated on a large scale metropolitan basis in Los Angeles, Detroit and San Diego. The rain gauges are owned and operated by various government and private organizations including the USWB, but the data are processed through and filed with the central agencies.
    a) Chicago—Rainfall data from 12 rain gauges are transmitted via leased telephone lines to the Metropolitan Sanitary District main office building

at 100 East Erie Street where they are recorded on Bristol recorder charts visually displayed in a manned control center.

b) Detroit—Rainfall data from 13 rain gauges are to be telemetered into a data logging center located in the Water Board Building at 735 Randolph Street. The data logger will report at 5-minute intervals how many tips occurred during the previous 5-minutes, and the total amount of rainfall accumulated since the beginning of rainfall. The information will be automatically typed on sheets as it is processed through the data logging center. The 13 rain gauges were installed in December 1968 and the telemetering system is expected to be completed in early 1969.

c) Philadelphia—Rainfall data from three city owned experimental rain gauges can be transmitted to the Water Department Load Control Center (LCC) at 29th and Cambria Streets. The signal is transmitted over leased telephone lines to Water Department microwave stations and from there to the LCC. The rainfall record is recorded on charts at the LCC.

d) San Diego—Rainfall data from six USWB rain gauges are transmitted to County of San Diego offices at 5555 Overland Avenue, where the data are recorded on tape. The tapes are processed through a computer and then sent to a USWB regional office in Salt Lake City.

A summary of the rain gauge network information for 15 of the 20 largest cities is presented in Table 14-1.

TABLE 14-1

Summary Information for Rain Gauge Networks in 15 of the 20 Largest Cities

City	Pop. (Approx)	Rank in Top 20 Cities	Area, Sq. Mi	Number of Recording Rain Gauges		Greatest Distance Between Rain Gauge in Metr. Netw. (Miles)	Type of Rain Gauges			Charts Available	Uses of Data	Rain Gauge Owner	Method used to Collect Data	Remarks
				Within City	City and Environs		Weighing	Tipping Bucket	Float					
Chicago	3,675,000	2	225	7	21	44	2	19	--	Yes—Met. San. Dist., 100 E. Erie, Chicago	In the operation of the Chicago Inland Water System	MSD—19 USWB—2	Data from 12 gauges telemetered to 100 E. Erie St. Data (charts) from remaining gauges acquired in field at rain gauge site	Rainfall data for 79 storms reduced to 5-minute intervals
Los Angeles	2,727,000	3	70	42	162	70	150	5	7	Yes—Los Angeles Flood Control District, P.O. Box 241B, Terminal Bldg., Los Angeles	Operation of drainage works, water conservation studies, design, determining water yield, flood control, water supply, irrigation	Los Angeles Flood Control District—89 Others—73	Data (charts) from all gauges acquired in field at rain gauge site	70 miles is for gauges shown on Figure 1LA which does not include 7 gauge located in area north of map boundary

City	Pop. (Approx)	Rank in Top 20 Cities	Area, Sq. Mi	Number of Recording Rain Gauges		Greatest Distance Between Rain Gauge in Metr. Netw. (Miles)	Type of Rain Gauges			Charts Available	Uses of Data	Rain Gauge Owner	Method used to Collect Data	Remarks
				Within City	City and Environs		Weighing	Tipping Bucket	Float					
Philadelphia	1,964,000	4	126	29	30	20	30	--	--	Yes—R&D unit of Phil. Water Dept., Municipal Building	Data primarily used to upgrade storm drainage design	City—26 Drexel Inst.—1 Franklin Inst.—1 USWB—2	Data from 3 experimental gauges can be telemetered to central location. Data (charts) from remaining acquired in field at rain gauge site	
Detroit	1,739,000	5	139	17	61[1]	55	17	--	--	Yes—Wastewater Systems Section, Detroit Metropolitan Water Services	Analyzing drainage complaints Check design curves Prepare storm reports	Detroit—16 Other—45	Data (charts) from all gauges acquired in field at rain gauge site	In early 1969, 13 additional tipping bucket gauges will be capable of transmitting data to control center Detroit gauges only
Houston	1,013,000	6	444	21	33[2]	31	9	--	24	Yes—USGS, Room 174, 2320 Labranch, Houston	In support of study conducted by USGS in cooperation with City	USGS—29 USWB—4	Data (charts) from all gauges acquired in field at rain gauge site	17 of the 24 float gauges are the USGS S-R type gauges

City	Pop. (Approx)	Rank in Top 20 Cities	Area, Sq. Mi	Number of Recording Rain Gauges		Greatest Distance Between Rain Gauge in Metr. Netw. (Miles)	Type of Rain Gauges			Charts Available	Uses of Data	Rain Gauge Owner	Method used to Collect Data	Remarks
				Within City	City and Environs		Weighing	Tipping Bucket	Float					
Baltimore	933,000	7	83	5	9^3	15	9	--	--	Yes—1967 to present at Baltimore City and County Offices. 1950 to 1967 with Professor J.C. Schaake, MIT, Cambridge, Massachusetts	From 1950 to 1967 data were used by the Johns Hopkins Storm Drainage Research in support of their project	County—3 City—4 USWB—2	Data (charts) from all gauges acquired in field at rain gauge site.	The storm drainage research project terminated in 1967 and rain gauges were taken over by the City and County
Cleveland	924,000	8	76	3	5	18	--	--	--			USWB—2 Cleveland Heights—2 Cleveland—1	Data (charts) from all gauges acquired in field at rain gauge site.	
Washington DC	802,000	9	61	8	9	10	9	--	--	Yes—1965 to present at System Planning Division, Department of Sanitary Engineering, Presidential Building	1. To upgrade storm drainage design curves 2. Design of relief sewers 3. In investigating complaints of flooding	District—7 USWB—2	Data (charts) from all gauges acquired in field at rain gauge site.	

City	Pop. (Approx)	Rank in Top 20 Cities	Area, Sq. Mi	Number of Recording Rain Gauges		Greatest Distance Between Rain Gauge in Metr. Netw. (Miles)	Type of Rain Gauges			Charts Available	Uses of Data	Rain Gauge Owner	Method used to Collect Data	Remarks
				Within City	City and Environs		Weighing	Tipping Bucket	Float					
St. Louis	769,000	10	62	1	7	18	8	--	--	Yes—1960 to present at Metropolitan St. Louis Sewer District, 2000 Hampton Avenue, St. Louis	Principally for use in design	Metropolitan St. Louis Sewer District—6 USWB—2	Data (charts) from all gauges acquired in field at rain gauge site.	
Milwaukee	760,000	11	95	14	14	15	4	10	--	Yes—Sewer Engineering Division, Bureau of Engineering, DPW, Municipal Building	Principally to upgrade storm drainage design curves	City—13 USWB—1	Data (charts) from all gauges acquired in field at rain gauge site.	Tipping bucket gauges are operated seasonally
Dallas	697,000	13	75	20	34	24	31	--	3	Yes—USGS, Fort Worth, Federal Center Bldg., P.O. Box 6976	In support of study conducted by USGS in cooperation with City and Corps of Engineers	USGS—31 USWB—2 C of E—1	Data (charts) from all gauges acquired in field at rain gauge site.	

City	Pop. (Approx)	Rank in Top 20 Cities	Area, Sq. Mi	Number of Recording Rain Gauges		Greatest Distance Between Rain Gauge in Metr. Netw. (Miles)	Type of Rain Gauges			Charts Available	Uses of Data	Rain Gauge Owner	Method used to Collect Data	Remarks
				Within City	City and Environs		Weighing	Tipping Bucket	Float					
New Orleans	675,000	15	50	12	12	11	2	10	--	Yes—from 1962 to present at Sewerage and Water Board Offices in City Hall	To upgrade storm drainage design curves	City—10 USWB—1	Data (charts) from all gauges acquired in field at rain gauge site.	The actual area of New Orleans is 363 sq. mi. but drained area only 50 sq. mi. Network located entirely in drained area
San Antonio	654,000	16	189	7	16	18	7	1	8	Yes—USGS, 7077 San Pedro, San Antonio. Network just installed during summer and fall of 1968	In support of study conducted by USGS in cooperation with the Texas Water Development Board	USGS—16	Data (charts) from all gauges acquired in field at rain gauge site.	Five of the float gauges are the USGS S-R digital recording type
San Diego	648,000	17	270	2	29	78	29	--	--	Yes—Department of Special District Services, County of San Diego, 5555 Overland Avenue, San Diego	In support of studies to provide optimum design of drainage works and to operate a flood forecasting system	USWB—13 S.D. County—12 Other Gov't Agencies—3 Pub Utility—1	Data from six USWB gauges telemetered to 5555 Overland Avenue. Data (charts) from all gauges acquired in field at rain gauge site.	The USWB rain gauge charts are sent to USWB Regional Office in Salt Lake City and are not available at San Diego County

City	Pop. (Approx)	Rank in Top 20 Cities	Area, Sq. Mi	Number of Recording Rain Gauges		Greatest Distance Between Rain Gauge in Metr. Netw. (Miles)	Type of Rain Gauges			Charts Available	Uses of Data	Rain Gauge Owner	Method used to Collect Data	Remarks
				Within City	City and Environs		Weighing	Tipping Bucket	Float					
Seattle	563,000	20	88	15	15	15	--	15	--	Yes—Sewerage and Drainage Section, Seattle Engineering Department, Municipal Building	1. Upgrade storm drainage curves 2. Study storm trends 3. Supply rainfall data to other City Departments	City—15	Data (charts) from all gauges acquired in field at rain gauge site.	City has developed unique way to reduce data automatically without any manual chart manipulation

[1] Located in three-county area of Wayne, Macomb & Oakland.

[2] All but one gauge located in Harris County.

[3] All located in Baltimore County.

[4] All but four gauges located in Dallas County.

Chapter 15

COMBINED SEWER SEPARATION USING PRESSURE SEWERS

FEASIBILITY AND DEVELOPMENT OF A NEW METHOD FOR SEPARATING WASTEWATER FROM COMBINED SEWERS

October 1969
Federal Water Pollution Control Administration
Department Of The Interior
New York, New York

1.0 ABSTRACT

This report is concerned with the separation of community wastewaters and runoff from rainfall and snowmelt in areas presently served by combined and intercepting sewers. Separation is accomplished by withdrawing the wastewater fraction of flows from existing plumbing systems and passing it through a sequence of added systems components as follows: 1) a storage, grinding and pumping unit within each building; 2) pressure tubing fished from the unit through each existing building sewer into the existing combined sewer; and 3) pressure piping inserted in that sewer and extending to the existing intercepting sewers that carry the wastewaters to treatment and disposal works. Runoff from rainfall and snowmelt, thus unencumbered by wastewaters, is removed from the community through the residual passageways of the one-time combined sewer system, which has thus become a combination of a new pressure conduit system within an old gravity conduit system.

The feasibility of this scheme of separation, the selection of available systems components and the development of required new systems components are described in this report on the basis of information drawn from 25 project reports and technical memoranda.

The feasibility of storing, grinding and pumping sewage from individual residences has been established; and standard comminuting and pumping equipment will be satisfactory for serving larger buildings. Acceptable types of pressure tubing are available that can be pushed and pulled through existing building drains and sewers. Pressure conduits can be suspended inside combined sewers that can be entered by workmen. There are combined sewer areas that can be separated most effectively by a version of the method investigated, but generally pressure systems will cost more than new gravity systems. New capabilities developed appear to be of potentially greater use for applications other than separation, such as new construction including utility corridors, and introduce viable alternatives for design of wastewater sewerage.

This report was submitted in fulfillment of Contract Number 14-12-29 between the Federal Water Pollution Control Administration and the American Society of Civil Engineers.

2.0 SUMMARY

2.1 Introduction

Effective water pollution control may require separate or separated collection systems, *viz.,* wastewater systems that collect and deliver to treatment works the water-borne wastes of household and industry and stormwater systems that collect and deliver to nearby watercourses the runoff from rainfall and snowmelt. Wastewater treatment works remove or modify waste substances in the carrying water to a required degree before discharging the effluent to receiving waters within or not far from the communities in which the waste matters are generated.

For historical reasons, about one fifth of the nation's population is presently served by combined systems of sewerage[1] that collect both wastewater and storm runoff in a single set of sewers. To keep the wastewater component carried by combined collecting system out of rivers, lakes, and tidal estuaries, the dry-weather flows of combined sewers are intercepted before they reach the terminals of the sewer system and are diverted to treatment works. Only during rainstorms and snowmelts that swell the flows of combined sewers beyond the capacity of the interceptors is a mixture of wastewater and runoff from rainfall and snowmelt discharged into the bodies of water that are otherwise protected by the intercepting system. In the northern United States about 3% of the total annual volume of sewage and substantial volumes of wastewater solids are scoured from the sewer system during heavy runoff and are discharged to receiving watercourses through stormwater overflows.

After the turn of the last century, the danger and nuisance of combined-sewer overflows were abated in many new municipal sewerage schemes by the construction of two separate systems of sewers: a wastewater system and a stormwater system. However, most of the existing combined systems were continued in service together with their interceptors except that in some instances additions to these systems were built as separate systems in which the sanitary sewers alone were connected to the existing combined sewers.

In recent years, a greater public awareness of the value of a clean environment and unpolluted natural waters, together with a desire to eliminate urban blight, has focused interest on the prevention, storage, and treatment of overflows from combined sewers. In most cases, overflows can be prevented by restricting the use of the existing combined sewers to the removal of stormwater alone, or to the removal of wastewater alone. Whichever is done, it is necessary to carry the physical separation of the two systems into tributary private properties.

What is often referred to as *conventional separation,* therefore, normally involves considerable inconvenience to and disruption of normal community life by construction activities on private properties as well as on public thoroughfares. The estimated construction cost for conventional separation of the combined sewer

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[1] American Public Works Association, Project 123, for the U.S. Department of the Interior, Federal Water Pollution Control Administration. December 1967. *Problems of Combined Sewer Facilities and Overflows.*

systems in the United States that serve about 36 million people is \$48 billion,[1] averaging close to \$1,300 per person served. Because it recognized the high cost and disruptive nature of conventional separation, the U.S. Congress provided funds for the development of alternatives as part of the scope of the "Clean Water Restoration Act of 1966" (P.L. 89-753).

2.2 Project Concept

As conceived by Professor Gordon M. Fair of Harvard University, the method of sewer separation with which the present report is concerned would incorporate into existing combined sewer systems relatively small-diameter pipes that would convey the wastewater fraction under pressure to existing interceptors. The one-time combined sewers would be retained as the conveyors of stormwater that would discharge to receiving bodies of water either directly through existing outlets or indirectly after passage through stormwater retention or treatment tanks or underground facilities of a similar nature.

Structurally, the proposed system would begin at a grinding and pumping unit within each building served by the system. Where possible, the unit would prepare the wastewaters for delivery to the system through small-diameter tubing inserted in the building sewer and connected to a conduit inserted in and attached to the interior of the existing combined sewer. The main trunks of the branching network of pressure conduits would discharge into the existing interceptor that would convey only wastewaters to treatment works. The existing building sewers and combined sewers would deliver to receiving water bodies only stormwater runoff from rainfall and snowmelt, together with such groundwater as entered the system from the soil.

In the creation of the proposed separate wastewater system, construction activity and traffic disruption would be greatly reduced by using the pipe-within-a-pipe concept, or, where necessary, by installing combined sewers. If total costs were less than for conventional separation, the scheme would constitute a viable alternative to conventional separation. By excluding seepage waters from pressurized reaches, the hydraulic loads on interceptors and treatment works would be reduced accordingly. In addition, an inherent potential advantage of pressure sewerage is that the piping is free from the limitations of gravity systems that must constantly slope downward no matter what the surface topography.

To minimize and, insofar as possible, prevent the clogging of tubing, conduits and auxiliary fittings, Professor Fair's concept included the grinding of sewage solids and pressurization in a single assembly at each building in which surges of peak flows would be attenuated by storage of incoming flows from the building served. Each residential building or structure with similar flows, therefore, would have a "storage-grinder-pump" unit.

2.3 *Project Scope*

With the support of an FWPCA contract, the ASCE conducted an investigation of the feasibility of Professor Fair's concept. The present report contains the results of that investigation, which had the benefit of the counsel of an advisory committee called the Project Steering Committee. The principal project concerns were: 1) the development of an assembly of workable and dependable systems components; 2) a study of the physical feasibility of hypothetical pressure systems introduced into existing combined sewerage districts; and 3) a cost analysis of the hypothetical systems.

Because there was little, if any, direct information on pressurized sewer systems of this kind, it was necessary to develop required design criteria and procedures, construction methods and installation techniques, workable systems devices and parts, general system designs and other supporting information. The following example illustrates one of the required developments. Although engineering interest in pressure sewerage for a variety of applications is by no means new, lack of a suitable household storage-grinder-pump unit stood in the way of the effective utilization of pressure systems in residential areas. Accordingly, the development of a suitable storage-grinder-pump unit became a focal point of interest of the project.

Assistance was sought from organizations and individuals that were experienced in related fields. In the course of the work, assistance was received from over one hundred individuals in almost fifty organizations.

The Project Steering Committee recognized the principle that research findings should be useful for future pressure sewer applications both within and beyond the immediate Project concept and that, to this end, the Project studies should be as broad as time and funds would permit. The studies accomplished in this spirit are documented in detail in a series of 25 technical memoranda and reports totaling close to 1,500 pages.

3.0 FINDINGS AND CONCLUSIONS

3.1 *Findings and Conclusions on the Feasibility of Grinding, Storing and Pumping Domestic Sewage*

The feasibility of storing, grinding, and pumping sewage from individual residences has been established. The final phase of development of the household storage-grinder-pump unit and its associated field-testing is in progress in a follow-on project by the New York State Department of Health under an FWPCA grant.

In reference to the operation of comminuting and pumping installations serving commercial buildings, multiple dwellings, and other large sources, it is concluded that the use of standard equipment on services of this scale in pressurized systems will be satisfactory.

It is recognized that there is still room for the development of suitable storage, grinding and pumping equipment for duties lying between those of individual households and large sources. However, some recently introduced assemblages appear to be suitable.

3.2 Findings and Conclusions on Systems of Pipes Inserted in Existing Conduits

The question of inserting and securing tubing and piping in combined sewers required evaluation of: methods of threading tubing through building drains and sewers and suspending conduit in street sewers; and the effect of inserted conduits and their hangers on the hydraulic capacity of intruded combined sewers.

The Project studies and demonstrations showed that pressure tubing can be pushed and pulled through existing building drains and sewers, and that acceptable types of tubing are available.

A single-piece molded plastic hanger was developed for suspending pressure conduits inside combined sewers and found to be structurally adequate. However, it was discovered that field-insertion of conduits in hangers bonded in place was an awkward and complicated task. In this connection, furthermore, laboratory flow tests demonstrated that within a certain range of ratios of the conduit diameter to the sewer diameter, the protrusion of the portion of the hanger adjacent to the conduit unduly decreased the hydraulic capacity of intruded combined sewers. *It is concluded that since the main feature of the hanger is its bonding system, a conventional metal-strap loop and suspension rod can be used satisfactorily in conjunction with the original plastic hanger seat, which will facilitate installation and minimize the diminution of intruded combined sewer hydraulic capacity.*

Hanger installation was found to require direct access to the interior of combined sewers, thus limiting the installation of conduits to sewers that can be entered by workmen. This conclusion restricts the pipe insertion concept to about one-seventh of the total length of combined sewers in major cities and to a still lower proportion of such sewers in smaller communities. However, there are sectors of communities with large combined sewers and so much congestion of underground utilities and street traffic that installation of an additional system of sewers is not economically feasible unless the required piping is installed in the existing sewers. This is not to say that separation is the only way to reduce pollution from overflows. Alternatives do exist and are being investigated currently.

3.2 Findings and Conclusions on the Cost of Separation by the Project Scheme Versus Conventional Means

The feasibility and cost of the Project scheme were investigated by designing pressurized sewer systems for three areas of reasonable size that are representative of many existing combined sewer systems, as follows: 1) a 53-acre commercial downtown area in Boston, Massachusetts; 2) a 157-acre mainly residential area in

Milwaukee, Wisconsin; and 3) a 323-acre predominantly residential area in San Francisco, California. For purposes of comparison, conventional separation of the test areas was studied by consultants in the cases of Boston and Milwaukee, and by the Department of Public Works in the case of San Francisco.

Much background information had to be collected or developed before suitable designs could be prepared. Examples of required information are: expected residential and commercial sewage flows; conventional equipment and controls for storing, grinding, and pumping sewage from commercial buildings and other large sources; search for and testing of suitable tubing, piping and fittings, and methods for their installation; identification of the configurations of collection systems amenable to proper operation and maintenance; satisfaction of requirements for overall system pressure controls and pumps; and determination of minimum solids transport velocities.

It is estimated that construction costs for separation of these three test areas by the ASCE Project method might cost about 50% more than their separation by the conventional method of laying a second system of gravity conduits. These estimates did not take into account the inconvenience and loss of business that would be associated with conventional separation.

Whereas the unit cost of separating the plumbing systems of buildings might be lowered by including all conversions in a single contract, the structurally variegated requirements and special situations bound to be encountered in such work would probably operate against a substantial cost reduction. This is an important consideration because this work represents some 40% to 60% of the total cost of conventional separation of the three study areas. Because the cost of building separation was quite similar for the two methods in these instances, the *net* competition in construction costs between pressure and gravity systems was thereby restricted to the remaining 60% to 40% of the total cost of conventional separation.

Operation and maintenance would presumably be more expensive for a pressure system. Estimates of annual costs for the Milwaukee study area, including operating costs and amortization of construction costs, were about 85% higher for the pressure scheme than for conventional separation.

It is not possible without further study to determine the typicality of the cost estimates for the three study areas on a national basis. *Because much care was exercised in selecting these areas as reasonably typical, the results obtained suggest that the Project scheme will generally cost more than conventional separation.* However, although the Project scheme was not found to offer a general and direct means to lower the cost of sewer separation, it was recognized that *there are special situations in which existing systems or parts of systems can be separated most effectively by the ASCE Project method or a suitable modification of that method.*

3.3 Findings and Conclusions on the Physical Feasibility of the Project Scheme for Other Applications

Inherent in the successful first-phase development of the household storage-grinder-pump unit is the promise of the technological feasibility of pressurized sewer systems for residential areas. Completion of this development in the field in the near future, coupled with information on materials, procedures and design criteria collected or developed in the ASCE Project, provides a further degree of freedom in the design of sewer systems. *Uses for this new knowledge appear to be potentially greater for applications to purposes other than the separation of existing combined sewer systems.*

There is good reason to believe that many residential communities or subdivisions of communities will eventually have to replace septic tanks with public sewerage. Some of them will find that pressure systems meet their needs best. Typical examples are: residences on steeply sloping shores; areas encumbered by physical barriers such as escarpments and swamps; isolated pockets of low land; areas of undulant terrain; buildings from which wastewaters must be lifted to the level of existing gravity sewers; and areas of dispersed occupancy such as semi-rural areas.

Most of the examples cited have some bearing on new sewerage construction. In the leap-frog development of suburban areas, for instance, installation of light-weight pressure conduit by plowing techniques at shallow depth becomes relatively inexpensive and easy. Such pressure systems can then serve isolated new developments and subsequently be linked into the community system, whether it be gravity or a pressure system.

A promising future use of pressure systems is the full exploitation of utility corridors, called "utilidors." These corridors beneath city streets conserve and make efficient use of underground space, particularly where streets are burdened with heavy traffic, both vehicular and pedestrian. They also simplify maintenance, repair and replacement of the utilities they shelter. Inclusion of gravity sanitary sewers in utilidors would force the placing of the utilidors at the grades required for the sewers. This is a profoundly restrictive requirement except where needed sewer slopes and ground-surface slopes happen to be reasonably parallel. The use of pressure sewers would lift this constraint from utilidors in the same way as it commonly does from water mains. Utilidors, incidentally, are important features of cities in the far north where permafrost otherwise imposes severe restrictions on water supply and sewerage.

Lastly, some thought has been given in the Project study to the potential of extending the Project concept to the selective isolation, grinding, and wastewater transport of essentially all readily decomposable organic waste substances from households and industries to existing, enlarged, or integrated new waste treatment works.

Chapter 16

NON-METROPOLITAN DENSE RAIN GAUGE NETWORKS

January 1970
ASCE Urban Water Resources Research Program
Technical Memorandum No. 11
New York, New York

1.0 INTRODUCTION

Findings of the first year of the Office of Water Resources Research (OWRR) project on a "Systematic Study and Development of Long-Range Programs of Urban Water Resources Research" (report on OWRR supported portion of Phase I of ASCE Program) were published by ASCE in *Urban Water Resources Research*, September 1968.[1] The quotations that follow are from pages 3 and 7 of that report.

> *Considerable attention was given to several technical aspects of storm drainage. Major goals are: to arrive at general mathematical descriptions of the rainfall-runoff process combined with a basis for quantifying pollution loadings from storm sewer systems applicable nationwide; to develop adequate technical knowledge for reliable planning of water quality and quantity exchanges between the several urban water service functions for multi-purpose development; and to facilitate and improve the planning, design and management of drainage works.*
>
> *Considerable potential synergistic benefits could be obtained in comprehensive, multi-purpose development of urban water, on a scale of involvement that in the aggregate could be much greater than for river basins. Reliable planning of water quantity and quality exchanges between the several urban water service functions is hampered particularly by a process. The primitive status of urban hydrology is the consequence of an absence of a meaningful body of field data. Techniques needed directly and indirectly for simulation in water resources planning, development and management are largely latent for a lack of suitable data to test and develop them. The input for the water cycle is precipitation, and little is known about it in the urban context. As opposed to data needed for simulation model development, a form of historical storm data is needed as inputs for planning, development and management.*
>
> *The following research should be pursued simultaneously:*
>
> *a. Plans being formulated by the USGS for the acquisition of rainfall-runoff-quality data and its dissemination should be implemented as soon as possible, because model development is dependent on data availability and several years of data may be required for process mastery.*

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[1] U.S. Department of Interior. September 1968. *Urban Water Resources Research*, First Year Report to the Office of Water Resources Research. ASCE, New York, New York.

b. Existing mathematical models for simulating the rainfall-runoff-quality process should be under continual testing and improvement, starting with the small amount of data now available.

c. Research on metropolitan storms should be pursued not only to develop inputs suitable for future uses, but also for contemporary management and operation of works.

Research on metropolitan storms as recommended in item c., above, requires a reservoir of reliable rainfall data from dense rain gauge networks preferably located in urban areas. The extent of rain gauge networks and the history of data available in the 20 largest cities were documented in ASCE Urban Water Resources Research Program Technical Memorandum No. 9.[2] Rain gauge networks are operated by 15 of the 20 largest cities and their locations are noted on Figure 1. General conclusions drawn in Technical Memorandum No. 9 are: data from most city rain gauge networks are collected and retained on file in original chart form, and only selective summary tabulations of storm rainfall data are commonly available. Notable exceptions to the latter conclusion are Detroit, San Diego and Seattle. In Detroit, rainfall data from 13 recently installed tipping bucket rain gauges is transmitted via telephone lines to a data logging center where the rainfall data are automatically typed at 5-minute intervals. In San Diego, rainfall data from six digital-type rain gauges are transmitted via telephone lines to San Diego County offices where the data are recorded on magnetic tape. Processing the tapes through a computer results in printout sheets listing rainfall amounts at 15-minute intervals. In Seattle, rainfall data from 20 tipping bucket gauges are recorded on computer punch cards and a program has been written that permits obtaining rainfall intensity values at 1-minute intervals. In addition, in Chicago, rainfall data at 5-minute intervals has been recorded on tabulation sheets for 79 storms dating from June 1932 through July 1963 for a rain gauge network encompassing 12 to 21 rain gauges. The rain gauge networks in Chicago, Detroit, San Diego and Seattle are described in detail in footnote 2.

Thunderstorms are of particular interest in the study of metropolitan storms. Generally, individual rain cells of thunderstorms range up to about four miles in diameter and have a lifetime of about 25 minutes.[3] Therefore, networks consisting of recording rain gauges spaced less than about four miles apart with data reducible to a time interval of 15-minutes or less are of particular interest. A rain gauge network is referred to as "dense" in this technical memorandum if on the average there is one gauge per 16-square-miles or less. Time synchronization of rainfall data from a network of gauges is difficult. The Illinois State Water Survey has concluded that data from recording rain gauges with internal recording mechanisms powered by clocks with weekly chart speeds can be realistically and reliably reduced to 30-minute intervals, 24-hour charts to 15-minute intervals, and 6-hour charts to 5-minute

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[2] Tucker, L.S. March 17, 1969. "Raingage Networks in the Largest Cities." *ASCE Urban Water Resources Research Program Technical Memorandum No. 9.* New York, New York.

[3] Travelers Research Center, Inc. September 1968. "Considerations for Characterizing the Time and Space Distributions of Metropolitan Storm Rainfall."

intervals. On this basis, use of rainfall data at 15-minute intervals should be restricted to rain gauges with internal recording mechanisms powered by clocks with chart speeds of 24-hours or less. Accurate synchronization can best be obtained when all rainfall data are simultaneously transmitted to a data logging center where the information is recorded with the aid of data processing equipment.

Rainfall data from dense networks in metropolitan areas available in an amenable form for the study of metropolitan scale storms is limited, and it will be necessary to use augmented rainfall data from dense networks in non-metropolitan areas throughout the United States: to document the history of data available from these networks; and to indicate where additional information about the networks and data can be obtained.

The information obtained describing the networks could not have been gathered without the cooperation of the operating agencies involved. The specific sources of information are acknowledged at the beginning of each section describing a network or networks. The organizations that provided invaluable assistance and made this technical memorandum possible are the Agricultural Research Service; Bell Telephone Laboratories; Illinois State Water Survey; Tennessee Valley Authority; U.S. Geological Survey; U.S. Weather Bureau; University of Arizona; and University of California at Davis.

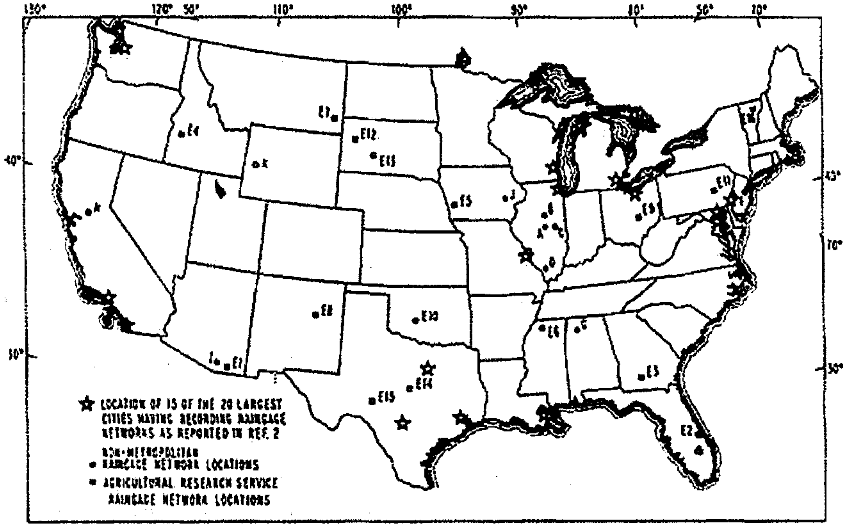
2.0 SUMMARY

This section summarizes information on 13 dense recording rain gauge networks located in non-metropolitan areas of the United States. Of the networks described, six are operated by the Illinois State Water Survey and one each by the Bell Telephone Laboratories, Tennessee Valley Authority, U.S. Geological Survey, U.S. Weather Bureau, University of Arizona, University of California at Davis, and the Agricultural Research Service (ARS). The 13 networks are located in eight states, see Figure 1. Data are reduced and recorded on punch cards or magnetic tape for most of the networks. Rainfall data for all networks are available, although arrangements for obtaining data will vary considerably with the source, and inquiries should be made to the organizations involved. Only six of 13 networks are still in operation.

Fifteen additional dense rain gauge networks operated by the ARS are not reported herein, but a companion report to this technical memorandum is being prepared by ARS personnel that will describe all 16 of the ARS rain gauge networks in detail. The locations of the 16 networks are shown in Figure 1. All 16 of the ARS networks are presently in operation.

Locations of the 15 of the 20 largest U.S. cities operating rain gauges as reported are also shown in Figure 1.

A summary of information on the 13 dense rain gauge networks is presented in Table 16-1. More detailed information on each of the networks in Table 16-1 is presented in succeeding sections of this technical memorandum.



NON-METROPOLITAN NETWORKS

A –	Goose Creek Network	Illinois State Water Survey	E9 –	Coshocton, Ohio	Agricultural Research Service
A –	East-Central Illinois Network	Illinois State Water Survey	E10 –	Chickasha, Oklahoma	Agricultural Research Service
A –	Central Illinois Network	Illinois State Water Survey	E11 –	Klingerstown, Pennsylvania	Agricultural Research Service
B –	Panther Creek Network	Illinois State Water Survey	E12 –	Newell, South Dakota	Agricultural Research Service
C –	Boneyard Creek Network	Illinois State Water Survey	E13 –	Cottonwood, South Dakota	Agricultural Research Service
D –	Shawnee Network	Illinois State Water Survey	E14 –	Riesel, Texas	Agricultural Research Service
E1 -	Tombstone, Arizona	Agricultural Research Service	E15 –	Sonora, Texas	Agricultural Research Service
E2 –	Vero Beach, Florida	Agricultural Research Service	E16 –	Danville, Vermont	Agricultural Research Service
E3 –	Tifton, Georgia	Agricultural Research Service	F -	Holmdel, New Jersey	Bell Telephone Laboratories
E4 -	Boise, Idaho	Agricultural Research Service	G -	Bear Creek, Alabama	Tennessee Valley Authority
E5 -	Treynor, Iowa	Agricultural Research Service	H -	Davis, California	University of California
E6 -	Oxford, Mississippi	Agricultural Research Service	I -	Tucson, Arizona	University of Arizona
E7 -	Ekalaka, Montana	Agricultural Research Service	J -	Iowa City, Iowa	U.S. Weather Bureau
E8 -	Santa Rosa, New Mexico	Agricultural Research Service	K -	Badger Wash Basins	U.S. Geological Survey

FIGURE 16-1. Location of Non-metropolitan Dense Rain Gage Networks, and 15 of the 20 Largest Cities Operating Rain Gauge Networks

TABLE 16-1

Summary Information on Rain Gauge Networks Described in this Technical Memorandum

Network Name	Network Location	Organization Operating Network	Equipment Number and Kind of recording rain gauges	Chart Speed (one revolution per indicated time period)	Approx. Size of Network Sq. Miles	Approx. Ave age Distance Between Gauges, Miles	Period of Record		Intensity Data Reduced from Charts	Form of Reduced Data	Remarks
							From	To			
Goose Creek	Central Illinois	Illinois State Water Survey	33- Weighing type with internal chart recorders	Mostly 24-hour	50	1 ⅓	1951	1951	Yes	Data tabulated at 1-minute intervals on IBM cards for 21 storms	
			50- Weighing type with internal chart recorders	6-hour	100	1 ½	1952	1953	Yes	Data tabulated at 1-minute intervals on IBM cards for 29 storms	
			50- Weighing type with internal chart recorders	Mostly 24-hour	100	1 ½	1954	1954	Yes	Data tabulated for total storm amounts only on original log sheets	
East Central Illinois	Central Illinois	Illinois State Water Supply	49- Weighing type with internal chart recorders	Mostly 24-hour	400	3	1955	1968	Yes	Data tabulated on IBM cards, mostly at 15-minute intervals	
Central Illinois	Central Illinois	Illinois State Water Survey	196- Weighing type with internal chart recorders	Mostly 24-hour	1600	3	1968	Present	Yes	Data tabulated on IBM cards, mostly at 15-minute intervals	

Network Name	Network Location	Organization Operating Network	Equipment Number and Kind of recording rain gauges	Chart Speed (one revolution per indicated time period)	Approx. Size of Network Sq. Miles	Approx. Ave age Distance Between Gauges, Miles	Period of Record		Intensity Data Reduced from Charts	Form of Reduced Data	Remarks
							From	To			
Panther Creek	Central Illinois	Illinois State Water Survey	25- Weighing type with internal chart recorders	Mostly Weekly	100	2	1950	1954	Yes	Data tabulated for total storm amounts only on original log sheets	
Boneyard Creek	Urbana Illinois	Illinois State Water Survey	11- Weighing type with internal chart recorders	Mostly Weekly	10	1	1949	1967	Yes	Data tabulated on IBM cards, mostly at 30-minute intervals	
Shawnee	Southern Illinois	Illinois State Water Survey	44- Weighing type with internal chart recorders	Mostly 24-hour	700	4	1964	Present	Yes	Data tabulated on IBM cards, mostly at 15-minute intervals	
Chickasha	South-western Oklahoma	Agricultural Research Service	228- Weighing type with internal chart recorders	90% 24-hour	1130	3	1961	Present	Yes	Data recorded on computer punch cards	The ARS operates 15 other dense recording rain gauge networks in the U.S. Chickasha network serves as an example. All 16 ARS networks will be described in detail in future ARS 41 series publication

Network Name	**Network Location**	**Organization Operating Network**	**Equipment Number and Kind of recording rain gauges**	**Chart Speed (one revolution per indicated time period)**	**Approx. Size of Network Sq. Miles**	**Approx. Ave age Distance Between Gauges, Miles**	**Period of Record**		**Intensity Data Reduced from Charts**	**Form of Reduced Data**	**Remarks**
							From	**To**			
Crawford Hill	Holmdel, New Jersey	Bell Telephone Laboratories	96- Continuous measuring device developed by Bell Labs. See remarks	See Remarks	50	0.8	1966	1968	See Remarks	Data recorded on magnetic tape at 10-second intervals	Special gauge developed by Bell Labs continuously measures water flowing down an inclined plane between capacitors data is transmitted to Crawford Hill Lab and recorded on magnetic tape
Upper Bear Creek	Northwestern Alabama	Tennessee Valley Authority	25- Weighing type with internal chart recorders	Weekly	143	2 ½	1962	Present	No		
Davis	Davis California	University of California	29- Weighing type with internal digital tape recorders	24-hour	68	1 ½	1966	1969	See Remarks	Data transferred from punched tapes to computer punch cards	Data at gauge punched on tape at 5-minute intervals in form of accumulated rainfall to nearest 1/10 inch
Atterbury	10 miles SE of Tucson	University of Arizona	9- Weighing type with internal chart recorders	5-12-hour 4-24-hour	18	1 ½		Present	See Remarks	See Remarks	Rainfall data from all storm events are in process of being transferred from charts to computer punch cards

Network Name	**Network Location**	**Organization Operating Network**	**Equipment Number and Kind of recording rain gauges**	**Chart Speed (one revolution per indicated time period)**	**Approx. Size of Network Sq. Miles**	**Approx. Ave age Distance Between Gauges, Miles**	**Period of Record**		**Intensity Data Reduced from Charts**	**Form of Reduced Data**	**Remarks**
							From	**To**			
Iowa	Iowa City	U.S. Weather Bureau	9- Weighing type with internal chart recorders		63	3	1941	1962	Yes	Hourly rainfall data available on computer cards from USWB in Ashville	
Badger Wash	West. Colorado	U.S. Geological Survey	11- Weighing type with internal chart records, tipping buckets, digital	192-hours	3	<1	1954	Present	No	Daily rainfall data available in USGS Water Supply Paper No. 1532-B (1954-58), WSP 1532-D (in press, 1959-1966)	Recorders operated during summer months April-October

Chapter 17

AVAILABILITY OF RAINFALL-RUNOFF DATA FOR PARTLY SEWERED URBAN DRAINAGE CATCHMENTS
(NOTE: ONLY INTRODUCTION AND SUMMARY ARE PROVIDED. SEE CHAPTER 13 FOR FULLY SEWERED CATCHMENTS)

March 1970
ASCE Urban Water Resources Research Program
Technical Memorandum No. 13
New York, New York

1.0 INTRODUCTION

The purpose of this technical memorandum is to identify available rainfall-runoff data for partially sewered urban drainage catchments to facilitate model development by researchers. Considerable detailed attention has been given to requirements for mathematical modeling of rainfall-runoff-quality in the first report by ASCE for OWRR.[1] The very limited amount of available rainfall-runoff data for fully sewered catchments has been identified in an earlier technical memorandum.[2] For the purposes of this technical memorandum, a "partially" sewered drainage catchment is defined as one containing open channel watercourses together with developed secondary drainage systems consisting of street gutter drainage and/or roadside ditches and underground storm sewers.

An ASCE report for the Geological Survey[3] focused upon improvement in design and management of urban storm drainage. The report emphasized the need for research on urban storm drainage hydrology, stating that the central goal of such research should be maximum transferability of results. Maximum transferability can be achieved through simulation of the rainfall-runoff-quality process via mathematical models. Unfortunately, a reservoir of data readily adaptable to the analysis of rainfall-runoff-quality relationships does not exist. As an interim measure, available rainfall-runoff data can be used in initial model formulations, such as with the Stanford Watershed Model.[4]

The need for obtaining comprehensive rainfall-runoff data that would cover a range of hydrologic conditions and development patterns was emphasized in a Progress Report by the ASCE Task Force on "Effect of Urban Development on Flood Discharges" published in January 1969.[5] Further, the Progress Report pointed out a

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[1] Methods of Analysis Task Committee. September 1968. "Considerations for Modeling Urban Rainfall-Runoff-Quality Processes."

[2] Tucker, L.S. March 3, 1969. "Availability of Rainfall-Runoff Data for Sewered Drainage Catchments." ASCE Urban Water Resources Research Program, Technical Memorandum No. 8, New York, New York.

[3] ASCE Urban Water Resources Research Program. April 1969. *Basic Information Needs in Urban Hydrology*. A study for the Geological Survey, U.S. Department of the Interior, New York, New York.

[4] Crawford, N.H. and R.K. Linsley. July 1966. "Digital Simulation in Hydrology: Stanford Watershed Model IV." Department of Engineering, Stanford University, Technical Report No. 39.

[5] Task Force on Effect of Urban Development on Flood Discharges, Committee on Flood Control. January 1969. "Effect of Urban Development on Flood Discharges—Current Knowledge and Future Needs." Progress Report. ASCE *Journal Hydrological Division*, Volume 95, No. HY1.

need to develop hydrologic models for the analysis of changes due to urban development, and the need to examine theoretical approaches and to seek fundamental understanding of the physical processes that affect rainfall and runoff. Eagleson[6] has presented sound theoretical foundations for urban hydrology research in a recently published text.

The interaction and interdependence of sewered catchments and receiving open channel drainage is discussed in ASCE Program Technical Memorandum No. 12,[7] together with some of the environmental and technical factors that influenced the adoption and design of open channels for drainage in Milwaukee.

The USGS has catalogued sources of flow data and certain water quality data from watersheds in metropolitan areas,[8] but precipitation gauges exist on only a very few of the gauged watersheds cited. The drainage catchments described herein are limited to those partially sewered catchments where both rainfall and runoff data are collected.

It has been stated that the emphasis in initial research should be process mastery for developed urban drainage catchments. Therefore, the catchments described herein are all highly developed urban catchments. Impervious cover is used as a measure of the degree of development, and a highly developed catchment has been arbitrarily defined as one whose total imperviousness constitutes 20% or more of the total catchment area. The term impervious cover as used herein includes all roofs, driveways, parking lots, sidewalks and streets in a drainage catchment. In many instances, the degree of imperviousness is only very roughly known.

A summary of information on partially sewered catchments for which rainfall-runoff data has been collected is given in Section 2.0.

The development of the information contained in this memorandum was made possible through the generous cooperation of the agencies and organizations involved in data acquisition and analysis. Most of the rainfall-runoff gauging projects described herein are operated by the U.S. Geological Survey. Invaluable assistance was received from USGS offices in: Austin, Ft. Worth and Houston, Texas; Denver, Colorado; Fairfax, Virginia; Lansing, Michigan; Mineola, New York; and Raleigh, North Carolina. Equally important assistance was provided by Duke University, Illinois State Water Survey, Public Works Research Institute of the Japanese Government, Purdue University, and Utah State University.

All developed urban drainage catchments for which rainfall-runoff data have been, or are being collected that were brought to the writer's attention within the timeframe available for writing this report are described herein. Oversights are entirely possible,

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[6] Eagleson, P.S. 1970. *Dynamic Hydrology*. McGraw-Hill Book Company, New York, New York, 462 pp.

[7] Prawdzik, T.B. February 1970. "Environmental and Technical Factors for Open Drainage Channels in Milwaukee." ASCE Urban Water Resources Research Program Technical Memorandum No. 12, New York, New York.

[8] Schneider, W.J. 1968. *Water Data for Metropolitan Areas.* Water Supply Paper 1871, Geological Survey, U.S. Department of the Interior, Washington, DC.

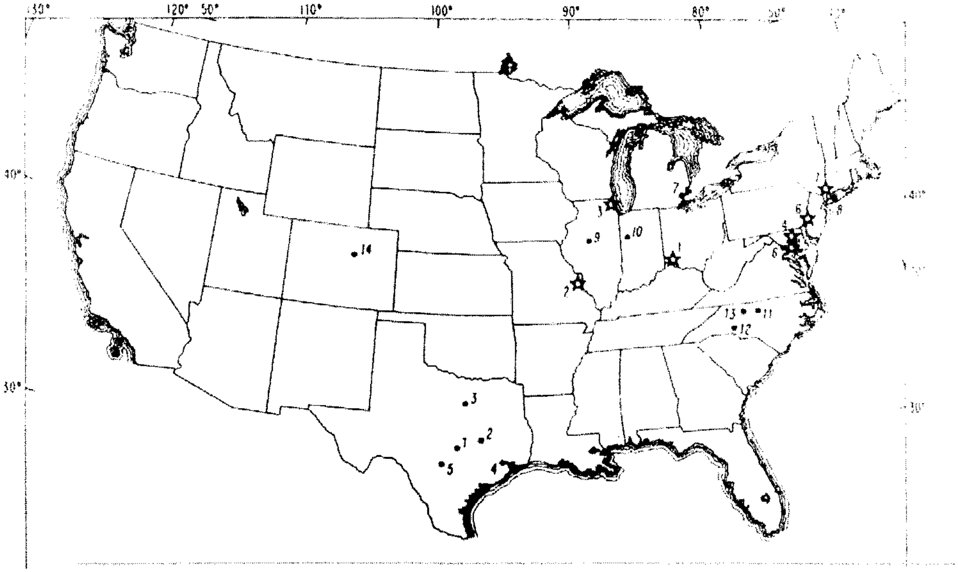
and the writer assumes full responsibility for any omissions, however unintentional and inadvertent.

2.0 SUMMARY

This section summarizes information on the availability of rainfall-runoff data from 64 developed partially sewered urban drainage catchments in the U.S. and eight in Japan. The 64 catchments in the U.S. are concentrated in eight states, and the location of these instrumented catchments are noted in Figure 17-1. Rainfall-runoff data for most of the catchments are available, but in a variety of forms. Arrangements for obtaining data should be made with the local organizations involved, identified at the end of each section.

The location of cities with instrumented, completely sewered, urban catchments previously reported are shown on Figure 17-1.

A summary of information on the 64 instrumented, highly developed, partially sewered urban drainage catchments in the U.S. is presented in Table 17-.1.



Location of Cities with Partially Sewered Drainage Catchments Described in this Technical Memorandum

1 - Austin, Texas
2 - Bryan, Texas
3 - Dallas/Fort Worth, Texas
4 - Houston, Texas
5 - San Antonio, Texas
6 - Arlington/Fairfax/Falls Church, Virginia
7 - Detroit, Michigan
8 - Westbury/Syosset/Hempstead, Long Island
9 - Champaign/Urbana, Urbana
10 - West Lafayette, Indiana
11 - Durham, North Carolina
12 - Charlotte, North Carolina
13 - Winston-Salem, North Carolina
14 - Denver, Colorado

Locations of Cities with Completely Sewered Drainage Catchments

1 - Cincinnati, Ohio
2 - St. Louis, Missouri
3 - Chicago, Illinois
5 - Washington, DC
6 - Philadelphia, Pennsylvania
7 - New York, New York

FIGURE 17-1. Locations of 64 Instrumented U.S. Urban Drainage Catchments Summarized in Table 17-1.

TABLE 17-1

Summary Information on Developed (Impervious Cover 20% or greater) Urban Drainage Catchments Described in this Technical Memorandum

Catchment	Location	Size, Sq. Mi. (Acres)	Total Impervious Cover, %	Operating Organization
Ross-Ade (upper)	West Lafayette, Indiana	(29)	38	Purdue University
Ross-Ade (lower)	West Lafayette, Indiana	(392)	37	Purdue University
Purdue Swine Farm (upper)	West Lafayette, Indiana	(178)	21	Purdue University
Westbury	Westbury, Long Island, NY	(15.0)	32	USGS, Mineola, New York
Syosset	Syosset, Long Island, NY	(28.8)	35	USGS, Mineola, New York
East Meadow Brook	Hemstead, Long Island, NY	10	--	USGS, Mineola, New York
Boneyard Creek	Champaign-Urbana, Illinois	4.70	44	USGS, Urbana Illinois St. Water Survey
Waller Creek (upper)	Austin, Texas	2.31	29	USGS, Austin, Texas
Waller Creek (lower)	Austin, Texas	4.13	21	USGS, Austin, Texas
Burton Creek	Bryan, Texas	1.33	26	USGS, Austin, Texas
Turtle Creek	Dallas, Texas	7.98	--	USGS, Ft. Worth, Texas
Bachman Branch	Dallas, Texas	10.0	--	USGS, Ft. Worth, Texas
Cottonwood Creek above Forest Lane	Dallas, Texas	8.5	--	USGS, Ft. Worth, Texas
Dry Branch at Fain Street	Ft. Worth, Texas	2.15	--	USGS, Ft. Worth, Texas
Cottonwood Creek Tributary at Seminary So. Shopping Center (at Mkt Railroad)	Ft. Worth, Texas	0.97	--	USGS, Ft. Worth, Texas
Sycamore Creek at Interstate Hwy. 35-W	Ft. Worth, Texas	1.35	--	USGS, Ft. Worth, Texas
Joes Creek	Dallas, Texas	7.51	--	USGS, Ft. Worth, Texas

Catchment	**Location**	**Size, Sq. Mi. (Acres)**	**Total Impervious Cover, %**	**Operating Organization**
Floyd Branch at Forest Lane	Dallas, Texas	4.17	--	USGS, Ft. Worth, Texas
Ash Creek at Highland Ave.	Dallas, Texas	6.92	--	USGS, Ft. Worth, Texas
Coombs Creek at Sylvan Ave.	Dallas, Texas	4.75	--	USGS, Ft. Worth, Texas
Cedar Creek at Bonnie View Rd.	Dallas, Texas	9.42	--	USGS, Ft. Worth, Texas
Forney Creek	Dallas, Texas	1.84	--	USGS, Ft. Worth, Texas
Bering Ditch at Woodway Dr.	Houston, Texas	2.96	27	USGS, Houston, Texas
Hunting Bayou Tributary at Cavalcade Street	Houston, Texas	1.03	29	USGS, Houston, Texas
Hunting Bayou at Falls Street	Houston, Texas	3.42	21	USGS, Houston, Texas
Stonybrook Ditch	Houston, Texas	0.50	36	USGS, Houston, Texas
Bintliff Ditch at Bissonnet St.	Houston, Texas	4.29	26	USGS, Houston, Texas
Brays Bayou	Houston, Texas	88.4	40	USGS, Houston, Texas
White Oak Bayou	Houston, Texas	92.0	35	USGS, Houston, Texas
Sims Bayou	Houston, Texas	64.0	30	USGS, Houston, Texas
Halls Bayou	Houston, Texas	24.7	30	USGS, Houston, Texas
Greens Bayou	Houston, Texas	72.7	25	USGS, Houston, Texas
Leon Creek Tributary	San Antonio, Texas	1.19	--	USGS, San Antonio, Texas
Salado Creek Tributary at Bee Street	San Antonio, Texas	0.45	--	USGS, San Antonio, Texas
Alazan Creek at St. Cloud St.	San Antonio, Texas	3.26	--	USGS, San Antonio, Texas
Salado Creek at Bitters Road	San Antonio, Texas	0.62	--	USGS, San Antonio, Texas
Olmos Creek at Farm	San Antonio, Texas	0.33	--	USGS, San Antonio,

Catchment	Location	Size, Sq. Mi. (Acres)	Total Impervious Cover, %	Operating Organization
Road 1535				Texas
Little Pimmit Run Tributary at Arlington	Arlington County, Virginia	0.41	28	USGS, Fairfax, Virginia
Little Pimmit Run at Arlington	Fairfax County, Virginia	2.31	20	USGS, Fairfax, Virginia
Long Branch	Fairfax County, Virginia	0.94	30	USGS, Fairfax, Virginia
Piney Branch	Vienna, Virginia	0.29	30	USGS, Fairfax, Virginia
Tripps Run	Falls Church, Virginia	1.78	25	USGS, Fairfax, Virginia
Tripps Run near Falls Church	Fairfax County, Virginia	4.55	25	USGS, Fairfax, Virginia
Tripps Run Tributary near Falls Church	Fairfax Count, Virginia	0.50	25	USGS, Fairfax, Virginia
Four Mile Run	Alexandria, Virginia	14.4	20	USGS, Fairfax, Virginia
Penn Daw Outfall	Alexandria, Virginia	0.82	20	USGS, Fairfax, Virginia
Red Run	Near Detroit, Michigan	36.5	25	USGS, Lansing, Michigan
Third Fork Creek	Durham, North Carolina	1.68	--	Duke University & USGS Raleigh, North Carolina
Briar Creek Tributary 7	Charlotte, North Carolina	(333)	20	USGS, Raleigh, North Carolina
Little Sugar Creek Tributary at Brookcrest Drive	Charlotte, North Carolina	(538)	21	USGS, Raleigh, North Carolina
Brushy Creek Tributary at Attuck Street	Charlotte, North Carolina	11.9	--	USGS, Raleigh, North Carolina
Brushy Creek Tributary 2	Charlotte, North Carolina	(352)	37	USGS, Raleigh, North Carolina
Tar Branch	Charlotte, North Carolina	(378)	28	USGS, Raleigh, North Carolina
Burke Branch	Charlotte, North Carolina	(794)	20	USGS, Raleigh, North Carolina
Dye Creek	Durham, North Carolina	(518)	32	USGS, Raleigh, North

Catchment	Location	Size, Sq. Mi. (Acres)	Total Impervious Cover, %	Operating Organization
				Carolina
Goose Creek	Durham, North Carolina	(947)	25	USGS, Raleigh, North Carolina
Third Fork Creek Tributary	Durham, North Carolina	(333)	20	USGS, Raleigh, North Carolina
Rocky Creek Tributary	Durham, North Carolina	(288)	26	USGS, Raleigh, North Carolina
Dry Creek Tributary	Littleton, Colorado	0.98	--	USGS, Denver, Colorado
Sanderson Gulch Tributary	Lakewood, Colorado	--	--	USGS, Denver, Colorado
Concourse D Storm Drain	Stapleton Airport, Denver, Colorado	0.19	100	USGS, Denver, Colorado
Tuck Drain	Northglenn, Colorado	0.67	33	USGS, Denver, Colorado
Hill Crest Drain	Northglenn, Colorado	0.285	29	USGS, Denver, Colorado
Kennedy Drive Drain	Northglenn, Colorado	1.05	49	USGS, Denver, Colorado

Chapter 18

SYSTEMS ANALYSIS FOR URBAN WATER MANAGEMENT

September 1970
Prepared For ASCE Urban Water Resources Research Program
Prepared By Water Resources Engineers, Inc.
Walnut Creek, California

1.0 INTRODUCTION

1.1 Background

The American Society of Civil Engineers (ASCE), through its Urban Hydrology Research Council, is currently performing a study to develop a long-range program of urban water resources research. A current contract between ASCE and the Office of Water Resources Research (OWRR), U.S. Department of the Interior, continues a program initiated in 1967 between ASCE and OWRR. The first phase of the investigation had included a charge to:

> *Conduct a pre-feasibility study to determine possible effectiveness, cost, and time requirements for a comprehensive systems engineering analysis of all aspects of urban water.*[1]

Water Resources Engineers, Inc. (WRE), of Walnut Creek, California, was privileged to have been one of two organizations selected to perform the above sub-task of the 1967-1968 study as an ASCE sub-contractor.[2] Another sub-task of the original program was a similar pre-feasibility study conducted by two other organizations to determine the possibility of a related general analysis of the *economic* aspects of urban water.

Under the 1967-1968 sub-contract, WRE determined that a systems engineering analysis of all the metropolitan water sub-systems was indeed feasible. The study identified a 5-year, $15.9M program of applied research necessary to bring systems engineering analysis to bear on all the sub-systems simultaneously, at a high level of spatial and temporal detail, and at the highest possible level of analytical sophistication. WRE did not speculate at that time on what additional manpower and budget would be necessary to complete the corollary economic systems analysis.

Cognizant of the crucial need to maintain an impetus in the urban water analysis area, ASCE was permitted to include a modest task of initiation of overall systems simulation in its current, second phase contract with OWRR,[3] budgeting a maximum of $30,000 to undertake, through sub-contract, a very preliminary attempt at both

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[1] Contract No. 14-01-0001-1585 between ASCE and OWRR.

[2] Water Resources Engineers, Inc. September 1968. "Comprehensive System Engineering Analysis of All Aspects of Urban Water, A Pre-feasibility Study." Appendix H in *Urban Water Resources Research.* First Year Report to OWRR, ASCE Urban Hydrology Research Council. Clearinghouse for Federal Scientific and Technical Information No. PD 184 318, W69-03506.

[3] Contract No. 14-01-0001-1992 between ASCE and OWRR.

engineering simulation and economic systems analysis. It has been WRE's privilege to serve as ASCE's sole subcontractor for this most challenging and important task, and to serve OWRR, the funding agency, in providing a demonstration of the systems concept as a viable guide for planning, implementation, and operation of metropolitan water systems and for fundamental research as well.

1.2 Systems Analysis—Goals for Urban Water Management

Urban water managers probably view their roles variously with respect to responsibilities and priorities for their daily tasks, but most would likely agree that providing a water supply, protecting property from excessive storm runoff, or removing wastes are central goals. Nonetheless, they might also agree that the technical achievement of these objectives is not their most demanding duty. The more compelling problems, the ones that require most of their time and resourcefulness, are the provision of these services: a) economically; b) reliably; c) in accordance with the socially acceptable customs of water use in the region; and d) within the political and legal constraints limiting the use of sources of supply or receiving waterbodies. An essential attribute of a successful urban water manager, then, is a comprehensive appreciation of the many ramifications of the physical works alternatives he/she might wish to implement.

Systems analysis, which is a body of formalized evaluation techniques, provides the capability of building frameworks or models that will allow the urban water manager to view in advance the consequences of his decisions. From his own appreciation of the acceptability of these predicted consequences, he can judge the feasibility of various technical alternatives.

We know that various applications of systems analysis methods have been very helpful to urban water managers in recent years for planning and operating specific physical sub-systems. Notable among these tools have been digital computer models for balancing heads and flows in distribution system networks and others for the determination of the least-costly waste treatment process to satisfy constraining water quality standards in receiving waters.

But beyond the technical rigor that systems analysis has provided, it also has the yet untapped capability of allowing us to anticipate consequences comprehensively. It should be possible to construct systems analysis models that will allow evaluation of the consequences of many urban water sub-systems operating together, and it should be possible to operate social, demographic, political, and economic models along with the technical evaluation models to evaluate as quantitatively as possible the many "non-technical" ramifications of the physical alternatives. These steps into systems comprehensiveness have yet to be taken. It is our contention that man's level of understanding has advanced to the point that virtually all the required comprehensive models of technical and non-technical aspects could be constructed through amalgamation of existing knowledge. *Only the amalgamating remains to be done* to realize the tremendous capability of viewing almost countless alternatives of water supply, water treatment, water distribution, waste collection, waste treatment, waste disposal, and the various possible uses and reuses of water and the effects of all these on the environment.

It would seem, then, that the goals of urban water management with respect to the use and further development of systems analysis tools such as those initially explored in this report would be:

1) To improve and expand planning capability through provision of a comprehensive water-planning model that simulates long-term behavior of all urban water sub-systems and their alternates operated to meet demands in future periods.
2) To improve implementation and operational control through provision of a much more detailed model that would simulate entire system behavior and individual sub-system behavior at the sub-systems normal level of time and space operation.
3) To assist decision-making by supporting development of demographic, economic, and legal-political models that would aid in evaluation of the "non-water" consequences of physical works alternatives.

The major purpose of the study reported here was to indicate that these goals are attainable. Additional benefits of attaining them include the economies of scale in analysis time and manpower that systems analysis tools provide, the high degree of transferability of these models from application in one community or borough to application in others, and the framework that these models provide to which future knowledge can be added as researchers discover new truths about our water environment and new techniques for representing these truths.

1.3 Project Objectives

This study was performed by engineers, directly for the ASCE, and indirectly for urban water managers who tend to be engineering oriented. Consequently, it was primarily a technical enterprise. The major objective was to demonstrate that a comprehensive model could be constructed in general-enough format to allow its application to many urban water systems, while at the same time the urban water sub-system sub-models comprising the larger model could be operated independently to describe behavior of specific urban sub-systems. Additionally, we wished to demonstrate that economic systems analysis could be conducted compatibly with the engineering systems analysis by constructing an economic evaluation model that used technical output of the engineering model as basic input information.

To attain these demonstration objectives the study was divided into the following four tasks:

1) Formulate a comprehensive, but coarse-element, mathematical model of a hypothetical Urban Water Resources System, structured essentially as shown

in Figure 18-1, to indicate the interrelationships among inflows, storage volumes, outflows, and qualities in the various sub-systems.

2) Demonstrate the feasibility of producing a meaningful mathematical model of an Urban Water Resources System by operating the model formulated in Task 1 above for a set of hypothetical urban water sub-systems.
3) Describe a more detailed mathematical model of the Storm Sewer sub-system as indicated in Figure 18-1 and demonstrate the facility for such a model for: a) accepting water inputs from preceding sub-systems (i.e., Urban Hydrology or Rainfall); b) changing quantity and quality within the sub-system; and c) producing outputs of quantity and quality to be accepted as inputs by succeeding sub-systems (Receiving Water Environment).
4) Formulate and demonstrate an Economic Systems model that will permit the evaluation of physical works projects, such as alternative storm sewer sizes and routes, intended to meet urban water objectives that may be subject to both technical and budgetary constraints.

1.4 Summary of Results

The following chapters and the appendix describe the results of this very small first step toward systems analysis of "all aspects of urban water." Section 2 discusses the concept of systems analysis as a tool for urban water management. In a fairly non-technical way, Section 3 describes the comprehensive simulation-model developed during the study for the specific hypothetical system shown in Figure 18-2. For greater detail in modeling, Section 4 describes a much more expensive but more refined stormwater-modeling project being performed for the Federal Water Quality Administration. Section 5 describes the economics modeling undertaken for demonstration purposes during the study. Section 6 contains the results of the hypothetical demonstrations of both the technical and the economic models. Section 7 presents our major conclusions and recommendations. In condensed form these include:

1) Long-range planning is likely to be benefited most by comprehensive modeling of real metropolitan water systems.
2) Economics modeling for the entire urban water resources system will be most helpful in allocating costs of sub-systems to their user's according to the user's relative amount of benefits. The ability to quantify the "intangible" effects of various physical works alternatives, even in a relative way, appears to hold promise for assessing environmental impacts and the values gained or lost by such projects.
3) Continuation of technical engineering modeling should be directed to application of simultaneous equation solution models or network theory to sub-systems in a real metropolitan area.
4) Economic modeling research should be directed to derivation of explicit relationships between water user requirements for both quantity and quality,

and the benefits or losses that accrue to these users as a result of satisfying or failing to satisfy their demands.

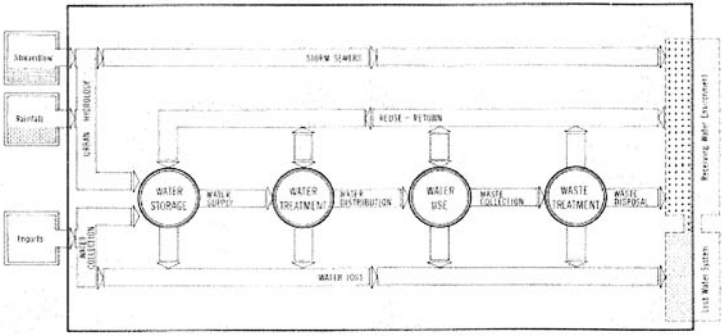

FIGURE 18-1. The Urban Water Resources System

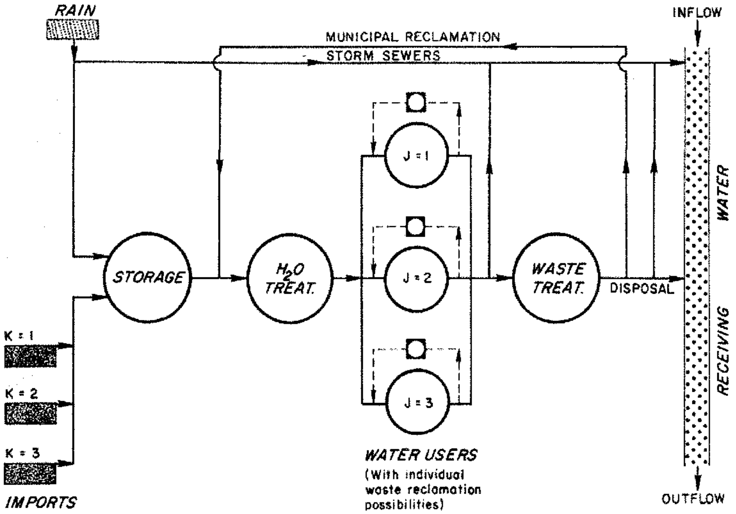

FIGURE 18-2. Urban Water System for Demonstration Case

1.5 Organization of the Study

The work reported here was conducted by WRE personnel in the firm's main office in Walnut Creek, California. Dr. Michael B. Sonnen was responsible for day-to-day direction of the study, formulation of the technical and economics models, and reporting of results to ASCE. Dr. Larry A. Roesner performed the review of the detailed storm sewer system-modeling project with Federal Water Quality Administration (FWQA). Mr. William R. Norton provided computer programming assistance and substantial engineering background to assure that programs were solving properly stated problems in correct ways.

1.6 Acknowledgments

WRE would like to thank the Urban Hydrology Research Council of ASCE for support of this sub-contract. Particularly, we would like to acknowledge the continued and active interest in this project's output by Murray B. McPherson, Director of the ASCE Urban Water Resources Research Program.

2.0 SYSTEMS ANALYSIS AS A METROPOLITAN WATER MANAGEMENT TOOL

Modern techniques of systems analysis, either for management and coordination of huge programs of interdependent endeavors or for technical evaluation of thousands or millions of yet un-built physical works alternatives, offer no panacea for solving problems. They do not assure that correct answers, much less optimal answers, will be found. They can be used foolishly, incorrectly, or in procedures whose basic validity is open to interpretation and about which reasonable men can disagree. Even so, systems analysis may offer the only hope for sensible solutions to many problems of the metropolis. Man has dreamed of going to the moon and has basically known how to get there for centuries, but he lacked the necessary analytical arsenal until the past decade.

We have some knowledge and even marvelous dreams about flow in pipes, microbiology in waste treatment plants, possible solid and liquid waste productions associated with various changes and rates of changes in population, and the values that society loses through pollution or gains through water treatment; but if we cannot come to analyze or optimize these bits of knowledge or insight in a coordinated fashion, which means through systems analysis, they may not be resolved properly; or where they are tackled piecemeal, the overall solution is likely to be very inefficient.

It is heartening, therefore, to see OWRR and ASCE make a determined effort to bring systems analysis to bear on metropolitan water problems. There are four reasons why water management will be well served through the results of projected systems research:

1) Metropolitan water management alternatives that were far too numerous and too complex to analyze previously will become candidates for tractable evaluation.
2) Problems ranging in time and space detail from the most minute dimensions of operation and maintenance to the broadest level of planning detail will be resolvable through the same or related tools.
3) The tools produced through systems analysis research can be transferred with a modicum of adjustment for application to problems in other regions, other cities, other plants, or other pipes than those for which they were originally developed.
4) Because the first three reasons are valid; the concepts of systems analysis offer tremendous economies of scale for analysis of the nation's or the world's metropolitan areas. The first demonstration of comprehensive systems engineering and economic analysis may be fairly expensive; but it will be miniscule in comparison to the value of the facilities the analysis will help plan, particularly in comparison to the value of facilities in the many cities that will ultimately be served by the same analytic capability.

2.1 Comprehensive Treatment of Alternatives

It is not too difficult to formulate a sewage treatment process that removes 85% of the BOD and 90% of the suspended solids from a given waste. It is not too hard to conceptualize a storage reservoir whose output equals its input less its change in storage. It does become a little trying, however, to select from among numerous possible water treatment alternatives the one whose efficiencies for removal of many constituents will or will not satisfy the quality demands of various classes of urban water users whose wastes may be recycled in part and may be discharged in part to a receiving water that is used as one of the community's water supply sources, as well as by an entirely different class of water users whose quantity and quality demands are likely very different again. It can also be more than just slightly perplexing to discover that the optimal degree of community waste reclamation will depend on the amount of supply to the individual users which in turn depends on the amount of firm supply available to the community, with the latter provided partly on the basis of the community's ability to reclaim its wastes.

Succinctly, systems analysis techniques allow for both theoretical soundness and incorporation of a large range of alternatives in an evaluation virtually without conflict from numbers of interdependencies or algebraic complexity. It has become true that engineers, like pure scientists, are constrained in their vistas of alternative solutions by reasonableness alone. Systems analysis techniques are available to deal with many, though not all, of the interdependent, looped, "non-linear" problems imaginable.

2.2 Variability of Time and Space

Earlier, WRE described the time and space problems associated with development of comprehensive metropolitan water models. The essence of the problem is illustrated in Figure 18-3. There is usually a reasonable amount of time for the resolution of planning problems related to requirements expected years from now. However, the water manager would undoubtedly prefer to begin immediately analyzing the most detailed problems associated with operation and maintenance of existing water resources systems. Unfortunately, it is necessary and normal in modeling and in other endeavors to begin with the grossest representations and to refine and hone these early model versions into more elaborate, sophisticated, and precise descriptions. Hence, in the current project and in the field of systems engineering analysis of water problems generally, we have been forced to begin with fairly gross analyses. The most detailed analyses to date have been performed on single subsystems such as water distribution networks or receiving waters. While such models are able to deal with the details of small time intervals (or steady-state analyses of detailed spatial networks), the comprehensive model of an entire urban system presented here has been tested only with monthly flows and quality levels, and with a limited number of representative process elements.

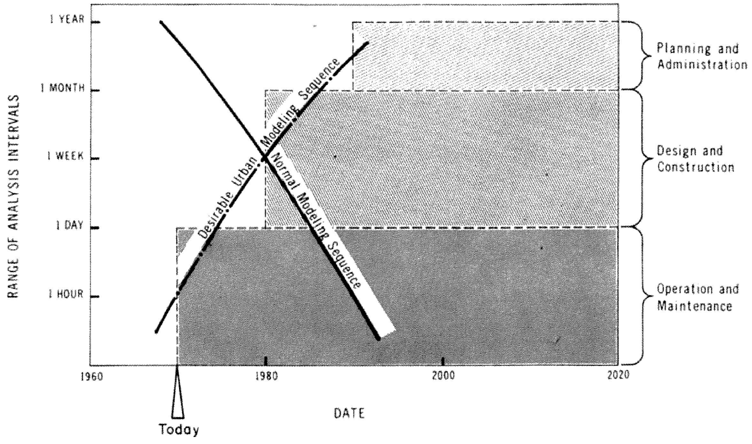

FIGURE 18-3. Stages of development and levels of detail to be encountered in systems analysis of the urban water resources system.

An objective of this study, however, was to demonstrate that sub-system models that permit fairly detailed time and space descriptions could be linked to simulate short-term behavior of coupled sub-systems. Indeed, the FWQA stormwater project, described in more detail later, has been successful in integrating an urban runoff model, a sewer transport model, and a receiving water model that matches prototype data within 15% of observed data and predicts peak flows within five minutes of the observed fact. These models operate on roughly 15 second to five minute analysis intervals. The ability to link detailed models is thereby demonstrated. It is pertinent to point out, however, that this (these) model(s) cost about $.5M to develop, operate(s) on three sub-systems linked only superficially to the mainline treatment-use-retreatment set of urban water sub-systems, and deals with economic factors not at all. Again, however, the concept of very detailed simulation of complete urban water resources systems is shown to be valid; but time and further research and development funding remain as requirements to bring the considerable power of this concept to full fruition.

2.3 Transferability of Concept

It remains to be demonstrated conclusively that metropolitan water models can be transferred from application in one region to applications in others. It can be intuited *a priori,* however, that models that account for water demands, deliveries, and allocations in the State of Texas, for example, should be applicable with minor modifications to problems in the State of Kansas or the Boroughs of New York.

Certainly, it is common knowledge that sub-system-level models such as distribution network analysis programs, bay and estuary models, and reservoir temperature models can be applied in one prototype situation after another with only changes in data to describe the local physical situation and coefficients to specify differences in curve forms.

The largest determinant of transferability is obviously the generality of the solution technique employed. The technical evaluation model developed during this project follows a step-wise, one-equation-at-a-time solution path. It leads to an answer at the end of 92 solved equations. Hence, although the answers for all 92 equations are kept, nonetheless, the model is structured so that a very particular problem is solved, and it is *not* structured for easy manipulation to solve another problem even for the same metropolis. Indeed, it may be easier to use this model to solve the same problem in a second metropolis than to solve a different problem in the same one. This need not be the case for all models. There are more sophisticated simultaneous equation solution techniques that would allow more simple transference of the unknowns in one sub-system to be known quantities in the next application used to solve for answers in a different sub-system. However, the type of solution adopted for this project insured a tight accounting for all variables with a minimum of programming complexity. It was used as an expedient to demonstrate the concept within the limited total time available. A more sophisticated, if preferable, solution might have required much more time to perfect.

2.4 Economies of Analysis Scale

Comparisons of cost for studies that use systems techniques in one case and not in another tend to be a bit unfair to the systems application. Quite often it will cost more to perform a water quality management study, let's say, with mathematical models of the physical alternatives coupled with formal optimization models to aid in the selection from among the many alternatives than if no systems tools are employed. The point that is often missed in this comparison is that much more "study" gets done when the systems tools are used. We avoid assiduously saying that systems analysis allows us to do things we could never do before, but emphasize that it does allow us to do things we never *would* do otherwise. Solution of several hundred simultaneous equations, for example, was never worth the man-years of manual effort required to know the state of the system at several hundred points, especially if all that would result would be a solution for January when we wanted the solution for every month for fifty years. Consequently, we skeletonize pipe networks, pick single "design" storms, evaluate three or four treatment processes; or otherwise pre-select or limit the number of alternatives and the complexity of the evaluation procedure to be employed.

It is very possible, then, that a metropolitan water analysis will cost more if systems techniques are employed than if they are not. The adage about paying a little more but getting so much more, though, is certainly applicable. The Federal Water Quality Administration's Stormwater Management Project, for example, in which WRE participated, required about $.5M for development and verification of a generalized simulation model that would convert rainfall and sanitary waste loads into combined sewer outflow hydrographs and receiving water responses. This investment included verification for urban watersheds in four different U.S. cities (for which data were provided by FWQA). WRE is now assisting the City of San Francisco, California with an analysis of the combined sewers in the entire City of San Francisco. Following the San Francisco Project, the modified model could be used to simulate systems behavior in other metropolitan areas for a fraction of the original cost.

Other experience with models for urban water sub-systems, such distribution pipe networks, further substantiates that applications to additional sub-systems can be miniscule in cost compared to the original development and first application costs. The whole point of developing these tools is to save time, money, and manpower in later applications.

3.0 INITIATION OF COMPREHENSIVE TECHNICAL MODELING

3.1 Purposes Of A Metropolitan Water Model

It surely must be the dream of every urban water manager to know what his entire system is doing at a given moment and to know at the same time what to do next when some condition changes. His hopes for improved certainty must vacillate between his needs for year-to-year prognostications of future demands and his moment-to-moment monitoring of flows, qualities, efficiencies, and manpower. For example, as soon as an engineer *plans* a new water treatment plant, he starts thinking about how to *design* the plant's hydraulic and quality components to best meet anticipated demands. Virtually simultaneously, he contemplates how the plant will *operate,* how it will remove and hold each contaminant, where its wastes might go, and what effects they may have on their destined disposal site. So the target of technical capability at which engineering researchers, consultants, and managers keep aiming is an elusive, expandable and contractible amalgam of time-and-space-variant principles, and the data to corroborate them.

Moreover, the amalgam should be expandable in a jurisdictional or functional sense as well. For example, residential, industrial, and public uses of water in a single community must compete for water service in much the same way that whole communities in a large metropolitan region compete with each other. So a comprehensive urban water model could aid not only in the technological evaluation of physical works alternatives, but also in the investigation of social, political and economic alternatives for locating, constructing, operating, financing, and sharing these facilities.

In summary, an urban water model could have the following purposes:

1) Permit evaluation of the needs for, and design and operation of, alternative physical works facilities associated with any and all aspects of urban water resources.
2) Support from a technical standpoint the social, political, and economic evaluation of alternative facilities or programs of facility construction, operation, or maintenance.
3) Provide a framework to which engineering and related researchers could add new knowledge, new data, and new interrelationships, with this framework serving as the focal point for their individual efforts in what otherwise might remain diverse and unrelated fields of research.
4) Allow comprehensive but systematic sensitivity testing of the response of each and every urban water sub-system to failures, interruptions, improvements, or simply changes in any other sub-system.

The overall goal of this model, then, would be to improve our insight to the workings and optimization of a complexly connected set of water-related parts. It is not too much to expect that the amalgamated set of sub-models called the Urban Water Model might one day form a sub-model itself in a much larger Urban Model comprised of other sub-models to simulate and evaluate such sub-systems as health, housing, transportation, education, communication, public safety, welfare and others. The present project, however, could consider only the very first steps of building the Urban Water Model.

3.2 Formulation of a Coarse Element Model

3.2.1 Variety of "Typical" Systems

The first step, and a constraining one as it turned out, was the selection of *the* urban water system to serve as the framework on which to build our model. Our somewhat Utopian goal was an urban complex of users with sub-systems serving them, sub-systems receiving their wastes, and possibilities for recycling of wastes. Additionally, we wanted the water transfers and quality changes of the metropolis to affect a "downstream" receiving water.

We started building with a single urban water user, as shown in Figure 18-4, being served from a single source, with alternatives in water treatment to provide him with adequate quality. The problem of selecting the proper treatment from known quality demands, QD_i for the $_i$th constituent, known raw water quality, QR_i and the removal efficiencies, $REMOVE_{mi}$ was too simple. So we added several possible raw water sources and considered what import mixture and treatment device in tandem would be optimal (see Figure 18-5). That was still too simple. So we moved to multiple discrete users, and decided not to guarantee satisfaction of any users demands until and unless a benefit *and* cost analysis indicated that his demands were justifiable (Figure 18-6). But that approach already had been used,[4] and the whole metropolis would not be involved. One can jump immediately, or at least quickly, then, from these rather simple bases to a much more detailed and complex set of circumstances such as depicted in Figure 18-7. Here there are multiple sources of raw water, multiple storage reservoirs, numerous alternative treatment plants, several water users,[5] many possible waste treatment alternatives, and several disposal schemes available for placing discharged wastes in any of several receiving waters. Moreover, this framework includes numerous possibilities for feedbacks or reclamation. One of the waste disposal alternatives, for example, is the discharge of treated wastes to one of the original raw water sources, a not unusual contemporary circumstance. Also, the possibility for recycle and retreatment of wastes by individual water users is included.

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[4] Sonnen, M.B. February 5-6, 1969. "Influence of Water User Requirements on the Relative Importance of Raw Water Characteristics." *Procedures—Eleventh Sanitary Engineering Conference on Influence of Raw Water Characteristics on Treatment.* University of Illinois, pp. 119-136.

[5] Or possibly whole communities of water users.

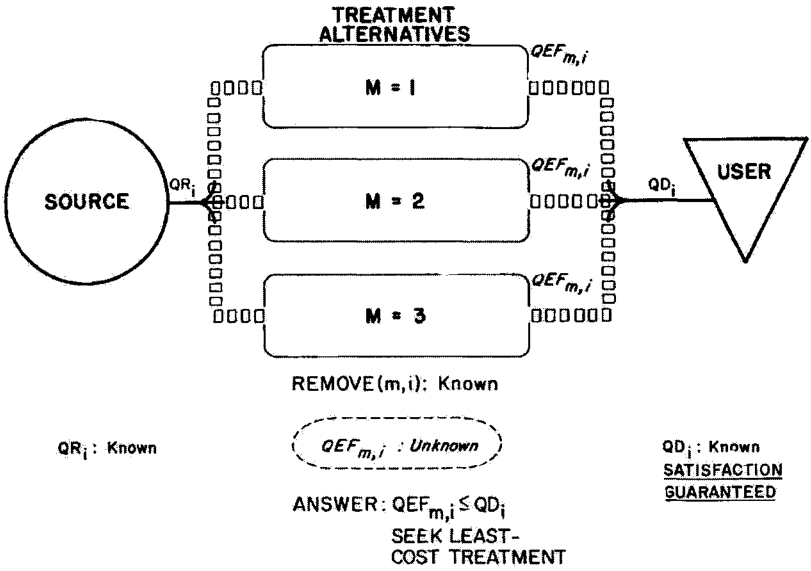

FIGURE 18-4. Simple Treatment Problem

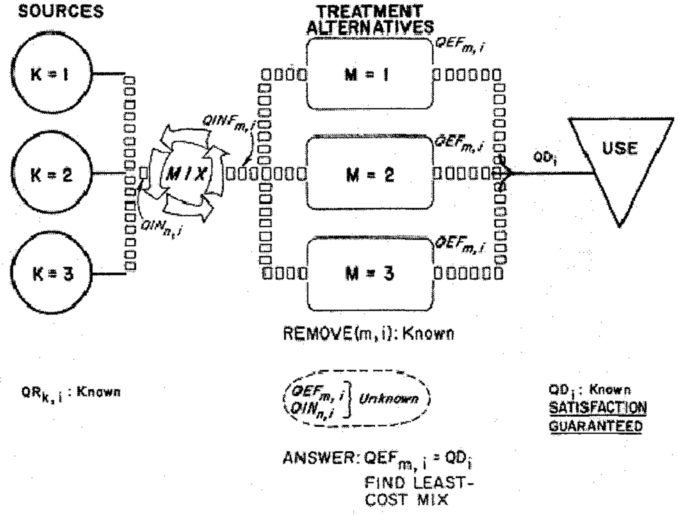

FIGURE 18-5. Multiple Source Problem

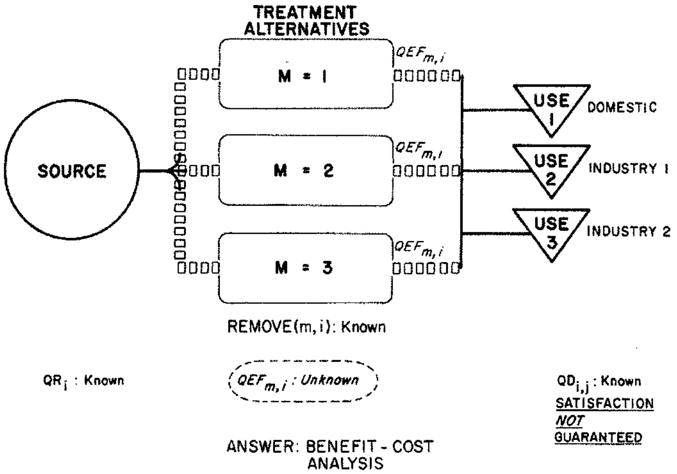

FIGURE 18-6. Multiple Use Water Treatment Problem

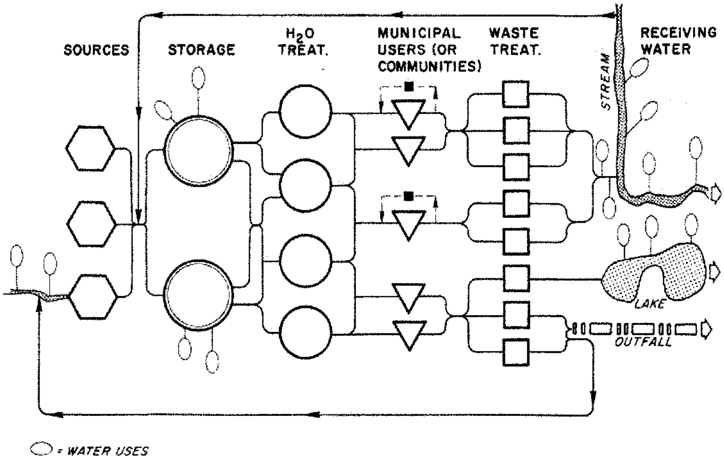

FIGURE 18-7. A Possible "Real" Urban Water System

As compelling and real-to-life as this hypothetical case appeared, it was too complex for a forthright demonstration and too involved for a low budget, first-step initiation of simulation. Consequently, the diagram was simplified considerably in some respects and expanded somewhat in others to the hypothetical set of urban sub-systems and interactions shown earlier in Figure 18-2.

3.2.2 Hypothetical Urban System and Simulation Model Adopted

Figure 18-2 indicates that numerous multiplicities were eliminated from the more-or-less "real" case of Figure 18-7. The demonstration system adopted included only multiple import sources and multiple water users. Only one facility was assumed for all other sub-systems. That is, it was assumed that all other sub-systems existed at the beginning of the problem and the simulation would be constrained by their dimensions and performance characteristics. The problem to be solved for this set of circumstances was quickly reduced to whether some mixture of quantities and qualities from the various import sources could or should satisfy the demands of any or all of the urban water users. Moreover, an objective of the demonstration was to keep track, through the simulation of the overall system behavior, of the quantity and quality of water at any point in the hypothetical metropolis during each time step.

The most complex element of the demonstration was the feedback that was included. The recycle possibilities for the individual users and the reclamation loop were incorporated both because they are likely physical possibilities now and in the future, and because they represent trying but important analytical non-linearities to be endured and overcome.

3.2.3 Solution Technique

Early in the project, a decision was made to construct a simulation model that solved the required equations step-wise rather than simultaneously. (It is likely that a simultaneous equation solution could have been developed within the same time and budget, but the possible problems associated with formation of the proper matrices and the probable increase in computer costs militated against that choice.)

The model finally constructed, then, was a bit unusual as a result of the feedbacks or non-linearities and our attempt at the same time to deal with these through step-wise rather than simultaneous solution. The problem to be solved was:

GIVEN:

a. The physical interrelationships depicted in Figure 18-2,

b. the monthly rainfall quantities and qualities,

c. the monthly stream inflows,

d. the monthly demands of all three users; and

e. a deterministic set of possible import quantities and qualities.

FIND:

a. Whether the predetermined import scheme was adequate to satisfy all demands; and

b. the monthly values of quantity and quality at each sub-system in Figure 18-2, including the outflow of the receiving water.

Stated more succinctly, the model was to simulate in monthly steps what happens throughout the hypothetical urban water system as a result of various attempts to meet demands.

In Figure 18-8, the general flow of the solution is indicated. Because the recycle possibilities existed, the solution began at the water use sub-system and started "backwards" toward supply in search of the necessary amount required from import. When the solution reached just "upstream" of the water treatment sub-system, the solution was stymied without knowledge of the amount and quality of the municipally recycled water. So the solution returned to the water use sub-system, added an increment of quality constituents and subtracted an increment of quantity to account for consumptive losses, and moved toward and through the waste treatment sub-system. The amount of effluent that was to be recycled in each month was taken as a percentage of the effluent available. The amount of water required from storage to meet the demands was then determinable.

As the very next series of steps, the solution calculated the amount of runoff to storage and the amount of discharge through the storm sewer sub-system as a percentage split of the given monthly rainfall. From that information, the model calculated the flow and quality leaving the receiving water environment. In addition, at that time, it determined the amount of water that would be available in storage as a result of the inflows from rainfall and the first trial quantity of imported water. If sufficient water was available to meet demands, both from prior storage and from current import, the quantity simulation was assumed to be valid and the quality simulation was made for that time period. Then the quantity simulation for the next month was begun back at the water use sub-system. If the amount of water available from import and prior available storage was *not* sufficient to meet demands, the deficit was allocated among the users according to their original relative demands, and new demands were calculated. Then the solution for that time period was repeated with the adjusted, if not completely satisfied, demands. After the second quantity solution, the quality solution was performed.

In the quality solution, facility was included to handle any number of conservative or non-conservative constituents. Quality was changed by "treatment" that was specified as removal efficiencies of the water and waste treatment sub-systems for each of the constituents. The individual reclamation processes for each user could have the same or different removal efficiencies for various constituents. The sequence of the quality solution was identical to that described above for the quantity solution.

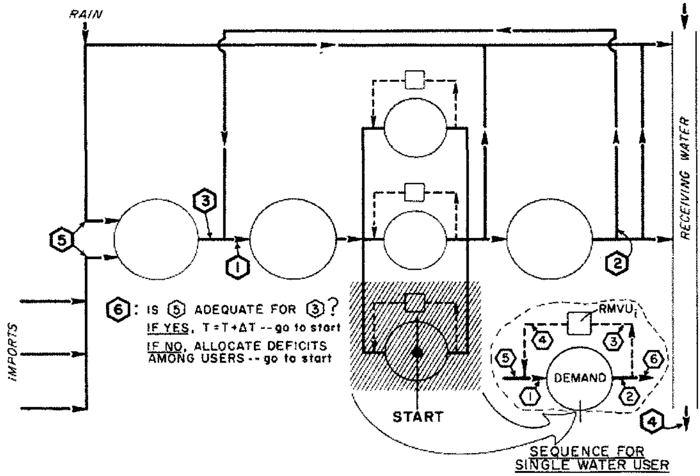

FIGURE 18-8. Sequences of Solution for Technical Evaluation

The description above is fairly general and includes only the basic steps of the solution. Figure 18-8 also shows the more detailed path followed in solving for the required amount of inflow to be supplied to each user, as a function of the amount of waste reclamation that each user practices. Section 6 presents results of some specific analyses made with this model and the economic model, described in Section 5, superimposed on it; and the computer program is presented in its entirety and documented in the appendix. It is sufficient to say here that a model was developed that in step-wise fashion simulates the movement of water through the urban environment from water supply sources to the receiving water. The solution sequence was complicated somewhat by non-linearities, but the demonstration of ability to model comprehensively "all aspects of urban water" has been made.

3.3 Special Problems with Time and Space

A major objective of this project has been fulfilled, namely to demonstrate the ability to model behavior at all points in the urban water resources system. However, in this project, we have modeled the urban water system as it "operates" on a monthly basis. Quite obviously, this is unrealistic for some sub-systems. Storm sewers notably, do not carry averaged monthly flows, but perform their entire and intended function in a matter of hours or days at the most. So, in the model developed in this study, there are no dynamic terms in the equations; the solutions for each sub-system are linked, steady state solutions in which the steady-state solution at time $t + \Delta t$ is dependent in part on the solution at time t.

Moreover, for the same reasons, there are some gross representations in space as well. The water distribution system has been "skeletonized" down to one line; the receiving water is represented as a single, fully mixed box rather than as a series of linked nodes with individual characteristics; storage and treatment facilities have been handled as single entities, notwithstanding the likely circumstance that there would be several such facilities throughout the community supplying water of varying quality to a given user.

Very briefly, the model developed in this study produces only a gross simulation of urban water system behavior. In the next chapter, we want to explore the feasibility and current successes of increasing the detail of simulation to represent realistically the time varying behavior of each sub-system.

4.0 REVIEW OF SUB-SYSTEM MODELING

The model described in the previous chapter illustrates that it is possible to construct a connected model of an entire urban water system. It is now to be shown that it is also possible to interrelate the urban sub-systems in general enough format to have a connected model applicable to many sub-systems, while at the same time the separate sub-system models would be capable of operating on specific detailed information for particular facilities, perhaps at various levels of time and space detail. Specifically, we will describe two projects of sub-system modeling in which WRE has been involved. Both of these deal with stormwater runoff, its transport in sewers, and its discharge to a receiving water.

4.1 FWQA Stormwater Management Project

The stormwater model described here is being developed for the FWQA through a joint effort of WRE, Metcalf and Eddy Engineers, and the Department of Environmental Engineering of the University of Florida. The overall objective of the project as stated in the contract is:

> *To develop a comprehensive mathematical model, capable of representing urban stormwater runoff phenomena, both quantity and quality, from the onset of precipitation on the basin, through collection, conveyance, storage, and treatment systems, to points downstream from outfalls which are significantly affected by storm discharges.*

At a project stage of somewhat more than 90% complete, the models required have been developed and preliminarily verified. Currently (summer 1970), they are being given final tests with data from various urban areas throughout the nation. *The* model developed in the FWQA project has been structured from three component sub-models:

1) A runoff model (rainfall-urban hydrology);

2) A transport model (storm and combined sewers); and
3) A receiving water model (streams, lakes, and estuaries).

4.1.1 The Runoff Model

The runoff model has been developed by WRE and Metcalf and Eddy. WRE has developed the quantity portion of the model, and Metcalf and Eddy has produced the quality portion of the model. Individually this model converts rainfall hyetographs to runoff hydrographs at various specified points at the lower ends of sub-catchments, and it also produces "pollutographs" of BOD and suspended solids concentrations over time at these same points. By itself, then, this model serves a very useful purpose, namely to describe the time and space variation of runoff quantities and qualities in urban watersheds. As a part of the FWQA stormwater model it also produces these outputs in punched card or other formats to be used directly by the transport model.

Specifically, the runoff model produces hydrographs of sub-catchment outflow, Q_O, from a simultaneous solution of Manning's equation and a continuity equation. Thus:

$$Q_O = \mathrm{V_O\ w\ D_e} \tag{1a}$$

where:

$$V_O = \frac{1.49}{n} D_e^{2/3}\ \mathrm{S_O}^{1/2} \tag{1b}$$

V_O is the outflow velocity of the runoff, w is the sub-catchment width normal to the flow direction, D_e is the "effective runoff depth," S_O is the ground slope, and n is a roughness coefficient.

The discharge, Q_O, as defined by equation (1a), is allowed to flow from the sub-catchment for a time period Δt, in seconds, and then the continuity equation is applied for the sub-catchment to calculate the depth and ultimately the flow, in the next time step:

$$D_{t+\Delta t} = D_t + (R - I)\Delta \mathrm{t} + \frac{Q_i - Q_O}{A}\Delta \mathrm{t} \tag{2}$$

where:

$D_t = D_e + D_O, D_O$ is the equivalent depth of depression storage (1/16 to 1/4 inch), R is the rainfall rate on the sub-catchment, I is the infiltration rate from the sub-catchment, Q_i is the sum of inflows from adjoining sub-catchments, and A is the area of the sub-catchment.

Repetitious application of the steady-state flow equation and the satisfaction of the continuity equation produce a quasi-non-steady or seemingly time-varying runoff hydrograph for each sub-catchment in the entire drainage area. These sub-catchment hydrographs are then routed through gutters and small drainage channels with the same solution technique.[6] This technique, known as the *kinematic wave solution,* can also be used to route flows in sewers and open channels where the assumption of temporal uniform flow is valid. An example of the output from the runoff model, computed in an application to a 13-acre test tract in Baltimore, Maryland[7] is shown in Figure 18-9.

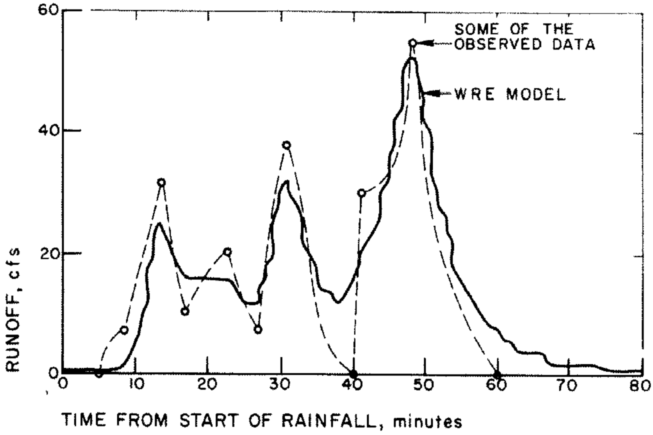

FIGURE 18-9. Calculated and Observed Runoff Hydrographs—FWQA Project

The runoff quality model operates on the runoff hydrographs at the sewer inlets, combining them with antecedent "dirt and dust" conditions, street sweeping practices, and land use patterns. The model produces "pollutographs" for BOD and suspended solids at the inlets. The results of a study by the American Public Works Association (APWA) on the water pollution aspects of urban runoff[8] are used to estimate the rate of pollutant accumulation. This daily value multiplied by the number of days since the last significant rainfall or street cleaning, gives the total amount of pollutant on

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[6] For gutter routing wD_e is replaced by the cross-sectional area of the flow in the gutter, and D_e is replaced by the hydraulic radius of the gutter.

[7] Tucker, L.S. August 1, 1968. *Northwood Gaging Installation, Baltimore-Instrumentation and Data.* ASCE Urban Water Resources research Program Technical Memorandum No. 1. Clearinghouse for Federal Scientific and Technical Information No. PB 182 786, W69-03507, 16 p.

[8] American Public Works Association. 1969. *Water Pollution Aspects of Urban Runoff.* FWPCA Water Pollution Control Research Series WP-20-15, USGPO, 272 p.

the ground at the start of the storm. The runoff hydrograph is used to determine the rate at which the pollutant enters the inlet.

Admittedly the quality portion of the runoff model is crude. It produces pollutographs only at the storm sewer inlets, and it gives no information about the areal distribution of pollutants throughout the sub-catchment. The rate expressions are empirical, and the basic data are subject to all the limitations of the APWA study, including the fact that all the data were collected in a single city (Chicago). Nevertheless, there is currently no better method available for predicting the quality of surface runoff, and indeed there are few if any other data than those collected in Chicago with which to verify any new models. All things considered, the computed pollutographs shown in Figure 18-10 for the 386-acre Laguna Street sub-catchment in San Francisco are excellent reproductions of the observed phenomena. Figures 18-9 and 18-10 represent the quantity and quality information that can be transferred from the runoff model to the transport model.

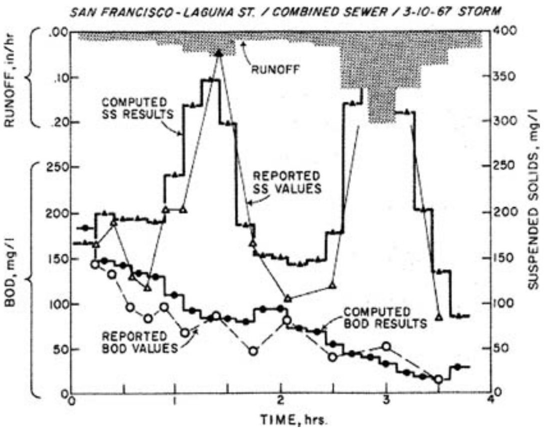

FIGURE 18-10. Quality Output for Runoff Model, FWQA Project

4.1.2 The Transport Model

The University of Florida has been almost exclusively responsible for development of the transport model. Metcalf and Eddy has been responsible for development of some treatment options that allow removal of pollutants by certain percentages specified for particular treatment processes. The function of the transport model is to route the total of: 1) storm inflow hydrographs; 2) any water added by Infiltrations; and 3) any sanitary wastes or "dry weather" flow through the main sewers and interceptors to the point of discharge to the receiving water.

Infiltration is estimated with a regression equation derived by multivariate analysis of data from the Johns Hopkins University Residential Sewerage Research Project.[9] The amount of infiltration is calculated as a function of antecedent soil moisture, the size of the largest pipe in the sub-catchments, the type of soil involved, the type of pipe joint, and the total length of sewers in the sub-catchment.

Inputs from the waste collection sub-system, i.e., the sanitary waste loads, are generated from a set of linear regression relationships developed by Howe and Linaweaver[10] to estimate average annual domestic water *demands.* The principal independent variables in these equations are the assessed property value and the number of people per dwelling. Although these average flows have been modified to reflect diurnal variations (on an hourly basis), seasonal variations are not taken into account. In addition, no provision is made for losses or consumption that occur during use. Dry weather wastewater flows for non-residential areas, such as commercial and industrial establishments, are estimated from unit water use values for each type of industry.

The quality of the dry weather flow is determined from available records of wastewater quality or estimated from widely accepted, general per capita wastewater loadings. Areas with garbage grinders or air conditioners are treated slightly differently. Industrial and commercial wastewater strengths are estimated on an individual basis as with the wastewater quantities.

The solution technique for quantity routing is nearly identical to that in the runoff model, except that the slope of the energy grade line is used in Manning's equation instead of the channel or pipe slope. Quality routing is accomplished by application of the principle of continuity of mass at nodal points in the sewer network. Both the quantity and quality solutions progress downstream until a junction, or node, with a new branch is reached. The solution then begins at the upstream end of that branch and brings the solution down that arm to the junction where the flows and masses from all tributary branches are added, and the solution proceeds further downstream.

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[9] Geyer, J.C. and J.J. Lentz. 1963. "An Evaluation of the Problems of Sanitary Sewer Design." *Final Report of the Residential Sewerage Research Project.* Department of Environmental Engineering Science, Johns Hopkins University.

[10] Howe, C.W., and F.P. Linaweaver, Jr. First Quarter 1967. "The Impact of Price on Residential Water Demand and Its Relation to System Design and Price Structure." *Water Resources Research,* Volume 3, No. 1, pp. 13-32.

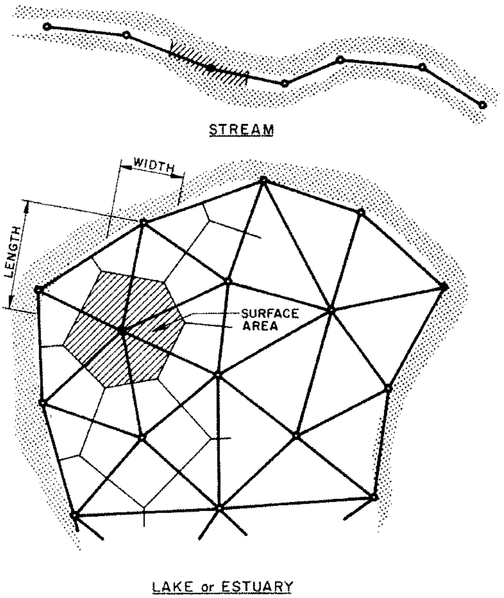
The stepwise progression of the solution in the downstream direction has two significant limitations. First, the model will not route runoff through sewers with relief connections to adjacent sewers, or through looped sewers. Second, any backwater effects caused by downstream flow control devices such as pumping stations, submerged outfalls, surcharged conduits, weirs or inflatable dams cannot be accommodated by the model, and hence these conditions must be handled by means external to the hydraulic solution itself.

Results from the transport model (and the receiving water model) have only recently been obtained and verified for demonstration areas throughout the nation. These are being reduced and prepared for presentation at the time of this writing and hence are not available for reproduction here. However, it can be reported that the transport model produces hydrographs and pollutographs in similar form to those from the runoff model and makes these available as input data to the receiving water model.

4.1.3 The Receiving Water Model

The receiving water sub-system model, like the others, has two components—a flow model, and a quality model. The flow model simulates the hydrodynamic behavior of a water body receiving stormwater inflow. The water body can be a lake, stream, or estuary.

For purposes of hydraulic computation, the water body is represented by a network of nodal points connected by channel segments. The nodal points and segments, such as those shown in Figure 18-11, are idealized hydraulic elements characterized by their surface area, cross-sectional area, length, friction coefficient, and so forth. Equations of motion and continuity can thus be applied to each element and solved simultaneously to produce a history of stages, velocities, and flows throughout the receiving water. Inputs from the stormwater transport model are made at nodal points in the pattern that are near, or correspond with, sewer outfalls.



Variables Associated with Nodes:
- Surface Area
- Depth
- Volume
- Head

Variables Associated with Flow Paths:
- Length
- Width
- Friction
- Velocity
- Flow
- Cross-section Area

FIGURE 18-11. Geometric Representations of Receiving Waters

4.1.4 Application and Expansion in San Francisco, California

Currently, WRE is engaged in developing for and with the City and County of San Francisco, a comprehensive model to describe behavior of the existing combined sewer network of the entire city. The major objective of the study is to provide a tool for the City to use in formulating a Master Plan for sewage disposal and pollution control in the near future. Countless alternatives exist that could be screened; hundreds of millions of dollars worth of facilities may be installed; and time is short in which lasting and important decisions must be made. The potential value of the computerized model in this application should be obvious.

Whereas the need for comprehensiveness in coverage of the entire City and County exists in this case along with a need to be fairly detailed and exact in the evaluation of alternatives, the project for the City must address itself to certain development aspects beyond those of the FWQA project. However, the experience gained in the

FWQA study, to say nothing of the assurances that the first study provided with respect to the feasibility of such modeling, has led directly to the San Francisco effort being much less expensive, but many times as expansive.

It is pertinent to say, nonetheless, that considerable money beyond the modeling budget is being spent in the San Francisco study to collect data and indeed to equip sampling stations with the newest available monitoring and telemetry facilities. The need for basic data to verify models such as those described, and perhaps far more importantly, just to provide knowledge of sub-system behavior, remains critical. Data collection is a topic explored nowhere else in this report, but it has been our experience that it is the most critical phase of urban water planning, design, or operation. It is also one of the most costly and paradoxically it is one of the most poorly understood. Data collection far too often is begun in a Pavlovian response to discovery that nothing is known. Only rarely is it begun, managed, and sustained in response to analyses that indicate specifically what needs to be known. The San Francisco experience will hopefully be an exception to the rule, and hopes are heightened because models that require specific bits of information at discrete times are being built concomitantly with the data acquisition efforts.

4.2 Compatibility of Diffused Models

We have attempted to show that persons in Gainesville, Palo Alto, and Walnut Creek could develop and test independently three mathematical models designed to simulate behavior of three linked physical entities, and that they could then bring them together, give the first one inputs (rainfall) and have the second and third produce outputs (hydrographs and pollutographs) in proper response. This has been done, then, for the rainfall-runoff-storm sewer-receiving water sub-system of Figure 18-1, it has been done with sufficient theoretical rigor to produce simulated responses that correspond well with prototype behavior. That is, the time-and-space detail has been fine enough to produce realistic responses.

In the comprehensive model described in Section 3, we were not able, for obvious reasons of budget and time limitations, to produce as realistic a collation of operational sub-models. It is our conclusion from the stormwater project experience, though, that sub-models describing detailed behavior of *all* the urban water sub-systems could be developed in such a way that they could be linked to form a single model of "all aspects of urban water," that is, all the sub-systems. To do that, however, special effort will be needed to provide proper interfacing of the sub-models.

Even the way in which a sub-system model is formulated may depend on the performance of the sub-systems on either side of the sub-system in question. Critically, the sub-model must be capable of accepting input from prior or adjacent sub-system models, and it must produce outputs that are acceptable as inputs by subsequent sub-models. In the initial step of identifying all inputs from and outputs to connecting sub-systems, moreover, consideration should also be given to sub-systems that might be optionally connected, either as alternative facilities or as facilities that function only part of the time. For example, outputs from an "urban runoff model" will always form inputs to a "stormwater transport model," but output from a "waste collection model" may or may not become inputs to the "stormwater transport model," depending on whether or not combined or separate sewers are involved. Hence, sub-system model interconnections should be formulated in such a manner that the connections are easy to make or break.

Once input and output parameters have been defined, the time scales associated with them must be specified. That is, over what time interval will each sub-model solve its particular problem, and what time averaging or interpolating will be necessary to transfer information in the proper timeframe for the next sub-model? For example, the "stormwater transport model" may calculate flows and concentrations at an outfall for every 15 seconds of real time, but the "receiving water model," which is likely to deal with much larger network representations, may require input data only once per minute. Hence, the transport model results may have to be averaged before transfer to the receiving water model.

It might seem simplest to use "instantaneous" values for the inputs and outputs, since sub-models designed to operate on these values could produce answers at any desired level of detail, i.e., answers averaged over any desired time interval. However, it is more desirable to have each sub-model work at time scales most commensurate with prototype behavior, since substantial savings in computer time (and sub-system detailing) can be realized by using the largest time intervals possible. So if the urban water model could be operated on a daily basis, let's say, until a storm occurred, and we wished to simulate hour-by-hour or minute-by-minute behavior of the storm sewer system, perhaps the model could be formulated to "shift gears" to accommodate the momentary need for finer detail. "Shifting gears" is not a new or unique idea in this application. Maass, et al.[11] used this technique to route floods through a river basin. In that case a shift was made from monthly simulation to operation on a 6-hour basis until the flood had been routed through the system, at which time the monthly operation was resumed. It should be sufficient to conclude here that models for water distribution network analyses, receiving water D.O. sag-curve analyses, water and waste treatment simulation, and storage and transfer models for free-surface flow situations have been developed and applied individually by countless investigators, and that with a certain amount of adaptation effort these models can be linked to perform simulations of the entire metropolitan water milieu operating on any appropriate time scale to deal with planning, design, or operation alternatives.

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[11] Maass, A., M.M. Hufschmidt, et al.. 1962. *Design of Water Resource Systems—New Techniques for Relating Economic Objectives, Engineering Analysis and Governmental Planning.* Harvard University Press, Cambridge, 620 p.

5.0 INITIATION OF ECONOMIC SYSTEMS ANALYSIS

5.1 Economics of Water Resources Development

For perhaps 70 years now the major type of economic analysis applied to water resources planning alternatives has been benefit-cost analysis. The objective of the exercise has been to determine what size or "scale" of project, what height of dam at a given location will produce the maximum difference between the benefits to be derived and the costs to be incurred to accrue those benefits. As we all recall, projects whose benefits were greater than costs (benefit-to-cost ratio was greater than 1.0) were deemed to be justified or at least justifiable. There is some evidence[12] that Congressmen used the numerical size of these ratios as something of a guide when appropriating funds for such facilities.

The Water Resources Council[13] is currently reviewing the traditional analysis procedure with the purpose of providing guidelines for a broader project evaluation framework to include consideration of such objectives as regional development, environmental preservation, and well being of the people, in addition to economic efficiency with respect to national income. The outcome of this review is still pending; it can be anticipated, however, that more analysis, rather than less, will be required for the multi-objective optimization indicated. Systems analysis will surely be of service as a tool to facilitate the evaluations necessary.

Curiously, there is another objective often invoked whose analytical description may be quite different from that for economic efficiency. This objective, minimization of cost, is *most* often, though not always, employed for the water *quality* management goal. The normal ploy is to adopt a dissolved oxygen standard set by others and then to find the type of single waste treatment plant or the mixture of various scattered waste treatment plants that will just attain the standard for the minimum expenditure of dollars for construction and operation of said plants.

5.2 Applications of Systems Economic Analysis to Water-Oriented Sub-Systems

In 1962, Maass, et al., produced an extraordinary book in which the authors made a valiant attempt to deal in the emerging language and analytics of the computer with the traditional water resources problems of water supply, flood control, and navigation. Their purpose was to demonstrate that dollar benefits could be counted for each purpose of development and that these could be compared with the costs of these development schemes with lightning speed and theoretical rigor. They were more than just mildly successful. In 1966, others of the Harvard Water Group[14] published a later volume in which the growing tools of the simulation field were even

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[12] Haveman, R.H. 1965. *Water Resource Investment and the Public Interest, An Analysis of Federal Expenditures in Ten Southern States.* Vanderbilt University Press, Nashville, TN, 199 p.

[13] United States Water Resources Council—Special Task Force. June 1969. *Procedures for Evaluation of Water and Related Land Resource Projects.* Washington, DC, 122 p.

[14] Hufschmidt, M.M. and M.B. Fiering. 1966. *Simulation Techniques for Design of Water-Resource Systems.* Harvard University Press, Cambridge, 212 p.

more explicitly laid out, and computerized accounting techniques for benefits and costs were presented in some detail.

Somewhere in those early days of 1962, someone perhaps still within the Harvard Water Group,[15] decided to bring water quality into the systems economic analysis picture, and from the very beginning the objective was posed in terms of minimization of cost to meet a quality standard in the receiving water.[16]

Since 1962, there have been numerous applications of systems analysis to problems of the economics of water development and quality management. Notable ones are those of Hall;[17,18] Schweig and Cole;[19] Kally;[20] Loucks;[21] Buras and Schweig;[22] Butcher, Haimes, and Hall;[23] Mobasheri and Harboe;[24] and Young and Pisano.[25] These workers have dealt with optimization of both costs and net benefits for various sizes, routes, and stages of water development projects. Both linear and dynamic programming have been used. Recent developments by WRE and the Texas Water Development Board[26] have resulted in multi-stage, multi-reservoir optimization through techniques less costly and less time consuming than linear or dynamic programming.

In basin-wide water quality management, numerous authors have dealt with the least-cost way to meet stream standards, almost universally those for dissolved oxygen.[27,28,29,30,31,32,33] Dysart and Hines[34] have produced a dynamic programming

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[15] Thomas, H.A., Jr., and R.P. Burden. February 15, 1963. *Operations Research in Water Quality Management.* Final Report to Bureau of State Services, Division of Water Supply and Pollution Control, USPHA.

[16] Lynn, W.R., J.A. Logan, and A. Charnes. June 1962. "Systems Analysis for Planning Wastewater Treatment Plants." *Journal WPCF,* Volume 34, No. 6, pp. 565-581.

[17] Hall, W.A. September 1961. "Aqueduct Capacity Under an Optimum Benefit Policy." *Journal of the Irrigation and Drainage Division,* ASCE, Volume 87, No. IR3, Part I, pp. 1-11.

[18] Hall, W.A. July 1964. "Optimum Design of a Multiple-Purpose Reservoir." *Journal of the Hydraulics Division,* ASCE, Volume 90, No. HY4, pp. 141-149.

[19] Schweig, Z., and J.A. Cole. June 1968. "Optimal Control of Linked Reservoirs." *Water Resources Research,* Volume 4, No. 3, pp. 479-497.

[20] Kally, E. March 1969. "Pipeline Planning by Dynamic Computer Programming." *Journal of WPCF,* Volume 38, No. 12, pp. 1883-1897.

[21] Loucks, D.P. August 1969. *Stochastic Methods for Analyzing River Basin Systems.* Report to OWRR, (Project No. C-1034), Department of Water Resources Engineering and the Water Resources and Marine Science Center, Cornell University, 313 p.

[22] Buras, N. and Z. Schweig. September 1969. "Aqueduct Route Optimization by Dynamic Programming." *Journal of the Hydraulics Division,* ASCE, Volume 95, No. HY5, pp. 1615-1631.

[23] Butcher, W.S., Y.Y. Haimes, and W.A. Hall. December 1969. "Dynamic Programming for the Optimal Sequencing of Water Supply Projects." *Water Resources Research,* Volume 5, No. 6, pp. 1196-1204.

[24] Mobasheri, F. and R.C. Harboe. February 1979. "A Two-State Optimization Model for Design of a Multipurpose Reservoir." *Water Resources Research,* Volume 6, No. 1, pp. 22-31.

[25] Young, G.K. and M.A. Pisano. February 1970. "Nonlinear Programming Applied to Regional Water Resource Planning." *Water Resources Research,* Volume 6, No. 1, pp. 32-42.

[26] Water Resources Engineers, Inc. August 1969. *A Completion Report on System Simulation for Management of a Total Water Resource.* Prepared with and for the Texas Water Development Board, Austin, Texas, 132 p.

[27] Deininger, R.A. 1965. "Water Quality Management—The Planning of Economically Optimal Pollution Control Systems." *Proceedings of the First Annual Meeting of the American Water Resources Association,* American Water Resources Association, Urbana, Illinois, pp. 254-282.

[28] Goodman, A.S. and W.E. Dobbins. December 1966. "Mathematical Model for Water Pollution Control Studies." *Journal of the Sanitary Engineering Division,* ASCE, Volume 92, No. SA6, pp 1-19.

[29] Kerri, K.D. December 1966. "An Economic Approach to Water Quality Control." *Journal AWWA,* Volume 61, No. 3, pp. 115-118.

model for studying least-cost solutions to quality management problems involving both thermal pollution and dissolved oxygen. which obviously are linked.

For choice of particular treatment processes, systems analysis techniques have been applied to the staging problem, optimization of trickling filter operation, and simulation and optimization of the step-aeration process. Evenson et al.,[35] and Shih and DeFilippi[36] have independently developed dynamic programming models to select the least-costly mix of treatment components to meet an effluent BOD requirement.

Despite the successes in the past with applications of systems techniques to particular water resources economics problems, there appears to be a long way to go to bring systems economic analysis to bear on entire urban water systems comprised of numerous sub-systems that have various owners (buyers) and beneficiaries who may or may not be those same owners. In other words, the relationship between benefits and costs is not so explicit as we have assumed it to be in the past, and the guarantees of satisfaction of demands that least-cost methods presume are not always valid in the metropolitan planning context. Whereas it may remain the goal of City A to meet its own water demands in perpetuity, it may not be in the best interest of Metropolis C comprised of City A and City B, to meet 100% of the demands of City A during times of drought. It may also be more beneficial relative to costs, for a community to treat its wastes to a high degree and to recycle the wastes rather than to import water from an objectionable source. These questions suggest that benefit *and* cost analysis are needed for all the urban sub-systems and that it would be advantageous to be able to superimpose the analysis of both benefits and costs on a simulation result derived for evaluation of physical performance. An initial superposition of this type was one of the major objectives of the present study.

Before outlining benefit and cost functions used, we discuss below the concept of benefit-cost analysis and why, in spite of its weaknesses, it remains the most logical framework in which to view our decision for choice among physical works alternatives.

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[30] Liebman, J.C. and W.R. Lynn. Third Quarter 1966. "The Optimal Allocation of Stream Dissolved Oxygen." *Water Resources Research*, Volume 2, No. 3, pp. 581-591.

[31] ReVelle, C.S., D.P. Loucks, and W.R. Lynn. February 1969. "Linear Programming Applied to Water Quality Management." *Water Resources Research*, Volume 4, No. 1, pp. 1-9.

[32] Thomann, R.V. and M.J. Sobel. October 1964. "Estuarine Water Quality Management and Forecasting." *Journal of the Sanitary Engineering Division*, ASCE, Volume 90, No. SA5, pp. 9-36.

[33] Thomann, R.V. Third Quarter 1965. "Recent Results from a Mathematical Model of Water Pollution Control in the Delaware Estuary." *Water Resources Research*, Volume 1, No. 3, pp. 349-359.

[34] Dysart, B.C., III and W.W. Hines. January 1969. *Development and Application of a Rational Water Quality Planning Model*, Georgia Institute of Technology, Water Resources Center, WRC-0668, 182 p.

[35] Evenson, D.E., G.T. Orlob, and J.R. Monser. November 1969. "Preliminary Selection of Waste Treatment Systems." *Journal WPCF*, Volume 41, No. 11, Part I, pp. 1845-1858.

[36] Shih, C.S. and J.A. DeFilippi. April 1970. "System Optimization of Waste Treatment Plant Process Design." *Journal of the Sanitary Engineering Division*, ASCE, Volume 96, No. SA2, pp. 409-421.

5.3 Benefit-Cost Analysis as a Decision Framework

5.3.1 Background

The benefit-cost method :of analysis for evaluating alternative courses of action dates from the turn of the century and originated with the Army Corps of Engineers.[37] Its underlying philosophy has been that the one project, from among several, that has the highest ratio of total annual benefits to total annual costs is the one that should be constructed, provided, of course, that the ratio for this particular project is greater than 1.0. If benefits do not exceed costs—that is, the ratio is less than 1.0—the project is not economically justified. This method has been used in this country primarily to evaluate water-resource and transportation alternatives.[38] Senate Document 97,[39] as well as other sources,[40,41] indicates that it will be used more and more widely.

5.3.2 Limitations

Prest and Turvey[42] have provided a rather thorough survey of the benefit-cost method of analysis and its strengths and weaknesses. Some of their findings will be reviewed here.

First of all, the technique is limited both in principle and in practice. In principle it should be understood to be a technique for making decisions within a predetermined policy framework, and it is limited by this policy, which encompasses considerations of diverse types including those of a social and political kind. The use of the benefit-cost ratio method for evaluating alternatives is not the policy itself; it is only a tool to help implement the policy; and the economic objectives must be chosen beforehand. Second, the technique is not adequate to handle very large-sized projects that could of themselves cause alteration of the price system throughout the entire economy. Whereas these conditions are hardly likely to obtain for water resources projects currently being planned in this country, nonetheless they might arise in some of the developing nations. Prest and Turvey point out that even in such cases as those the technique may not fail completely, but the manipulators of the method should stay acutely aware that this limitation exists.

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[37] National Research Council. 1966. *Waste Management and Control,* Publication 1400, National Academy of Science, Washington, DC, 257 p.

[38] Kneese, A.V. 1964. *The Economics of Regional Water Quality Management.* The Johns Hopkins University Press, Baltimore, Maryland, 215 p.

[39] President's Water Resources Council. June 4, 1964. "Policies, Standards, and Procedures in the Formulation, Evaluation, and Review of Plans for Use and Development of Water and Related Land Resources." *Senate Document No. 97, 87th Congress,* Second Session, May 29, 1962, U.S.G.P.O., Washington, DC, 13 p. and Supplement No. 1, "Evaluation Standards for Primary Outdoor Recreation Benefits," 9 p.

[40] Dixon, J.W. 1964. "Water Resources, Part I. Planning and Development." In *Handbook of Applied Hydrology,* Ven Te Chow, Editor-in-Chief, McGraw-Hill Book Company, Inc., New York, New York, pp. 26-1 to 26-29.

[41] Weinkauff, H.C.C. and C.P. Lindner. March 8-12, 1965. "Current Concepts of Water Resources Project Formulation and Their Application to Hydro-Development in the Southeast." *ASCE Water Resources Engineering Conference,* Mobile, Alabama, Conference Preprint 139, 27 p.

[42] Prest, A.R. and R. Turvey. December 1965. "Cost-Benefit Analysis: A Survey." *The Economic Journal,* pp. 683-731.

Brandt[43] rather succinctly criticized the technique's main weaknesses when he said that considering projects yielding a benefit-cost ratio of greater than 1.0 and then choosing the one with the highest value from among the alternatives not only invites "overestimation of benefits and underestimation of costs, but also it provides no sensible criterion for a comparison of many projects as to their contribution to the growth of the social product." This last assertion is another way of saying that the technique does not encompass the entire policy objective for considering the development of the project in the first place. Having a benefit-cost ratio calculated for each of the alternatives does not end the search for which of these should be undertaken. The remainder of the priority rating, indeed the choice itself, must be left to the public, its representatives, or industrial management, whoever is involved and by whomever the analyst is employed. It is, of course, of vital concern to whoever is left to make the choice that an economic analysis of this sort be made, but there may be facts or attitudes available to these policy makers that will lead them to choose from among the alternative solutions one that does not have the highest benefit-cost ratio but that will at the same time best meet their objective.

There is also apparently some tendency for analysts to sanctify the number 1.0. Planners interested in the building of dams, canals, or other water projects, while conscientiously trying to be objective in their evaluations, may be eyeing the cost column while enumerating and evaluating the benefits. In private conversations with WRE personnel some water planners and hydrologists have admitted that if a benefit-cost ratio for a project turns out to be 0.9 or as low as 0.7, they will sometimes repeat the calculations to see if something was overlooked or left out. Sometimes benefits are recalculated, other times the design flood computations are checked to see if the predicted event could really be so large. This philosophy is not altogether to be condemned, particularly in light of the many rather hazy estimates that may be made. It is an interesting observation, however, that projects whose benefit-cost ratios are reported usually have a calculated ratio of 1.2, 1.7, 2.6, or some similar value. It may be reasonably factual that these are the values, but never has WRE found a reported benefit-cost ratio of 65, 317 or anything nearly so convincing. Still many projects must be clearly justifiable. On the other hand it is likely that some project has been undertaken that had a calculated benefit-cost ratio of greater than 1.0, but a value that must have stretched the resourcefulness of the planner in its attainment.

Just enumeration of costs and benefits presents problems, even before we begin to evaluate them. The definition of the project itself, the project life, and any externalities or secondary benefits that are to be included are all decisions that are not easily made. Usually the scope and nature of the project will be clear, but occasionally there is overlap between the proposed project and others belonging to or controlled by the same authority. For example, if the project to be evaluated is a dam, its existence may affect water levels, power outputs, and other factors at downstream dams operated by the same authority. It may or may not be that these

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[43] Brandt, K. 1961. "Water Pollution Control and its Challenge to Political Economic Research." *Proceedings, The National Conference on Water Pollution.* December 12-14, 1960, U.S. Government Printing Office, Washington, DC, pp. 474-484 and Discussion, p. 491.

effects at the downstream dams are part of the proposed project and hence the costs and benefits accruing there should or should not be included in evaluating the new project, depending on a policy decision that must be made first.

Secondary benefit evaluation is a critical area where double counting can occur. The distinction to be remembered here, which is not a simple one to make in many cases, is whether market prices already reflect the marginal social benefits or costs that we think will result from the project, in which case they should not be included; or whether the market has no way of including the benefits from the particular output of the project being considered in which case these benefits should be included. For example, the benefits from industrial water supply can be evaluated for a rubber tire manufacturer as the price he is able to get for his tires plus the secondary benefit to be derived from selling still more tires that his first profits allowed him to make because he could then buy another conveyor belt, minus his costs for the supplied water and the conveyor. The secondary benefits are his extra tire sales less the cost of his conveyor and not the extra automobile sales minus tire shipping and auto assembling costs and the cost of the conveyor. The market demand for tires already reflects the value of extra cars and the costs of shipping and assembling. It does not reflect the value of the water itself and the benefits to be gained therefrom, and so these values alone must be imputed.

We will have more to say later about the particular problem of time in an economic analysis, but it has relevance to the enumeration problem of benefit-cost analysis. Assignment of project life, as we shall see, depends on many things, is often a rather subjective estimate; and the error that a wrong choice would manifest depends on the interest rate adopted, the larger the interest rate, the less the effect. It is obviously another policy decision that must be made outside of the benefit-cost analysis itself, although it has a direct bearing on the outcome.

So much for enumeration problems, which are formidable. Evaluation problems are at least as troublesome, but they are more obvious, and hence require less discussion. Perhaps the most obvious one is that some users or beneficiaries can only have the value of their benefits estimated through the alternative-cost method, whereas benefits for others can be estimated or measured directly, but perhaps only incompletely. The question arises whether the benefits for both users estimated two ways are really in their true relation to one another. More perniciously, questions arise regarding the reconciliation of dissimilar reliabilities, vulnerabilities, estimation errors and other physically probabilistic variances among alternative projects, and even within elements of a given project.

The other truly difficult problem is the one of assigning intangible values. Prest and Turvey have written that these values must be included in the prose of the analysis since there is no way to include them in the arithmetic. (In the method derived during this project, WRE has attempted to include intangibles within the arithmetic.)

Other problems requiring considerable evaluation judgments include the measurement of "utility," in general; the comparability of utility among different users; allowances for market imperfections and externalities which can only be partially known; the choice of an appropriate discount rate; and wrestling with socio-economic uncertainties such as major political or military upheavals or natural disasters, which Prest and Turvey correctly point out, "none of us can predict..."

5.3.3 Strengths

Faced with all of these weaknesses why do we persist in using this technique? First of all, and very simply, there is nothing better available. Secondly, it forces those responsible to quantify benefits and costs as well as can be done, rather than resting on vague qualitative judgments or personal hunches. This, Prest and Turvey have said, "is obviously a good thing in itself; some information is always better than none." Thirdly, this technique does lead us to some understanding or knowledge of the prices that consumers are willing to pay, and it may cause questions to be raised about existing pricing policies and other soul-searchers that might otherwise never be questioned. Then too, "Even if [the] analysis cannot give the right answers, it can sometimes play the purely negative role of screening projects and rejecting those answers that are obviously less promising." And finally, "...insistence on cost-benefit analysis can help in rejection of inferior projects, which are nevertheless promoted for empire-building or pork-barrel reasons."

5.4 Successes with Comprehensive Accounting in this Study

Apparently, we were considerably over-optimistic at the outset of the study with regard to our intended economic modeling output. It was our original intention to produce an executive program that would accumulate costs and benefits, appropriately discounted over time, for all the urban water sub-systems. Moreover, this program was to direct the execution of sub-programs or sub-routines that were to be provided for each sub-system. These sub-routines were to accumulate the benefits and costs associated with each sub-system independently. So we wanted to produce a model that would evaluate the benefits and costs of various storage system alternatives, various treatment alternatives, various sewer arrangements, and so forth, and that at the same time would accumulate the benefits and costs associated with entire urban water systems comprised of the many alternative sub-systems.

For purely demonstration purposes, we also wanted the benefits of each sub-system to be derived either as positive quantity or quality gains to the beneficiaries or as averted quantity or quality losses that would not have to be suffered by these same users. Table 18-1 indicates the sub-routines that would have been used for each sub-system.

As it turned out, we were forced by time, budget, and circumstances to settle for considerably less than the ideal. We have been able to compute the benefits and costs for two sub-systems and to accumulate these properly to demonstrate what we intended to be the accumulation for the entire urban water system. Rather than developing an entirely separate program to perform the economic analysis, however, we have included the economics in a sub-routine to the technical evaluation model.

The economic calculations are performed in the sub-routine that organizes and performs the printout of results from the technical model. This allows the accounting for each sub-system to be done while the technical results for that sub-system are still available within the machine.

TABLE 18-1

Sub-routines to Have Been Used for Evaluation of Benefits and Losses for each Urban Water Sub-system

Urban Water Sub-System	**Affected Users**	**Sub-Routines**				
		Benefits		**Avert Losses**		**Cost**
		Quantity	**Quality**	**Quantity**	**Quality**	
Urban Hydrology	Urban	X		X		X
Storm Sewers	Urban			X	X	X
Water Supply	Urban	X		X		X
Water Distribution	Urban	X			X	X
Waste Collection	Urban	X			X	X
Waste Disposal	Stream	X			X	X
Reuse-Return	Urban	X		X		X
Storage	Urban	X		X		X
Water Treatment	Urban		X		X	X
Water Use	Urban	X	X			X
Waste Treatment	Stream		X		X	X

The major purpose of sub-routine POUT is to summarize the technical results, print headings at the tops of appropriate pages, and cause the monthly technical results to be written in the proper places. However, since the benefit calculations we intended to perform were to be functions of the technical output of the physical sub-systems, this seemed to be the ideal place as well for calculation of the economic performance of information. Very basically, we wished to calculate benefits as a function of: 1) an annual amount of water demand satisfied or quality demand satisfied, U; and 2) a unit price, P, or the amount of money that each user or beneficiary would be willing to pay for the corresponding amount of use. That is:

$$B = U * P$$

Where:

B = annual benefit from satisfaction of quantity or quality demands

U = amount of annual demands satisfied

P = unit price or gain per unit of satisfied demand.

The values of P and U for the two sub-systems we analyzed, then, were calculated (or read as input data) in the sub-routine POUT. Also calculated were the annual costs of the sub-systems as functions of their capital and operating costs, which in turn were also functions of the technical output.

5.4.1 Economic Functions for the Water Use Sub-system

The water use and water treatment sub-systems are the two that were analyzed during this project. We chose these two so that we could use one whose benefits would be predominantly water quantity benefits and the other as one having primarily water quality benefits. For the water use sub-system the values of U were calculated from the monthly[44] demand values for each user. For months (years) in which demands were met the use levels were taken as the demands:

$$U_j = TDMD_j * DELT$$

where:

U_j is the use level for user $_j$; $TDMD_j$ is the flow demand exerted by user $_j$; DELT is the time step during which the demand is exerted—in this case 30 days. For cases in which deficits occurred, the amount of loss or nonuse was taken as the quantity of the deficit:

$$U_j = -(QDEF_j * DELT)$$

In this case $QDEF_j$ was the amount of the flow deficit for the $_j$th user, and the negative sign caused the benefit to be accumulated as a loss rather than a gain.

For the water use sub-system, in which there were three separate users ($_j$), the values of P, or price, were assumed to be one dollar, two dollars, and fifty cents per 1,000 gallons in cases of both—adequate supply and deficit.

Costs for this sub-system were comprised of capital cost and operation and maintenance cost, each appropriately discounted for time and interest rate. Capital costs were assumed to be piping costs for each user and the retreatment or reclamation plant costs. Both types of costs were assumed to be incurred in the first

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[44] As a computational convenience, the annual economic results were computed from the monthly technical results as though each month's quantity and quality outputs were average annual values—another indication of the ability to "shift gears" in a time sense.

year and again in the seventh year of the demonstration project life. These costs were brought to present worth with the proper single payment present worth factor. We assumed constant amounts for these costs for each user.

Operation and maintenance costs were taken to be functions of the amount of water used by each user and the amount of reclamation he practiced. That is:

$$OMC_j = (COMP_j * QUSE_j) + (COMR \;\; * QJRT_j)$$

where:

OMC_j = operation and maintenance costs in a given year for user $_j$, \$/yr;

$COMP_j$ = coefficient to account for annual cost of water use for each user, \$/mgd;

$QUSE_j$ = average annual demand for user $_j$, mgd;

$COMR_j$ = coefficient to account for annual reclamation cost for user $_j$, \$/mgd;

$QJRT_j$ = average annual reclamation flow for user $_j$, mgd.

For each alternative investigated, the two coefficients were given as data, and the flow values were calculated in the technical evaluation model.

5.4.2 Economic Functions for the Water Treatment Sub-System

This sub-system was chosen primarily because its benefits are associated with water quality.[45] The beneficiaries are the same three water users, but now benefits are calculated for each user on the basis of whether or not the water treatment plant is able to meet each user's demands for concentrations of numerous quality constituents.

The U values then must be calculated for each user and each constituent of interest to him. When the imported water supplies and collected runoff are of sufficiently good quality to be treated to a level suitable to the most demanding user, then the U's are calculated as the amount of water used:

$$U_{ji} = K * DELT * \frac{CSUP_i - CUIN_i}{CSUP_i} * QUSE_j$$

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[45] A very interesting benefit problem dealt with by Sonnen in other places is that of *waste* treatment in which the beneficiaries are downstream water users not responsible for the costs to protect their quality desires.

where:

U_{ji} = unit benefit to the $_j$ th user from having his quality demand for the $_i$th constituent met;

K = a coefficient;

$DELT$ = time period;

$CSUP_i$ = concentration of constituent $_I$ in the storage sub-system effluent;

$CUIN_i$ = concentration of constituent $_i$ supplied to the three users (set by the most stringent demand from among the three users);

$QUSE_j$ = amount of flow supplied to each user.

For cases in which the supply was not able to be treated to adequate levels, a loss was calculated as:

$$U_{ji} = -K * DELT * \frac{CV_i - CUIN_i}{CSUP_i} * QUSE_j$$

where:

CV_i is the concentration of the stored runoff and imported water. This case obtains only when CV_i is greater than $CUIN_i$ so U_{ji} in this case is always negative, implying a loss. Just for the sake of demonstration, we also included facility for users to gain a bonus benefit when their quality demands were more than met, in this case U_{ji} by the percentage by which a particular user's demand was less-than the quality of the water actually supplied.

Costs for the water treatment sub-system again were comprised of both capital and operation-and-maintenance costs. The capital costs in this case were assumed to occur only in the first year and were

$$CAPQ = 10{,}000 * (VFUL / DELT)$$

where:

$CAPQ$ = capital cost, \$;

$VFUL$ = storage capacity of the storage reservoir, mg;

$DELT$ = time period, 30 days.

The quantity $VFUL / DELT$ was taken as a measure of the design capacity of the treatment plant.

Operation and maintenance costs were taken to be functions of the constituent in question, the efficiency of the process for the removal of that constituent, and the amount of the water supplied in the time period:

$$OMCQ_t = TRT_i * RMOV_i * QSUP_t$$

where:

$OMCO_t$ = O&M costs in a given time period, $_t$;

TRT_i = a coefficient for each constituent, $_i$;

$RMOV_i$ = efficiency of the water treatment plant for the removal of constituent, $_i$

$QSUP_t$ = amount of water supplied to the treatment plant in time period, $_t$.

5.4.3 Repetitive Time and Interest Calculations

Calculations were made in a separate sub-routine (PBEN) for the present worth factors and capital recovery factors necessary to reduce all benefit and cost data to present values and equivalent annual values over the project life. For each time period, then, this sub-routine was called to calculate present values of benefits, for example, as

$$BPW_t = [B(F_t - F_{t-1})] + BPW_{t-1}$$

where:

BPW_t = present worth of benefits for a given sub-system for all the periods through time, $_t$;

B = benefits for the sub-system in time $_t$;

F_t = present worth factor for uniform annual series-to-time $_t$;

F_{t-1} = present worth factor for uniform annual series-to-time $_{t-1}$;

BPW_{t-1} = present worth of benefits for the sub-system through the last time period.

Similar repetitive calculations were made for costs with single-payment present-worth factors used to discount future capital costs and with uniform annual present-worth factors to discount future operation and maintenance costs.

5.4.4 Intangible Benefit Calculations

The economics programs developed here were also given the capability to increase the monetary benefit values by an amount representative of the "intangible" or inestimable, non-monetary benefits. This was done simply by calculating "total" benefits as:

$$BT = B(1 + R)$$

where:

BT = total benefits;

B = monetary benefits ;

R = a factor equal to or greater than zero.

In this study, we merely used constant values of R from zero to 4.0, depending on the user, the supposed importance of the user to the hypothetical metropolitan area, or the relative importance of a particular quality constituent to a particular user. The concept of such multipliers, however, and possible values for them, have been presented in detail elsewhere.[46,47,48]

6.0 RESULTS OF INITIAL COMPREHENSIVE MODELING

6.1 Technical Modeling

Early in this project the technical evaluation model was developed and applied to the hypothetical urban system shown earlier in Figure 18-2. The demonstration analysis has included: evaluation of six alternative combinations of user demands; and yields of imported water from three possible sources, each containing five quality constituents of varying interest to each of three water users. Each water user was given wastewater reclamation facilities having removal efficiencies for each constituent ranging from 0 to 100%. A period of 14 months was simulated.

The six flow combinations and the quantity demands used in the demonstration case are listed in Table 18-2. Quality demands for the three users are listed in Table 18-3. In addition to the considerable variability already built into the demonstration it was

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[46] Sonnen, M.B. 1967. *Evaluation of Alternative Waste Treatment Facilities*. University of Illinois, Sanitary Engineering Series, No. 41, Urbana, 180 p.

[47] Water Resources Engineers, Inc. September 1969. *Evaluation of Alternative Water Quality Control Plans for Elkhorn Slough and Moss Landing Harbor*. Presented to the California State Water Resources Control Board and the Central Coastal Regional Water Quality Control Board, 63 p.

[48] Water Resources Engineers, Inc. December 1968. *Waste Water Reclamation Potential for the Laguna de Santa Rosa* Report to the California State Water Resources Control Board, 86 p.

also of interest to know how the simulation would be changed by employing different removal efficiencies for the five constituents provided at the water and waste treatment plants and at each user's reclamation facilities. In Table 18-4 the combinations of removal efficiencies that were tried in separate computer runs are listed. The three removal efficiencies were taken as constant for all five constituents at each treatment site, although facility exists for accepting separate removal efficiencies for each treatment location, including each user's wastewater reclamation plant. (The amount of waste reclaimed by each user varied with time and ranged from 0 to 80% of the individual user's waste flow).

Table 18-5 summarizes the major findings of the quantity simulation. The monthly amounts of water in the storage reservoir are tabulated for the six demand-yield alternatives described in Table 18-2. The first, fourth, and fifth cases, n, did not provide enough water to satisfy the imposed demands, and deficits occurred in the later months. As may be noted, the other three cases, n = 2, 3, and 6, were able to satisfy all quantity demands, but with differing amounts of storage, particularly in the later months when demands were increased (see Table 18-2).

Table 18-6 indicates the variation in quality results experienced in the four trial simulations with the various removal efficiencies indicated in Table 18-4 and the six demand-yield cases from Table 18-2. The table lists nitrate concentrations merely because this was the one constituent among the five whose objective was violated most often. We have chosen to show the nitrate values in the receiving water for the 14th month merely to illustrate the extremes in spatial and temporal results at the ends of the simulations.

TABLE 18-2

Variations in Firm Yields and Demands Used in Demonstration Modeling

Case n	Import Source k	Import Source Firm Yield Q_{nk} (mgd)	User j	Demand* $QUSE_{nj}$ (mgd)
1	1	1	1	10
	2	50	2	25
	3	0	3	5
2	1	1	1	10
	2	50	2	25
	3	50	3	5
3	1	1	1	10
	2	50	2	25
	3	100	3	5
4	1	1	1	10
	2	50	2	50
	3	0	3	5
5	1	1	1	10
	2	50	2	50
	3	50	3	5
6	1	1	1	10
	2	50	2	50
	3	100	3	5

* Demands for each user were doubled for the last seven months of the simulated 14-month period.

TABLE 18-3

Quality Demands for Each User

User j	Quality Demands				
	Hardness	Coliforms	Nitrate	BOD	TDS
1	100 mg/L $CaCO_3$	5 MPN/100 ml	1 mg/L	1 mg/L	200 mg/L
2	70	1	1	1	150
3	120	0.1	1	1	400

TABLE 18-4

Combinations of Removal Efficiencies for all Constituents Used in Technical Model Demonstration

Simulation	Water Treatment Efficiency, %	Water Treatment Efficiency, %	Reclamation Efficiency, %
1	100	0	0
2	0	100	0
3	0	0	100
4	50	50	50

TABLE 18-5

Water Volumes in Storage Sub-System During Simulated Demonstration Period (million gallons)

Time Period (month)	**Demand-Yield Case, n**					
	1	**2**	**3**	**4**	**5**	**6**
1	881	881	881	267	267	267
2	1,201	1,201	1,201	3	3	3
3	502	502	502	174	174	174
4	1,263	1,263	1,263	337	337	337
5	291	291	291	192	192	192
6	655	655	655	0*	1,342	2,767
7	1,001	1,001	1,001	0	1,009	938
8	299	299	299	0	443	1,800
9	0*	1,074	2,499	0	0*	2,636
10	0	482	411	0	0	677
11	0	79	11	0	0	1,885
12	0	1,257	2,618	0	0	319
13	0	815	682	0	0	1,544
14	0	349	223	0	0	2,663

* Amount of deficit in each time period was recorded as well.

TABLE 18-6

Nitrate Concentrations of Receiving Water in 14th Month of Simulation (mg/L)

Simulation	**Demand-Yield Case, n**					
	1	**2**	**3**	**4**	**5**	**6**
1*	-	-	-	-	-	-
2	9.21	9.21	9.21	8.69	8.69	8.69
3	10.72	10.72	10.72	10.66	10.66	10.66
4	9.97	9.97	9.97	9.67	9.67	9.67

* Nitrate information not printed for this simulation.

It is interesting to note that for all simulations the fourth, fifth, and sixth cases of demand-yield combinations were productive of better receiving water quality than were the first three. For the first three cases, the demands for water by the urban users were much lower. It was also observed that in the early months in which demands were lower- than in later months, the receiving water quality was still worse. In this particular case, then, increased urbanization (or at least greater urban water demands) led to an improvement in receiving water quality. This was, of course, a direct result of the quality of the imported water that was presumed to be rather bad in the low yield source and much better in the higher yield sources. It is also interesting to note that 50% removal at all three treatment sites produced better quality than 100% reclamation alone, but worse quality than 100% waste treatment alone.

The clear implication of this demonstration is that hundreds or even thousands of alternative combinations of demands, yields, rainfalls, streamflows, treatment efficiencies, and sub-system sizes and locations could be simulated. The Urban Water Economics Model that has been described in Section 5 is intended to provide the capability to make rational decisions about "good-better-best" from among the technical alternatives evaluated with the comprehensive Urban Water Model that simulates physical behavior alone.

6.2 Economic Modeling

The economic demonstration was made for only the first case (n=l) of import source yields and user demands listed in Table 18-2. Moreover, only two quality constituents, hardness and coliform organisms, were used in the demonstration. These limitations were imposed merely to lessen computer costs and volumes of output. It seemed expedient for purposes of demonstration merely to include only those technical quantities necessary to show a range of possible variation.

Four analyses were made for both the technical simulation of the hypothetical urban system and the economic analysis superimposed on those solutions with the schedule of removal efficiencies for the three types of treatment, as shown in Table 18-7.

TABLE 18-7

Economic Modeling Demonstration Cases

Economic Simulation Case	Water Treatment Efficiency, %	Reclamation Efficiency, %	Waste Treatment Efficiency, %
1	50	50	50
2	0	50	50
3	50	50	0
4	5	50	0

Table 18-8 presents the results of the economic evaluation for the two sub-systems: Water Use and Water Treatment. It can be noted that the Water Use sub-system, whose benefits and costs were all quantity-dependent, had the same costs (C), monetary benefits (B), and total benefits (BT) for all four cases. The Water Treatment sub-system, on the other hand, had benefits and costs that were presumed to be equality-dependent, and hence the results are different for all four cases. It is interesting that the results indicate that water treatment is certainly economically justifiable, but that the addition of marginal waste treatment to reclaim municipal wastes (Case 1) produces essentially the same economic results as if no waste treatment were provided (Case 3).

TABLE 18-8

Results of Economic Modeling for Individual Sub-Systems

Economic Simulation Case	Water Use Sub-System				Water Treatment Sub-System			
	C	B/C	BT/C	BT-	C	B/C	BT/C	BT-C
1	0.037	30.0	72.7	2.64	8.25	1.73	2.11	9.18
2	0.037	30.0	72.7	2.64	0.056	-0.9	-1.10	-0.12
3	0.037	30.0	72.7	2.64	8.25	1.75	2.13	9.33
4	0.037	30.0	72.7	2.64	0.056	-4.59	-5.39	-0.36

Table 18-9 gives the accumulated results for the Water Use and Water Treatment sub-systems combined. Here it is interesting that the benefit-to-cost ratio is much higher for Cases 2 and 4, while the net benefits are much higher for Cases 1 and 3, which indicates again that the choice of economic objective is a critical step in determining which physical works alternative is preferred and hence should be constructed. It is worth noting, too, that the analysis of the two-part system has indicated some choice must be made while the Water Use sub-system results were totally indifferent to the cases tested. Also, even though the Water Use results were case-independent, the BT/C ratios for the two-part system can be seen to have not been dominated by the Water Treatment sub-system results. The maximum BT/C ratio occurred for Case 3 in the individual Water Treatment analysis (Table 18-8) while the maximum BT/C ratio for the entire system occurred for Case 2 (Table 18-9).

TABLE 18-9

Results of Economic Modeling for Water Use and Water Treatment Sub-Systems Combined

Case	C	B	BT	BT/C	BT-C
1	8.29	15.4	20.1	2.43	11.8
2	0.092	1.07	2.62	28.3	2.52
3	8.29	15.5	20.3	2.45	12.0
4	0.092	0.85	2.38	25.7	2.29

It can be concluded that whereas many more sub-systems, constituents, and users could have been examined, nevertheless, the results produced in this project are demonstrative of the concepts and indicative of the substantial gains that can be made in analysis capability. We believe that the next step that should be taken is to apply these principles and programs to a real urban region. In addition, we believe that a certain portion of that effort should be directed to the development and feasibility evaluation of applying more sophisticated systems analysis techniques, such as linear and dynamic programming optimization, coupled with simulations of physical and chemical behavior. Whereas optimization procedures have been developed primarily to determine least-cost solutions, there is no question but that they can be altered or constrained to deal with benefits as well. It is recommended that the next step in urban water systems analysis include application of particular types of these techniques, such as the out-of-kilter algorithm linear programming technique, already applied in other situations.

7.0 CONCLUSIONS AND RECOMMENDATIONS

This initial step in developing systems analysis tools for evaluation of a metropolitan water environment has been extremely enlightening. It has demonstrated that mathematical models can be built to simulate the behavior of the strung-together water sub-systems in the urban setting, from rainfall and runoff, through storage, use, and several varieties and steps of treatment, to discharge to the receiving water and subsequent exportation to the next urban "basin." Moreover, the study has shown that economic-financial analyses for each and all of the sub-systems involved can be conducted both separately and simultaneously for all alternative plans being scrutinized for technical feasibility.

But, still further, the study has indicated numerous areas of technical and even philosophical weakness. For example, after many years of investigation, the explicit relationship between rainfall and runoff in an urban setting is not known. We may never know it, and systems analysis may not help us to get much closer to knowing it. But systems analysis tools can accept and operate with the most complicated empirical hydrologic formulations that man will ever devise. It is likely that we will never understand precisely how water is used, consumed, and deteriorated by various urban water customers, but we can simulate fairly well the observed results of what apparently is happening in their homes and industries and on their lawns and streets. After years of controversy and bickering, we still do not know, nor are we able to even formulate well, what water is worth to individuals and groups. We have been almost completely befuddled in attempts to associate economic values with quality constituents existing at various concentrations in water. We have resorted to equating the value of these things to the cost of bricks and pipes and machines required to change the concentration from one level to another; but the likelihood of these component valuations expressing the true, absolute worth of quality at any level must be remote. Nevertheless, it is very important to realize that *such weaknesses prevail independently from* the *type of project analysis employed.* Neither systems analysis nor any other approach will make such weaknesses go away, and they certainly should not be hidden or overlooked in any complex evaluation.

The general public is more aware of the interconnections and interdependencies among societal and physical elements than ever before. There is a widespread recognition that single-purpose solutions resulting from single-purpose analyses can be costly at minimum and disastrous in the extreme. Terms such as "ecology" and "environment" imply to the layman the concept of a comprehensive or integrated viewpoint or approach to problem solving. Systems analysis is merely a body of formalized procedures for quantitatively pursuing that approach, and thus it has the potential virtue of treating problems in the whole rather than artificially in assumed independent, separate parts. The present study, merely a beginning for the urban water field, clearly indicates that this potential can indeed be reached, given the time, funds, and determination required. A particularly attractive feature of any such general approach, of course, is its extremely high value of transferability: development costs are commonly reclaimed in the first few applications.

Specifically, from this exercise we can conclude the following:

1) Existing systems analysis methods can be applied to simulate behavior of entire specific urban water systems and to evaluate the benefits and costs associated with every sub-system involved, individually and collectively.
2) The simple, step-wise simulation program developed in this study is adequate for description of fairly gross continuity balances. Even if we were to solve the same problem again still in a step-wise fashion, however, we would more than likely start with the "upstream" imports and rainfalls and solve the problem in the direction of the water users, checking when we got there whether or not their demands were met. If they were not met, we would go back and readjust the imports or the water treatment or waste treatment efficiencies, and then re-solve toward the demand point. The problems of non-linearities would remain, but the solution would be a bit more straightforward. A large disadvantage of this second solution sequence, however, is that the "unknown" treatment efficiencies may, in fact, be known, constrained by the existence of already-operating plants.
3) The solution to the quandary immediately above is obviously a simultaneous equation solution. A logical continuation of the present work would be the development of a much more sophisticated simulation-optimization model built on the basis of network theory and applying the "out-of-kilter algorithm" approach to what is really a linear programming problem. This technique can be used to simulate behavior on a continuous basis at a level of time detail finer than the monthly simulation reported here. The technique solves simultaneously for the "least-costly" way of delivering water from each point to the next in a network of investigative links (sub-system), the "cost" constraints being established either as technical limitations, imposed penalties for not meeting demands, or simply the cost of facilities or services.
4) The potential users that would be most favored by further development of these tools would most likely be urban water planners and managers. (Most design will probably continue to be done on an individual sub-system basis for some time). Planning and scheduling expansion of sub-systems for the future will probably be aided most by the technical simulation models and their use in sensitivity analyses, although they can also be used to simulate daily behavior for the manager responsible for allocating manpower and equipment to service numerous urban water sub-systems.[49] The economic systems analysis capability has its greatest potential in planning, where choice must be made among alternative staged development programs including components of several or many sub-systems. An extension of this work needed today (and needed for some-time in the past) would be an allocation model that specifically divides cost responsibility among the beneficiaries. Certainly, there are models today that do the arithmetic to allocate tax assessments among beneficiaries of urban facilities, but these use criteria such as property area or frontage, and not an explicit description of each user's benefit from the services rendered. Other extensions of the economic

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[49] WRE concluded this in its first report as well.

modeling that are quite within the realm of possibility include further amenity, attitudinal benefit evaluations and ecologic, environmental impact studies, which would include such measures within the economic arithmetic.

From what we have learned, and the conclusions we have drawn, we recommend the following steps for OWRR's continuing program of urban water resources research:

1) Focus on metropolitan agencies having comprehensive responsibility for at least two water-related functions throughout their regions and who, at the same time, are faced with planning and implementing in the near future a program of expansion or renewal of the their urban water sub-systems. Such agencies would likely be receptive from both staff-preparedness and impending problem viewpoints to the application of comprehensive systems techniques for alleviation of their planning load and, at the same time, for expansion of the spectrum of alternatives that can be investigated.
2) Institute a research-application project for one or more such agencies, with the work supported by consultants as necessary, to develop the most sophisticated systems models possible for metropolitan region in question, consistent with the agency's specific physical system arrangements and limitations and its social and economic planning objectives. However, strong emphasis should be placed on noting at every possible point in the development and application program those ways and means of transferring the results to other metropolitan regions.
3) Support individual projects whose *major* goal is to determine more precisely or more explicitly the behavior of particular urban sub-systems, but at least whose *minor* goal is to formulate these solutions in a fashion that will make them amenable to systems analysis adaptation and assimilation into comprehensive analyses covering more than one sub-system.

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Chapter 19

PROSPECTS FOR METROPOLITAN WATER MANAGEMENT

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Sponsored by: Office of Water Resources Research

1.0 ABSTRACT

ASCE is assisting in outlining and developing a coordinated national program of urban water resources research, including the provision of guidelines for long-range studies on urban water problems. The first phase of work was concluded with the publication of two reports.[1,2] This study of metropolitan water management, including an assessment of related research needs, is a part of the second phase of work which is sponsored exclusively by the Office of Water Resources Research, U.S. Department of the Interior.

2.0 OBJECTIVE, BACKGROUND, SCOPE AND FORMAT

2.1 Report Objective

The objective was to conduct a study of metropolitan water management, taking into account: expectations for the environment of future cities; necessary or desirable new administrative arrangements for future public services management; and the expected impact and interaction of emerging technology, urbanization trends, social goals and related matters.

2.2 Intended Audience and Study Purpose

The intended audience for this report includes managers, planners and designers of programs and facilities, and researchers in urban water resources.

The purpose of the study is to search for ways to make more effective use of technology in the improvement of urban living along with more efficient allocation and use of metropolitan total fiscal resources.

2.3 ASCE Urban Water Resources Research Program

The ASCE Program was initiated by the Urban Hydrology Research Council of ASCE. A Steering Committee designated by the Council gives general direction to the Program: S.W. Jens (Chairman); W.C. Ackermann; J.C. Geyer; C.F. Izzard; and D.E. Jones, Jr. Program Director is M.B. McPherson, at Harvard University.

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[1] American Society of Civil Engineers. September 1968. *Urban Water Resources Research.* A study for the Office of Water Resources Research, U.S. Department of the Interior, New York, New York, over 600 p.

[2] American Society of Civil Engineers. April 1969. *Basic Information Needs in Urban Hydrology.* A study for the Geological Survey, U.S. Department of the Interior, New York, New York, 112 p.

The second phase of the Program, sponsored by the Office of Water Resource Research, includes: reviewing findings of the initial report[3] to OWRR; conducting a study of metropolitan water management (reported herein); developing, collaborating and participating in research by municipalities and other organizations; initiating the design of a comprehensive simulation model for an engineering-economic systems analysis of all aspects of urban water; and conducting a feasibility study of metropolitan water intelligence systems. As for the first phase, Dr. George F. Mangan, Jr., is the technical liaison representative for OWRR on the second phase.

2.4 Engineering Foundation Research Conferences

The ASCE Urban Hydrology Research Council has conducted three Engineering Foundation Research Conferences: in 1965, 1968[4] and 1970. The first phase of the ASCE Program was essentially an implementation of the basic recommendations of the 1965 Conference, and the 1968 Conference was arranged and conducted in such a way as to provide complementary inputs for the two first-phase reports. The third Conference, held at Deerfield Academy, Deerfield, Massachusetts, July 27-31, 1970, was on "Urban Water Resources Management." One feature of the 1970 Conference was a deliberate design for providing inputs to the present report, and several of the research needs emerged from the 1970 Conference. The first draft of this report was distributed to all attendees in advance of the Conference to provide a subject reference background and to facilitate assistance in the resolution of research needs.

2.5 Pre-Report Assistance

Twenty-one eminent specialists were consulted at the beginning of the study, for the purposes of seeking general guidance and identification of the most vital references for conducting the study. The duration of the conferences ranged from an hour to a full day. Because all of the conferences were conducted before the writing of this report was commenced, it is obvious that none of the co-operators are responsible in any way whatever for any of the contents. Much more time would have been required for the study, and a far less comprehensive coverage would have been achieved, without the help of the persons consulted. ASCE and the writer are, therefore, greatly indebted to the following individuals for their invaluable guidance at the initiation of this study (affiliations cited were in effect at the time of the conferences, and order of names is alphabetical by surname):

- Mr. Harvey O. Banks, Consulting Engineer, Belmont, California
- Dr. Kurt W. Bauer, Executive Director, Southeastern Wisconsin Regional Planning Commission, Waukesha, Wisconsin

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[3] August 9-13, 1965. *Engineering Foundation Research Conference—Urban Hydrology Research* Report. Proctor Academy, Andover, New Hampshire. Co-sponsored by the Urban Hydrology Research Council, ASCE, p 26, mimeo.

[4] American Society of Civil Engineers. 1969. *Water and Metropolitan Man.* Report on Engineering Foundation Research Conference. Co-sponsored by ASCE Urban Hydrology Research Council, Proctor Academy, Andover, New Hampshire, August 12-16, 1968. New York, New York, p. 90.

- Commissioner Samuel S. Baxter, Water Department, Philadelphia, Pennsylvania
- Professor Lyle E. Craine, National Water Commission, Arlington, Virginia
- Professor Leonard B. Dworsky, Director, Water Resources and Marine Sciences Center, Cornell University, Ithaca, New York
- Professor Gordon M. Fair, Division of Engineering and Applied Physics, Harvard University, Cambridge, Massachusetts (Deceased 1970)
- Mr. Allen O. Friedland, Chief Sanitary Engineer, Department of Public Works, City and County of San Francisco, California
- Commissioner Herbert A. Goetsch, Department of Public Works, City of Milwaukee, Wisconsin
- Professor Duane Hill, Water Resources Center, Georgia Institute of Technology, Atlanta, Georgia
- Mr. Robert C. Levy, City Engineer, City and County of San Francisco, California
- Professor Ray K. Linsley, Department of Civil Engineering, Stanford University, Stanford, California
- Professor Roscoe C. Martin, Department of Political Science, Syracuse University, Syracuse, New York
- Mr. Herbert McCullough, City Engineer, City of Milwaukee, Wisconsin
- Mr. William D. McElwee, Southeastern Wisconsin Regional Planning Commission, Waukesha, Wisconsin
- Professor P.R. McGauhey, University of California, Richmond, California
- Mr. Albert J. Richter, Advisory Commission on Intergovernmental Relations, Washington, DC
- Professor Stanley Scott, Institute of Governmental Studies, University of California, Berkeley, California
- Professor Robert L. Smith, Department of Civil Engineering, The University of Kansas, Lawrence, Kansas
- Mr. S.M. Tatarian, Director, Department of Public Works, City and County of San Francisco, California
- Professor Gene E. Willeke, Department of Civil Engineering, Stanford University, Stanford, California
- Professor Joseph F. Zimmerman, Graduate School of Public Affairs, The State University of New York at Albany, New York

2.6 Study Scope

Metropolitan water management affects, is affected by, interconnects or interacts with, just about every activity of urban life. The literature on metropolitan problems is voluminous and growing rapidly. A thorough literature search would have taken much longer than the time available for the whole study. On the other hand, there are central recurring themes, opinions, observations and findings in the multitude of publications extant. As a realistic expedient, the writer sought out specific selected references having a high probability of authority or comprehensiveness or consensus of viewpoint. In this connection, the consultations cited in the section above were of inestimable help. Emphasis was placed on current or very recent references to insure a reflection of contemporary thinking.

Specialists in the many areas touched upon in this study might regard the coverage afforded their hallowed precincts as too simplistic and overly condensed. But this report is a synthesis; a root fundamental of synthesis is simplicity; and the midwife of simplicity is brevity.

2.7 Report Format

Individual readers will encounter sections that survey material with which they are intimately familiar or which contain more detail on a particular aspect than they will wish to ponder. For these reasons an attempt has been made to make each section a reasonably independent entity, with the thread of discussion and development following a continuous chain from section to section.

2.8 General Information Sources

Several bibliographic summaries of works on metropolitan issues and problems deserve mention.

The Public Administration Service has published four volumes containing listings of over 16,000 references on all aspects of metropolitan government: about 5,000 references for 1925-1955; 2,500 for 1955-1957; 6,000 for 1958-1964; and 3,000 for 1965-1967. From 1955 through 1967 the average number of references cited has exceeded 800 per year rather uniformly, as against an average of less than 200 per year for the three previous decades. The latest volume[5] "is devoted to literature published from January 1965 through December 1967. Like its predecessors, it focuses on problems of metropolitan communities and on proposed and attempted solutions. What was noted in the earlier volumes about this literature remains true: that the problems, functions, and structures of governments continue to be examined closely by authors writing in the field while studies of metropolitan politics continue to be relatively neglected. In contrast with this situation, experts are devoting their

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[5] Public Administration Service. 1969. *Metropolitan Communities: A Bibliography,* "With Special Emphasis upon Government and Politics." Supplement: 1965-1967, Chicago, IL, p. 272. Sponsored and compiled by the Institute of Governmental Studies, University of California, Berkeley; compilers—Barbara Hudson and Robert H. McDonald.

attention increasingly to the socio-economic background of metropolitan communities."

The Graduate School of Public Affairs of the State University of New York at Albany publishes a *Metropolitan Area Annual.* These volumes help to provide an overview of metropolitan developments in recent history, by including articles on federal, state and local actions that have had a significant impact on metropolitan areas. Each issue also provides directories of addresses of state agencies for local affairs, metropolitan planning commissions, metropolitan councils of governments, and other organizations of interest to those engaged in metropolitan studies. Additionally, each issue contains an annotated citation of metropolitan surveys and a selected bibliography for the year. The 1969 volume[6] has 812 annotated metropolitan survey citations and lists 425 references.

The Graduate School of Public Affairs of the State University of New York at Albany also publishes annually a *Survey of Metropolitan Planning* and issues bi-monthly a *Metropolitan Area Digest.*

In Massachusetts, the Institute of Human Sciences of Boston College has recently begun publication of a new semi-annual periodical, *Urban and Social Change Review.* The journal has two principal aims: to help widen and eventually eliminate the current bottleneck in information flows between the researcher or scholar and the urban decision-maker or practitioner; and to provide and promote some integration in the broad field of urban science and action.[7]

3.0 INTRODUCTION, SUMMARY, CONCLUSIONS, RECOMMENDATIONS

3.1 Introduction

Urban problems transcend the boundaries of the metropolis and encompass almost the full range of issues confronting society. To address urban problems is to brave almost every conceivable domestic social, economic and political question.

The nation is confronted with demands for the resolution of new social values and reversal of the trend in worsening environmental quality; and these are interrelated. For some time, we have tended to use the earth as if we were the last generation. The social and physical decay of many central cities has been accompanied by suburban growth at the expense of adjacent rural areas. Increasing urban populations and technological activity and the associated spread of urbanization have exacerbated environmental pollution. Over two thirds of the nation's population now lives in metropolitan areas. Concurrent, continuous increases in urbanization and population are expected. The evolution of the United States from an agrarian to an urban society

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[6] Graduate School of Public Affairs. 1969. *1969 Metropolitan Area Annual.* Edited by Iliana S. Hastings and Wendell G. Lorang, Jr., State University of New York at Albany, p. 229.

[7] Graduate School of Public Affairs. January—February 1970. *Metropolitan Area Digest.* Volume XIII, No. 1, State University of New York at Albany, New York, p. 12.

has seemingly outpaced the capacity for adjustment of our prevailing institutions of government.

The greatest task of local governments in the future is expected to be in the provision of new services. Environmental quality must be improved and maintained despite a primitive knowledge of environmental pollution. Urban water is an essential, finite resource, and its effective conservation and management can and must result in the enhancement of the quality of living in metropolitan areas. The role of water in metropolitan areas is thereby inextricably linked with other urban services and the major functions of local and area-wide governmental entities.

The total direct and indirect costs to the public for urban water resource facilities are expected to be on the order of $15-billion per year over the next several years, most of which will be borne at local levels of government. Water and wastewater services alone now account for about a tenth of local government expenditures.

The key descriptor for the future is intensification: of people, of activity of all kinds, and of competition and conflict. In addition to its vital roles of supporting human life and providing environmental necessities and amenities, the manner in which urban water is managed can be expected to have an ever-greater influence on national social stability and economic growth in the decades ahead.

3.2 Summary, Chapter 3: Urbanization: Growth, Problems and Trends

The character and growth of urbanization is discussed in this chapter. More intensive tensions of all kinds are encountered in the largest metropolitan areas where, perversely, a major share of all national population growth is expected to occur. About a third of the population already live in the score of largest metropolises. The amount of land occupied by the larger metropolitan areas is expected to double over the next three decades. Environmental deterioration has been linked with economic growth. Very few meaningful generalizations on economic, social, and racial disparities can be drawn about metropolitan areas and, paradoxically, diversity is the only feature common to them all. The phenomenon of growing urban frictions is manifest worldwide. Problems of administration in U.S. metropolises are often attributed to fractionalized government: the existence of a large number of independent jurisdictions. The absences of a tangible national urban policy and a viable national urban water policy have been deplored. Waste management is challenged by the need to reduce pollution in the face of exponentially rising waste production. The urban crunch will get worse before it gets better if for no other reason than the natural time lag between symptom and remedy.

3.3 Summary, Chapter 4: Human Biological and Social Needs; The Roles of Science, Technology and Private Corporations

The watchword for the 1970s is more likely to be "stability" than "growth," with the goal of maintaining equable order and a free and maximized diversity without unduly perturbating the economy. Because the biological adaptability of human beings lags behind their facile capability for social accommodation, there are inherent dangers from the continuous intensive bombardment of urban populations by adverse environmental stimuli. Although pollution is numbered among these adverse stimuli, of some importance is the satisfaction of the human hunger for associations with nature, which is itself delimited and denigrated by pollution. Thus, pollution has a multiplying adverse effect on man. Each ecocidal crisis compounds the development of others. Not only ecological but also social, economic and political spillovers must be taken into better account. The dissipation of interstitial or overhead energy in human activity seems to be increasing at the expense of productive energy. The linear view of the world in the public mind must be dispelled by facts showing the exponential changes of reality. Occupational specialization in science and technology has inhibited necessary communication with the public and the accounting for interdependencies and spillovers. Technology, simultaneously viewed as a savior and a threat, must and will increasingly reflect a greater social awareness by technologists. For a number of reasons, American business is expected to develop a much greater social involvement, with corporations becoming major vehicles for helping to bring about social change.

3.4 Summary, Chapter 5: Metropolitan Physical and Governmental Structure

Metropolitan physical and governmental structures are considered in this chapter. Urban development is marked by a diversified mixture of property and services ownership. Water represents merely a sub-system of the total, complex metropolitan system. Government, the established form of political administration, is traced from colonial times to the present, with attention accorded the derivation of local government powers and relevant attributes of the federal system. All powers from the state level downward reside with the states. The diffusion of authority or decentralization of decision-making in the federal system is exemplified by the existence of 50 states with over 80,000 local governments, including counties, municipalities, townships, school districts and other special districts. Some features of local units of government are explored, together with the roles of the branches of government, the place of "home rule" and the character of local government charters. In view of the fractionalized government in metropolitan areas it is not surprising that there is no prototype in this country of a single unit of general local government for a metropolitan area that provides all local government functions and services. Metropolitan political structure can be divided into four basic elements: the government of the central city; the local governments of the outlying suburbs; the special authorities dealing with particular functions; and the mixture of federal, state and local units that combine for the achievement of particular programs. The ability to influence metropolitan public decisions has a pluralist rather than elitist pattern. A highly simplified flow chart for visualizing the urban political system is presented.

3.5 Summary, Chapter 6: Adaptations for Unifying Governmental Structure in the Metropolis

With regard to the performance of area-wide functions, the ineffectiveness of local governments is laid to: fragmentation and overlapping of governmental units; disparities between tax and service boundaries; state constitutional and statutory restrictions; and metropolitan areas overlapping of state lines. General single-tier and two-tier approaches for governmental unification are discussed. Sixteen methods of local adaptation are ordered roughly by the severity of their general effect on local government, the less impactive half being primarily procedural and the other half being primarily structural. Each method is defined; and a specific example is cited where appropriate, with particular attention given to water-related functions. Special-purpose units are classified as school districts and public authorities, with the remainder designated "special districts." The chapter closes with a reminder of the vertical axis of common programs that link together federal, state and metropolitan local officials.

3.6 Summary, Chapter 7: Prospects for Governmental Reorganization in Metropolitan Areas

There is a widespread consensus of opinion among authorities on government that there probably is no one ideal pattern of local government structure for metropolitan communities. This view appears to be consistent with the recognition of the fact that the only true commonality shared by metropolises in this country is diversity. Not only is it expected that different metropolitan areas will use different methods of adaptation for unifying governmental structure, but some methods will be used in concert to supplement one another. Moreover, milder means, which have greater political feasibility, can serve as stepping-stones to the adoption of more comprehensive approaches later or may simply serve to relieve pressures for greater unification. Research findings have led to a growing realization among many responsible observers that the most crucial social problems of the metropolis are at best peripherally related to local government organization. Additionally, an arsenal of remedial measures less severe than reorganization have already been proposed by the Advisory Commission on Intergovernmental Relations for Utilization by the States to ease metropolitan disparities and pressures. Two-tier unification approaches that appear to have better prospects for the most general use are the urban county, the multi-purpose district or metropolitan service corporation, and the council of government. Over a hundred councils of government have been formed thus far, at a rate that was greatest in the late 1960s. The political feasibility of the urban county is greater for single-county metropolitan areas, which are generally the smaller ones. Multi-purpose districts, in addition to councils of government, are the more likely prospect for the larger, multi-county areas. The chapter closes with consideration of the intergovernmental roles of the national and state governments and some factors that should be considered for any regional approach to the amelioration of area-wide problems.

3.7 Summary, Chapter 8: Contemporary Metropolitan Water Management

The term "water management," widely used only in the last decade, has not yet acquired a specific meaning. Although rational control of water is its objective, there is no common understanding of needed policies and institutions for achieving rational control. Management functions can be roughly segregated into a formulative or strategic level, usually legislative, and an administrative or tactical level, usually executive. Public policy, the definition of public purpose by responsible authority, can be divided into three types: legislative, management and working policy. Enigmatically, administrative efficiency suggests moving decision-making on comprehensive issues (such as environmental quality) upward through the administrative hierarchy, while contemporary public demands for improved administrative accountability call for greater accessibility of the decision-maker to the individual citizen. Governmental techniques for influencing water use and development, in progressive stages, are: intelligence operations; identification and planning; regulation of water use; water resource development; and regional water distribution and disposal. Water management is an advanced stage of governmental involvement, usually characterized by the first four or all five of these stages. Management responsibilities are diffused horizontally within metropolitan areas and vertically from the local through the state and national levels. Chapter 8 reviews the status of river basin planning and development, state water resources planning and programs, and water management in metropolitan areas. In connection with the last, 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. This chapter closes with a review of the status of water quality management and a discussion of the aesthetics, amenities, and safety features associated with surface waters.

3.8 Summary, Chapter 9: Expected Directions in Metropolitan Water Management

Findings of the preceding chapter suggest substantial opportunities for justifying integrated metropolitan water management on the strength of the interdependent and interrelated nature of all aspects of urban water and of land development. Geographic service unification is often propounded as a means of capturing greater economies of scale, and functional coordination is seen as the mechanism for achieving higher organizational efficiency. The public sense of a progressive loss of individual identity and a companion apprehension of organizational bigness could conceivably constitute a formidable liability in attempts to unify the management of metropolitan water. There is widespread questioning of all institutions.

Concern over pollution has extended to an anxiety over the total environment. National policy has not been resolved to a degree consistent with the seriousness of environmental pollution as presently envisaged, and the kind and quality of metropolitan water management that will emerge in this decade will be a function of the kind and quality of policy that emerges.

The issue of "metropolitan government," discussed at some length in earlier chapters, is reviewed in Chapter 9. The theory of retaining the units of local government but attempting to find methods for coordinating their efforts seems to endure and prevail. Thus, the chapter emphasis on incremental alteration, voluntary cooperation, grant-in-aid inducements and tax reforms, all with an eye to political feasibility. There is broad consensus on physical metropolitan area planning. However, the incorporation of social planning is highly controversial, and there appears to be no reason to expect any change from the traditional planning approach on social matters in the next few years. The attitude towards distribution of local government functions appears to be hardening on a two-pronged approach: assignment of each function to the lowest and smallest governmental entity qualified to perform it, with assignment of area-wide functions to metropolitan agencies. This orientation emphasizes the achievement of certain socio-political goals rather than increased administrative efficiency through reallocation of functions. In fact, the quality of public services may well be deemed more important than cost minimization through improved administrative efficiency for at least some functions.

Chapter 9 also recollects the existence of regional water differences and suggests function allocation and water management criteria. The prospects for more widespread administration of water and wastewater services on an area-wide basis will be improved if considerations of public health, other water uses, planning, and guiding sound development, are brought into the picture—because comprehensive approaches cannot be justified on an economic basis alone. It appears that the present trend towards more "blue-green" development favors, beyond the normal provision of amenities, the satisfaction of some local aesthetic and even recreational needs.

As an alternative, supplement or complement to integrated water management, possibilities for integrated waste management are considered. In view of the prevailing pessimism on the probability for widespread integrated water management, the immediate prospect for such a more ambitious scheme at larger scale appears to be dim, although there are municipal precedents where organizational orientation has been shifted from a purpose to such a broad process alignment, and integrated waste management might well be widely practiced at some point in the future.

Two problems that have to be overcome in order to enlarge service areas are the method of financing by levies and/or charges, and the reconciliation of disparate service and financing jurisdictions. Large private corporations, like many large cities, are confronting bureaucratic sluggishness that more than offsets economies of scale. As a consequence, companies will decentralize to achieve greater flexibility, a forecast seemingly consistent with expected directions in metropolitan governmental organization. Efficiency in terms of economy of scale is as difficult to define and quantify in public services as in the variegated private sector and generalizations from specific cases are made at extreme hazard. The trend towards "regionalization" of public wastewater treatment facilities is based heavily on considerations beyond or in addition to economies of scale.

Chapter 9, and hence the text of this report, draws to a close with a warning on the inherent bias of specialists and a proposed way for the states to better utilize the services of firms of consulting engineers. In the closure to Chapter 9 it is concluded that despite the enthusiastic advocacy of integrated area-wide metropolitan water management commonly advanced by water resource experts, because of the paucity of facts demonstrating a superior socio-political-economic efficaciousness for a more comprehensive approach, the only verdict that appears appropriate is the Scottish verdict of not proven.

3.9 Conclusions

Integrated management would presumably include: water supply, treatment and distribution; wastewater collection, treatment and disposal; drainage and flood mitigation; and perhaps water-based recreation and aesthetics. Area-wide management would presumably include all or nearly all of the geographical territory of a metropolitan area over the foreseeable future. No example exists in the United States of a metropolitan agency which satisfies both of these presumptive definitions and all but a very few cases are far from the mark. That is, integrated area-wide management of metropolitan water resources is a concept, not a contemporary actuality.

As this report emphasizes, there are water resource specialists who see in integrated area-wide management the ultimate for capturing economies of scale and for sharpening administrative efficiency. Another view, which reflects a concern for the increased vulnerability to disruption that attends extended interdependence, holds that service reliability might be better secured by the desegregation of administrative jurisdictions, responsibilities and authority. "...the highly complex 20th century city is the most vulnerable point in man's terrestrial sphere."[8] The middle ground is represented by those who favor several large fairly independent (and therefore secure) zones within a metropolis, with area-wide interests served by means of some sort of coordinative mechanism.

This range of views is shared by particular specialists in other fields and services, notably in governmental science and transportation. However, the pros and cons of integrated area-wide management versus the status quo versus zonal administration have been only very subjectively evaluated, and existing coordinative mechanisms are seldom touted with anything like an enthusiastic consensus. Further, even champions of diffused autonomy seem to favor some form of area-wide revenue and cost sharing. More importantly, at this stage of public awareness, few fail to

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[8] November 2, 1970. "The City as Battlefield: A Global Concern." *TIME.*

acknowledge the interconnectedness within "the environment" or "the ecology" of a metropolis. Thus, we find widespread acceptance of an area-wide approach to the planning function of metropolitan water management, while opinion is divided on implementation and even more so on operation.

Turning now to management tools, engineering-economics systems analysis on a metropolitan scale of all aspects of urban water resources has been found to be feasible,[9] the design of a comprehensive simulation model has been initiated,[10] and follow-on development is under way by one or more metropolitan agencies. Because the degree of detail required for system accounting and simulation is least for the planning function and greatest for the operating function, continued capability development will favor short-term returns for planning and long-term returns for operation. Especially important is the fact that present urban water exigencies emphasize the resolution of management strategies over tactical devices, and the prospect that short-term benefits predominantly will accrue to the planning function is, therefore, particularly fortuitous. (Of course, returns to implementation and operation will also be realized, but these will be mostly cumulative over time.) Looking at a much larger scale, public demands for multi-sided solutions to broad problems or collections of problems, supplanting compartmentalized solutions to limited problems, have been directed at all three levels of government in a quickening tempo. Requirements here, too, emphasize management strategy, and early successes in urban water planning could very conceivably lead the way for some other services.

The purpose of further development of systems analysis as a tool should be the enhancement of management capability through the addition of new techniques that provide a greater insight with respect to the facilities and services for which managers are responsible. The replacement of old techniques is not an issue because the fundamental objective should be to maximize the value of that which managers already know. "Much has already been done, but much more must still be done with current management tools wielded by existing institutions."[11] The degree of involvement by upper management in the enterprise will determine its eventual success because the entire thrust of the exercise <u>must</u> be management-oriented. Use of a comprehensive systems approach will provide an overall means for organizing management decision-making processes and the technical work, and will serve as a framework for quantification techniques such as dynamic programming and cost-benefit analysis.

Wherever integrated area-wide management is reasonably approximated, operating tactics will become more complicated; and for efficient operation much more and better field intelligence will be required, on an instantaneous-availability basis. Of

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[9] American Society of Civil Engineers. September 1968. *Urban Water Resources Research.* First Year Report to the Office of Water Resources Research, U.S. Department of the Interior, New York, New York.

[10] Water Resources Engineers, Inc. September 1970. "Systems Analysis for Urban Water Management." A report prepared for the Office of Water Resources Research, U.S. Department of the Interior, through the ASCE Urban Water Resources Research Program, Walnut Creek, California.

[11] Council on Environmental Quality. August 1970. *Environmental Quality.* First Annual Report, GPO, Washington, DC. Transmitted to the Congress.

grave concern is the maintenance of public service-standards in the face of increased operating complexity. Private corporation production reliability-standards are analogous to public service-standards. As industry managers seek to gain greater productivity through more integrated operations and the economies of scale their reliability standards intensify as they try to insure the success of their enormous financial commitment,[12] and here, too, the quality of process control assumes heightened importance.

The term "metropolitan water intelligence system" was coined by this writer[13] to describe the hardening concept of multi-service automatic operational control, wherein field intelligence on all aspects of urban water might be acquired, including: precipitation; stream stages and flow rates; water and wastewater treatment facilities; water demands and distribution system rates and pressures; settings of regulating structures; quality parameters for watercourses and impoundments, and within conveyance systems; and the status of special facilities such as recreational ponds and lakes. Possibilities exist for incorporating non-water-related service intelligence, including traffic and air pollution monitoring, because these are affected by precipitation and the trend for their control is towards a centralized operation. In its ultimate form, the intelligence system would be in the closed-loop mode and would be computer centered. Using field intelligence as inputs, the computer decision program would resolve best service-least operating cost options, taking into account estimated reliability and risks, and would actuate field regulating and control facilities to approach elected option states. Feedback features would be such as to permit manual supervisory intervention at any time.

Serious consideration has been given to the development of an automatic operational control of water distribution systems capability,[14,15] but even this single-service objective has not yet been achieved in practice; however, tangible progress is being made.

The foregoing remarks on management tools imply a rough correlation between the required degree of tool sophistication and the level of complexity of the institution being managed. While ultra-concepts of tools and institutions may be more readily perceived than imperfect actualities and in some ways might be more easily defended, as with governmental reorganization in metropolitan areas (Chapter 7) "there are halfway grounds which permit the maximum of freedom consonant with the minimum order necessary to transaction of the public's business."[16] A remarkable phenomenon of the last decade has been a gradual shifting almost everywhere from a

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[12] The Editors. October 17, 1970. "The Trillion-Dollar Economy." A Special Report in *Business Week.*

[13] American Society of Civil Engineers. April 1969. *Basic Information Needs in Urban Hydrology.* A Study for the Geological Survey, U.S. Department of the Interior, New York, New York.

[14] Brock, D. April 1963. "Closed-Loop Automatic Control of Water System Operations." *Journal American Water Works Association,* Volume 55, No. 4.

[15] McPherson, M.B. November 1967. "Water Distribution System Simulation for Automatic Operational Control." *Water and Sewage Works,* reference number.

[16] Martin, R.C. 1963. *Metropolis in Transition: Local Government Adaptation to Changing Urban Needs.* GPO, Washington, DC.

project, mission or unifunctional approach towards a process approach, e.g., greater appreciation for systems analysis methods, formation of environmental conservation and protection agencies, the ecology movement, and emerging views on governmental revenue-sharing.

Evidence of the multifarious character of our metropolises appears throughout this report. Because the only commonality of metropolitan areas appears to be their diversity and very few probably share absolutes, it is important to develop techniques, tools, and methods with some deliberate attention accorded the transferability of their basic features. Specificity greatly intensifies in their application to a given metropolis. "In addressing such distributed or regional problems as transportation, air pollution, waste disposal, control and prevention of crime, what is typically needed is a combination of centrally directed research broadly oriented to the problems, and locally directed R & D that views the problems in terms of specific local constraints."[17]

There is a modest history of quixotic attempts to harmonize tools and urban problems by downgrading or otherwise redefining problems until they meet the limitations and constraints of the tools. Emphasis should surely not be on the reformation of problems themselves but on the various processes in ways that will assist managers to do more effective management. That is, to reiterate somewhat, success should be defined not by the artful elegance of a tool but by the extent of its usefulness to the manager served! The Urban Research Policy of the National League of Cities specifies that: "Urban research must relate to urban decision-making..... effective application of such research must include positive involvement of local officials in the planning, conduct and translation to use."[18] *Ecce signum*—behold the sign.

Past levels of research have been abysmally low relative to both the scale of investments and the scope of planning and management responsibilities. The greatest concern will increasingly be on the quality of water. This concern is intimately related to acknowledged imperatives of aesthetic enhancement, expansion of recreational opportunities and more extensive availability of waterfronts for public uses. The need for a much greater research effort, already obvious, will accordingly escalate to a more crucial stage as planning and management inevitably become more complex.

"Technologies of project planning, engineering, and construction for urban public facilities have lagged throughout the world due to limited commitment of resources for research and development."[19]

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[17] National Academy of Sciences-National Academy of Engineering. 1969. *The Impact of Science and Technology on Regional Economic Development*. NAS, Washington, DC.

[18] National League of Cities. 1970. *National Municipal Policy* (adopted at the 46th Annual Congress of Cities, December 1-4, 1969, San Diego, California). Washington, DC.

[19] Walsh, A.H. 1969. *The Urban Challenge to Government: An International Comparison of Thirteen Cities*. Frederick A. Praeger, New York, New York.

3.10 Federal Government Research Priorities

The recommendations of the ten-year program of federal research developed by the Committee on Water Resources Research of the Federal Council for Science and Technology that were included in a well-known 1966 report[20] are being revised.[21] Although major areas of research concentration for the next half-decade are presently under study, five specific problems have already been singled out which "warrant immediate increased research support." All five problems relate to metropolitan water resources. Abstracted portions of the Council's appraisals of these five problems follow.

<u>Managing metropolitan water systems</u>. (The term "systems" refers to all urban aspects). " ...The problem encompasses: An inadequate hydrologic database, relatively crude models for design of regional systems, uncertain public and private water resource investment policy, political and institutional obstacles to effective planning and management, shifts of population groups within the metropolitan area, and integration of water systems with other elements of urban planning... "

<u>Improving regional water resource planning and management</u>. " ...Two areas in particular have emerged as significant research needs: A procedure for analyzing and evaluating competing social objectives; and a procedure for deriving maximum beneficial use of the powerful technique of systems analysis in the management process....."

<u>Controlling pollution caused by heated water discharges, oil, and sediment</u>. "The national program of water quality improvement faces many difficulties including the still unsolved problems of eutrophication, agricultural land drainage control, combined sewer overflows, storm sewer pollution, and industrial wastes. In addition, three problems of great significance have emerged in recent years: thermal pollution, oil pollution and sediment pollution ..."

<u>Protecting the public health</u>. " ...The research need extends particularly to the following: Development of new and improved analytical methods; improvement of epidemiological and toxicological water hygiene surveillance techniques; (and) development of new and improved treatment methods to remove impurities which are unaffected by currently available treatment technology ..."

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[20] Federal Council for Science and Technology. February 1966. *A Ten-Year Program of Federal Water Resources Research,* Committee on Water Resources Research, GPO, Washington, DC.

[21] Federal Council for Science and Technology. December 1969. *Federal Water Resources Research Program for Fiscal Year 1970.* GPO, Washington, DC.

Predicting ecologic change. "Predicting the response of water-related ecosystems to man-induced environmental change is fundamental to planning for resource use—but our ability to make such predictions is crude. The subtlety and variety of ecologic responses to man-made environmental change are great, indeed. They emphasize the need for a generalized, comprehensive theory which could serve as a framework for detailed qualitative and quantitative predictions as to how ecosystems may be expected to react to specified alterations in the environment... "

The Council's purview extends to all water resources research performed or sponsored by the federal government. Although state and local governments and private corporations and organizations also sponsor and/or perform a sizable portion of the total urban water research effort, federal leadership, such as in the focusing on priorities exemplified above, is of crucial significance. Leadership, and performance everywhere, is inhibited by the fact that "national objectives in water resources development have not been precisely enunciated."

For the recommendations that follow, and conclude this chapter, no attempt is made to suggest or even imply which agencies of the three levels of government or which private organizations should sponsor and/or perform the research. Public works officials of principal cities have cautioned the writer on two requirements peculiar to the acquisition of financial support for research from local governments: the proposed work must be at a level that can be sold to those review authorities with fiscal veto power, and the results must include a contribution to the resolution of immediate problems. There are organizations such as the American Public Works Association Research Foundation that can pursue research on subjects of nationwide interest with the financial support of an *ad hoc* consortium of departments from several cities. At the state level, the water resource research institutes located in universities of each of the fifty states and Puerto Rico (formed under the Water Resources Research Act of 1964 and its amendments) can facilitate urban water research of a statewide interest. The new leadership in urban water research at the national level, particularly by the Department of the Interior, should be met in spirit if not in kind by local and state governmental agencies, and there is evidence that this is beginning to happen. Limitation of the duration of projects is a liability of present research support requirements because it results in a discontinuity of research; and inabilities to fund long-term projects are said to inhibit speculative research of the kind needed to attack some of the more difficult problems.

3.11 Recommendations: Introduction

A considerable number of research needs relating to urban water management were catalogued in earlier ASCE reports. Those recommendations have recently been updated and extended in a much broader framework.[22] The recommendations that follow assume the above-mentioned needs citations as prologues, and emphasize particulars that evolved in conducting the present study and that were brought out or

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[22] Office of Water Resources Research. (in press [1970]). *An Urban Water Resources Research Program.* U.S. Department of the Interior, Washington, DC.

amplified in the July 1970 Engineering Foundation Research Conference on "Urban Water Resources Management." "By far the most attention was devoted to the frustrating problems in decision-making, especially those stemming from the competition between technical and social alternatives and solutions... Of basic concern to many conferees was the manner in which technical and social alternatives tend to dominate the decision-making and management systems to the exclusion of environmental and ecological alternatives and solutions."[23] An eminent conference participant noted that the first draft of the present report overlooked specific research recommendations on ecological consequences of human activity.[24] As an expedient, recommendations have been subjectively divided into three parts: ecological-environmental, institutional-organizational, and fiscal-economic.

While the subjects singled out below should be indicative of some principal research needs, the reader will infer numerous related needs from concepts explored throughout the succeeding chapters.

3.12 Recommendations: Ecological-Environmental

" ...Water, land, and people should interact in ways that either preserve natural ecological balances or that compensate properly for their alteration by producing new and workable equilibriums... By very definition, an urban area is a disruption of the natural environment. The larger or greater the urbanization, the greater the disruption ..."

The Engineering Foundation Research Conference on "Urban Water Resources Management" revealed dramatically that for all practical purposes we really know very little about "ecosystems, the urban ecosystem, and the urban human ecosystem."[25]

"Until this point in history the nations of the world have lacked considered, comprehensive policies for managing the environment."[26] Good management depends on knowledge about the functioning of ecosystems, but present knowledge is considered to be very scanty. "Research work should be promoted to assess methods of management which avoid deterioration and impoverishment of natural resources."

Listed first among the seven major present and future environmental needs defined by the Council on Environmental Quality is "a conceptual framework" that hinges on the answers to such questions as: "How much value do Americans place on the natural environment as against the man-made environment of cities? How much do people value aesthetics? Do they agree about what is aesthetically desirable? These, and a

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[23] Wright, K.R. November 1970. "Conference Held on Urban Water Resources Management." *Civil Engineering.*

[24] Reiterated in private communication of August 4, 1970 from William Whipple, Jr.

[25] From unpublished conference summary comments by David R. Dawdy, July 31, 1970.

[26] United Nations Educational, Scientific and Cultural Organization. 1970. *Use and Conservation of the Biosphere.* UNESCO, Natural Resources Research X, Paris, France.

host of similar questions, must be raised when trying to align priorities for coping with environmental decline."

Specific public behavioral enigmas are the questions of how to obtain compliance to meet a societal goal already reinforced by public policy, as in water quality control, and how to obtain acceptance, as for reuse. What are feasible alternative measures for securing compliance and acceptance? In broader terms, improved techniques are needed for the resolution of multi-faceted conflicts among claimants to the resource.

Behavioral sciences, such as psychology and sociology, frequently have been urged to supply basic data and background information on public value-systems involving natural resources. More specifically, hope has been expressed that the social sciences can help in conceptualizing the water pollution problem in ways that will contribute to the identification of those types of physical, economic and social knowledge that are "basic to intelligent policy."

In the planning and development of water resources, the engineer frequently laments the inhibiting influences of nebulous policy and an ambiguity of social values, whereas social scientists often imply, in something of a counterproposal, that development and presentation of wider spectra of project and program alternatives by engineers would help them structure value paradigms that would in turn impact on the resolution of public policy. This seeming circle of shifted responsibility might be broken by jointly engaging in experiments to explore the character of alternative plans in given places which, as sets, have successfully led to resolution of implemented plans and programs, and which, therefore, by definition, have contained socially and politically feasible elements. Thus, indirect, empirical measures of social values as they relate to urban water could thereby be derived.

On the other hand, results of research under way on attitudes of individuals and various groups towards pollution, the environment and nature, could lead to more basic understandings of value systems for water applications than findings from exclusively water-oriented attitudinal studies. Regardless of the type of studies pursued, incorporating the testing of specialists' attitudes could be very enlightening, including attitudes of biologists, social scientists and engineers. The "eyes of the beholder" are often those of such specialists because they help form public opinion on issues affecting their field. In fact, inclusion of motivational studies might help in differentiating areas of bias and skewed emphasis of individual specialists and whole disciplines. A common mistake is to assess the attitude/motivation of the "public" and the "decision-maker" and take for granted that the assessors and the specialized users of the assessors' results will not distort information by modulating it through their own value systems.

Human occupancy in metropolitan areas exhibits a dichotomy of dispersion and concentration. Numerous urbanologists, noting the inordinately high cost of extending underground services in sprawling suburbs, have frequently advocated the rigid regulation of water main and wastewater sewer extensions as a device for discouraging excessive dispersion and encouraging compliance with regional plans.

Modest use of this approach has encountered unspectacular success in the United States, reflecting an aversion to social planning (Chapter 9). In contrast, the Province of Ontario is imposing controls governing land-use and population density on its 900-odd communities: "Initially, control will be exerted by giving or denying permission to localities for water and sewage systems."[27] Few states seem to be prepared to take such comparatively drastic measures, and one view holds that "water supply policy is a very weak weapon in the struggle for more rational land-use" ... and "it is futile to expect to do indirectly what cannot be done directly."[28] However, there are socio-biologic problems from the concentration of people in intensively urbanized centers, the relief of which does not necessarily depend on major change of state government postures.

Study of "population density and overcrowding as functions of human behavior and well-being" has been cited as a "vital ecological need," with civil engineers "sorely in need of such guidance in pursuing their professional activities in land-use planning and project development."[29] This view is shared at least in terms of its public health aspects: "Although research on crowding, congestion, and isolation in physical space is a field not usually considered the responsibility of health agencies, there is evidence that this represents a major health problem, particularly in terms of behavioral disturbances."[30]

The same public health task force has found an urgent need for a national forecasting program "capable of timely and effective warning of technology-induced perturbations of any environmental factor which may have health implications." Whereas existing forecasting schemes emphasize emission sources and levels rather than extent of human exposure, the program recommended would include community hazard evaluation systems that would feature the use of some of the newer mathematical models for predicting the transport, storage and reactions of pollutants in the environment.

3.13 Recommendations: Institutional-Organizational

Several years ago, an analysis of urban growth patterns and trends noted that the physical scale or extent of area-wide coverage at which a significant degree of

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[27] "Ontario Maps the Way to Metro Reform." *Business Week.* November 21, 1970.

[28] Walker, G.M., Jr. and N. Wengert. July 1970. "Urban Water Policies and Decision-Making in the Detroit Metropolitan Region." *Office of Research Administration Report 01351-1-F.* University of Michigan, Ann Arbor, Michigan.

[29] Wisely, W.H. December 1970. "A Word with the Executive Director About ... Population and People." *Civil Engineering.*

[30] Task Force on Research Planning in Environmental Health Science. 1970. *Man's Health and the Environment—Some Research Needs.* U.S. Department of Health, Education and Welfare. GPO, Washington, DC.

integration of various services and functions can or should take place remained to be identified.[31]

Much better knowledge is needed now on the types of governmental organization that would be suitable for managing the entire urban water spectrum, together with proper methods for developing charges for each service. The desirable, politically feasible organizational arrangements for comprehensive water management in metropolitan areas appear to be quite limited in number. The appropriate focus for research, therefore, should be on the assets and liabilities of water management in terms of size of geographical territory and number of water-related functions to be administered, giving due regard to vulnerability to disruption induced by interdependencies. There is a possibility that, for some years, integrated area-wide management of metropolitan water resources may continue to be a unifying concept or principle, while failing to become a formal organizational actuality.

Intergovernmental and metropolitan organizational devices for supplying urban water should be catalogued, and the kinds of financing employed, institutional arrangements used, etc., should be studied—there are probably hundreds of arrangements nationally, running across and through units and levels of government.[32] A good study example is the recent analysis of Detroit Metropolitan Water Services.

The Advisory Commission on Intergovernmental Relations has "recommended that each state undertake a comprehensive study of all special districts to ascertain their number, type, function, and financing."[33]

"There is a paucity of published material on transfers of metropolitan functions to the States."[34]

Speaking of multiple-purpose river-basin developments: "Retrospective studies (post-audits) of existing projects are rare. A few are now under way. They should disclose how multi-use principles, built into physical structures, are matched against actual management practices. Resistance is great against such inquiries. They should be pursued, not only to disclose error, but also to provide realistic experience to strengthen future plans.[35] It has been argued that urban policies and goals are less specifiable than those for river basins, and that new or revised urban bases are

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[31] Wurster, C.B. 1963. "Form and Structure of the Future Urban Complex." *Cities and Space, the Future Use of Urban Land.* Edited by L. Wingo, Jr., Resources for the Future, Inc. via the Johns Hopkins Press, Baltimore, Maryland.

[32] The essence of a suggestion by Professor Roscoe C. Martin during an interview at Syracuse, New York, November 10, 1969.

[33] Advisory Commission on Intergovernmental Relations. October 1969. *Urban America and the Federal System.* Report M-47, Washington, DC.

[34] Advisory Commission on Intergovernmental Relations. June 1962. *Alternative Approaches to Governmental Reorganization in Metropolitan Areas.* Report A-11, Washington, DC.

[35] Wolman, A. February 1970. "Multiple Purpose River Development." *Journal American Water Works Association*, Volume 62, No. 2.

needed,[36] which suggests why reservations are often expressed on the degree of parallel between river basin management and metropolitan water management. Regardless, the proposal for post-audits certainly is applicable to local government projects."

"The resources in all institutions of higher education should be utilized in analyzing and developing proposals for coping with urban problems. Universities and colleges in the states should redesign educational offerings to make them more directly relevant to current concerns."[37]

3.14 Recommendations: Fiscal-Economic

The content of papers presented at a conference sponsored by the Committee on Urban Economics of Resources for the Future embraced a variety of urban water resource issues and several areas of needed research were recognized. A few examples will be cited. "There has been surprisingly little systematic empirical investigation of the effect of user charge financing on the output of services."[38] Studies of the relations between requirements, capacities and growth of urban areas have seldom been based on engineering data.[39] (An interviewee at the beginning of the present study suggested analysis of the relations between types and costs of services in terms of occupancy densities and density gradients and land-use patterns.) " ...there is no body of studies of demand for urban public services... Need rather than demand has been the more traditional basis of planning for public services."[40] "There is no provision in the American system, in either the private or the public sector to supply basic or applied research to the problems of cities."[41] However, broad programs of urban water resources research are emerging, particularly in the U.S. Department of the Interior.

"... There are inadequate data on supply costs for the multiple dimensions of water services such as peak and off-peak usage, fire protection, and for customer groups... Economists can provide guidelines for attaining an efficient pricing of urban water and improving existing rate-setting practices."[42] A summary of findings from an ongoing research project revealed that: "Useful demand models for aggregate municipal demand were developed, but the effect of price could not be measured."[43]

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[36] Flack, J.E. September 21, 1970. "Urban Water—Multiple Use Concept." A paper presented at the *American Water Works Association-WPCA Rocky Mountain Sections Annual Meeting.* Vail, Colorado.

[37] The American Assembly, Columbia University. 1969. "The States and the Urban Crisis." *Report of the 36th American Assembly,* October 30—November 2, 1969, Harriman, New York.

[38] Netzer, D. 1968. "Federal, State, and Local Finance in a Metropolitan Context." *Issues in Urban Economics.* Edited by H.S. Perloff and L. Wingo, Jr., Resources for the Future, Inc., via the Johns Hopkins Press, Baltimore, Maryland.

[39] Hirsch, W.A. "The Supply of Urban Public Services." *Issues in Urban Economics,* ibid.

[40] Margolis, J. "The Demand for Urban Public Services." *Issues in Urban Economics,* ibid.

[41] Campbell, A.K., and J. Burkhead. "Public Policy for Urban America." *Issues in Urban Economics,* ibid.

[42] Mann, P.C. September 1970. "A New Focus in Water Supply Economics—Urban Water Pricing." *Journal American Water Works Association,* Volume 62, No. 9.

[43] Boland, J.J. July 28, 1970. "Price-Demand and Demand-Cost Functions for Municipal Water Systems." A paper presented at the *Engineering Foundation Research Conference on "Urban Water Resources Management."* Deerfield, Massachusetts.

As noted by interviewees at the beginning of the present study, a liability of urban water services is their "low visibility" compared with other public services. The manner in which the public regards urban water resources thusly injects an unfavorable bias in attempts to make trade-offs between water and other urban services and in attempts merely to define the rank and role of water among all services.

The often-gross misallocation of water supplies among users and between users has been traced to discriminating and subsidized pricing patterns, and defective systems of water rights.[44] The latter suggests a "clear need for research into the legal aspects of the ownership of water and their influence on the management of water in the best socio-economic and political manner. Jointly pursued research by the Bar, engineers, sociologists, economists, planners, etc., to bring the law abreast of the best current technologic knowledge could probably favorably influence urban water management."[45]

The private investor, governmental legislator and executive alike need to be able to find out how one unit of government compares with another in the adequacy and cost of services being provided.

"In effect what is needed is a 'Dun and Bradstreet' for local government 'functional health' as there has been one for fiscal health for a long time. To achieve greater objectivity the Commission has suggested that this effort should be undertaken by a non-governmental organization and should look toward the establishment of optional standards, the collection and analysis of data, and periodic publication of comparative figures."

" ... public service differentials between political entities are surely of notable proportions and figure significantly in selecting a home site. Similarly, to the business firm, water and sewage disposal, police and fire protection, transportation, and education are factors of production, directly or indirectly, and the tax rate is their price. Surely, intercity comparisons are made by business firms selecting a location for a new plant or store, not just on a price basis, but also to find the most appropriate mix of public services for a particular operation."[46] This suggests the desirability of extending the scope of the recommendation immediately preceding to a regional scale.

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[44] Jens, S.W. May 1963. "Some Notes on Approaches and Viewpoints of Private Agencies to Metropolitan Water Management." A paper presented at the *Engineering Foundation Research Conference on "Urban Water Resources Management.* Deerfield, Massachusetts. (Referring to views expressed in *Water Supply: Economics, Technology, and Policy*, by J. Hirshleifer, J.C. DeHaven, and J.W. Milliman. The University of Chicago Press, 1960; which is summarized briefly in "Water Supply Economics, Technology, and Policy," by J.C. DeHaven. *Journal American Water Works Association*, Volume 55, No. 5, May 1963).

[45] Private communication of August 7, 1970 from S.W. Jens.

[46] Thompson, W.R. 1965. *A Preface to Urban Economics*. Resources for the Future, Inc., via the Johns Hopkins Press, Baltimore, Maryland.

"Whatever the future may bring, a good rate of return can be realized now from an investment in research into intrametropolitan area, intergovernmental economic relations, ranging from questions of fact and equity, such as the balance of external costs and benefits between political subdivisions to questions of policy and efficiency, such as the economics of pricing intergovernmental services."

Upgrading the quality of public servants and making government "more business-like" have long been advanced to surcease fiscal pressures. Noting the very large and still growing budgets of state governments and public concern about the effectiveness of more expensive state programs, the adaptation of federal budget analysis and budget programming techniques in the planning and monitoring of state programs has been advocated for Massachusetts.[47] These techniques are "concerned with planning for, identifying, and measuring the results of programs that spend public money. And they are concerned not just with how much more money is needed this year, but with what was accomplished with last year's money and what the program is apt to cost two to five years hence." To what extent are federal budget control techniques being used by states and to what extent could they or should they be adapted by state water resource, environmental and conservational agencies? This appears to offer a good opportunity for translating some of these nationally financed developments for the benefit of the states.

"One of the problems most in need of thorough, comprehensive study in the field of municipal government is that of tort liability, or general liability, insurance carried by hundreds of municipalities." ... However, in 1955 the "15 largest cities had no liability insurance, since they are in a position to meet their obligations for torts out of their regular annual appropriations."[48]

By way of re-emphasis: technical research on urban problems must be accompanied by improved procedures in management and finance; but it is concluded that although basic research may provide a real breakthrough in urban water knowledge there is need to apply more fully what is already known.

4.0 CONTEMPORARY METROPOLITAN WATER MANAGEMENT

4.1 Introduction

Government is the established form of political administration. The authority for all levels of government in the United States resides in the Constitution. Through the nineteenth century water needs were met mostly by local units of government as one of several housekeeping functions permitted them by the states, the state role at that time being largely regulatory. As the country grew and became a more interdependent whole, water problems and conflicts enlarged in geographic scale and economic scope and the national government increasingly asserted its superior legal authority to assuage conflicts and to facilitate water resource development. The

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[47] Casselman, R.C. November 28, 1970. "State Needs a Score-Keeping System." *The Boston Globe.*

[48] Baxter, S.S. March 1968. "Effects of Urbanization." *Water Resources Bulletin,* Volume 4, No. 1.

arbiter role of central governments in resolving water conflicts can be traced at least as far back as ancient Mesopotamia. The word "rivalry" derives from the Latin word for "stream," and the philosophical roots of our adversary system of law extend back to ancient civilizations.

Jurisdictional authority over water is complicated by the migratory nature of the resource and by public dispute over the cost of certain water services that have defied quantitative pricing.[49]

“Historically, the activities of most of the states in the water resource field have been largely confined to one or more of the following: administration of the acquisition and adjudication of water rights, basic data collection, investigations, planning, review of federal reports, and pollution control and other activities to safeguard the public health and safety ... A careful study of the United States Constitution, federal statutes, and Supreme Court decisions leads inescapably to the conclusion that the dominion of the states extends only to situations where Congress has specifically delegated the authority to control and administer water resources to the states or where Congress has not yet chosen to act. The proportion of the nation's water resources which is clearly under the complete jurisdiction of the states is relatively small.”[50]

Paradoxically, non-agricultural demands for water services and capital investments in water facilities predominantly exist in the metropolitan areas but the broad base of authority resides elsewhere: primarily with the national government and secondarily with the states. However, a number of metropolitan areas reach well beyond their borders for a supply of water and the impacts of air and water pollution burdens of a metropolis often spillover into communities and other zones some distance away.

State activities have focused on allocation, regulation and facilitation of local activity, with some states recently giving attention to overall water resources planning and the development of water projects that were beyond the capabilities of local units. "The federal government has been responsible for most multi-purpose river basin developments. Federal agencies also loom large in navigation, flood control, irrigation, sewage treatment assistance, pollution control and, more recently, in water use for recreational purposes."[51]

4.2 Water Management

“In the United States, as in Western Europe, the term water management has come into currency during the last decade, as efforts were made to obtain more complete and rational control of water use and development. Yet the term has not been given

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[49] Smith, R.L. November 1967. “Total Management of Water Resources.” *Journal American Water Works Association*, Volume 59, No. 11.

[50] Banks, H.O. January 30-31, 1968. “State Agency Water Resources Research Needs.” Report on *Third Annual Water Resources Research Conference*, sponsored by OWRR at Washington, DC.

[51] Advisory Commission on Intergovernmental Relations. 1966. *Metropolitan America: Challenge to Federalism*, Report M-31, Washington, DC.

specific meaning, and there is no common understanding of policies and institutions requisite to achieving such a goal."[52]

Management functions can be divided into two levels, formulative and administrative:[53]

> Formulative Level—establishing overall goals; and arriving at policies for their attainment.
>
> Administrative Level—forecasting future needs; making plans for meeting forecasts; mobilizing resources to carry out the plans; and actually supervising or operating the means or facilities developed.

Water resources fall within the public sector directly or by public franchise, and hence under the political structure of government. The formulative level has to do with policies or expressions of the state will, usually a legislative prerogative. Policy is the definition of public purpose by responsible authority. The implementation of policy is usually an executive, or administrative function. At all levels of government there are continuous heavings and haulings between the legislative and executive branches on specific questions of policy formulation. Thus, although "administration" is commonly defined as "professional management," the distinction can be drawn only at some hazard.

There are ambiguities, also, in defining types of policy. There are fundamentally three types of policy, which might be characterized as: the ethical premises of management, "legislative policy," the broad non-ethical rules laid down by top management; "management policy," and other rules, "working policy."[54]

In broad terms, the formulative level is strategical and the administrative level is tactical. As in military operations, strategy and tactics, while normally divided, often cross over echelons. In water management, there is often a strategy-tactics overlap in the areas of pricing, planning for community growth, and planning for management growth. Water-rate (use charge) structures are affected by and influenced by legislative and administrative offices at both the local and state levels. Community growth planning is affected by legislative and management policies of all levels of government. Planning for management growth is also affected by all levels of government, by discontinuities from the tenuous tenure of elected officials who exercise at least some degree of control over management policies.

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[52] Craine, L.E. 1969. *Water Management Innovations in England.* Resources for the Future, Inc., via the Johns Hopkins Press, Baltimore, Maryland.

[53] Sampson, R.J. October 1969. "Public Ownership Constraints on Utility Management." *Journal American Water Works Association,* Volume 61, No. 10.

[54] Simon, H.A. 1965. *Administrative Behavior.* Free Press Paperback 92893, the Macmillan Company, New York, New York.

A decision is a settling or terminating by giving judgment on a matter, on a course of action. Decisions are made in relation both to legislative policy and to management policy. Administrative efficiency is best served when the responsibility for weighing the importance of a given function against the importance of other functions is given to one who is responsible for all the functions involved or none of them, because function-identification modifies (biases) the decisions of managers. Thus, administrative efficiency calls for moving decision-making on comprehensive issues (such as environmental quality) upward through the administrative hierarchy while contemporary public demands for improved administrative accountability require that decision-makers must be more accessible to the individual citizen. How to achieve closeness while simultaneously moving to more remote positions appears to be an outstanding enigma of our time, but not truly a new issue.

"Government's objective in water management is, first, to choose the combination of water services to be produced from specified resources, and second, to provide those services efficiently. Given this two-pronged objective, water management becomes a process comprising the following activities: (1) appraising demand and supply; (2) assessing benefits and costs of alternative ways of meeting demands; (3) choosing actions which promise to meet demands most efficiently within given policy constraints; (4) implementing chosen actions; and (5) carrying out such continuing operations as are indicated."

Figure 19-1 gives governmental techniques for incrementally influencing water use and development: in terms of increasing integration of techniques; and greater evolution in scope and complexity. In the United States, "there is a philosophic presumption that government intervenes in water resources only to the extent necessary to assure public satisfaction with water services provided. The burden of proof is on government for any expansion of its role." Examples quickly come to mind of states that have at least partially penetrated Stages 4 and 5 in Figure 19-1. Agencies such as the Corps of Engineers and the Bureau of Reclamation have at least a partial Stage 4 involvement, and an authority such as TVA fits the Stage 5 category. Water management is an advanced stage of governmental involvement and is generally characterized by Stages 4 and 5. Local units of government may or may not employ all the techniques of Stages 4 and 5. In an attempt to clarify the distinction between their role and that of state authorities, water managers at the local level sometimes refer to themselves as "water utility managers[55] when speaking of their water supply and distribution responsibilities.

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[55] Baxter, S.S. December 1968. "Manager's Role in the Decision-Making Process." *Journal American Water Works Association*, Volume 60, No. 12.

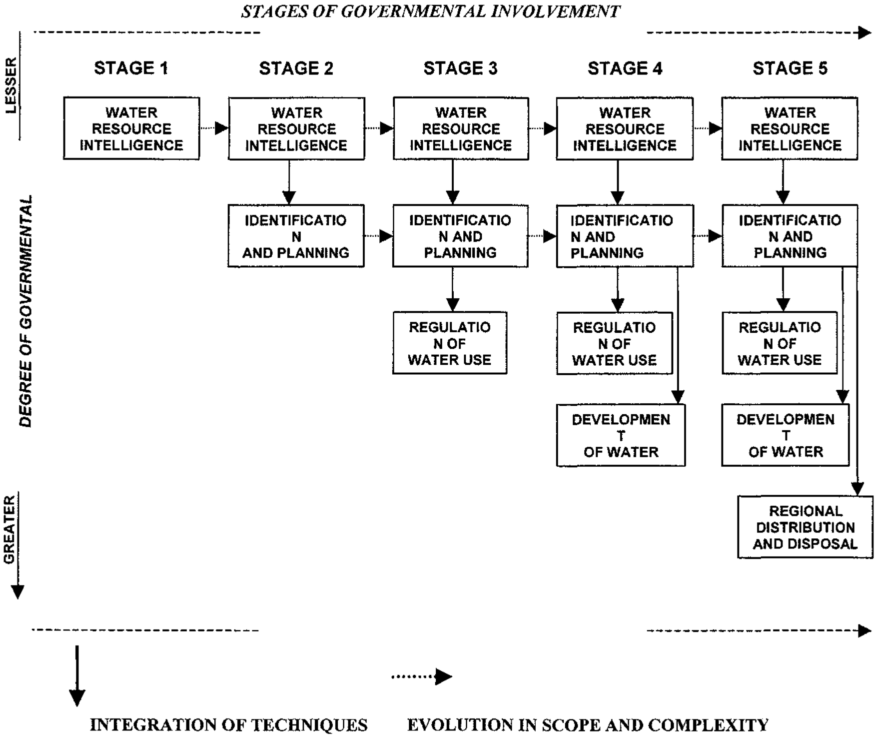

FIGURE 19-1. Governmental Techniques for Influencing Water Use and Development

(Reproduced from *Water Management Innovations in England*, by Lyle E. Craine, Resources for the Future, Inc., via The Johns Hopkins University Press, Baltimore, Maryland, 1969, page 19.)

The National Water Commission, created by the U.S. Congress in 1968 (PL 90-515), is an advisory panel that is, in effect, a citizen's counterpart to the Water Resources Council, an official agency of the federal government. The Commission held a series of public conferences in 1969 to obtain views and advice on its tentative program of special studies and related basic questions.[56] Attendees were often awed and dismayed by the proliferation of organizations, agencies or departments of government, at all levels, concerned with the conservation, development or utilization of water resources.[57] Thirty-eight federal agencies are involved in some aspect of water resources activities, of which about 18 are active in water resources research.[58] In nearly all great metropolises around the world every important level of government participates in each major public-service category and execution of most urban programs depends on actions by several governments, because of the national importance of large urban areas, their needs for scarce expertise, and their requirements for enormous capital investment.[59]

4.3 River Basin Planning and Development

The history of water resources development in the United States is characterized by the evolution of the multiple-purpose development concept, the focus on the river basin as a basic unit for planning and development, the formulation of cost-benefit relations for testing feasibility of projects, and the promulgation of competitive federal agency functions.[60] In planning, a "lead" agency "is supposed to corral the views, purposes, and projects and to assemble all of them into one euphemistically described 'integrated plan'..... Perhaps completely successful planning of the magnitude that we in the United States indulge in is impossible. Although persistent efforts are strenuously made to strengthen local participation, in many instances the results are more formal than substantive."

An appeal has been made to develop better means for the planning of solutions to water management problems that can encompass a greater flexibility in meeting immediate needs without the risk of irreversible long-term disadvantages, more opportunity of choice among alternatives, and a clearer presentation of the consequences of courses of action proposed.[61] Because the public hearing is too often utilized on a "take it or leave it" basis for a specific plan, the promotion of research into human preferences for use of water resources is also advocated.

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[56] The National Water Commission. December 31, 1969. *Annual Report for 1969.* GPO, Washington, DC.

[57] ASCE. February 1970. "Water Commission Testimony." *Hydraulics Division Newsletter.*

[58] Renne, R.R. July 1968. "Status of Water Research in the United States." *Journal American Water Works Association*, Volume 60, No. 7.

[59] Walsh, A.H. 1969. *The Urban Challenge to Government: An International Comparison of Thirteen Cities.* Frederick A. Praeger, New York, New York.

[60] Wolman, A. February 1970. "Multiple Purpose River Development." *Journal American Water Works Association*, Volume 62, No. 2.

[61] White, G.F. 1969. *Strategies of American Water Management.* The University of Michigan Press, Ann Arbor, Michigan.

Title I of the Water Resources Planning Act of 1965 authorized the creation of the Water Resources Council, whose functions are to make a national assessment of water conditions, establish a national pattern of federal water policy, create Title II Federal-State River Basin Commissions and make grants to the states for comprehensive water resources planning.[62] Title II provides for a collaborative system of water planning among federal, state and local governments. The Council completed its first national assessment of water conditions in 1968,[63] and has reviewed several types of river basin management arrangements.[64] The main recommendations of a special task force of the Water Resources Council included a call for broadening of the traditional framework of project analysis beyond economic efficiency to include the additional criteria of regional development, environmental enhancement, and the wellbeing of people.[65,66] A case study of the effects on intergovernmental relations and local politics in the Delaware River Basin from the 1960's drought highlights the role of the Commission in meeting that emergency.[67]

4.4 State Water Resources Planning and Programs

A study[68] of five selected state organizations in terms of their capabilities, in an organizational sense, to perform essential functions and consequently to achieve planning objectives, revealed that they were hampered by inadequate staff and funds, a tangled network of boards and committees, and in the absence of comprehensive statewide planning, state water resource planners have been handicapped in implementing the administrative machinery necessary to integrate planning objectives. In the study it was noted that many states recently have taken steps to implement statewide water planning programs of their own, although they had previously leaned heavily on the federal government for water planning and development. Alternative organizational approaches recommended for accomplishing necessary coordination and cooperation include the consolidation of natural resources functions in a single state agency or the establishment of a central council of department heads.

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[62] Caulfield, H.P., Jr. 1966. "North American Water Supply Problems and Their Solution." Section of Mineral and Natural Resources Law, American Bar Association, *Proceedings*.

[63] Water Resources Council. 1968. *The Nation's Water Resources*. GPO, Washington, DC.

[64] Water Resources Council. August 1967. "Alternative Institutional Arrangements for Managing River Basin Operations." Washington, DC.

[65] Water Resources Council. June 1969. "Procedures for Evaluation of Water and Related Land Resource Projects." Report by a Special Task Force. Washington, DC.

[66] Kalter, R.J., *et al.* February 1970. "Federal Evaluation of Resource Investments: A Case Study." Water Resources and Marine Sciences Center, and Department of Agricultural Economics, WRMSC Technical Report No. 24. Cornell University, Ithaca, New York.

[67] Hogarty, R.A. 1970. *The Delaware River Drought Emergency*. Inter-University Case Program No. 107. The Bobbs-Merrill Company, Indianapolis, Indiana.

[68] Hoggan, D.H. December 1969. "State Organizations for Water Resources Planning." *Journal American Water Works Association*, Volume 61, No. 12.

By November 1969, 26 states had an "office of community affairs,"[69] with many of them formed only a few years ago. This trend would seem to auger well for the consolidation of state natural resources functions in a single agency, or for the greater inclusion of metropolitan water considerations in community affairs offices for more integrated planning.

As implied above, several states are giving attention to overall water resources planning and the development of water projects that are beyond the capabilities of local units of government. State activities focus on allocation, regulation and facilitation of local activity. Various states have: undertaken comprehensive water management planning and development; coordinated water programs at the various levels of government; gathered hydrologic data and engaged in other research activities; facilitated local organizational and financial arrangements for the provision of water and sewer service in urban areas; provided technical assistance and training programs for local water and sewage agencies; aided in planning local water facilities; provided loans and grants for the construction of local water and sewer utilities; and developed urban water supplies.[70]

Within the framework of policies and controls established by state governments, local governments exercise paramount responsibility for urban water supply and sewerage; and the chief role of the states has been in the allocation of water supplies, the regulation of the use of waters including pollution control, and the regulation of planning and construction of local water and sewerage facilities.[71]

Special districts intimately connected with state natural resource development programs pose some of the most difficult problems in state-district relations, particularly in water resources, namely: drainage, irrigation, flood control, and some water supply districts.[72] A certain degree of supervision over the activities of sanitary districts is required because of the overriding responsibility of the states for water pollution control. Most special districts involved with water and related park and recreation functions are eligible to receive federal grant funds and federal operational or regulatory programs affect a number of these functions, requiring close intergovernmental cooperation and perhaps operating programs in some instances.

4.5 Metropolitan Water Management

Most of the aspects of metropolitan water management activities are listed in Table 19-1.

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[69] January-February 1970. "Indiana Becomes Twenty-Sixth State to Create Community Affairs Office." *Metropolitan Area Digest*, Volume XIII, No. 1. Graduate School of Public Affairs, State University of New York at Albany, New York.

[70] Advisory Commission on Intergovernmental Relations. October 1962. *Intergovernmental Responsibilities for Water Supply and Sewage Disposal in Metropolitan Areas*. Report A-13, Washington, DC.

[71] Advisory Commission on Intergovernmental Relations. September 1963. *Performance of Urban Functions: Local and Areawide*. Report A-22, Washington, DC.

[72] Advisory Commission on Intergovernmental Relations. May 1964. *The Problem of Special Districts in American Government*. Report A-22, Washington, DC.

TABLE 19-1

Aspects of Metropolitan Water Management Activities

Urban Water Uses:

- Water supply (domestic—including drinking water, commercial, industrial, agricultural and for fire protection);
- Conveyance of wastes (from buildings and industries);
- Dilution of combined and storm sewer system effluents and treatment plant effluents (by receiving bodies of water);
- Water-oriented recreation (and fish management);
- Aesthetics (such as landscaped creeks and ponds in parks and parkways);
- Transportation (commercial and recreational);
- Hydropower generation; and
- Dissipation of heat from thermal power plants.

Protection of Urban Areas from Flooding:

- Removal of surface water at source (by conduit systems or canals);
- Conveyance of upstream surface water through the area (by conduit systems or canals);
- Barricading banks, detaining or expressing flow of natural streams to mitigate spillover in occupied zones of floodplain (by levees, dikes, upstream storage or canalization);
- Flood warning and "dynamic" floodproofing of structures; and
- Floodplain management.

Manipulation of Urban Water:

- Groundwater recharge (using processed water or stored surplus surface water);
- Recycling of water (reuse of effluents for water supply, recreation, etc.); and
- Low flow augmentation (water transportation, water supply, etc.)

TABLE 19-1 (continued)

Aspects of Metropolitan Water Management Activities

Pollution Abatement in Urban Areas:

- Conveyance of sanitary sewage and industrial wastes in separate sewer systems;
- Interception and treatment[73] of sanitary sewage and industrial wastes (from separate sanitary sewer systems or dry weather flow from combined sewer systems);
- Interception and treatment of storm sewer discharges or combined sewer overflows (by means of detention storage facilities);
- Reinforcing waste assimilation capacity of receiving waterbodies (by forced aeration, low flow augmentation or other means, to raise ambient dissolved oxygen content);
- Treatment of sanitary wastes as point-of-origin (household and building treatment plants);
- Disposal of sludge from treatment plants; and
- Interception of runoff of mineral sediments from construction and natural areas.[74]

Water-Land Planning:

- Water as a controlling influence on residential development and on transport and industrial location;
- Reduction of peak drainage runoff rates by proper land-development design;[75]
- Utilization of the "blue-green" development concept, employing ponds with open space, for storm flow detention and recreation, to enhance urban property values and decrease property depreciation rates, thereby increasing long-term tax revenues; and
- Pursuit of long-range planning that incorporates community social values inherent in dynamic historical, physical and biological processes, responding to real variabilities of the environment, realizing both opportunities and limitations for human use, while utilizing the concept of complementary land use and identifying least-social-cost locations for development.[76]

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[73] October 1969. *A Primer on Waste Water Treatment.* FWQA, U.S. Department of the Interior, GPO, Washington, DC.

[74] Powell, M.D., W.C. Winter, and W.P. Bodwitch. March 1970. *Community Action Guidebook for Soil Erosion and Sediment Control.* National Association of Counties Research Foundation, Washington, DC.

[75] Jones, D.E., Jr. August 1967. "Urban Hydrology—a Redirection." *Civil Engineering.*

[76] McHarg, I. April 1969. "What Would You Do With, Say, Staten Island." *Natural History*, Journal of the Museum of Natural History, Volume 78, No. 4.

TABLE 19-1 (continued)

Aspects of Metropolitan Water Management Activities

Interfacial Public Services:

- Snowstorm and rainstorm traffic routing;
- Street cleaning scheduling;
- Snow removal strategies;
- Lawn irrigation conservation; and
- Air pollution control.

(For a general discussion of some of these aspects and their interrelationships, see: *Water as an Urban Resource and Nuisance*, by H.E. Thomas and W.J. Schneider, Geological Survey, U.S. Department of the Interior, Water in the Urban Environment Series, Circular 601-D, GPO, Washington, DC, 1970.)

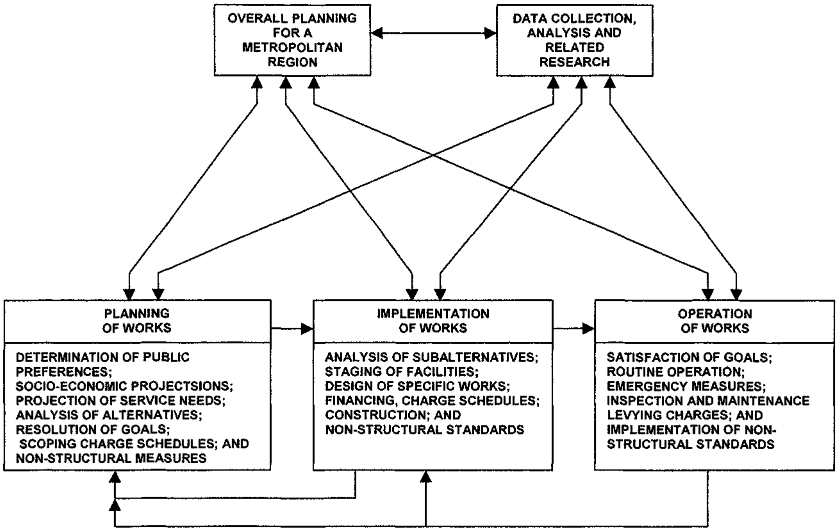

FIGURE 19-2. General Metropolitan Water Management Functions

Figure 19-2 shows the general functions of metropolitan water management. Overall planning for a metropolitan region will be discussed in the next chapter. Data collection, analysis and related research is currently pursued independently by individual local government departments and metropolitan special districts, with only limited collaboration on a national scale. Using the terminology of the earlier section on "Water Management" in this chapter, the planning, implementation and operation of works is primarily at the "administrative level;" however, water managers have a "formulative" capacity with regard to "management" policy and goals. Water and sewerage in large municipalities are commonly administered under either two separate departments, a single department or as separate or combined divisions of departments of public works. The most common organizational arrangement is, therefore, by major purpose or purposes. Departments providing water and sewerage are line departments because they render services to the public directly.[77] Major purpose line departments have the advantages of immediate service identity by the public and professional and occupational identity by employees.

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[77] Phillips, J.C. 1960. *Municipal Government and Administration in America.* The Macmillan Company, New York, New York.

Metropolitan area planning commissions are almost exclusively staff agencies, and not line agencies, because they are non-operating advisory organizations.

Figure 19-3 depicts the major physical components of the urban water resources system. This schematic representation indicates the interdependence and linkages between water supply-treatment-distribution and wastewater and stormwater collection-treatment-disposal. There appears to be a greater inclination towards the integration of all these services in single municipal departments.

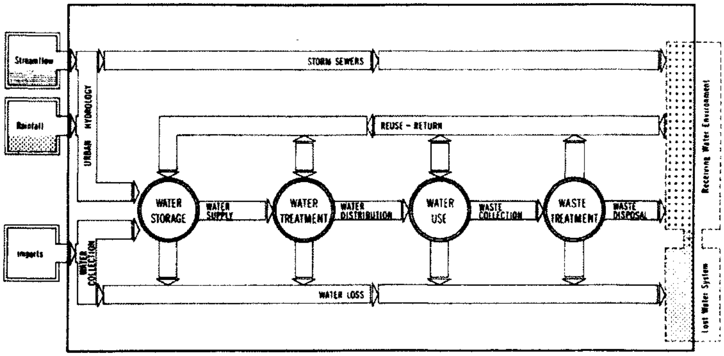

FIGURE 19-3. Major Physical Components of the Urban Water Resources System[78]

A process department is one where all activities using common techniques or skills are assembled in the same department under one direction, such as city personnel or city accounting departments. New York City has transferred water and sewerage functions to a newly formed Environmental Protection Administration, removing these functions from utility and quasi-utility orientations and combining them with related environmental control functions such as air-pollution abatement. This reorganization effectively changed water management from a purpose to a process arrangement, and in the changeover the role of water thus became subordinated to a larger scale of responsibility.

Commitments of large-scale capital investments for urban infrastructure, including permanent public-service facilities, are crucial prerequisites for response to urban problems. In major metropolitan areas throughout the world, nationwide competition for scarce capital resources, national commitment to development planning or to control of economic fluctuations, and superior national tax resources, have

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[78] Reproduced from Appendix H, by Water Resources Engineers, Inc., in *Urban Water Resources Research*. ASCE, New York, New York, September 1968, p. H-10.

cumulatively resulted in higher levels of government playing an important part in decision-making for urban activity requiring massive public-investment funds. American cities moved from a state of considerable autonomy and strong local identity toward one of interdependence and subordination to the national society during the second quarter of the present century.[79]

Thus, concern for the environment encourages the fostering of a greater horizontal interdependence at the metropolitan-local level, while vertical interdependence in the federal-state-metropolitan axis seemingly endures unchanged. This schizoid pressure adds one more serious complication for embattled mayors of major cities to cope with. It is, therefore, not surprising that some of these mayors in seeking to change over to a process-form of planning and/or organization from a long-established purpose-arrangement are appealing still more strongly for process-oriented rather than purpose-oriented federal and state grants-in-aid.

4.6 Metropolitan Area Planning

By 1923, every metropolis of 300,000 or more inhabitants had accepted planning as a municipal function. By the 1960s, development in metropolitan planning reflected a convergence of federal, state, and local efforts, as well as practical and scholarly interests. Of the 216 SMSAs in 1964,150 had some form of metropolitan planning. An increase in state permissive legislation, the "701" planning grant program (including its extension to councils of government), and Section 204 of the Metropolitan Development Act of 1966, stimulated development of area-wide planning agencies.[80] By late 1968, 208 of 230 SMSAs were served by 171 "area-wide" planning agencies authorized to handle the advance review and comment required by Section 204 for certain federal grants to local governments in metropolitan areas: slightly over a third were single county or city-county agencies; slightly over a third were regional planning commissions made up of two or more counties; over a fifth were COGs; and the small remainder was made up of metropolitan transportation and state planning agencies.

In outlining a profile of the average metropolitan planning agency, *circa* 1963, it was concluded that the typical agency probably had prepared some kind of comprehensive development plan, but only occasionally had it conducted studies on flood control, storm drainage, water pollution, waste disposal, and other area-wide environmental problems.[81] The large SMSAs have been particularly active in these areas over the last several years.

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[79] McKelvey, B. 1968. *The Emergence of Metropolitan America*, 1915-1966. Rutgers University Press, New Brunswick, New Jersey.

[80] Advisory Commission on Intergovernmental Relations. October 1969. *Urban America and the Federal System*. Report M-47, Washington, DC.

[81] "The Metropolitan Planning Agency: A Profile of the Average." *National Survey of Metropolitan Planning*. GPO, Washington, DC. 1963, in *Government of the Metropolis* (selected readings). Joseph F. Zimmerman, Holt, Rinehart and Winston, Inc. New York, New York. 1968.

Three distinct advantages accrue to local units of government that share in metropolitan planning: an opportunity to participate in decisions affecting both the region and the local community; availability of metropolitan studies and projections useful as background for local decisions; and general technical assistance.[82]

Planning commissions, like COGs, possess only advisory powers, and this is one reason COGs normally assume a planning function or can readily absorb an existing planning agency.

Effective regional planning requires that the planning agency be geared into a system having the disposition and power to guide development taking into account metropolitan needs. "Development of such a system or systems is one of the great social needs of the century."[83] This desire is echoed by many planners. "Unless oriented toward the power and capacity <u>to get things done</u>, the planning process is a costly exercise in futility, self-deception, and deception of others."[84]

Planning is regarded as a technique for consciously adjusting governmental action to the pressure of urban problems in order to guide urban growth and to provide public services and goods in the mixes desired. In this context: planning is one method of formulating comprehensive policies that correlate diverse public functions and geographic sectors in the metropolis; it can adapt the time dimensions of policy to the time dimensions of the phenomena with which it deals; it can be used to broaden the information basis of decisions by analyzing the probable effects of alternative actions; and urban-planning institutions, particularly when they have dynamic leaders, can become sources of innovation that advocate new solutions and combat the inertia of ponderous government machinery. Thus, planning as an institution is inextricably interrelated with other aspects of local government. "Articulated systems of planning have counteracted structural fragmentation of authority for the metropolis (whether horizontal or vertical fragmentation). When used for general policy control, such systems have also facilitated decentralization of other governmental roles. At the same time, detailed, centralized planning is associated with the tightest hierarchies in the world. Planning is a style, after all, that can be utilized for as many purposes as there are planners."

Many observers in the social and biological sciences deplore the almost overwhelming utilization of the potential of urban development planning for the provision of physical facilities as against socio-psycho-biological considerations. "Since environmental factors profoundly condition most aspects of daily existence, and in particular the biological and psychological development of children, the most urgent need in urban planning is a better knowledge of what human beings require biologically, what they desire culturally, and what they hope to become. In this, as in

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[82] "Who Benefits from Metropolitan Planning"? *The Effectiveness of Metropolitan Planning.* GPO, Washington, DC, 1964; in Zimmerman, *ibid.*

[83] Bebout, J.E. 1966. "Decision-Making in Metropolitan Areas." *Metropolitan Problems.* Drew University, Madison, New Jersey.

[84] Gross, B.M. 1967. "The City of Man: A Social Systems Reckoning." *Environment for Man: The Next Fifty Years*, edited by W.R. Ewald, Jr., Indiana University Press, Bloomington, Indiana.

all important aspects of life, the know-how is less important than the know why; unfortunately, technical considerations are practically always given precedence over human factors."[85]

Two recent books should be consulted for: an overview of the regional planning process;[86] and an historical account of municipal planning.[87] A directory of metropolitan planning commissions is available, together with an accounting of new developments on councils of government.[88] "The Societal Role," "The State of the Art," and "The Professional Planner's Role" in urban planning are the three divisions of a book published for the American Institute of Planners.[89]

The Urban Planning and Development Division of the American Society of Civil Engineers has prepared a useful planning guide;[90] and the Division's Journal contains papers on urban water resources planning from time to time.[91]

> Example: *Metropolitan Planning Agency.* At least from the standpoint of water resources, the Southeastern Wisconsin Regional Planning Commission has been one of the most productive and progressive of such agencies in the country.
>
> Examples: *Research on Metropolitan Water Planning.* The Office of Water Resources Research has supported several studies on institutions for metropolitan water planning. These include: an exploration into the tenuous action and communication linkages between river-basin and metropolitan-area planning agencies;[92] a state-of-the-art analysis of metropolitan water planning;[93] and a compendium of case studies on metropolitan water-land planning practices.[94] An allied study under OWRR sponsorship investigated potentials for water transfers in the greater New York City area.[95]

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[85] Dubos, R. 1968. *So Human an Animal.* Charles Scribner's Sons, New York, New York.

[86] Hufschmidt, M.M., editor. 1969. *Regional Planning: Challenge and Prospects.* Frederick A. Praeger, New York, New York, 396 p.

[87] Scott, M. 1969. *American City Planning Since 1890.* University of California Press, Berkeley, California, 745 p.

[88] *1968 Survey of Metropolitan Planning.* (Issued Annually.) J.F. Zimmerman, ed., Graduate School of Public Affairs, State University of New York at Albany, New York.

[89] 1970. *Urban Planning in Transition.* Edited by E. Erber for the American Institute of Planners. Grossman Publishers, Inc. New York, New York.

[90] ASCE. 1969. *Urban Planning Guide.* Manuals and Reports on Engineering Practice No. 49. New York, New York.

[91] Rockwell, M.L. August 1968. "Water Resources as an Element in Urban Planning." *Journal Urban Planning and Development Division.* ASCE, Proc. Volume 94, No. UP 1.

[92] Kelnhofer, G.J., Jr. July 1968. *Metropolitan Planning and River Basin Planning: Some Interrelationships.* Water Resources Center, Georgia Institute of Technology, Atlanta, Georgia.

[93] Sheaffer, J.R., S.A. Starr, G. Davis, and A. Richmond. 1969. *Metropolitan Water Resource Management.* Center for Urban Studies, University of Chicago, Illinois.

[94] Raven-Hansen, P. June 1969. *Water and the Cities: Contemporary Water Resources and Related-Land Planning.* Abt Associates, Inc. Cambridge, Massachusetts. (Case studies: Boston, Massachusetts; Chicago,

4.7 Water and Wastewater

Through the expansive 1920s, the central cities continued to enjoy an advantage in most aspects of municipal engineering. "Their water and sewer systems were more adequate and their police and fire departments more thoroughly organized." This advantage has persisted in varying degrees to the present. Suburban water and sewage problems in most metropolitan areas are of post-World War II origin. Referring to water distribution and wastewater collection systems: "without question, the suburbs are the critical aspect of the metropolitan water problem. The lag of investment is concentrated in the suburbs."

Water and sewage service in core cities is characterized by centralized systems, nearly all in public hands. "Outside the central city, municipal systems, contract arrangements, utility districts with a service area ranging in size from a sub-development to perhaps an entire suburban county, private companies, and individual systems coexist. Often there are enclaves within central cities and the service areas of metropolitan agencies."

The interrelation and interdependence of water and wastewater and the competition and conflict between multiple jurisdictions have intensified with the growth of metropolitan areas. There is also a greater variety of uses of water, particularly for recreational purposes in metropolitan areas. Much has been written about the possibilities for controlling directions of suburban growth by limiting water and sewer service. "Highways, water mains, sewer trunk lines, and even electric power and gas utilities still limit the direction of settlement and induce clustering. These constraints, however, provide nowhere near the limitations that applied in the days before widespread automobile ownership." Today, the provision of aesthetic inducements, such as in the "blue-green" development concept (Table 19-1), appear to offer more viable means of control.

One type of public service is used or consumed by society as a whole, rather than by an individual, with the service benefits most likely to be enjoyed only if the service is provided on a community-wide basis, such as sanitation, parks, police and fire protection. With a utility service, direct benefits can be enjoyed exclusively by the individual consumer, who pays for the number of units of service he uses, such as gas, electricity and telephone, and entrance-fee recreation. A third type of services falls between these two, in that the direct benefits can be largely enjoyed by individual customers, and priced accordingly, but they also can have a substantial spillover of indirect benefits to the community, such as water, sewerage, drainage, libraries and hospitals. Although the distinction cannot be pressed too far, the inherent spillovers of water, sewerage and drainage have precisely the social and political attributes that astute water managers will attempt to exploit.

Illinois; Columbia, South Carolina; Detroit, Michigan; Little Rock, Arkansas; Los Angeles, California; New Orleans, Louisiana; Pittsburg, Pennsylvania; San Antonio, Texas, and Seattle, Washington.)

[95] Zobler, L., G.W. Carey, M.R. Greenberg, and R.M. Hordon. April 1969. *Benefits from Integrated Water Management in Urban Areas—The Case of the New York Metropolitan Region.* Barnard College, Columbia University, New York, New York.

The most effective and economical service areas for metropolitan distribution of water, sewerage, and transportation, are geographic rather than political, encompassing several units of general local government. "Use of a special district permits such a functional service area to be defined, usually by interested parties, without regard to the boundaries of existing units of general government." However, as noted in several earlier chapters herein, special districts contribute to the balkanized state of government in many metropolises.

The Advisory Commission on Intergovernmental Relations was established by Public Law 380 in 1959. Since its establishment, the Commission has devoted continuing attention to problems of intergovernmental relations in metropolitan areas, where activities of all three levels of government operate in close proximity. Policy reports have dealt with alternative methods of governmental organization, planning, federal and state relations with local governments, and the administration of several federally aided urban programs. Most reports deal with broad types of problems that directly or indirectly involve a range of governmental activities. Certain Commission reports have focused on particular public functions, such as one on political and intergovernmental aspects of urban water supply and sewage disposal problems. A later report contained recommendations for sharing responsibilities for water supply and sewage disposal between the three levels of general government, and additional related proposals are subsumed in various, other reports of the Commission. A recent Commission report dealing with urban problems summarizes previous recommendations but contains no new policy proposals. That report contains a case study of water and sewerage, which includes a summary of earlier findings and proposals and a very good overview of the contemporary situation.

4.8 Water Quality

About eight-tenths of the country's population is served by some 23,000 water systems.[96] "Almost all the larger cities draw their water from surface sources." A national survey under way by the Environmental Control Administration of the U.S. Department of Health, Education and Welfare has disclosed that urban water needs in metropolitan areas are often met by bewildering tangles of parallel and competing systems, large and small, making continuous appraisal and quality control very difficult.[97] In effect, the plethora of systems not only defies attainment of comprehensive metropolitan management, but also poses problems for public health surveillance.

In the last quarter of the nineteenth century, much of the epidemic incidence of several gastro-intestinal diseases, including typhoid fever, was traced to water. In roughly the first quarter of the 1900s, effective water treatment processes were

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[96] February 15, 1970. "AWWA Viewpoint Presented to NWC." *Willing Water*. American Water Works Association, Volume 14, No. 3.

[97] Johnson, C.C., Jr. January 15, 2970. "Is Your City's Drinking Water Really Safe"? *Nation's Cities*, December 1969; reproduced in *Willing Water*. American Water Works Association, Volume 14, No. 1.

adopted as means for checking diseases of a waterborne character. Thus, water quality deterioration in the United States first received attention as a public health problem. The dramatic reduction, almost extinction, of epidemics traceable to waterborne disease in the face of extensive urban growth attests to the dedication and efficiency of the water works industry during this century. However, preliminary evaluations of the Environmental Control Administration suggest that "we have reached a point where we can no longer afford to take the purity and safety of public water supplies for granted ... In the case of water hygiene, as in all of the many environmental problems that face our nation and the world today, if we must wait for epidemiological studies of human illness to convince us of the hazards, it may well be too late."

In this country, wide recognition of the deteriorating quality of natural waters and large-scale efforts to control waste discharges date essentially from the end of World War II.[98] The main issues in contemporary water quality management are considered to be the resolution of the necessary or desirable: number of quality parameters to be used, the level and degree to which each should be controlled, and their definition in numerical terms; means and measures to be employed in reaching specified quality objectives; and institutional or organizational management arrangements.[99] Water quality spillover effects occur between uses, between units of local government in a metropolis, and often well beyond the discernable limits of a metropolitan area.

"The concept that management of the quality of water is quite as important as its physical management is now widely recognized and advocated in the United States. However, it is of such recent origin that no precise definition of the term has been established and widely accepted, principally because the jurisdiction in which the responsibility for management falls is exceedingly fragmented." The most recent philosophy of public objectives relates to the nature of water as a resource, with an emphasis on its capability for meeting a variety of social objectives. Such a re-orientation would move away from an almost exclusive emphasis on control of quality-depreciative agents as an end in itself. It is too early to assess with any realistic certainty the probable effect of several pieces of congressional legislation enacted over the past few years, starting with the Water Quality Act of 1965.

4.9 Aesthetics, Amenities and Safety

"In most urban areas there is concern with inundation by great floods and also by local storms. There is widespread concern with amplifying the opportunities for water-based recreation ready of access to urban residents, and with enhancing the conditions of urban living. A third focus of concern is improvement of systems for

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[98] Kneese, A.V., and B.T. Bower. 1968. *Managing Water Quality: Economics, Technology, Institutions.* Resources for the Future, Inc. via the Johns Hopkins Press, Baltimore, Maryland.

[99] McGauhey, P.H. 1968. *Engineering Management of Water Quality.* McGraw-Hill Book Company, New York, New York.

metropolitan area water supply and carriage of water-borne wastes. These concerns interact with management of the water resources of a river basin. "[100]

The greatest concern will increasingly be on the quality of water. This concern is intimately related to acknowledged imperatives of aesthetic advancement, expansion of recreational opportunities and more extensive availability of waterfronts for public uses.

Storm drainage is more of an amenity than a protector of health and safety. Various aspects of storm drainage management have been discussed in another ASCE report.[101] The local government department responsible for wastewater collection is normally also responsible for stormwater collection systems. About a fifth of the U.S. population is connected exclusively to combined sewers, and of the 14 largest cities, 10 are partially or wholly served by combined systems of sewerage.[102] The total lengths of underground conduits dwarf those of open watercourses in metropolitan areas.

Total lengths in the City of Milwaukee as of the beginning of 1970 were:[103]

Sanitary Sewers	-	685 miles
Storm Drainage Sewers	-	820 miles
Combined Sewers	-	550 miles
River Lengths	-	37 miles
Waterfront Lengths	-	8 miles

Opportunities are taken whenever possible to incorporate aesthetic advantages in the design of permanent open drainage channels wherever they are opted over underground conduits. The City has initiated an extensive program of waterfront improvements, along its rivers and on its Lake Michigan frontage, giving thorough attention to aesthetic values.[104] Hydrologic complications, flood mitigation, water shipping commerce, direct recreational usage and water quality control have been considered in an attempt to arrive at a balanced program.

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[100] Eaton, E.D. May 16, 1968. "Water Resources Planning Strategy." *New England Council of Water Center Directors, Water Resources Planning Conference.* Boston, Massachusetts.

[101] ASCE. April 1969. *Basic Information Needs in Urban Hydrology.* A study for the Geological Survey, U.S. Department of the Interior, New York, New York.

[102] ASCE. October 1969. *Combined Sewer Separation Using Pressure Sewers.* FWPCA, U.S. Department of the Interior, Water Pollution Control Research Series, ORD-4, New York; and M.B. McPherson. December 1967. "ASCE Combined Sewer Separation Project Progress." *Civil Engineering.*

[103] Prawdzik, T.B. February 1970. "Environmental and Technical Factors for Open Drainage Channels in Milwaukee." ASCE Urban Water Resources Research Program Technical Memorandum No. 12.

[104] Milwaukee River Technical Study Committee. 1968. *The Milwaukee River: An Inventory of Its Problems, An Appraisal of Its Potential.* City of Milwaukee, pp. 163.

Milwaukee illustrates the interdependence of all water resource aspects as well as their linkages with many other features of urban life.

The safety of people and properties occupying flood-prone areas is the concern of every level of government, and federal, state, special district and local governments are involved in the development of an ever-greater arsenal of remedial or mitigative measures and policies. Extensive programs of integrated floodplain management are particularly crucial for metropolitan areas that have little topographic relief, such as greater Chicago.[105] There are large cities that early dedicated most of their floodplains over to parkways, such as Philadelphia, or developed extensive networks of major drainage channels in step with urban development, such as Los Angeles.[106] Of course, new suburbs have more than occasionally aggravated flooding with subsequent induced damages in many major cities, because most major cities are located on large streams or bodies of water, and being topologically at the "bottom" are hydrologically subservient. Floodplain politics is highly controversial.[107]

As this chapter suggests, there appears to be a good case for integrated metropolitan water management because of the interdependent and interrelated nature of all aspects of urban water and of land development.

(Special Note).[108]

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[105] Sheaffer, J.R., D.W. Ellis, and A.M. Spieker. 1970. *Flood-Hazard Mapping in Metropolitan Chicago.* Geological Survey, U.S. Department of the Interior, Water in the Urban Environment Series, Circular 601-C, GPO, Washington, DC.

[106] Rantz, S.E. 1970. *Urban Sprawl and Flooding in Southern California.* Geological Survey, U.S. Department of Interior, Water in the Urban Environment Series, Circular 601-B, GPO, Washington, DC.

[107] Drew, E.B. April 1970. "Dam Outrage: The Story of the Army Engineers." *Atlantic.*

[108] Just as this report was about to go to the printer, the following reference became available: "Metropolitan Water Management—Case Studies and National Policy Implications," by Urban Systems Research and Engineering, Inc., Cambridge, Massachusetts, January 1971, a report for the National Water Commission.

Abstract—"A study of the planning, decision-making and implementation practices used in urban water management was made in 12 metropolitan areas. Four areas—Boston, Massachusetts; Lubbock, Texas; Milwaukee, Wisconsin; and Seattle, Washington were examined in detail. Water supply, sewage collection and treatment, water quality control, and water-based economic and recreational activities were examined for a range of objectives, technical and institutional approaches, exogenous influences, future problems and trends, and the impact of federal policies. Specific recommendations for federal action to improve financing, personnel practices, operations, availability of data and analyses, and the institutional and legal framework of water planning, decision making and implementation are made."

Chapter 20

FEASIBILITY OF THE METROPOLITAN WATER INTELLIGENCE SYSTEM CONCEPT

(INTEGRATED AUTOMATIC OPERATIONAL CONTROL) (NOTE: PARTIAL VERSION OF ORIGINAL)

December 1971
ASCE Urban Water Resources Research Program
Technical Memorandum No. 15
New York, New York

1.0 CONCEPT DEFINITION AND STUDY SCOPE

1.1 Concept

The term "metropolitan water intelligence system" was coined by this writer[1] to describe the hardening concept of multi-service automatic operational control, wherein field intelligence on all aspects of urban water might be acquired, including: precipitation; stream stages and flow rates; water and wastewater treatment facilities; water demands and distribution system rates and pressures; settings of regulating structures; quality parameters for watercourses and impoundments, and within conveyance systems; and the status of special facilities such as recreational ponds and lakes. Possibilities exist for incorporating non-water-related service intelligence, including motor vehicle traffic and air pollution monitoring, because these are affected by precipitation and the trend for their control is towards a centralized operation. In its ultimate form, the intelligence system would be in the computer centered closed-loop mode. Using field intelligence as inputs, the computer decision program would resolve best service-least operating cost options, taking into account estimated reliability and risks, and would actuate field regulating and control facilities to approach elected option states. Feedback features would be such as to permit manual supervisory intervention at any time.

This report is the last of three with an emphasis on urban water management. The first[2] attempted to take into account: expectations for the environment of future cities; necessary or desirable new administrative arrangements for future public services management; and the expected impact and interaction of emerging technology, urbanization trends, social goals and related matters. The second[3] dealt with operating problems and had as a major purpose the framing of a background for assessment of applicability and need of the intelligence system concept. Findings from these two reports are drawn upon for some of the conclusions reached in Section

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[1] American Society of Civil Engineers. April 1969. *Basic Information Needs in Urban Hydrology*. A study for the Geological Survey, U.S. Department of the Interior, New York, New York, p. 81.

[2] McPherson, M.B. December 1970. *Prospects for Metropolitan Water Management*. Report for the Office of Water Resources Research, U.S. Department of the Interior. ASCE, New York, New York.

[3] McPherson, M.B. September 1971. "Management Problems in Metropolitan Water Resource Operations." ASCE Urban Water Resources Research Program Technical Memorandum No. 14. ASCE, New York, New York.

3. The present report deals predominantly with the physical capability of existing technology.

In reading this report it must be borne in mind that the findings are those of a feasibility investigation. An exhaustive analysis of only one feature, such as automation of river water-quality monitoring, alone, would have required more time and funds than were available for this relatively low priority ASCE program study. Consequently, heavy reliance has been placed on documented cases appearing in the literature, for the follow-on convenience of the reader, and examples, only, of different types of control schemes have been cited. However, the examples used are mainly instances of the most advanced technology. A project is under way elsewhere with the objective of developing detailed intelligence system capability.[4]

1.2 Why Automation?

Expected extensive increases in urbanization and the concomitant expansion of facilities and services will require improved methods of administration and management. As competition for water and intensification of urban sprawl increase, metropolitan multi-purpose developments and organizational devices for their management will probably evolve to effectuate exchanges of water qualities and water quantities between the several urban water service functions. While the strategy for planning such developments will surely be more complex, operating tactics will also become more complicated, and for efficient operation much more and better field intelligence will be required, if present service standards are to be maintained, and on an instantaneous availability basis.

"... A multiplicity of related factors have contributed to the development of the increased demands that are being placed upon the water resource system. Among these are: (1) increased demands for potable water for domestic use by a rapidly growing population; (2) substantial shifts in population concentrations including urbanization and migration to water-short areas in the Western United States; (3) expansions of industrial uses of water, particularly for the production of electric power; (4) intensification of irrigation practices in the agricultural sector; and (5) increased leisure time that has expanded the desire for the availability of water resources for recreational purposes ..."[5]

Estimated annual average public costs for water, wastewater and drainage facilities and services through 1980, in 1967 dollars, are as follows[6]:

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[4] "Metropolitan Water Intelligence System Project." Department of Civil Engineering, Colorado State University, Ft. Collins, Colorado, sponsored by OWRR. Principal Investigators are Professors M.L. Albertson, and G.L. Smith, and L.S. Tucker is Project Manager. Initial attention, 1971-1972, is on combined sewer system overflow control.

[5] Hill, D.W. and R.L. Meek. June 30, 1970. "An Exploration of Components Affecting and Limiting Policymaking Options in Local Water Agencies." Environmental Resources Center, Colorado State University, Ft. Collins, Colorado. Completion Report Series No. 22, 115 pp., p.1.

[6] American Society of Civil Engineers. September 1968. *Urban Water Resources Research.* First Year Report to the Office of Water Resources Research, U.S. Department of the Interior, New York, New York, pp. 12-16.

Waterworks-			
	Distribution facilities construction		$ 1.55 billion
	Treatment plant construction		0.52 billion
	Operation, capital outlays over and above construction, and debt interest		3.00 billion
		Subtotal	$ 5.07 billion
Wastewater-			
	Sanitary sewerage construction		$ 1.65 billion
	Treatment plant construction		1.03 billion
	Operation, capital outlays over and above construction, and debt interest		1.30 billion
		Subtotal	$ 3.98 billion
Drainage-			
	Storm sewer construction		$ 2.50 billion
	Operation, capital outlays over and above construction, and debt interest		(Unknown)
		Total	
		At Least	**$ 11.55 billion**

Data on expenditures for other urban services are not available, such as for flood control and water-oriented recreation. "However, it is evident that average expenditures by local governments on urban water resources will be $12 billion or more annually through 1980."[7]

The estimated replacement value of public water, wastewater and drainage facilities, at 1965-1966 construction cost levels, is more than $112 billion.[8]

Interest in automatic operational control has been roughly in proportion to total service costs (see listing above), with waterworks leading. Comparative commitments will be evident in succeeding sections of this technical memorandum.

1.3 The Systems Approach

Engineering-economics systems analysis on a metropolitan scale of all aspects of urban water resources has been found to be feasible[9], the design of a comprehensive

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[7] Office of Water Resources Research. 1971. *A National Urban Water Resources Research Program.* U.S. Department of the Interior. GPO, Washington, DC, 54 pp., p. 2.

[8] ASCE. Urban Water Resources Research. op. cit., p. 14.

[9] ASCE. *Urban Water Resources Research. op. cit*, PP. 9, 35-38, and Appendices G through J.

simulation model has been initiated[10], and follow-on field development by others is underway. Because the degree of detail required for system accounting and simulation is least for the planning function and greatest for the operating function, continued capability development will favor short-term returns for planning and long-term returns for operations. Most local government departments and metropolitan special districts with responsibilities in water resources are primarily operations-oriented, and, hence, have a greater involvement in operational monitoring/surveillance and automatic control than in planning.

The complexity of the metropolitan water resources system can be appreciated by examining Figure 20-1, "Water Balance Model[11]." The various functions and facilities enumerated therein are the potential objects of operational control. Initiation of a metropolitan systems analysis would start with a gross (perhaps monthly or seasonal) accounting of water quantities and qualities on a mass balance. Operational control is concerned with real-time occurrences, and its effectuation depends upon much less gross (a range of every 15-minutes to daily) mass balances emphasizing status accounting of water quantities and qualities. Thus, as traditional systems analysis is extended in timeframe detail towards a real-time base, any seemingly artificial distinctions between advanced systems analysis and advanced operational control essentially disappear. For this reason, most analysts would regard automatic operational control as merely a facet of systems analysis, when the latter is regarded in a comprehensive sense.

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[10] Water Resources Engineers, Inc. September 1970. "System Analysis for Urban Water Management." Report prepared for the Office of Water Resources Research, U.S. Department of the Interior, through the ASCE Urban Water Resources Research Program, Walnut Creek, California.

[11] This illustration was generously made available courtesy of Parsons, Brinckerhoff, Quade and Douglas, Inc., Engineers; 111 John Street; New York, New York 10038, via P.H. Gilbert, Assistant Vice President, Water Resources.

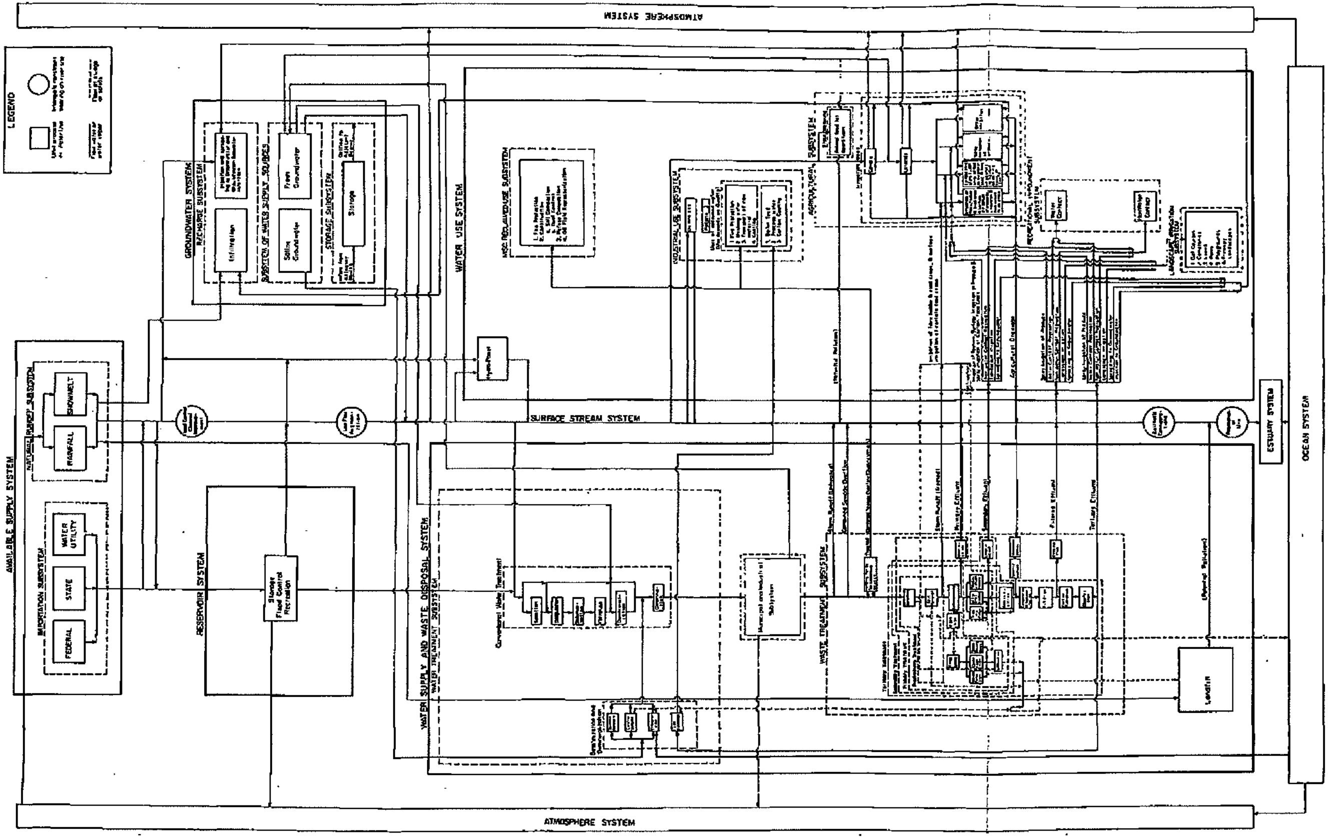

Figure 20-1. Water Balance Model

All this is to say that gross accountings via mass balances taken through more refined stages and the testing of alternative choices in planning and for implementation would parallel, and ultimately converge on, the needs and objectives of real-time operation. Distinctions are superfluous at this time because automatic operational control is presently more advanced than is comprehensive systems analysis in urban water management. Of considerable importance is the fact that all types of systems approaches, if of adequate breadth, can be complementary and mutually reinforcing.

1.4 Reliability

Of grave concern is the maintenance of public service-standards in the face of increasing operating complexity. Private corporation production reliability standards are analogous to public service standards. As industrial managers seek to gain greater productivity through more integrated operations and economies of scale their reliability standards intensify as they try to insure the success of their enormous financial commitments[12], and here, too, the quality of process control assumes heightened importance.

Techniques of Reliability and Maintainability Analysis, outgrowths from those developed for military programs, have a considerable potential for use in urban water operations: in assessment of system reliability; and in development of maintenance requirements for use in the analysis of maintenance functions.

Reliability and maintainability analysis is, thus, a manifestation of the systems approach and constitutes still one more dimension of systems analysis, although it has had only introductory application thus far in urban water operations. Acceptance of the metropolitan water intelligence system concept, a vital consideration in its feasibility, will depend in part on satisfactory measures of reliability.

Reliability and maintainability analysis would appear to be the bridging tool between automatic operational control and systems analysis for planning because of its relevance in all timeframes, and its inclusion would clearly also be complementary and mutually reinforcing in connection with use of the other two tools.

2.0 AUTOMATIC OPERATIONAL CONTROL

Operational control spans a spectrum ranging between monitoring and complete, or "hands-off," automatic control. While a number of levels of development, each with several variations, could be delineated, there appear to be three basic stages: system monitoring, remote supervisory control, and automatic control. Advances in control capability for urban water resources have mostly evolved from systems for monitoring field variables.

Components of monitoring systems are illustrated in Figure 20-2. Not all elements shown are included in all monitoring systems. An example is "on-site variables display"

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[12] The Editors. October 17, 1970. "The Trillion-Dollar Economy." A special report, *Business Week*, p. 172.

for checking for fidelity of instrument response, instrument malfunction, etc., at individual instrument sites. In the absence of this feature instruments can be checked by maintenance personnel only by using auxiliary communication (radio, telephone) with the monitoring center. The simpler monitoring systems feature only data logging, visual display charting, and warning and alarm displaying. Some kind of computer capacity is required for data analysis, but this can be offline. Completely missing is any overt field control function. Figure 20-3 is a schematic description of an air quality monitoring system that embodies the features of Figure 20-2.

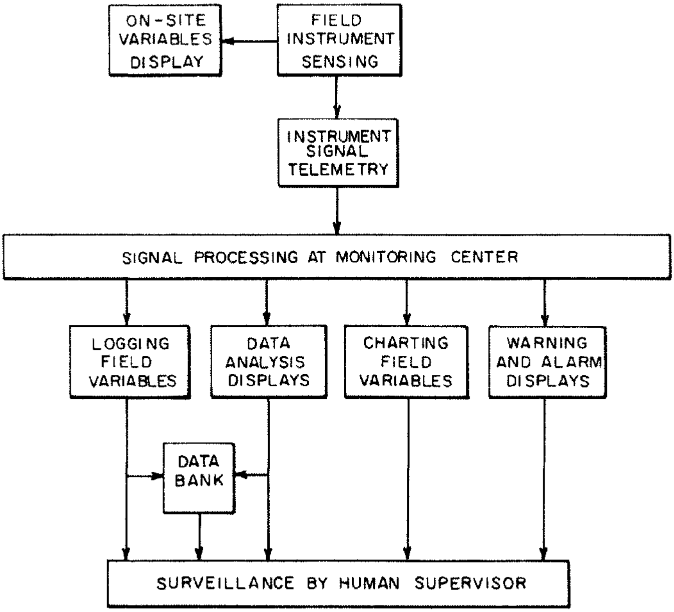

FIGURE 20-2. Monitoring Components

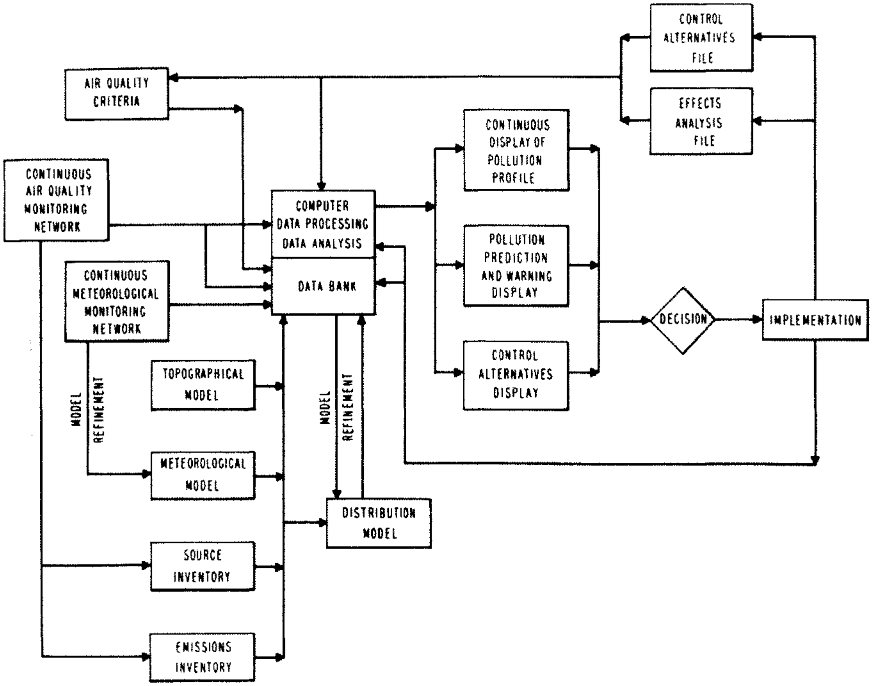

FIGURE 20-3. "Systems Schematic of a Continuous Air Quality Monitoring System" From Consulting Engineer, March 1968, p. 196.

An advanced form of remote supervisory control is represented in Figure 20-4, which is the content of Figure 20-2 plus some control features. "Control logic queries" refers to supervisor interrogation of computer programmed control logic, which can range from elaborate algorithms incorporating prototype response simulation through simple retrieval of command alternatives stored in the computer memory. As is the situation for "data analysis," computer access can be offline. Actuation of field control elements is performed remotely by manual supervisor command. In simpler systems, control element response to an actuation command can be ascertained indirectly by observing changes in affected field sensor signals. On the other hand, the "response signals telemetry" to the control center in Figure 20-4 provides direct control-loop feedback, with an opportunity to damp system actuation response instabilities by using guidelines pre-programmed as part of the control logic. Provision of optional "onsite display of responses" facilitates field checking of actuators by maintenance personnel for the same reason given above for "onsite variables display." Although telemetry of instrument, control, and response signals are shown separated in Figure 20-4, all three could use the same carrier facility. An appropriate analogy to Figure 20-4 would be flight simulators of the types used for "hands-on" training of airline pilots and astronauts. An interpretation of the data flow in remote supervisory control is given in Figure 20-5, all of which is compatible with the content of Figure 20-4.

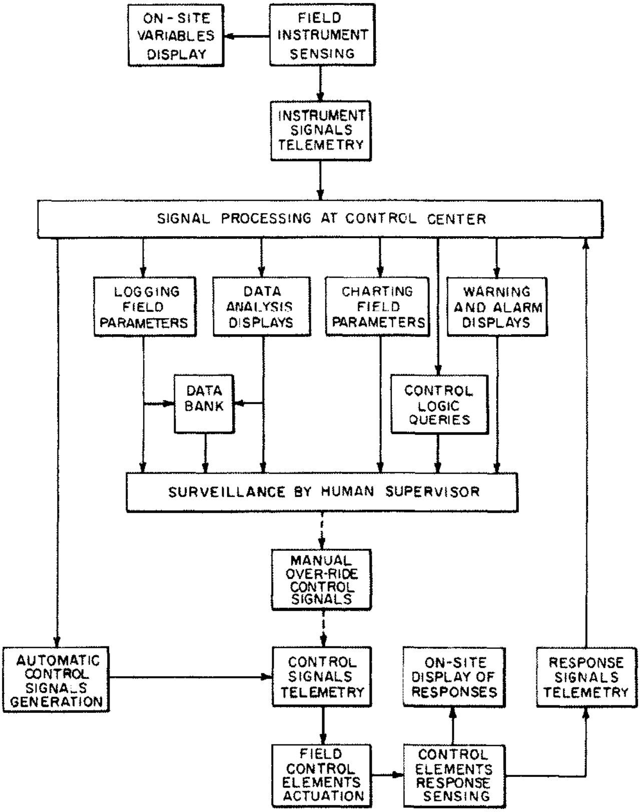

FIGURE 20-4. Remote Supervisory Control Components

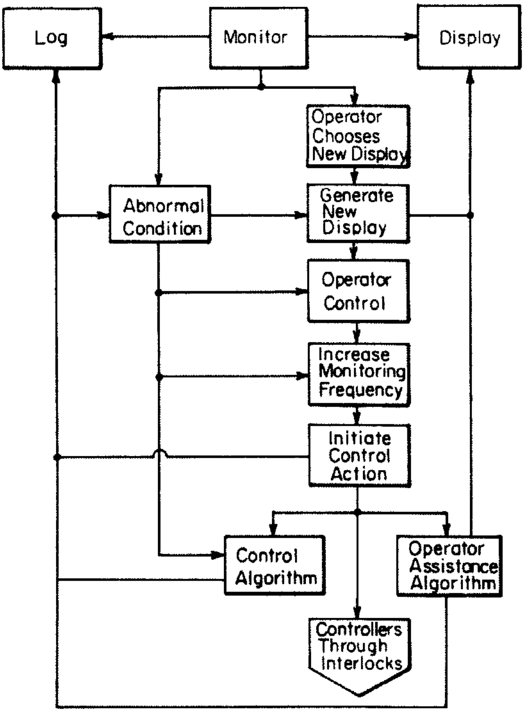

FIGURE 20-5. Remote Supervisory Control Data Flow (Reproduced from "Computer Applications in Distribution." By R.C. Neel, Journal American Water Works Association, Volume 63, No. 8, August 1971, p. 488.)

Automatic operational control components are shown in Figure 20-6, which is identical with Figure 20-4 except for two items: "manual control" has been replaced with "manual override;" and "automatic control signals generation" has been added. It has been said in a number of quarters that achievement of the latter capability would at least double the cost over that for remote supervisory control. Control logic must be greatly expanded, to the extent that the control system can be a full-fledged surrogate for the human supervisor, in what is sometimes termed "hands-off" operation, with supervisor intervention via manual override only in the event of control system failure or inadequacy. Control logic requirements are discussed in connection with water distribution systems in Section 6 and sewered drainage catchments in Section 12. Suffice it to say that there appear to be no examples of complete automatic operational control capability for management of any basic urban water resource function, but water distribution system developments seem to be getting close. However, generalizations are always drawn at extreme hazard. There are instances of some urban water services that have parts of components under complete automatic control. It appears that urban water operations are somewhat behind developments in space-vehicle automatic control, but so is every other civil application.

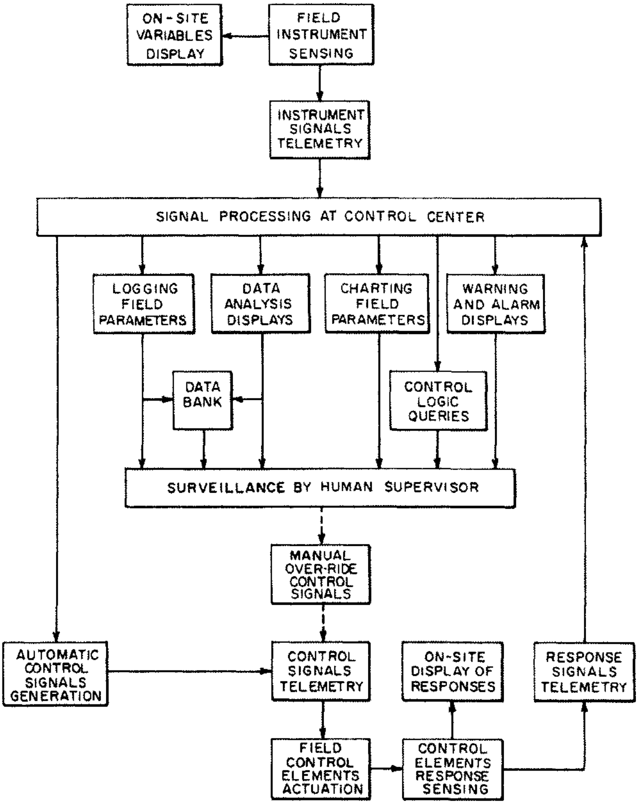

FIGURE 20-6. Automatic Operational Control Components

3.0 SUMMARY AND CONCLUSIONS

3.1 Summary

Summary Section 4 (Information Systems, Computers, and Associated Issues in Automation.) Present trends in American business management indicate that: the easier computer applications are well established but enthusiasm for grandiose schemes such as total management information and control systems has cooled as the tougher aspects have been encountered; investment in automation continues to escalate, yet pushbutton management is still a long way off for most companies; availability of the minicomputer has broken the logjam somewhat, by making more feasible piecemeal automation development, which is much less risky than precipitous total conversion; automation of manufacturing will affect competitive positions in foreign trade; steady growth in the ratio of consumers to workers and the ascendancy of services over industry as a national economic base indicate an even higher premium on productivity and an enlargement of internal-services personnel in industry; and the prognosis is that advances in automation are inevitable but the pace of development is slower than expected only a few years ago, now that management better understands the formidable obstacles that must be overcome. Greater flexibility in leasing arrangements for telemetry via telephone lines will tend to facilitate urban water automation and hence telecommunications are not likely to be a constraint on feasibility. Whereas use of broad-based information systems by local government is feared as a probable invasion of privacy and a threat to the rights of the individual, little public opposition is expected on special-purpose systems such as for urban water resources. The intelligence system concept affects principally the routine operations of works, where most of the personnel are concentrated, with occasional but comparatively infrequent information feedback linkage to top and middle managers. There are numerous compelling reasons for using the systems approach, of which the intelligence system concept is a part, not the least being improvement in the odds for achieving administrative goals by avoiding actions, each logical on its own merits, but that cumulatively are inadvertently counter-productive.

Summary Section 5 (Water Works Advances in Automation.) Although water works lead the urban water field in automation, with a history of development reaching back over at least four decades, complete automation is still quite a ways off. Widespread interest in adoption of remote supervisory control was evident by 1955, but development lagged behind advances in gas, electric, pipeline and mass transportation operations. Rapid obsolescence of instrumentation, telemetry terminal and data-processing equipment evidenced in the last decade has caused a degree of conservatism on automation to the extent that feasibility is now judged heavily on anticipated speed of cost recovery. Still, advances in automation continue, mostly motivated by spiraling labor costs and public demands for a higher quality of all local government services. While some see consolidation and area wide enlargement of facilities as a greater vulnerability to their disruption, others disagreeing completely feel that such an emphasis on the resulting increased complexity overlooks the substantial advantages of greatly increased operational flexibility afforded thereby. Literally hundreds of water works, including the smaller ones, have some degree of automatic operational control capability, albeit only monitoring in most cases.

Summary Section 6 (Water Distribution Systems: Analysis/Design Mode Versus Automatic Operational Control Mode.) The design mode features close near-solutions to hydraulic balances of distribution networks made possible by creating an identity of number of unknowns and number of equations, the result being a definition of the system piezometric surface for the instant of system operation simulated. An identity status is accomplished through employment of certain approximations and simplifying assumptions, the validity of which is indirectly reflected by the proximity of simulation results to observed prototype performance. Once simulation capability for the existing system is verified, the model is employed in the analysis of system improvements and extensions for projected needs. Actual operation and its analysis are complicated by the usual division of systems into service districts that are basically separate hydraulic entities yet intertwined. In severe contradistinction to the analysis/design mode, the control mode features a sampling of the piezometric surface at a few field points from which actual field occurrences must be inferred. That is, the number of unknowns greatly exceeds the number of applicable equations. Mastery of control logic is presently the only obstacle to the achievement of automatic operational control. Control logic is the software that would make possible a full-fledged surrogate for the control system human supervisor. Control logic includes determination of the piezometric surface, recognition and definition of any off-normal conditions inferred thereby, resolution of applicable least cost and/or best service criteria to be effected, actuation of field regulating devices for criteria satisfaction, and confirmation of success of actuation. Of several control logic schemes being developed, one would embrace simulation-fitting and option-testing holistically while another uses a building-block approach. Because of the large number of computer testing runs that have to be made in a brief period of time for a given system, use of rather simple relations to expedite replication of system transfer functions will probably be necessary.

Summary Section 7 (Water Distribution Systems: Examples.) Examples of automatic operational control activities cited range from an intention to initiate a system capability to a case where all pumping and pressure-regulating functions, but not the piping network, are under computer-directed control. It appears safe to assert that it will take some time, perhaps several years, before control logic advances permit "hands-off" operation of complete larger water systems. The examples cited illustrate the contention in Section 2 that capability generally evolves from monitoring into remote supervisory control and thence towards full automation. There is a substantial possibility that the amount of detail that the system supervisor/dispatcher will have to master may be much greater for automation simply because the control logic detail tends to be additional to the detail of remote supervisory control.

Summary Section 8 (Water Transmission and Treatment.) The ultimate in water works control systems would have suzerainty over water treatment plants and raw water supply facilities in addition to the distribution system. Suitable sensors and regulators for impoundment and transmission control exist, but here, too, control logic is incompletely developed. That is, problems center in software, not in hardware. A difficulty with water treatment automation is the extent of empiricism that remains in the understanding of relationships between raw water physical-chemical characteristics and treatment physical-chemical processes, particularly in connection with raw surface waters of highly variable quality. While some smaller plants are on remote supervisory control it is difficult to believe that operation of some of the largest treatment works could be entrusted to complete automatic control let alone remote supervisory control. This is not to suggest that central control systems for large water works will not be developed that can decide such things as the proportion of total system demand to be met by each plant. For smaller works, automation is partially frustrated in some processes by a lack of automatic analyzers to replace some of the laboratory tests that are normally manually performed.

Summary Section 9 (Wastewater Treatment Plants and Water Quality Monitoring.) Almost all municipal wastewater plants employ biological-physical processes. Several important plant-control laboratory biological analyses (such as for oxygen demand and bacteria) require protracted reaction-incubation times for completion and have thus far defied attainment of automatic control on anything approaching a real-time basis. Incomplete knowledge on plant biological-physical processes impedes development of control logic. Thus, impediments in wastewater treatment automation are in both hardware and software, and few automatic control procedures are being used. Near-term prospects are for computer-assisted supervisory control of the larger plants. Increasingly stringent water quality objectives and recent emphases on regionalized management may hasten developments in automation. Because of the close hydraulic ties between wastewater collection systems (separate sanitary sewers and combined sewers) and treatment facilities, their joint pollution-abatement performance is inextricably linked and both aspects must be considered together in developing any automatic control capability. As processed wastewater reuse becomes more widely practiced it may be expected that automation of treatment facilities will become more widespread because of a greater overall resource interdependence. Like air quality monitoring, the ultimate in water quality monitoring has been computer assisted intelligence processing but with no overt control function. Automation efforts have been concentrated on samplers and field automated sample analyses. As for treatment plants, some needed sensors are not available, but there are more sensor needs for water quality monitoring because more numerous parameters are involved. It appears doubtful that the design of metropolitan intelligence systems should be predicated on requirements for national or global pollution surveillance because of comparatively severe differences in total objectives and in timeframes. However, some features of metropolitan systems should be integrated in some way with regional programs, such as for water quality monitoring. Oceanographic and limnologic automatic quality monitoring capability has been lagging despite the fact that the majority of the nation's people reside in proximity to estuaries, coastal waters and the Great Lakes, waters that are among the most valuable of the nation's resources.

Summary Section 10 (Flood Mitigation.) Use of the term "flood control" has been deliberately avoided because within metropolitan areas little flooding is truly controlled. Even on total river basin systems, emphasis has been on forecasting, infrequently with offline operational simulation, rather than on direct effectuation of controls, and automatic control of a river regulating system is thought to be still a long way off. For the two metropolitan examples cited, present plans go no further than remote supervisory control. Two outstanding reasons are the rarity and short duration of flood events over the year and the necessity for around-the-clock field personnel regardless of ultimate automation because of the level of maintenance required for large-scale flood regulation facilities. Capability for real-time operation of urban stream control structures will take some time to develop because flows and stages must be simulated simultaneously with the occurrence of precipitation events, including simulation of alternative operating tactics and incorporation of mostly yet-to-be defined management operating strategies. A dearth of historic runoff records on urban streams, compounded by the rapid changes that have taken place in urban watersheds, has thwarted effective use of existing simulation models for planning, and the same impasse will be encountered when serious attention is afforded the operating mode. Concern over flood amelioration with its preoccupation with rainfall-runoff varies considerably from one metropolis to another whereas rainfall-runoff quality processes are of more vital concern nationally.

Summary Section 11 (Control Systems for Abatement of Pollution from Combined Sewer Systems.) Almost a third of the nation's population served by public wastewater collection systems is served by combined systems of sewerage. Overflows from combined sewers are thought to comprise a significant source of stream pollution, and EPA officials very recently noted that requirements for control of pollution from combined sewer overflows are rapidly becoming more stringent and that control of pollution caused by urban stormwater discharges is on the horizon. The latter reflects the widely held opinion that pollution from storm drainage is almost as severe as that from combined sewer overflows. Several demonstration projects are under way for combined sewer overflow abatement, all of which incorporate some form of in-system storage capacity upstream from regulators and auxiliary receiving treatment facilities. Because overflows occur over a very small part of a year, any facilities provided for in-system storage and/or treatment of potential overflows must be put on line almost instantaneously, a capability requiring some form of automatic operational control. Although some projects have automatic control as an ultimate objective, the necessary control criteria and control logic are yet to be developed. When abatement of pollution from stormwater conveyed by separate systems of storm drains is attempted, the difficulties to be overcome may be more severe because all such stormwater must be passed through ad hoc treatment facilities, there being no interceptor sewers in separate stormwater systems to divert small storm occurrence flows to perennially operated wastewater treatment plants as in combined systems.

Summary Section 12 (Sewered Drainage Catchment Control Logic Enigmas.) Possibilities for automatic control of separate systems of wastewater sewerage are concentrated at treatment plant influents and effluents and within the plants. While wastewater collection systems carry a perennial burden, underground drainage systems carry stormwater only a few days of the year. Of course, so do waterways, but waterway flood mitigation is of widely varying concern among metropolitan areas. Besides, the total length of rivers and waterfronts are everywhere a minute fraction of the total length of underground systems. Further, the present motivation for automatic control of underground drainage is predominantly for pollution abatement, and runoff- quality control is far more demanding in control criteria than is runoff alone. Additionally, there is widespread interest in multi-purpose drainage facilities that exploit opportunities for add-on water-based recreation that provide more effective protection of buildings from flooding, and admit use and reuse of stormwater for water supply. These all require storage utilization for effectuation, and some require special treatment plants, necessitating employment of some kind of control system for managing the sudden and brief impact of storm flows. The scale of the problem is almost overwhelming: most of the larger metropolises have scores and even hundreds of catchment areas and cumulative drain lengths of over a thousand miles. The discontinuous, low-incidence character of drainage flow, compounded by the random and largely unpredictable nature of precipitation, is a formidable challenge to automatic operational control and is one of the reasons why the art for water distribution systems is comparatively so far ahead. A monumental problem in the analysis/design of drainage systems is the choice of storms to be used. In the operating mode, any control system must not only respond almost instantaneously to the actual occurrence of rainfall but must anticipate the probable character of subsequent time and spatial changes barely before they occur. A desirable adjunct would be an incipient storm-occurrence forecasting capability. As in design/analysis, separation of abstractions from total rainfall to derive rainfall excess is necessarily highly subjective, and delimits the reliability of affected control-response components. While detailed rainfall-runoff quality modeling based on real field data is both desirable and logical for development of control criteria, existing models suffer substantially from a severe shortage of data for their calibration, verification and, particularly, for their realistic application to non-gauged catchments. The control mode can be, and probably must be, approached more pragmatically than design/analysis, and the ratio of art-to-science required for the automatic operational control of urban water services will probably be highest for storm sewer systems.

3.2 Previous Findings[13][14]

Ineffectiveness of local governments in the performance of area-wide functions has been laid to: fragmentation and overlapping of governmental units; disparities between tax and service boundaries; state constitutional and statutory restrictions; and metropolitan areas overlapping of state lines. In terms of comprehensive water resource development, the planning, implementation and operation of facilities and services are usually fragmented in both the central cities and in their metropolitan districts. For example, 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: namely, differences in hydrologic, service and revenue areas, and in political jurisdictions.

The private sector is an important participant in metropolitan water management. Some 15% of all municipal water systems are privately owned and operated, although they

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[13] McPherson, M.B. December 1970. *Prospects for Metropolitan Water Management.* ASCE, New York City, New York, pp. 2-3 through 2-8, and pp. 9-15 and 9-23.

[14] McPherson, M.B. September 1971. "Management Problems in Metropolitan Water Resource Operations." ASCE Urban Water Resources Research Program, Technical Memorandum No. 14, pp. 15, 30-31, 36, 49, 51, 52-54, and 61-62.

serve mostly small and medium-sized communities. About a third of all municipal water withdrawals nationally serve industrial and commercial uses. Industry (other than steam-electric power) draws about 10% of its water requirements from municipal systems and the remainder is self-supplied. In addition, cooling water for steam-electric power is essentially all self-supplied. The amount of self-supplied industrial withdrawals has not been separately assessed for metropolitan areas, but it is a significant part of the national total. Responsibility for urban water management is thus divided between public agencies and private interests.

Geographic service unification is often propounded as a means of capturing greater economies of scale, and functional coordination is seen as the mechanism for achieving higher organizational efficiency. The public sense of a progressive loss of individual identity and a companion apprehension of organizational bigness could conceivably constitute formidable liabilities in attempts to unify the management of metropolitan water.

Managers of almost every American industry have sought to gain productivity through economies of scale, such as by combining parts of manufacturing processes, analogous to consolidation of wastewater treatment facilities or integration of urban water functions. Industrial reliability standards intensify the more interdependent the operations and the greater the concentration of financial commitment. In the private sector, reliability affects profit and in the public sector, affects acceptance. In the public sector, sensitivity to quality and reliability of service can outweigh cost considerations, effectively diluting apparent economies of scale. Efficiency will be as difficult to define and quantify in public services as it is in the variegated private sector, and the case for bigness in public services probably rests more on quality than on cost. As in the private sector, there may be diseconomies of scale in administration ("bureaucracy"), that more than offset economies of scale in production and finance. Further, the scale of problems and social relationships in metropolitan areas continually shifts with growth and technological advance, so that no particular reorganization can meet needs once and for all.

Reuse of reclaimed wastewater by industry and agriculture and recharge of municipal groundwater supplies are potential means for providing auxiliary water supplies, eliminating or at least delaying extension of existing supply facilities of the conventional type. Although there is presently a lack of economic incentive for adopting reuse by both water retailers and wholesalers, it is expected that analyses for future supply extensions will need to take reuse alternatives into account. Of course, the formation of a single agency responsible for all water resource services metropolis-wide would fully internalize the reuse question and is a valid argument for institutional reform. However, it is only one argument in a multi-faceted proposition. Nevertheless, serious implementation of reuse practice requires an integrated approach; and whether this can be adequately accomplished without drastic institutional change remains to be demonstrated.

On a national level, problems of water quality are expected to dominate those of quantity, and there has been an historical absence of tangible motivations for more careful husbanding of water resources. There is no real economic incentive for better husbandry because water delivery is under-priced and the penalty nationally for pollution is nominal at best. Expected continuous metropolitan growth will surely greatly intensify water resource competition and conflict, and as a practical expedient a national policy favoring scarcity over exploitation might be expected to emerge sooner or later, whether arising from economic or ethical considerations or both.

The disparity between the approach to water and air pollution control and the approach to solid waste management is seemingly declining as greater emphasis is placed on what the environment should be rather than on specifications regulating matter discarded. Ascendancy of a management approach for protecting air, water and land resources favors integrated operations at the metropolitan level. A feature of integrated management that cannot be regarded too lightly is its modulating capability: by internalizing conflicts and reducing tension through less constrained trade-off bargaining; the capability for attenuation of extremes in physical strains afforded by larger scale operations where operating flexibility is greater; and economies in operation such as through centralization of laboratories and monitoring equipment and the pooling of specialized professionals and scarce technicians. The prospects for more widespread administration of water and wastewater services on an area-wide basis will be improved if considerations of public health, other water uses, planning, and guiding sound development, are brought into the picture—because comprehensive approaches cannot be justified on an economic basis alone.

New rules and regulations governing issuance of federal grants for construction of municipal treatment works stipulate that due consideration must be given to the advantages of regional (area wide) and basin sewerage facility planning, with municipalities joining together in cooperative regional treatment systems whenever feasible. Compatibility of local water pollution control plans with metropolitan and river basin water quality management objectives is also required. Joint management of water and wastewater services is on the increase, as is use of the service charge for financing combined water and wastewater operations.

There is much talk of giving the individual citizen a clearer choice among alternatives in water projects. However, present service charges too seldom approach true cost-recovery levels and the choice among alternatives is consequently proportionally obscured.

In order to enlarge service areas, two problems that have to be overcome are the method of financing by levies and/or charges and the reconciliation of disparate service and financing jurisdictions. Water and wastewater services are delivered services, as opposed to storm drainage, flood mitigation, recreation, waterfronts, etc., services the benefits of which are more unevenly diffused throughout the community. Water and wastewater services should approach as closely as possible to a true self-sustained fiscal arrangement, whether separately or jointly administered, for those cases where other means are currently employed, viz., property taxes or other indirect devices. Diffused public services, such as water-based recreation, should continue to be funded from general taxation, but local government budgets should break out and stipulate expenditures therefore, to facilitate overall water accounting and management. There are opportunities in almost every metropolis to ease pressures on local taxation by transferring part of the burden to service charges wherever this is reasonably logical.

This argument properly can be extended to certain types of user charges for pollution abatement, in the "pay-to-pollute" mode.

Over the past decade and a half the incidence of vandalism, and even probable sabotage, has greatly increased. Yet there are practical limits to the degree of security and shutdown prevention that can be provided. Managers of urban water resource works and public utilities must give serious attention to the vulnerability of system elements to disruption, and the provision of dual power feeders plus an auxiliary power source for water facilities has become a mandatory practice.

The escalating cost of manpower in municipal services has led to more extensive use of automatic controls and related devices for water services operation. However, the security of these aids is dependent upon the security of the electric power and communication systems that serve them.

"Integrated management" would presumably include: water supply, treatment and distribution; wastewater collection, treatment and disposal; drainage and flood mitigation; and perhaps water-based recreation and aesthetics. "Area-wide management" would presumably include all or nearly all of the geographical territory of a metropolitan area over the foreseeable future. No example exists in the United States of a metropolitan agency which satisfies both of these presumptive definitions and all but a very few cases are far from the mark. That is, integrated area-wide management of metropolitan water resources is a concept, not a contemporary actuality.

There are water resource specialists who see in integrated area-wide management the ultimate for capturing economies of scale and for sharpening administrative efficiency. Another view, which reflects a concern for the increased vulnerability to disruption that attends extended interdependence, holds that service reliability might be better secured by the disaggregation of administrative jurisdictions, responsibilities and authority. The middle ground is represented by those who favor several large fairly independent (and therefore secure) zones within a metropolis, with area-wide interests served by means of some sort of coordinative mechanism. This range of views is shared by particular specialists in other fields and services, notably in governmental science and transportation. However, the pros and cons of integrated area-wide management versus the status quo versus zonal administration have been only very subjectively evaluated, and existing coordinative mechanisms are seldom touted with anything like an enthusiastic consensus. Further, even champions of diffused autonomy seem to favor some form of area-wide revenue and cost sharing. More importantly, at this stage of public awareness, few fail to acknowledge the interconnectedness within "the environment" or "the ecology" of a metropolis. Thus we find widespread acceptance of an area-wide approach to the planning function of metropolitan water management, while opinion is divided on implementation and even more so on operation.

3.3 Conclusions

An ancient Greek sage is reported to have said, in effect that "to debate the question of virtue is not necessarily to be against it." Contrariwise, to investigate the feasibility of the metropolitan water intelligence system concept does not necessarily signify its advocacy nor does it mean its promotion as some kind of panacea. Considering the above summary of previous Program findings in this section, it is evident that a clear need for such a revolutionary tool cannot be demonstrated under prevailing local government institutional arrangements and national policies and priorities, although it appears to be a concept whose time will come.

Almost six years ago the Governor of California asked the state's aerospace industry to take a look at four big problems facing the state: transportation; pollution, water conservation and waste management; crime prevention; and information collection. Systems analysis was found to be of considerable potential for working on these problems, but the industry experienced one of its greatest difficulties in attempting to identify "the client." Even for urban water resources the multiplicity of governmental units at the local level that are "the client" at issue defy comprehensive systems analysis implementation, and intelligence systems comprise a component of that approach.

Chapter 21

METROPOLITAN INDUSTRIAL WATER USE (NOTE: SUMMARY OF ENTIRE DOCUMENT PROVIDED)

May 1972
ASCE Urban Water Resources Research Program
Technical Memorandum No. 16
New York, New York

1.0 SELF-SUPPLIED INDUSTRIAL WATER USE IN METROPOLITAN AREAS

The Water Resources Council addressed the question of water use by U.S. industries in its report on "The Nation's Water Resources."[1] Although the Council's report specified the total amount of self-supplied industrial water that was withdrawn, it did not delineate the use in metropolitan areas. The purpose of the analysis reported herein is to determine the amount of self-supplied industrial water withdrawn in the metropolitan areas of the U.S. Basic data from the year 1964 were used so that this study would be compatible with the Council's assessment.

1.1 General

Since 1964 is the base year used in this study, unless otherwise noted all figures will be for that year. Although absolute figures are different today, it is the basic comparison between self-supplied and total withdrawal in metropolitan areas that is important. The relative comparisons are considered valid.

The Standard Metropolitan Statistical Area (SMSA) is used to represent metropolitan America. The definition of an SMSA is an area delineated to include at least one city with 50,000 or more inhabitants. It includes the county of such a central city and adjacent counties that are found to be metropolitan in character and economically and socially integrated with the county of the central city. As of December 31, 1965 there were 224 SMSAs representing a 1960 total population of 116,968,692, which compares with a population of 179,323,175 in the same year for the entire nation.[2]

There is no publication or information readily available that contains a summary of the water withdrawn in the SMSAs. A 1968 report by the USGS[3] used 1963 Census of Manufacturers data to summarize self-supplied industrial water withdrawals by state and by region. The USGS report did not delineate self-supplied industrial water withdrawals in metropolitan areas. The U.S. totals reported in the USGS study are summarized in Table 21-1.

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[1] U.S. Water Resources Council. 1968. *The Nation's Water Resources.* Washington, DC.

[2] U.S. Department of Commerce, Bureau of the Census. April 1967. *County and City Data Book, 1967.*

[3] Murray, C. 1968. *Estimated Use of Water in the United States, 1965.* USGS Circular 556.

TABLE 21-1

Summary of 1965 USA Water Withdrawals From 1968 USGS Study

Total Self-Supplied Industrial Withdrawals in USA (mgd)			
	Other than Power	Thermal Electric Power	Totals
Ground	6,800	1,100	7,900
Surface	30,000	91,000	121,000
Saline	8,900	36,100	45,000
TOTAL	45,700	128,200	173,900
Public Withdrawals (mgd)			
Industrial and Commercial Uses		7,800	
Domestic Uses		16,000	
TOTAL		23,800	

It would be a major undertaking to reorganize the basic Census Bureau data to obtain corresponding water withdrawal figures for metropolitan areas. Consequently, an indirect method was developed using available value added[4] estimates and total gross industrial output figures for industrial groups for each SMSA to determine metropolitan industrial water withdrawals. One advantage of this method is that it could be possible to estimate future water withdrawals given future estimates of two economic indicators (value added and gross output in dollars for industrial groups). The method used to estimate water withdrawals within SMSAs is summarized in the next section.

1.2 Method Used to Estimate Industrial Metropolitan Water Withdrawals

The Census of Manufacturers data[5] obtained by the Bureau of Census gives water use figures for four standard industrial classification (SIC) groups consisting of manufacturing, mining and mineral processing, ordnance, and construction. In its

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[4] Value added by manufacturer is derived by subtracting the total cost of materials from the value of shipments and other receipts, and adjusting the resulting amount by the net change in finished products and work-in-process inventories between the beginning and end of the year (value added by manufacturer to a product between the time it becomes his responsibility until it is sold).

[5] U.S. Department of Commerce, Bureau of the Census. 1966. *Census of Manufacturers, Water Use in Manufacturing*, Volume 1, Washington, DC.

assessment, the Water Resources Council points out that water use by government facilities producing ordnance materials was not reported in the Census of Manufacturers data, but that water used by privately owned ordnance manufacturers was reported. However, withdrawals by government and privately owned ordnance plants were considered small enough by the Council to be ignored in their study. Both government and private ordnance withdrawals also are ignored in this study because of their relatively small water use. Because of the minor quantities of withdrawals involved in the construction industry, those withdrawals were not reported in the Council's assessment or in this study. The two remaining SIC categories only (mineral industries and manufacturing) are addressed in this report.

In a 1958 inter-industry study the Office of Business Economics of the Department of Commerce classified manufacturers into 51 sectors, and the basic data available for this study were categorized into these 51 sectors. Recent Census of Manufacturers data gives national water usage figures for 120 industrial groups. It was necessary to reorganize the 120 industry groups into the 51 manufacturing sectors to correspond with the 1958 inter-industry study and the available basic data. Ratios of total water withdrawal per dollar of value added based on the national data were determined for each of the 51 sectors. Data on value added for the mineral industry were not available, but the water withdrawals were related instead to dollar output.

The basic data comprised information on computer tapes consisting of total value added figures for each of the 51 manufacturing sectors and dollar output for the six mineral industry sectors for all non-New England SMSAs and every county in the USA. These basic data were obtained on magnetic tape from the Institute of Defense Analysis (IDA), Department of Defense. The value added and gross output data were determined for IDA for each county in a study by Fauccet in 1968.[6]

The IDA data tape includes SMSA summaries (except for the New England states),[7] and the total water withdrawal per industrial sector was determined for each SMSA by multiplying the coefficients by the value added or gross output. The magnetic tape received from IDA was put on the Colorado State University computer, and programs were written so the analysis of total water withdrawals for each SMSA could be made by computer.

The next step was to determine how much of the industrial water withdrawal in each SMSA was publicly supplied and how much was self-supplied. (A further delineation of self-supplied withdrawals into surface, ground and saline or contaminated sources was also made). It was not possible to obtain this information

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[6] Fauccet, J. 1968. "1963 Output Measures for Input-Output Sectors by County." Prepared for the Office of Civil Defense, Office of the Secretary of the Army, Washington, DC.

[7] In New England, each county in an SMSA was identified and the total water withdrawal per industrial sector was determined by summing water uses of appropriate counties. Some counties were included in more than one SMSA, and in these cases the county withdrawals were apportioned evenly to each SMSA.

for the mineral industry. However, according to Kaufman,[8] almost all of the water used by the mineral industry is self-supplied.

Ratios for the manufacturing industry were determined for each state of: public-supplied versus total withdrawals; self-supplied versus total withdrawals; and self-supplied surface, ground and saline versus total withdrawals. These ratios were based on Census of Manufacturers data. These are the ratios in the analysis that are most susceptible to error. The inherent assumption is that the same ratios are applicable statewide and for all industrial sectors within the state. In all likelihood these ratios vary within states and from industrial sector to industrial sector, but the relative values of how much water is obtained from public supplies and how much is self-supplied is considered reasonable.

The public-supplied and the self-supplied withdrawals for each non-New England SMSA were then determined by multiplying the above ratios times the total SMSA withdrawals. Withdrawals for New England SMSAs were obtained by adding the withdrawals of the several counties associated with each SMSA.

The Institute of Defense Analysis did not determine any figures for the thermoelectric power industry by county. Therefore, it was not possible to estimate the water withdrawals of the thermoelectric power industry by SMSA.

1.3 Results

The total water withdrawals by SMSAs for the mineral industries are contained in Appendix E.[9] The total water withdrawals by SMSAs for manufacturing sectors are contained in Appendix F. Names of SMSA's are given in Appendix B for the SMSA identifying code numbers cited in Appendices E and F. The results are summarized in Tables 21-2 and 21-3 for both the nation as a whole and for SMSAs only.

The USGS report estimated the total self-supplied industrial water withdrawals for 1964 (not including thermoelectric power) to be 45,700 mgd. The estimate arrived at in this study was 49,760 mgd or about 9% greater than that reported by the USGS. The self-supplied withdrawals for the entire nation were obtained in this study using the IDA data by the method outlined above. The only difference was that for this particular calculation all counties were summarized rather than only SMSAs. The USGS report was based on Bureau of Census data that used about 85% of the largest manufacturers in developing estimates. Although the 15% omitted would be small water users, it would account for some of the difference between the USGS total water withdrawal estimate and the larger value calculated in this study.

As indicated in Tables 21-2 and 21-3 the total industrial withdrawal (not including thermoelectric power) in SMSAs is 39,120 mgd. This is 70% of the total national

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[8] Kaufman, A. and M. Nadler. 1966. *Water Use in the Mineral Industry*. U.S. Bureau of Mines, Circular 8285.

[9] Appendices are NOT included in this document. For further information, please refer to the original document.

industrial withdrawal (excluding power) of 56,090 mgd. Of the 39,120 mgd of industrial water withdrawn (other than power) in SMSA's, 34,480 mgd (or 88%) is self-supplied.

TABLE 21-2

Summary of Water Withdrawals in USA, 1964 (mgd)
(Tucker, Millan and Burt)

Use Category	Total	Public	Self-Supplied Industrial			
			Surface	Ground	Saline*	Total
Mineral Industry:						
SMSA only	820					820
Total USA	3,390					3,390
Manufacturing Industry:						
SMSA only	38,300	4,640	22,100	3,930	7,630	33,660
Total USA	52,700	6,330	30,700	6,050	9,620	46.370
Total Industrial (Other than Power):						
SMSA only	39,120	4,640				34,480
Total USA	56,090	6,330				49,760
Total Thermal Electric Power in USA**	128,200		91,000	1,100	36,100	128,200
Domestic Withdrawals**	16,000	16,000				
Total USA Industrial, Power, and Domestic Withdrawals	200,290	22,330				
Total USA Self-Supplied Industrial						177,960

* Salt water or otherwise contaminated water.

** From USGS Report

The above is particularly significant when considering total metropolitan water withdrawals. The total domestic water withdrawal in 1964 reported in the USGS study was 16,000 mgd. Assuming that 75% of this was withdrawn in SMSAs (SMSAs accounted for 65% of the population in 1960) and supplied by public systems, then the total metropolitan water withdrawal (excluding power) would be about 51,120 mgd (39,120 mgd + .75 x 16,000 mgd). About 34,480 mgd were self-supplied industrial (other than power) SMSA withdrawals, or about 67% of the metropolitan water withdrawals (excluding power) were from self-supplied industrial sources.

The above figures, however, do not include withdrawals by electric utilities because data discriminating water used by electric utilities on an SMSA basis were not available. Total withdrawals by electric utilities as reported in the USGS study are summarized in Tables 26-2 and 26-3. The total thermoelectric withdrawals for the USA in 1964 were 128,200 mgd that were entirely self-supplied. The electric utilities, thus, represent about 64% of total water withdrawn for industry, power and electric uses in the United States and about 72% of the self-supplied industrial water withdrawals in the U.S.

TABLE 21-3

Summary of Industrial Water Withdrawals
In the USA, 1964 (mgd)
(Tucker, Millan and Burt)

Industrial Withdrawals in USA			
Self- and Public-Supplied		**Self-Supplied**	
Mineral Industry	3,390	Mineral Industry	3,390
Manufacturing	52,700	Manufacturing	46,370
Thermal Electric**	128,200	Thermal Electric:	
		Ground	1,100
		Surface	91,000
		Saline*	36,100
TOTAL	39,120	TOTAL	34,480

* Salt water or otherwise contaminated water.

** From USGS Report

Since all thermoelectric withdrawals are self-supplied, any occurring in SMSAs will only increase the percentage of self-supplied withdrawals within the SMSAs. For example, if 50% of the thermoelectric withdrawals occurred within SMSAs, the total self-supplied withdrawal within SMSAs would be 98,580 mgd (34,480 + 64,100) or 95% of the total SMSA withdrawals for industry of 103,220 mgd (39,120 + 64,100). The SMSA self-supplied industrial withdrawals would then be 86% of the total SMSA withdrawal (including domestic) of 115,220 mgd (39,120 + 64,100 + .75 x 16,000), a significant increase over the 67% when power was excluded.

The sensitivity of the percentage of self-supplied industrial withdrawals of total metropolitan water withdrawals to various assumptions of metropolitan thermo-power withdrawals is shown in Table 21-4. When the assumed percentage of SMSA thermo withdrawals is varied from 30% to 80% the self-supplied industrial as a percentage of total metropolitan withdrawals varies only from 81% to 89% (assuming 75% of domestic withdrawals occur in SMSAs). The sensitivity of the assumption that 75% of domestic water withdrawals occur in SMSAs is also shown in Table 21-4.

TABLE 21-4

Sensitivity of Self-Supplied Metropolitan Withdrawals To Various Assumed Metropolitan Thermo Withdrawals

Assumed Metro Thermo Withdrawals %	Self-Supplied Individual as % of Total Individual Withdrawals	Self-Supplied Individual as % of Total Metro Withdrawal	
		(1)	(2)
30	95	81	79
40	96	84	82
50	96	86	84
60	96	87	85
70	97	88	87
80	97	89	88

Assume 75% of Domestic Withdrawal in SMSAs
Assume 90% of Domestic Withdrawal in SMSAs

The results obtained in this study are not assumed to be exact values but rather are approximations. The accuracy for a particular SMSA depends to a large extent on how nearly the national average water use per industrial sector approximates the actual water use, and on the accuracy of the assumption that the ratio of self-supplied to total SMSA withdrawals is constant statewide and for all industries. The results for the total nation and for all the SMSAs are considered to be a good estimate.

1.4 Summary

The water withdrawals in 224 SMSA's were evaluated based on 1964 basic data provided by the IDA. The total metropolitan water withdrawal (not including thermo-power) was estimated to be 51,120 mgd. This amount includes manufacturing, mineral industries, and domestic uses. Of this total metropolitan withdrawal about 34,480 mgd, or 67%, is self-supplied by manufacturers and mineral industries.

The total metropolitan mineral industry and manufacturing withdrawals (domestic and power withdrawals excluded) were estimated to be about 39,120 mgd. About

88% of the metropolitan mineral and manufacturing water withdrawals are self-supplied.

There was no information available to permit making an estimate of the metropolitan water withdrawals for thermo-power. The total U.S. withdrawal for power was 128,200 mgd, all of which was self-supplied. Therefore, whatever the metropolitan power withdrawals are, they are self-supplied.

If it is assumed that 50% of the power withdrawals and 75% of the domestic withdrawals are in SMSAs, then self-supplied industrial withdrawals (mineral, manufacturing and power) represent about 86% of the total SMSA water withdrawal. Public controlled water supplies appear to account for a relatively small portion of the water withdrawn in metropolitan USA.

Chapter 22

RESIDENTIAL STREETS

1974
Objectives, Principles & Design Considerations
Published Jointly By: ULI; ASCE; NAHB

FOREWORD

Publication of this report under the joint auspices of the American Society of Civil Engineers (ASCE), the National Association of Home Builders (NAHB), and the Urban Land Institute (ULI) is a unique and significant action. This publication represents the combination of three entirely different perspectives in a united effort to provide guidance and counsel on a subject of common interest and importance—the setting of objectives, principles, and design considerations to be applied to the development of residential streets.

This report is the first of what is hoped will be a series of jointly sponsored and reviewed reports, and is the culmination of over two years of mutual involvement. During this time, the content and direction of the report evolved from an initial attempt to create a manual for residential street development complete with design standards for alignments, configurations, and construction details, to this report setting forth objectives, principles, and design considerations.

It is hoped this report will provide guidance for communities of all scales and differing locales, in adopting residential street design and development regulations that are responsive to local conditions in general and specific sites in particular.

ACKNOWLEDGEMENTS

An undertaking such as this had multitude of participants. The need for a report on residential street development practices originated in the NAHB Land Use and Engineering Committee under the guidance of David P. Rhame. The initial survey and report that was the basis for further involvement of ULI and ASCE was prepared by the NAHB Research Foundation and its staff. The various committees shown are comprised of members of the related organizations who had responsibility for the review and comments on various drafts. Staff members of these organizations were responsible for processing and interpreting review comments into the final document. A special note of thanks goes to D. Earl Jones, Jr., Chairman of the ASCE Review Committee, for his efforts in preparing a major revision of the manuscript as a result of the review process.

1.0 INTRODUCTION

1.1 History

In the beginning of the 1940s, American cities faced public dissatisfaction with thousands of miles of alternately dusty and muddy unpaved streets that were costly to maintain. Cities adopted incentives to stimulate street paving. Property owners virtually stood in line to have their streets paved on a shared cost basis.

After World War II, a home on an unpaved street presented a poor public image for its owner. In some areas, wide residential street pavements became prestige symbols for individuals, neighborhoods, or even entire communities.

Homebuilders and land developers quickly perceived the public's need and desire for paved streets. As a result of the increased market for new homes served by paved streets, unpaved streets in new residential developments soon became a rarity.

Administration of large municipal street paving programs and control of new housing developments necessitated adoption of municipal street construction standards and specifications. These standards and specifications usually were patterned after state highway department standards because of ready availability. Since that time, formerly rural crossroads have metamorphosed into established satellite communities, or large cities in their own right. As these new jurisdictions have emerged, street improvement standards have been borrowed from established neighboring cities.

Little opportunity for desirable research in the standards-setting process was available. Initially adopted standards often required upgrading to correct obvious performance deficiencies and reduce mounting municipal street maintenance costs, or to reflect changing state highway department practices necessitated by increasingly heavy truck traffic.

The early development of municipal street standards was a response to immediate problems. The original standards based on highway standards were often utilized and have since been followed very largely out of habit. Little finite statistical information or research has been focused on refinement of residential street improvement standards, although extensive studies of higher order streets have been done. The distribution of past research efforts was grossly imbalanced because residential street mileage constitutes the major portion of any urban street system.

Before 1940, municipalities controlled new housing areas through zoning ordinances and subdivision regulations. Municipalities regulated density by stipulating lot area and requiring property line setbacks. Those rigidly applied setbacks and yard dimensions produced repetitious street patterns and massively monotonous streetscapes. Such patterns have little redeeming aesthetic appeal and are becoming less acceptable to the home buying public.

Changing lifestyles, value perceptions, and construction costs in the early 1960s, stimulated development and promulgation of the cluster-housing concept, generally termed Planned Unit Development (PUD). Successful PUDs required cost-effective designs that balanced initial cost against amortization, operation, maintenance, and replacement costs. Development of PUDs necessitated satisfactory performance of street improvements, if only because of the long-term, potentially vocal involvement of organized PUD residents.

Cluster-planning and PUDs focused public attention upon open spaces and upon the landscape including the streetscape. The early focus has been a significant factor in today's environmental awareness.

By the early 1960s, serious attention had turned toward improved land planning for suburban areas. Among others, the ASCE, the Federal Housing Administration (now Department of Housing and Urban Development), the Building Research and Advisory Board, the Urban Land Institute, the architectural departments of many universities, and the NAHB produced studies and evaluations of innovative community and neighborhood design. The Planned Unit Development was evolved and refined along with density zoning. Yesterday's innovations now are accepted and implemented widely.

One factor in the acceptance and success of PUDs and cluster-housing was the freedom allowed planners and engineers to lay out and interrelate street patterns, open spaces, and building sites. The design freedom inherent in PUD planning made possible the preservation of natural vegetation and interesting topography, and produced more desirable living environments by permitting sound economies in land development with benefits accruing to the residents and the community.

1.2 Problems

A major obstacle to acceptance of more effective or innovative land development and improvement designs was the deep-rooted, traditionally rigid design approach. This has been particularly evident in street design; local governments still often pattern design and construction requirements after state highway design and construction practices. Such patterning is reasonable for major thoroughfares but often is inappropriate for local residential streets. Traffic characteristics, construction, maintenance, and performance needs on residential streets are vastly different from those on highways.

Rising land development and housing costs cannot be attributed solely to unnecessarily high street improvement standards. Nevertheless, such street standards may contribute significantly to the cost problem without assuring the best long-term value for either community or homebuyer.

Until 1967, the Federal Housing Administration (FHA) published *Street Improvement Standards* for all metropolitan areas and for many non-metropolitan communities. The reason for its discontinuance was that the problem of "no local standards" was being supplanted by new problems posed by excessively rigorous local standards.

FHA's *Street Improvement Standards* had been initiated about 1940 to upgrade local residential street paving practices. The guidelines established either FHA standards or local community standards, whichever was higher, as the minimum acceptable practice in FHA-analyzed land developments in a locale. Since one of FHA's two fundamental objectives was to improve housing conditions and the quality of the living environment, it is significant that a factor in the discontinuance of *Street Improvement Standards* was their unwillingness to endorse economically unjustifiable construction requirements.

To view the FHA decision in perspective, it must be recognized that by 1967 even small communities had adopted pavement design and quality standards and required paving all newly platted residential streets. The FHA *Street Improvement Standards* had served its purpose.

The home building industry supported FHA's early street improvement objectives because it also was concerned with enhancing the quality of the living environment. Streets and street furniture are highly visible and the successful builder/developer realized that the design and appearance of the streets could either enhance or detract from the subdivision. The builder/developer was aware also that although a competitive market required that construction dollars produce maximum buyer appeal, those dollars also had to provide long-term quality and durability. If they didn't, the builder/developer's reputation suffered.

The builder/developer often is the person most sensitive to street improvements costs. He sees them as a significant element in total housing construction costs and would like to minimize them. At the same time, he knows that inadequate or deteriorating streets can be a major cause of buyer dissatisfaction. He, therefore, has strong motivations to eliminate unnecessary improvement costs but still assure quality improvements. This self-interest closely parallels homebuyer and community interests. Everyone benefits from improvements that are functionally adequate, durable, and cost effective.

1.3 This Effort

The NAHB Land Use and Engineering Committee, and the NAHB Research Institute originated the development of this document in an effort to identify optimum residential street design and construction standards and practices. A national survey found few municipalities using performance-oriented land development standards. Diverse and seemingly inconsistent standards were found to be in effect throughout the country, and nationally recognized street design standards of the Institute of Traffic Engineers and others had been adopted in few communities.

The survey findings prompted the Committee's endorsement of a search for subdivision regulations emphasizing performance needs, similar to ordinances previously adopted by several municipalities. Such ordinances would have stated requirements in terms of objectives, principles, criteria, and illustrative rather than specification standards. These ordinances would have provided increased flexibility and opportunity for design professionals to meet varying conditions assuring reasonable levels of long-term performance.

The NAHB Research Foundation prepared the initial draft of this document after extensive research. The draft incorporated the current best standard practice in design and construction of residential streets.

The initial draft was circulated for review and comment to numerous individuals, agencies, and associations. Although recognizing unanimous consensus is virtually impossible, it was expected that the comments would allow NAHB to fine tune the recommendations to obtain the greatest degree of consensus.

1.4 The Result

Street improvement design guidance reflected herein could not be expressed entirely in performance terms, nor could any single set of design standards be suitable for all local conditions that might be encountered. This report is not intended to be a comprehensive design guide, but is intended to focus upon a number of significant considerations that may be overlooked occasionally. The information presented is considered appropriate for use as a general guide toward identification of optimally functional, durable, cost-effective residential street design, construction, and maintenance practices. Variations from the basic guides, which may be useful under some conditions, are described wherever feasible.

Hopefully, consideration of these guides by developers, consultants, and local officials will help achieve functional residential streets in good living environments for people and will help assure long-term public and private economies.

The complexity of the subject, the range of possible environmental design conditions, and the difficulties of achieving consensus regarding residential street design elements are recognized. Consensus is used advisedly because unanimous agreement is virtually impossible among persons having different interests and motivations. ASCE and ULI reviewers often expressed divergent opinions. Significant minority views accordingly are incorporated for consideration as variations to recommended practices.

To the degree that conservation is consistent with utility, safety, and reasonable user convenience, it is intended herein to reflect a conservation bias. A conservation bias may seem to favor minimum builder/developer investments, but it also benefits property owners who must ultimately pay for that investment and the community that must bear operation and maintenance costs. Consequently, evaluations reflected herein emphasize deferral or permanent avoidance of any capital investment that is unnecessary to assure functional utility, low maintenance cost, safety, and reasonable user convenience. The objectives sought are conservation of construction materials and labor, and conservation of the individual's personal energy as a pedestrian or cyclist, or his fuel energy as a driver. Although the builder/developer may be the first recipient of benefits from sound conservation measures, he certainly is not the last one. A conservation bias increasingly is an essential objective for almost all human activities if the quality of life in our nation is to be optimized and preserved.

1.5 The Need

ASCE, NAHB, and ULI perceive a significant need for guidance in the selection of appropriate residential street improvements, especially if widely publicized forecasts prove correct and construction during the next 30 years equals in volume all construction since 1776. Such a volume of construction would necessitate major growth in present cities and suburbs and short timeframe development of entire new cities. The ASCE Review Committee's focus, therefore, was upon the future, to free the cities of the future from many of today's evident problems.

As our nation approaches its bicentennial, many urban streets built over 20 years ago are sorely in need of complete reconstruction. To that mounting need, add the annually increasing inventory of street mileage to be maintained. This warns of a forthcoming maintenance crisis and emphasizes the need to initiate sound conservation measures.

It is hoped that anyone disagreeing with any part of this report will advise the publisher of their disagreement, including detailed reasons and alternative recommendations. Such information will help guide future revisions and enhance the document's value to communities.

2.0 OBJECTIVES

Residential streets comprise a major portion of urban street mileage, for which little up-to-date guidance has been published. Highways, freeways, expressways, and major thoroughfares present different needs, and different values, and have been extensively treated in many other publications.

Basic objectives of this document are:

- To provide guidance that is consistent with today's conditions and need, but anticipates—insofar as possible and practical—physical, social, economic, and transportation trends that may be significant in shaping tomorrow's urban lifestyles.
- To consider the special problems and needs of residential streets.
- To provide guidance toward and alternatives for safe, convenient vehicle, bicycle, and pedestrian movements and circulation within residential communities.
- To encourage creation of harmonious relationships between streetscapes and the residential environments they affect and serve.
- To make the street and its improvements contribute to increased individual safety and livability.

- To stimulate consideration of appropriate balances among initial construction costs, amortization costs, operating costs, maintenance costs, and replacement costs, thereby encouraging designs that will minimize total average annual costs.
- To recognize: 1) absolute convenience and total safety are unobtainable at any cost; 2) some inconvenience and minor hazards are inherent even in the best practical design; and 3) important economic savings may be accomplished with only minor inconvenience.
- To stimulate design decisions that will conserve materials, construction labor, construction equipment, land, and environmental values.

The current energy crunch and the need for long-term energy conservation is a consideration that will shape future development—e.g., increased development of public transportation requiring more intensive occupancy of urban land for economically sound operations; and increased use of Planned Unit Developments with their unique design and land conservation features.

- To stimulate adoption of consistent standards within a community to the extent that construction, operating, and maintenance economies will result, but not to the extent that the use or evolution of optimum designs and construction for unique or special conditions will be inhibited.

3.0 PRINCIPLES

The following statements are presented as principles in related categories.

3.1 General Principles

Major elements of the street system may be used to help define and buffer different land use areas enhancing their identity and cohesiveness.

- Street and pedestrian circulation patterns in a new residential area should be compatible with objectives established by a community's major street plan.
- Planned layout should minimize overall length of streets.
- A residential area should be conveniently accessible from major streets and highways.

Access points to major highways, thoroughfares, and arterial streets should be limited in number, given special design consideration, and, whenever possible, located where other features are not competing for driver attention. Cul-de-sacs and courts enhance privacy and the lack of through-traffic improves safety.

- Driveway entrances should be avoided on arterial streets and, wherever possible, on collector streets.
- Through-traffic on minor residential streets should be avoided.

- Residential streets should provide safe and convenient access to housing.
- Paved access must be available to fire, ambulance, and police vehicles to within 100-feet of the principal entrances to dwellings. Closer access, for infrequent use, may be across unpaved areas.
- All dwellings must be accessible by emergency and service vehicles.

3.2 Safety Principles

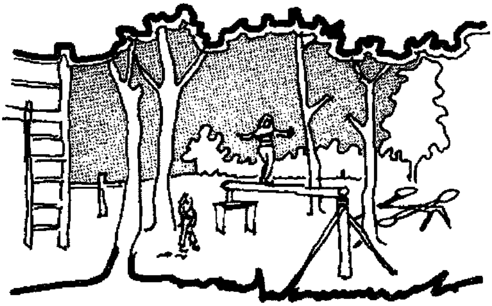
- Assure necessary lighting along streets and walkways.

The need for lighting may also be minimized by imaginative design wherever possible.

- Use of signs should be minimized and signposts should be unobtrusive.
- Informational signs should not compete with traffic control signs for driver attention.
- Use of break-away street furniture should be considered whenever possible and should be grouped for aesthetic as well as safety values.
- Sight distances should be consistent with probable traffic speed, terrain, alignments, and climatic extremes.

The surest way to improve pedestrian safety is to remove pedestrian traffic from areas of potential conflict with automobiles.

- Separate pedestrian, bicycle, and vehicular traffic.
- Limit through-traffic on residential streets.



Children will use streets to play if no alternative plan is provided for them

Street planning and dwelling unit siting should be coordinated to reduce the incidence of housing on through-streets.

- Assure visibility of parks, play areas, and interior block open spaces from the street.

3.3 Design Principles

- Horizontal and vertical street alignments should relate to the natural contours of the site insofar as is practical and should be consistent with other design objectives.
- Horizontal and vertical alignment of streets should be selected to minimize grading quantities.
- Wherever possible, street layouts should be planned to avoid excessive runoff concentration and the need for storm sewers.

Streets crossing drainageways often may be improved advantageously as dams, providing an urban pond that can enhance the environment and the neighborhood, as well as provide some flood storage to help attenuate peak runoff flows. Water quality and other factors must be considered in the decision equation. Reduced area for street pavement is only one source of potential economy; others should be considered. Small storm sewers are expensive in relation to their capacity, both to construct and to maintain, and often can be avoided by well-designed street layout.

- Street planning should relate to overall community planning.

Places, lanes, and cul-de-sacs should provide direct access to residential units; sub-collectors may provide more direct access to higher density uses such as townhouse or apartment-clusters—they may be used also as scenic drives and to locate minor retail and service facilities. Collector and arterial streets are suitable for the location of neighborhood or community-level retail and service facilities; they provide access for schools, mid-rise, and high-rise apartment structures.

3.4 Construction Principles

Pavement costs usually are a function of the volume of materials that must be handled—the thinner the total thickness of a pavement section, the less it usually costs.

- Pavement designs should be appropriate for the specific traffic load, sub-grade soil, surface drainage, groundwater, and climatic conditions existing at the pavement's location.

As the assured bearing strength of a soil increases, the thickness of pavement necessary for traffic load distribution to the sub-grade soil decreases. It may be optimally economical to improve or stabilize natural sub-grade soils beneath pavements. Dollars spent on sub-grade improvement may produce greater savings in pavement construction and maintenance costs. Lime stabilization of plastic bentonitic clay is an example. Full-depth asphaltic concrete or Portland Cement concrete pavements generally can provide necessary load distribution characteristics with a thinner total paving section than a granular base with a thin wearing surface.

The resultant saving in pavement thickness translates into significant savings in the volume of pavement materials handled, hence construction economies. The resultant pavement generally will be more durable, if only because it is more water-resistant. (These generalities are oversimplified for the purpose of suggesting approaches worth exploring in more specific situations.)

Pavements so designed may be either more expensive or less costly than those usually found in local government jurisdictions, but they should provide improved service and utility consistent with minimum community maintenance cost.

- The thickness of a pavement should be a function of both the assured load supporting value of the earth sub-grade beneath the pavement and the wheel load distribution characteristics of the pavement materials.
- Pavement edge treatments other than curb and gutter may be used where conditions permit adequate drainage and the roadway base will not be adversely affected.

Elimination of curbs will permit use of very slight longitudinal roadway gradients. In such practice, the roadway crown assures lateral roadway drainage to roadside or mid-block swales that can effectively provide runoff detention storage and sub-base drainage. The importance of detention storage for runoff management is increasing as urban areas expand.

3.5 Intersections

- Whenever possible, residential street layouts should be planned to avoid four-way intersections.

The total area of residential street pavements bears a direct relationship not only to initial cost, but also to amortization and maintenance costs.

- Residential street widths normally should be the minimum consistent with safety and adequate fulfillment of street function. Pedestrian accidents are somewhat proportional to street crossing travel distance.

Paved area generally should be minimized insofar as practical to increase permeable soil area and open green space within the community.

- Paved area within intersections should be minimized.
- Oblique intersections should be avoided.
- Safe sight distances at intersections should be assured.
- Turning lanes at heavily traveled intersections should be provided.
- To the extent feasible, the number of street intersections should be minimized.
- Larger corner radii should be avoided.

Large corner radii are a temptation to increase speed where visibility is poorest and conflicts are most likely. Residential street corner radii of about 15-20 feet have generally been found effective.

Intersections and driveways on the inside of a curve should be avoided particularly.

- Wherever possible, intersections on curves should be avoided.

3.6 Parking

- A balance between off-street and on-street parking should be established.

The relative costs of off-street and on-street parking should be carefully explored. Relative excavation quantities, runoff alternatives, maintenance costs, and percent of publicly owned land are important considerations.

- Dwelling unit entrances should relate to parking locations to assure convenient and safe access.

Plant selection should consider size at maturity, seasonal foliage differences, and maintenance needs including feeding, pruning, spraying, irrigation, and necessary replacement frequency.

- Large parking areas should be subdivided by meaningful and maintainable planting strips, bays, and islands to provide visual screening and thereby reduce adverse impacts on the aesthetic value of the landscape.

The neighborhood layout should assure parking on collector streets will be inconvenient.

- Parking should be adequate for both residents and guests.

3.7 Pedestrian and Bicycle Paths

Often it will be advantageous for reasons of public convenience, safety, initial and maintenance costs, ease and speed of access, and optimized land use to route pedestrian traffic outside of street rights-of-way.

- Pedestrian access to schools, shopping, and existing or possible public transportation load points should be convenient.
- Pedestrian and bicycle-way alignments should have a reasonable relationship to foreseeable movement desires, parking, and community facilities, and should be safe, secure, and attractive.

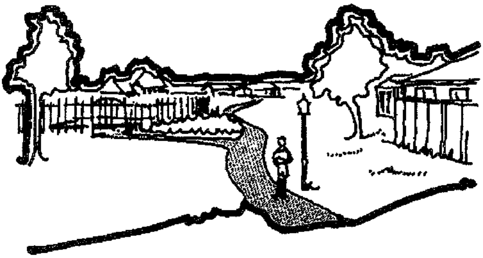
As public transportation assumes a significant role in areas where it has been nonexistent, serious consideration should be given to future fulfillment of passenger needs for shelter, and secure bicycle storage at public transportation loading points. Neighborhood planners should be thinking in terms of how such facilities might be provided at reasonable cost and without adverse aesthetic impact. Locations might be pre-selected and protected by appropriate easements, and also landscaped in anticipation of future needs.

- Potential pedestrian, bicycle, and vehicular conflicts should be minimized.
- Pedestrian and bicycle travel routes should be selected to have minimum practical change in grade (elevation) throughout their lengths.

One-square-mile of residential property often contains as much as 44 miles of sidewalk paralleling roadway pavements—currently about a $500,000 investment. The need for and the benefits from sidewalk investment on minor residential streets obviously should be carefully examined. Sidewalks should be eliminated where they are made unnecessary by an alternative pedestrian system with low traffic hazards. Installation of sidewalks may increase the impervious area of a land development by approximately 3%, which could have significance in urban drainage planning

Pedestrian walks should be provided to improve or assure public access at locations offering unusual overlooks or other particularly interesting physical features.

- Provisions for street side sidewalks should be a response to need rather than to arbitrary policy.



Paths and sidewalks should connect destinations

3.8 General Landscaping

- Natural landscape features should be preserved.

Natural terrain, terrain sculpturing, and/or low-maintenance plantings may be used to induce pedestrians and bicyclists to follow intended walks and paths.

- Select right-of-way landscaping and screen plantings to have mature size consistent with vehicular and pedestrian traffic visibility needs.

Street landscaping offers possibilities for improving neighborhood appearance and aesthetic value. The landscaping must be planned to contribute to safety and may offset needs for certain other capital improvements. Appropriate plantings can serve effectively as barricades or barriers to protect adjacent properties from vehicles and to catch out-of-control vehicles.

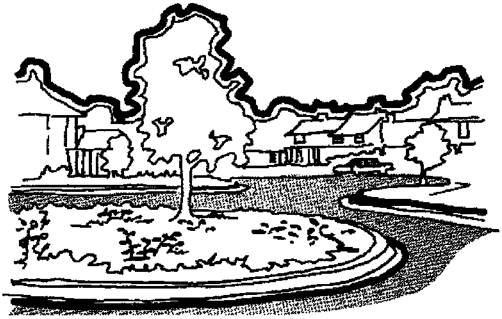
Split-level streets, meaningfully sized islands containing specimen trees, protective walls and embankments, and other innovative techniques should be encouraged to provide aesthetic enhancement and environmental preservation of residential areas.

4.0 DESIGN CONSIDERATIONS

4.1 Purpose

The objective in presenting residential street design considerations is to direct the reader's attention to a broad range of factors that should be considered in the selection process. The considerations are a mixture of criteria, specification standards, and performance experience, bearing upon practical urban residential street design concerns.

It should be clearly understood that a *minimum* defined to assure *functional* utility may be less than *desirable* from some standpoints. The difference between *minimum* and *desirable* generally is a matter for individual and group value judgments during the decision-making process.



Street landscaping can be functional and beautiful

4.2 Residential Street Classification

Residential streets historically have been classified into a single category. A continuing needs exists to develop a hierarchy of residential street classification emphasizing a greater sensitivity to the differences among residential streets.

Residential street classification decisions should be responsive to the amount and kind of traffic a street must serve—that is related to the number and kind of living units served, their spacing, and the transportation habits of the occupants. Transportation habits reflect such factors as vehicle ownership, number of vehicle trips generated by family members, and the time distribution of those trips. Transportation habits also are influenced by family composition, convenience of public transportation, convenience of service and shopping facilities, climatic characteristics, and the nature of family employment. Family income often correlates closely with transportation habits, but it can be a misleading indicator.

Street classification based on width and type of pavement cross-section is insensitive to varying user requirements. Such classification often results in standardized dimensions and designs for residential and other streets, and construction must be in accordance with one or another of those standards. This approach can seriously inhibit designs based upon actual need, and has the effect of forcing all construction to provide for the most rigorous preconceived conditions, which may be adequate or inadequate in actual practice.

Classifications based on traffic densities usually have been patterned after highway design practice. Although such a classification is more satisfactory for design purposes than some other systems, it too has significant shortcomings. Traffic density classification generally has not considered the very low range of traffic densities common on much of the street mileage in residential areas.

Traffic conditions can be predicted quite accurately for most new subdivisions as well as changes in the projected traffic patterns due to development of surrounding areas. The classification assigned a particular street can and should relate to its particular traffic projection.

Average Daily Traffic (ADT)—The average total number of vehicles traversing a highway or route on a typical day is a principal factor in the design or alteration of highways and arterial streets. For highway traffic volumes, traffic distribution throughout the day can be approximated from ADT, and has proven the ratio of peak hour traffic to ADT to be relatively constant. Peak loading factors, defining short duration high traffic volumes, especially influence highway intersection design and control.

ADT and peak loading factors are useful tools for urban traffic engineers who find them generally to be indicative of traffic usage and behavior on the more heavily traveled urban streets although often inapplicable to residential streets. For traffic loads in the 5,000-25,000 ADT range, ADT correlates closely with traffic behavior. Such volume requires speeds 30 miles per hour (mph) or higher and few interruptions. Such traffic usually is making trips longer than one mile, and has a particular destination. Somewhere below the 5,000 ADT level, the concept of projection by use of the ADT becomes unrealistic and inapplicable.

Recognizing that traffic volume (ADT) provides one available basis, despite its uncertainties and irrelevancies for classifying residential streets, a generalized classification could be described as follows:

Classification	Usual ADT Range
Place	0-100
Lane	75-350
Sub-collector	200-1,000
Collector	800-3,000
Arterial (or Higher)	Over 3,000

In reality, however, the actual ADT, particularly on collector streets that exist all over the U.S., is significantly higher than the upper limits shown. These streets are serving without public complaint and without a high incidence of accidents or other problems. This clearly demonstrates the use of ADT, as the principal basis for residential street design, will tend toward over design particularly at the collector street level.

In this light, the following functional definitions of street classifications are provided:

- **Place**—a short street, cul-de-sac, or court. The primary purpose of a place is to conduct traffic to and from dwelling units to other streets within the hierarchy of streets. Usually a place is a dead-end street with no through-traffic and limited on-street parking.
- **Lane**—a short street, cul-de-sac, or court, or a street with branching places or lanes. The primary purpose of a lane is to conduct traffic to and from dwelling units to other streets within the hierarchy. Occasionally a lane will connect with two or three small places or lanes. Usually, there is no through-traffic between two streets of a higher classification.
- **Sub-collector**—provides access to places and lanes and conducts traffic to an activity center or a higher classification street. It may be a loop street connecting one collector or arterial street at two points, or a more or less straight street conducting traffic between collector and/or arterial streets.
- **Collector**—functions to conduct traffic between major arterial streets and/or activity centers. It is a principal traffic artery within residential areas and carries relatively high volume. A collector has potential for sustaining minor retail or other commercial establishments along its route that will influence the traffic flow.
- **Arterial**—the major street in the hierarchy. It has a high ADT and is not intended to be a residential street. An arterial provides connections with major state and interstate roadways and has a high potential for the location of significant community facilities as well as retail, commercial, and industrial facilities.

In properly designed residential neighborhoods without through-traffic, travel distances from residences to collector streets are short, actual traffic speeds are low, lane capacity is not a controlling design factor, and inconvenience or minor delay is an inconsequential consideration. The brief delay or need to decrease speed, so resented by the highway or arterial street user and the cause of many high-speed accidents, is expected by and is acceptable to residential traffic. In this case, it is customary for the individual to drive protectively to avoid children and pets.

Momentary yielding to resolve minor residential traffic conflicts is practical at residential area speeds. Residential traffic yields to the driver backing from his driveway, or the backing driver yields, and no one is unduly delayed. When residential traffic is impeded by parked vehicles, approaching vehicles often both yield, and then proceed with caution.

ADT, therefore, is not considered the most meaningful index for minor residential street design because the traffic density and consequences of highway and arterial street speeds are not present and residential driving attitudes and habits are different.

Another basic problem with trying to use ADT for residential traffic load estimation is the uncertainty of residential trip generation characteristics. Actual counts in the Boston metropolitan area ranged from 11-22 trips per dwelling unit. Similar counts in 61 subdivisions in Connecticut ranged from 5.4-14.6 trips per dwelling unit with an average of 8.6. Studies reported by the Institute of Traffic Engineers found 8.7 trips per dwelling unit on weekdays and Sundays and an average of ten on Saturday. A spot check on a nine dwelling cul-de-sac in Arlington, Virginia, during the pre-Christmas, 1973 rush period, indicated 6.4 trips per dwelling unit, including 11 truck deliveries; the notable observation was only one moving vehicle was on the street at any one time.

The characteristics of local residential traffic and resident expectations probably are the most definitive considerations when selecting appropriate residential street standards. ADT provides only generalized guidance for decision-making.

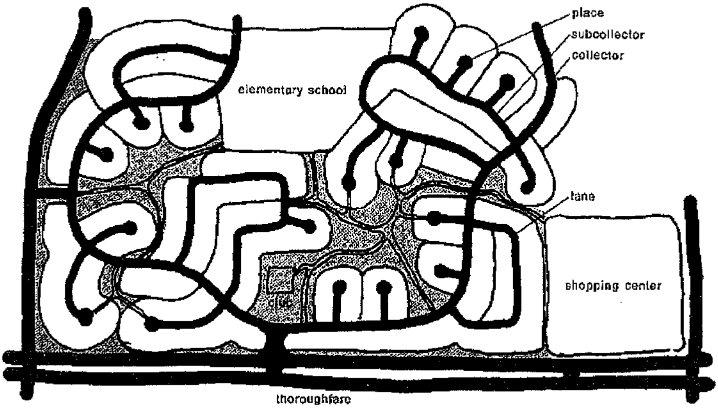
4.3 Factors That Influence Street Use

Factors other than ADT that should be considered in decision-making may include:

- Employment dispersal. For example, in one-employer areas, everyone may travel to or from work at approximately the same time and even along the same routes.
- Carpooling characteristics within the area.
- The degree to which public transportation is expected to be used.
- Vehicle ownership and use characteristics of present or anticipated neighborhood occupants.

- Family size and age characteristics expected within the neighborhood. The location and kind, size, and price or rent of the dwelling units will reflect these characteristics.
- Relative cost and availability of alternative travel modes.
- Proximity to shopping and other support or service facilities.
- The extent to which residents may tend to shop or engage in other activities as groups.
- Anticipated trends of vehicle-use frequency, size, ownership, occupancy changes, and cost and availability of maintenance materials, labor and equipment.
- Limitations imposed by the community's existing maintenance equipment—such as operating width.

Recognizing that some of the above factors are not easily quantifiable, observing patterns on existing residential streets that have a basic hierarchical system would be useful to determine local traffic characteristics.



4.4 Access

Access describes vehicular ingress and egress to traveled portions of the street. Access on minor residential streets within a system is primarily a consideration of driveway access to a dwelling unit. Increasing the number of driveway intersections with a street tends to decrease street capacity and reduce traffic speed. It increases the number of potential conflict points even though there seems to be no correlation with increased accident incidence.

Paved access must be available to fire, ambulance, and police vehicles to within 100-feet of the principal entrances to dwellings. Closer access, for infrequent use, may be across unpaved areas. Access to apartments must be paved to the principal entrance for the convenience of occupants.

The number of entrances to residential areas from arterial streets and highways should be minimized. Entrances should be designed for safety and convenient turning. If a sub-collector or collector street provides access to community facilities, retail areas, and arterial streets, it may require more than two moving lanes with turning lanes at major access or discharge points.

Using a generalized residential street classification system, with characteristics set according to the projected traffic character of planned streets and with consideration of influences from existing streets, will help plan an acceptable functional street system.

The insensitivity of a rigid classification system must be recognized and that system properly interpreted in relationship to a specific design solution. The following section sets forth more specific guidelines for the development of the elements of a functional system.

4.5 Street Alignment

The selection of linear or curvilinear street layout patterns should not be based upon personal preference. Decisions regarding street layout should result from evaluations of a variety of factors including topography, soil characteristics, geologic conditions, drainage patterns, potential runoff quantities, length and character of streets, types and locations of abutting land uses, and purpose of individual streets. Residential development should be beset neither by an endless vista of traffic ways encouraging through-traffic, nor by a spaghetti-like labyrinth that is irrational, incomprehensible, and confusing.

Adequate topographic mapping is a fundamental requirement for optimum residential neighborhood planning including street layout. The planner should be sensitive to development, construction, operations, and maintenance costs that often can be minimized by properly interrelating street layout to natural topography. Basic topographic maps, overlaid with supplemental information regarding soils, depths to rock, groundwater conditions, specimen tree locations, desirable natural features, peripheral occupancy characteristics, etc., provide the layout planner with integrated information upon which to base designs that are functionally, economically, aesthetically, and environmentally optimal.

Horizontal alignment of residential streets should be based on terrain, sight distance, and probable roadway speeds using the computation methods endorsed by the Institute of Traffic Engineers and using actual locally observed traffic speeds.

Vertical alignment of residential streets should assure that inclines generally can be negotiated during adverse weather conditions and sight distances are adequate for safety. No statement of maximum permissible grade seems sound, as flexibility must be provided in individual situations to assure reasonable interrelation of grading and drainage needs with important aesthetic values and local attitudes and customs.

Optimum roadway or pathway gradient for conservation of pedestrian, cyclist, or automotive energy is slightly downgrade against the prevailing wind. The ideal gradient and the prevailing wind should be deliberately considered as a planning factor when selecting locations for shopping centers, service centers, schools, and connecting walks or pathways. Judicious selection of pedestrian and cycle routes to follow contours, when feasible, can enhance the quality of life in a neighborhood in many different ways.

Neighborhood street layouts should be designed to prevent, or discourage, the use of local streets as shortcuts for through-traffic.

4.6 Intersection Design

Sight Distances at Intersections: Adequate sight distances at intersections should be maintained. Vehicles always should be visible as an absolute minimum, when they are located 75-feet from the centerlines of uncontrolled intersecting streets. Greater visibility is desirable. If it cannot be provided practically, intersection control must be considered essential.

Highway designers have avoided hilltop intersections because of their particular collision hazard, but an intersection slightly below a hilltop can be similarly hazardous. A residential intersection blinded by a hillcrest is also hazardous. However, if alternatives are limited, a residential intersection at a hillcrest is preferable to one located below a hillcrest because of the two-directional visibility and the fact that residential traffic is slower than that on highways.

Horizontal Alignment at Intersections: The preferred angle of intersection of intersecting streets is 90 degrees. The minimum angle of *pavement* intersection should be 80 degrees for non-arterial streets. No sound reason exists why pavements cannot be offset somewhat within the right-of-way to improve the intersection angle in difficult alignment situations. Two streets intersecting the same street (T-intersections) should be offset at least 125-feet (centerline offset) when practicable.

Vertical Alignment at Intersections: The gradient within 100-feet of intersections should not exceed 10% (or 2% in heavy snow areas) and every reasonable effort should be made to keep the gradient below 3%. The gradients of sub-collectors and collectors should be held below 2% and preferably below 1% at intersections.

4.7 Number of Lanes

Few local residential streets in properly planned neighborhoods without through-traffic require two moving lanes to serve the usual light residential traffic loads. An important exception to this is any situation in which bumper-to-bumper parking, un-relieved by frequent driveway or other breaks, might develop for any appreciable distance—e.g., over 100-feet. Collector streets will require one-to-two lanes in each direction, based upon expected traffic loads; if parking is to be permitted on collector streets, a parking lane or lanes may be necessary.

Sub-collector streets in properly planned residential neighborhoods rarely should require more than a total of two moving lanes. Parking should be discouraged on sub-collector streets because of the nature of their function and character of their traffic. If parking will not be discouraged effectively on a sub-collector street, a parking lane or lanes should be provided.

Residential streets can be divided into split, one-way streets to preserve a desirable natural feature or to minimize otherwise necessary grading of steep terrain. In either case, an emergency parking lane will be necessary if the divided length exceeds 100-feet. Nighttime visibility and rapid perception of the traffic division point must be assured. Paired one-way residential streets, even single frontage, may be necessary in steep terrain.

4.8 Right-of-Way Widths

Right-of-way widths for residential streets should be adequate to provide required street pavements. As needed, additional width may be provided to accommodate sidewalks, utilities, drainage facilities, landscaping, street furniture, and grading. Right-of-way width may be conserved where utilities are placed within easements that become part of rear-lot or front or side yard building setback space. Right-of-way width allowance for future street widening should be unnecessary in most well-planned residential neighborhoods. Where right-of-way width will include an allowance for future widening, its provision should be based on the community master plan and reliable projections of future use. If a street is subject to future widening, by virtue of its location and traffic routing, it should not be considered or treated as a residential street. It should be designed to have only limited direct residential access, and the ultimate right-of-way width should be dedicated initially.

The basis for right-of-way width policies should be examined by everyone involved in the street development process. Utilities may often be located advantageously outside of the street right-of-way, so that imperfect trench backfills and unforeseen pavement cuts will not hasten pavement deterioration. Pedestrian walkways may be safer, more serviceable, and of greater aesthetic value if they can be divorced from roadways both in alignment and grade. Grade transitions often can be effected on individual building sites rather than within public rights-of-way. A community realizes no tax revenue from a street right-of-way. Every needless square foot of street right-of-way also represents an avoidable maintenance liability.

Layouts of residential areas should be planned so no future need to widen residential or sub-collector streets will arise.

Collector streets present a different problem since they usually provide ingress to and egress from a neighborhood in two directions, and, therefore, cannot be dead-ended.

A need for a future widening allowance on collector streets could be argued because of their through characteristic, but widening of a collector street will change the character of a neighborhood and increase the attractiveness of the street to through-traffic. Such widening is viewed as undesirable—so every reasonable planning strategy should be utilized to preclude future widening needs for most residential area collector streets. The best strategy usually is to assure sufficient arterial street capacity outside of the neighborhood to accommodate foreseeable traffic needs.

Rights-of-way for pedestrian walks and bicycle paths are necessary except when the walk or path is located within a dedicated street or common area within a subdivision. Where walks or paths cross private property, an easement for its use should be recorded. When a walk or path right-of-way is necessary, it probably should extend at least 2 feet to either side of any walk or path surfacing. Where walk or path construction will require changing the natural grade more than a few inches, its right-of-way should be sufficiently wide to accommodate all necessary grading, planting, and construction. Where land values are significant, it may be worthwhile to consider acquisition only of land that will be covered by a walk or path plus temporary use of working space necessary during construction.

Street right-of-way width requirements are all too frequently fixed uniformly by local ordinances regardless of topographic or other variables. Obviously, a right-of-way must accommodate the various construction elements to be located within it, and usually must provide width for appropriate transition from pavement edge grade to adjacent private property grade. When grade differentials are minimal, and pedestrian walks and utilities are not located in the street right-of-way, *and* the street is curbed—usually no justifiable reason exists for a right-of-way width appreciably wider than the roadway pavement.

Preservation of or capitalization on some unusual natural feature of a landscape may dictate an irregular right-of-way width, as may special-effects landscaping. Particular advantages may accrue from preserving existing specimen trees provided they are not so old as to be a liability. Street trees generally should be planted on private property near—rather than within—the street right-of-way. Such a location is recommended because: 1) few communities have sufficient funds to support optimum tree feeding and other maintenance; 2) the nearer a tree is to roadway pavement, the more apt it is to be a hazard; 3) property owners are more likely to feed and maintain a tree that belongs to them; 4) root growth may displace and damage walks, curbs, and pavements; and 5) trees, through transpiration, will aggravate differential movements of unstable soils. In wide rights-of-way, these arguments tend to have reduced significance—the benefit of landscaping with trees, especially to create unusual effects, may outweigh the disadvantage of their location within the right-of-way. In any case, the location of shrubs and trees, either within or adjacent to street rights-of-way, should be controlled and maintained to assure ample sight distances for pedestrians and drivers.

4.9 Residential Street Pavement Widths

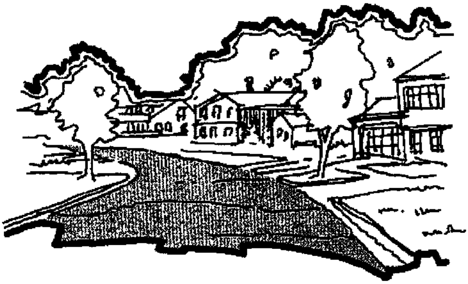
Residential street pavement width practices largely have evolved from moving lane, parking lane, and design speed concepts. Width needs often were set to provide for the largest vehicle that might foreseeably use the street. Such design approaches are effective for the arterial street situation but are difficult to justify for residential streets serving small numbers of homes. Decisions regarding street widths for residential streets, including those serving small numbers of dwelling units on cul-de-sacs, clusters, or loops, are worthy of careful consideration.

Selection of appropriate pavement widths must consider probable peak traffic volume, parking needs and controls, probable vehicle speeds, and limitations imposed by sight distances, climate, terrain, and maintenance needs. The minimum width that will assure reasonable satisfaction of all foreseeable needs normally should be selected for construction to minimize average annual street costs.

Arterial streets generally have an ADT greater than 3,000 and are used as major thoroughfares. They are designed and constructed to fulfill the needs of a particular public construction authority for a particular road or highway. Direct access from individual driveways to arterial streets should be prohibited, or at the very least strongly discouraged. Parking permitted on arterial streets should be of an emergency nature only. Design speed is an essential arterial street capacity consideration and usually will be at least 30 mph. Arterial street design is not considered herein.

Residential street pavement widths recommended for consideration—and their limitations—are as follows:

- A 36-foot-wide pavement is the most commonly observed width for collector and sub-collector streets—providing two moving lanes and on-street emergency parking lanes.



A 26-foot wide pavement

- A 26-foot-wide pavement is the typical residential street pavement width in many cities—assuring one freely moving lane even where parking occurs on both sides. Its performance in such use seems excellent. The level of resident inconvenience occasioned by the lack of two moving lanes is remarkably low. In fact, no appreciable difference in driver convenience generally is noted between a 26-foot wide and a 36-foot wide pavement unless the neighborhood layout permits travel distances in excess of three blocks between a dwelling and a collector street.
- A 24-foot-wide pavement technically is suitable for automobile parking on both sides of the street without impeding one-lane vehicle movement. A 24-foot width is typical for residential streets in some cities, but generally is considered too tight. Comparative accident incidence studies for 24-foot-wide minor residential street pavement are scarce and somewhat inconclusive; however, its use is not normally recommended although the proliferation of small automobiles may well make the use of 24-foot-wide pavement feasible.
- A 22-foot-wide pavement offers no significant advantage over a 20-foot-wide pavement. Although parking is feasible only on one side, a 22-foot width is sufficiently wide to tempt drivers to park on both sides—obstructing access that is a major disadvantage. The excess width is unnecessary for appropriate vehicular movements.
- A 20-foot-wide pavement is the minimum width that generally offers year-round utility and convenience where snow and ice control needs are foreseeable. Suitable for cul-de-sacs up to about 300-feet long—a 20-foot wide pavement will provide parking on one side with alternating vehicular traffic flow, is sufficiently narrow that drivers are not tempted to park on both sides, and has minimum construction, amortization, space, and maintenance requirements. Its biggest disadvantage is that it will not accommodate parking on both sides.
- An 18-foot-wide pavement is suitable only for use on short one-way loop streets serving no more than 15 dwellings, or on short cul-de-sacs, usually serving no more than five to seven dwellings. It is subject to the same limitations and inconveniences as narrower pavements, but to a somewhat lesser degree. A properly parked vehicle on an 18-foot-wide one-way pavement will not impede the largest truck that might seek to use the roadway.
- The 16-foot pavement is not suitable for cul-de-sacs having more than three dwellings, but offers acceptable utility on one-way loop streets in the following situations: 1) adequate off-street parking is assured; 2) the climate is mild and snow and ice control problems cannot be foreseen; 3) total loop length will not exceed about 500-600 feet, serving no more than about ten dwelling units; 4) adequate longitudinal sight distances can be provided; and 5) vehicle speeds may be reasonably expected to be in the 10-15 mph range. The 16-foot-wide pavement may be a practical loop

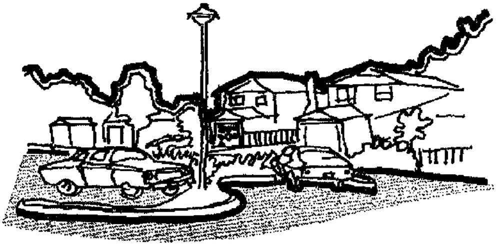
street alternative in difficult terrain where cross-pavement ground slopes are severe, where vehicle speeds should not exceed 10 mph and where other above-outlined considerations can be met. Under the various conditions outlined, the 16-foot-wide pavement can be functionally effective, but will result in a higher level of resident inconvenience than a wider pavement. Sixteen feet cannot be considered a desirable pavement width, but must be conceded to be acceptable under certain conditions, such as 16-foot-wide one-way streets created to avoid destruction of natural features.

- Even narrower pavements may be appropriate under specialized conditions.

4.10 Parking

4.10.1 On-Street Parking

Three options exist for providing on-street parking: 1) parking on one side of the street; 2) parking on both sides of the street; and 3) parking bays. Parking lanes require an 8-foot paved width. An equally wide retained gravel shoulder can be used in lieu of paved parking lanes and may have the advantage of reducing the rate of stormwater runoff. Such shoulders may help create a natural or rural appearance, but displaced gravel can be a nuisance on the paved area. Roadside shoulders are dependent upon sensitive landscaping to assure fulfillment of function and appearance objectives, and are dependent upon careful design and construction to assure permanence and avoid excessive maintenance.



A parking bay in a cul-de-sac

Normally, only parallel parking should be permitted on minor residential streets. Ninety degree, or angle parking rarely is suitable for single-family residential areas, as both require more paved space per vehicle and necessary maneuvers require more moving lane space than parallel parking. Angle parking generally is considered undesirable, more hazardous than parallel parking, and requires greater street maneuvering width than parallel parking.

Parking space, usually for angle parking, can be developed in the centers of cul-de-sac turnarounds. When landscaping assuredly will be maintained by a property owners' association, such an installation can be effective and attractive. When basic site limitations necessitate a long cul-de-sac—perhaps 1,000 to 1,200 feet—an intermediate turning circle may improve a difficult design situation. Providing the circle with landscaped central parking to improve the off-street/on-street parking rations may be desirable. Guidance regarding the appropriate dimensions and geometry for design of on-street parking can be found in the Institute of Traffic Engineers' publications.

4.10.2 Off-Street Parking

Off-street parking minimizes the need for parking lanes on the street. Off-street parking space needs vary with vehicle ownership and driving habits of the residents.

If on-street parking is precluded by narrow pavements, provision of off-street parking must assure no traffic-way encroachment by parked vehicles. Decisions regarding the balance between on-street and off-street parking provisions should be established after evaluating all needs, alternatives, objectives, and costs with the site designer, the maintaining authority, other affected public officials, and the developer.

All residential occupant parking should be off-street. Only visitor parking should overflow onto the street. Automobile ownership trends in recent years have been toward two-car families to the extent that about one-third of all American families own two or more cars. Recent fuel availability limitations may affect this trend. Building industry and local authorities should be alert to the implications of continuing changes in vehicle ownership and use trends.

Appropriate off-street parking facilities and requirements will vary among and within communities. Family size and ages, family income level, population density per acre, proximity to and adequacy of public transportation, schools and shopping, and many other factors affect determinations of off-street parking needs. These factors are not constant and should be evaluated to fit individual situations.

A modern phenomenon, especially noticeable since the advent of the energy crisis, is persons from outlying areas parking all day on closer-in residential streets near public transportation routes. The unforeseen parking load often is resented by adjacent property owners who traditionally consider the curb in front of their homes as their domain. The significance of this special parking problem has yet to be evaluated.

Parking on arterial streets is unwise from the vehicle owner's viewpoint, undesirable from the traffic capacity standpoint, and hazardous from the safety standpoint. To somewhat lesser degrees, the same statements are applicable to collector and sub-collector streets. The best resolution of the question seems to be to prohibit parking on arterial streets, primary collector streets, and public transportation routes, and physically to discourage parking by making it inconvenient on other collector and sub-collector streets.

Many parking studies purport to prove either on-street or off-street is safer, more economical, or represents better planning. Any vehicle entry into a lane of moving traffic is a potential accident hazard, especially when a backing movement is involved. The driver's condition, driving habits, and alertness are probably the primary safety elements on quiet residential streets, regardless of parking location.

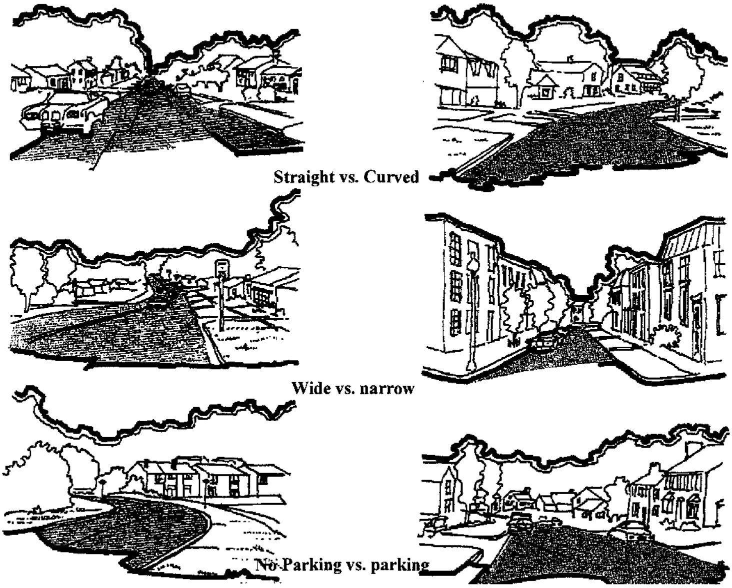
The relative safety of on-street and off-street parking on minor residential streets is debatable, but it is obvious that a vehicle parked off-street is less likely to be hit by a moving vehicle.

Increasingly important in residential area planning is the accommodation of recreational vehicles (RVs) to avoid creating a cluttered neighborhood appearance. Parking of trailers and other special purpose limited use vehicles should not be permitted on residential streets, in front yards, or between residences, because of the visual impact and fire hazard. Communities are discovering also that rear-yard RV parking is disadvantageous. Some developments provide secure area for RV storage and maintenance.

4.10.2 Speed

Traffic speeds on residential streets generally are affected by:

- Open width or clearance of the street. A street with wide lanes invites faster movements.



- Horizontal and vertical street alignment. Straight streets with long sight distances tend to invite increased speed.
- The number of access points to the street. A variety of obvious potential conflict points inhibit speeding.
- Number of parked cars or other obstructions on the street. Visual barriers effectively decrease traffic speeds, as each barrier may pose a potential conflict.
- Signs and signals at intersections, which are obvious speed controls within the immediate vicinity of the control device.
- Speed of other traffic on the street. Lanes and places, because of their usually short lengths, alignments, and negligible traffic load, tend to be self-regulatory.

Some traditional concepts of *design speed* for residential streets are questionable. Residential streets tend toward low vehicle speeds because of their short lengths, parked vehicles, driver concern for children and pets, and other reasons. Places and lanes particularly will have low traffic speeds. Traffic speeds on sub-collector and collector streets will be higher. Horizontal and vertical alignment control, selection of street widths, and other controllable design elements can influence actual traffic speeds.

5.0 SPECIAL DESIGN CONSIDERATIONS

The planning process in street design allows subdivision designers and local governments to use the primary considerations of ADT, traffic density, and access to determine the major design features of street systems.

Natural terrain, climate, soil types, and other factors must also be considered when setting standards and designing street systems. These conditions are often specific but may be local or regional in nature.

5.1 Gradients

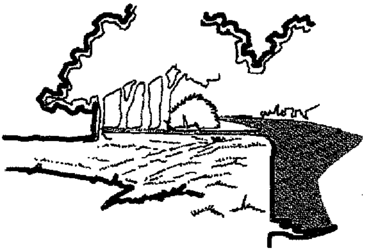
A minimum gradient on all curbed streets is necessary to prevent ponding of water. Also included in the design should be a cross-slope to move water off the traveled portion of the roadway. Minimum gradient usually is specified as 0.5%; however, in very flat terrain it has been reduced to 0.3%, and in some very difficult situations to as little as 0.1%. If successful performance is to be achieved, sub-grade compaction and construction gradient controls become very critical when gradients are flatter than 1%. Pavement cross-slope usually is specified at about one-quarter inch per foot.

In hilly terrain, no practical way may exist to avoid street gradients steeper than those considered maximum in less severe topography. Under such conditions, there may be no alternative to providing the least steep gradients possible. Particular effort should be directed toward securing the flattest possible gradients near intersections.

If unusually steep gradients cannot be avoided: 1) both uphill and downhill access to every property should be provided if at all possible; 2) parking in excess of usual needs should be provided at the foot of the hill in snow country; 3) pedestrian walkways or walkways with handrails may be essential; and 4) walkways in snow country should be limited to about a 5% longitudinal gradient by using flights of several steps intermittently along the length.

5.2 Drainage

Well-designed, maintainable roadside swales may be useful for runoff detention storage when: 1) curbs may be safely and soundly omitted; 2) groundwater conditions are not and will not become adverse; and 3) downstream advantages from attenuating peak runoff would exist. Detention storage installations may use small driveway culverts to meter outflow, provided they are built of well-aligned, durable pipe, and further, provided the community or maintenance association operates equipment designed for rapid clearance of culvert blockage. In northern climes, small driveway culverts are prone to wintertime freezing, are very difficult to clear of ice, and runoff storage in roadside swales may have limited practicality unless the soils are quite permeable. As small culverts are difficult to clear of ice, larger culverts with a weir placed above them often will solve the problem and still permit a metered flow. Curbed roadways provide drainage outfall for adjacent properties and compose about 95% of most urban storm drainage systems. Thoughtful curbed street layout and gradient planning will help route storm runoff in order to avoid undue concentration on street surfaces, often obviating the need for small storm sewers.



Drainage Swale

Sufficient topographic relief and integrated design of runoff routing and street layout may prevent excessive surface runoff concentrations and permit runoff discharge directly into major drainageways without need for intermediate runoff collection in storm sewers.

Downstream edges of pavements and downstream faces of embankments should be protected against erosion and scour. Upstream properties normally should have floors and openings to basements at least 18-24 inches above the street crossing the drainageway to prevent flooding.

5.3 Cut and Embankment Slopes

The economic significance of slope instability is much greater along residential streets than along rural highways. Highway slope failures generally involve a few hours of public inconvenience and a moderate maintenance outlay, whereas slope failures in developed residential areas generally involve utilities and street improvements, and often buildings—all are more costly to restore and generally involve considerable public inconvenience during restoration.

The kinds of situations in which slope or embankment stability may become a problem generally are identifiable within any geographic area. Competent geological engineering counsel can provide appropriate local guidance regarding the identification of potential stability hazards and practical methods for their mitigation.

5.4 Pavement Design

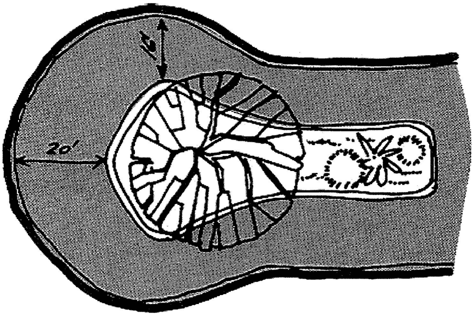
Portland cement concrete that will be exposed to the weather normally should have a minimum cement content of six bags per cubic yard of concrete. When weathering conditions will be severe, an even richer mixture is desirable. When a significant amount of alkali—calcium or sodium sulphates—is present and when deicing salts are customarily used, sulphate-resistant aggregates and specially formulated Portland cements should be used in concrete. The Portland Cement Association will gladly provide detailed guidance regarding specialized concrete weathering problems.

The Asphalt Institute recommends asphaltic concrete street wearing surfaces always be placed in at least a 2-inch thickness. In practice, many communities place a fine graded asphaltic concrete surface on one and one-half inch thickness with no apparent significant problems. Local community officials, state and county highway department officials, and Asphalt Institute representatives usually will make every effort to provide helpful information regarding local practices, available equipment, pavement performance, etc.

Strength of soil sub-grades for pavements usually should be evaluated for a nearly saturated soil condition, because: 1) capillary rise and upward vapor transmission from a groundwater table, or hydro-genesis phenomena where there is no groundwater table, usually will result in moisture gain beneath a paved area; 2) residential street design generally necessitates establishment of curb and pavement grades below adjacent properties, rather than above them as in highway practice, so that runoff flows to residential pavements rather than away from them; and 3) the strength of many soils is lower in a near saturated condition than at the moisture content specified for standardized bearing strength tests.

5.5 Turning Requirements

The residential dead-end direction reversal problem is basically an automobile problem and only secondarily a large vehicle problem. Turning requirements of special vehicles such as fire trucks and moving vans should not be the determining factor in the selection of the street turning radius because short-turning emergency vehicles can be acquired and less critical vehicles can maneuver during turns. Moving vans are not emergency vehicles and have time to negotiate turns—they are likely to use driveways to facilitate loading and unloading. Regardless of turning space dimensions, parked vehicles will reduce available turning space. Parking restrictions, signing, and enforcement may at times be desirable.



This variation on the standard cul-de-sac allows good vehicle turning movements within a minimum paved area.

5.6 Turn-Arounds

The advantage of clustering homes has led to use of a variety of configurations that include cul-de-sacs, courts, "Ts," "Ys," etc. Elimination of turn-around facilities is rarely justifiable. The usual length of streets leading to turn-arounds ranges from 400-600 feet. Design flexibility provided by a longer street often is desirable. When topography dictates longer dead-end streets, intermediate turn-arounds may be advantageous.

Turn-arounds for most places and lanes may be a turning circle. For streets with few dwelling units, the use of a "T" or "Y" turn-around is another solution.

When it is necessary to temporarily terminate a sub-collector or collector street, the use of a temporary turn-around is necessary. Curbs, if used, should be terminated before entrance to the turn-around. Such dead-ends generally should be barricaded to protect the public.

Turning facilities and their dimensions, suitable for residential cul-de-sacs, clusters, or other dead-end streets, have been subjects of widely divergent past practices. Except on short driveways serving individual dwelling units, every dead-ended roadway that might be used by large vehicles should be provided with a turning facility.

The design of turning facilities for residential streets should consider:

- Large delivery trucks, common 20 years ago, are uncommon today as many buyers carry their purchases and remaining deliveries are made with smaller vehicles.
- The residential dead-end direction reversal problem is basically an automobile problem and only secondarily a large vehicle problem.

- There is an accelerating trend toward use of small vehicles (sub-compacts) throughout our nation. Some of these sub-compacts have an outside turning radius of 17 feet.
- A smaller turning radius can now be achieved for emergency vehicles. These factors suggest that turning radii based on currently available vehicles may be reduced in the future.

5.7 Special Vehicle Turning Requirements

Parking in cul-de-sacs and driveway spacing presents unknowns that can negate the most carefully selected pavement edge radius objectives—necessitating use of driveway openings and maneuvering to reverse the travel direction of a large vehicle. An excessively large paved radius would be required to permit curbside parking and a perfectly executed direction reversal of a refuse packer without backing. Such a turn-around dimension is beyond the bounds of reason and exceeds all known urban requirements. Cities that require a circular paved turning area from 75-80 feet in diameter have had few resident complaints or operating problems relating to the adequacy of turning spaces. Backing movements always present problems because vision is limited. On a cul-de-sac, possibly visited by two trucks per week, normally there will be at least 30 automobile backing movements from each driveway. The comparative added risk from truck turning seems negligible when analyzed either theoretically or practically. A cul-de-sac with central parking will enlarge the pavement diameter and facilitate larger vehicle turning maneuvers.

In decreasing order of use frequency, residential streets are used by refuse packers, delivery trucks, snowplows, moving vans, and fire trucks. Consideration of individual characteristics and use-alternatives should help identify appropriate facilities and dimensions for different situations.

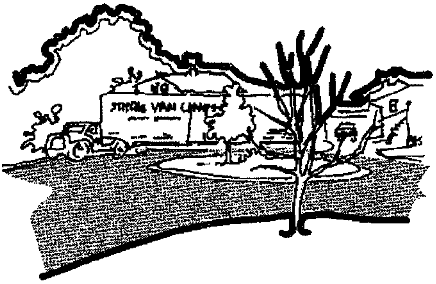
5.8 Fire Equipment

Fire equipment usage, although the least frequent of any type of large vehicle usage, is time-critical. Fire department drivers and crews are trained to relate addresses to access routes within their jurisdiction. It is unusual for fire equipment to inadvertently enter a dead-ended roadway. The likelihood of erroneous entry can be further diminished by appropriate street name signs, and signs indicating the dead-end. Very large trucks, such as a hook and ladder, do not usually travel to fire calls in single-family residential areas. Pumpers and ladder trucks that travel to such areas often have been quite large—especially when purchased for use in commercial or multi-family areas—and typically have had large turning radii. Little need for such large equipment in newly developing single-family residential areas exists. Many communities are beginning to use smaller sizes of ordinary trucks on which special equipment can be mounted. The high cost of specially constructed fire trucks, their frequently limited maneuverability, and their high maintenance costs have been accelerating the trend toward smaller vehicles with smaller turning radii for suburban residential use.

Some of the older fire equipment may have a turning radius as large as 50-55 feet. In any reasonably dimensioned dead-end, it is impractical for older fire equipment to reverse direction without backing movements. Newer trucks, especially conversions from major manufacturers' commercial line trucks, have smaller turning radii—generally in the 30-40 foot range. When fire trucks must reverse direction quickly, they have the advantage of utilizing the accompanying firemen for guidance, so backing movements pose few problems and little loss of time. When developing areas produce large numbers of cul-de-sacs or other dead-end, turn-around facilities, consideration should be given to local acquisition of fire equipment sized to fit the new residential areas. It is more economical to purchase trucks that fit a community's streets than to build all streets to fit infrequently used trucks.

5.9 Moving Vans

Moving vans represent very infrequent dead-end street use. Usually, they are articulated and are maneuvered extensively to achieve an optimum position for loading or unloading. Moving vans usually carry helpers who assist the driver's maneuverings, enabling him to turn the vehicle without significant problems in relatively tight spaces.



Large vehicles do find a way to fit

5.10 Snowplowing

Snowplowing in turn-arounds involves maneuvering almost regardless of the size of the plowing equipment used, so it should not become a ruling design consideration. Minor residential streets requiring snowplowing often might be cleared advantageously by smaller equipment than is generally appropriate for arterial streets and highways. As noted elsewhere, snow removal on minor residential streets often is unnecessary except in heavy snow areas, and is one of the lower priority snow clearance needs in a community.

5.11 Refuse Packers

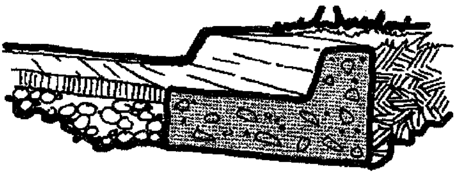
Refuse packers still operate on a residence-to-residence basis at walking speeds in some communities. Turning radii of refuse packers may be as wide as 55-57 feet, depending upon make and model. Some maneuvering, with the guidance of the loaders, generally will be needed to reverse a refuse packer's direction in almost any reasonably designed turn-around, especially if cars are parked on it. At walking speeds, and with the guidance of the driver's aides, backing movements should pose no problem on a cul-de-sac. Some communities are using small loading vehicles for house-to-house refuse collection to permit more efficient use of expensive refuse packing vehicle time—possibly creating a trend toward such operations. Turning requirements for refuse collection equipment are not seen as controlling design factors.

5.12 Curbing

Curb, or curb and gutter, serves to contain granular pavement base materials, prevent penetration of water beneath pavements, and prevent a raveling of the pavement edge by providing it with lateral support. These needs also can be fulfilled by installing thickened-edge pavements by use of anchored steel edgings flush with the pavement surface, and by use of flush Portland cement concrete headers along pavement edges. Alternatives to curbs are worthy of consideration when the sole purpose of the curb is to furnish pavement-edge protection.

A primary function of roadway curb is to discourage drivers, especially when parking, from encroaching beyond paved surfaces. Curbs provide a psychological barrier that is significantly greater than the physical impedance to the vehicle. Other barriers may be equally effective, such as railings, posts, wheel stops, and appropriate plantings. Edge plantings of such shrubs as multi-flora rose can provide the desired barrier, create an effective vehicle safety barrier as the plants mature, provide a delightful neighborhood character during the blooming season, increase privacy of adjacent residential properties, and effectively discourage on-street parking.

A residential roadway curb is unjustifiable as a safety barrier to protect sidewalk traffic because in a significant proportion of instances, vehicles striking most curbs will mount them out of control. Statistically, it is improbable that a vehicle will impact a curb at the precise place and moment that a pedestrian is adversely positioned, particularly on residential streets.



Straight battered curb

The rolled curb—an easily mountable curb—came into popular use before and shortly after World War II. Subsequent experience suggests such a curb's drainage handling—hydraulic—and some other characteristics are poorer than those of a straight battered curb. For example, if a moving vehicle hits a rolled curb, the driver is more likely to lose control than if he hits a straight battered curb. With present construction technology, neither curb requires the use of a face form, so construction costs are comparable. The straight battered curb is decidedly superior to the rolled curb for which reason the FHA and the ITE do not encourage use of the rolled curb.

5.13 Climate

When snowfall is deep and frequent, pavement widths may be slightly wider than when a 6-inch snowfall is a rarity. The added width is advantageous: 1) to permit some snow storage on the roadway; and 2) to permit plowing equipment to pass parked vehicles without conflict. In areas only rarely subject to a 6-inch snowfall, these advantages disappear.

While the width of a snowplow blade may be as great as 12 feet, narrower blades are routinely obtainable. The blade width should not be a controlling factor in minor residential pavement width determinations.

Snow discharge characteristics of snowplows are such that some snow falls back upon the edge of the plowed area after the plow passes that can reduce the effective cleared width by 1 or 2 feet. The importance of this narrowing is somewhat dependent upon street width and traffic characteristics—it is not a significant factor on minor residential streets.

Modern winter automobile tires have proved effective under snow and ice conditions for negotiating up to 6-inches of snow even on surprisingly steep grades. The individual driving under winter conditions has a moral, if not legal, responsibility to his neighbors to assure that his vehicle is properly shod. Community responsibility is to assure emergency routes, arterial streets, and collector streets, in that order, are opened to travel quickly. Snow removal on residential streets necessarily is given lower priority.

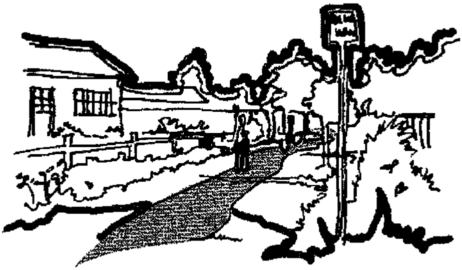
Local maintenance authorities, from the standpoint of cost and effectiveness, may wish to consider the advantages of a policy of not plowing residential streets when: 1) snowfall is less than 5-6 inches; 2) there is not drifting; and 3) additional snow is not immediately forecast. Under these conditions, snow windrowed by plowing may unnecessarily cause extra shoveling to extricate cars parked along the street or on adjacent driveways, and the windrowed snow may still be a local nuisance after almost all other snow has melted. Savings of fuel, equipment, and labor—often incurred as a result of overtime maintenance work—can accrue from non-plowing policies and public inconvenience may even be reduced.

Snow and ice control by application of various salts, even salts mixed with rust inhibitors, should be carefully evaluated. Average saline damage to vehicles may exceed $100 per vehicle, per year. Salts also cause or induce extensive damage to porous materials, especially Portland cement concrete bridges, culverts, walks, and curbs. Environmentalists concerned with downstream water quality will provide additional reasons for avoiding deicing salts. Use of abrasives seems preferable.

5.14 Pedestrian Circulation

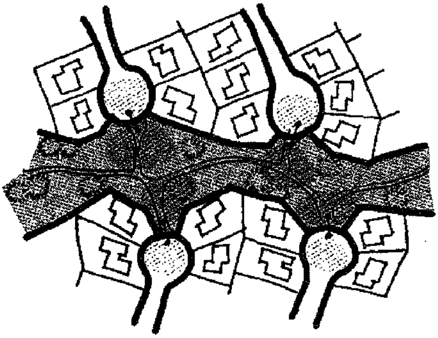
In general, the complete separation of vehicular from pedestrian circulation systems is desirable. To achieve such separation, a development's pedestrian system may contain three different types of sidewalks or paths:

- Paths or sidewalks, usually 2-3 feet wide, connecting individual dwelling units with off-street parking, parking pads, parking lots, parking bays, and refuse disposal areas;
- Local paths or sidewalks, usually 3-4 feet wide, connecting neighboring dwelling units;
- Paths connecting groups of dwellings with commercial centers and other community facilities. Width selection is dependent upon pedestrian and bicycle traffic requirements.



Pedestrian and bicycle paths

Sidewalks and paths should not be used for drainage. Only small amounts of surface drainage should flow across walks or paths. A minimum transverse—cross-slope of one-eighth inch per foot—toward the street when the walk is within a street right-of-way, is essential to provide walk surface drainage. The transverse slope should not exceed three-eighths inch per foot because greater slopes are hazardous under icy conditions and at night.



Good design is safe and visible

Pedestrian paths in well-drained soils may serve satisfactorily without any special surfacing. Where soils are soft or sticky when wet, pathway soils should be stabilized, surfaced or paved, as appropriate for the amount and character of their potential usage. An occasionally used footpath may serve adequately without improvement or with graveling. Heavy pedestrian usage or bicycle traffic usually requires a hard surface. It should be recognized that the paved surface added to an average subdivision for sidewalks and paths can increase the impervious cover by as much as 3% in the subdivision.

5.15 Walks and Paths

Sidewalks along some residential streets are necessary and desirable. The basic test of need should be expected use and sidewalk relationship as an element of a functional pedestrian system. Realistic evaluation often will reveal sidewalks on one or both sides of a minor residential street will be superfluous. When children are anticipated and paved private driveways will not be installed, sidewalks should be installed on at least one side of the street.

Public paths or sidewalks always should be located within a public right–of-way, a public easement, or a common area.

Sidewalks within a street right-of-way normally should be 4 feet wide. Sidewalks within the rights-of-way of short cul-de-sacs or courts may produce insufficient benefit to justify the cost. The added edge maintenance of such sidewalks entail may make them a greater liability than asset.

Common area paths or walks generally should be wide enough to provide either two pedestrian lanes or one pedestrian and one bicycle lane. A 4-foot width is tight for passing bicycles, although generally adequate; a 4.5-foot width is much more comfortable and a 5-foot width borders on luxury. The above widths assume no side obstructions or walls.

A path exclusively for bicycles need not be more than 3.5-feet wide and would be acceptable for all but the major elements of a bicycle path system. The major elements of a two-lane bicycle path should be approximately 5-feet wide.

Path and sidewalk street crossings should be located where there is good sight distance along the road. Curb cuts should be provided for users of wheelchairs, wagons, tricycles, and bicycles.

When heavily traveled paths or sidewalks cross busy streets, safety devices such as appropriate signs, signals, and painted crosswalks should be used. When paths or sidewalks intersect arterial streets, special signal controls, underpasses, or overpasses provide the safest method of pedestrian crossing.

Walks and paths in common areas or other locations away from streets generally should be lighted or integrated into the detailed area plan or layout enabling visual surveillance of the walk or path from the street. The opportunity for surveillance is an important factor relating to user safety and security.

5.16 Street Lighting

Street lighting provides safety, security, and convenience. It is expensive to install and uses significant amounts of energy. Energy savings cannot be justified when the tradeoff involves pedestrian security. Convenience, as the sole justification for lighting, may have to be sacrificed to energy conservation, especially in low-density settings where pedestrian traffic may be a rarity.

Street and path lighting should be selected to have a high illuminating efficiency and to provide no more illumination than is necessary to conserve energy. Unnecessary lighting should be avoided, but lighting essential for security or safety should always be provided. Continuing needs for energy conservation are undefined but appear certain. They probably will vary geographically, so local energy companies should be a basic source of specific guidance.

Street lighting and bicycle or pedestrian path lighting should be located or mounted to prevent light shining upon residential windows, or into the eyes of drivers, pedestrians, or bicyclists.

5.17 Material Conservation Needs

Our nation faces a long-term energy outlook that may affect the availability and cost of construction materials. The United States in 1970 had over 3,700,000 miles of roads and streets, over three-fourths surfaced, and is adding at least 30,000 miles a year of urban streets to this system. Material demands for construction and maintenance are a cumulative future commitment of maintenance materials, equipment, and labor. Continuing expansion of our street and road system is necessary to serve population growth. Unquestionably, conservation of construction and maintenance materials is essential.

6.0 CONCLUSION

As noted in the Introduction, as many miles of residential streets may be built in the next 30 years as have been built during the entire history of the United States. Our streets have evolved from the often narrow, dusty, unpaved residential streets of 30 or 40 years ago to wide paved streets with curbs, gutters, sidewalks, street lighting, and landscaping. The process of converting from gravel or dirt streets to current practice has largely relied on experience and expertise developed in the highway engineering field. This process has largely been the filtering down and adaptation of standards developed for highways having high volume, heavy vehicle use. In an era when the national propensity was to see limitless national resources, the approach to setting standards often was "when in doubt, pick the highest standard."

The purpose of this report is not to suggest that street design in the past has been "done all wrong" and should be completely redone, but rather to suggest objectives for our residential streets and principles to guide us toward optimum design standards. These objectives and principles are fundamentally common sense and should be readily acceptable. What is different about the objectives and principles is that they are specifically oriented toward residential streets, and they acknowledge that residential streets differ from commercial or industrial streets and from major arterials and interstate highways.

It is presumptuous to suggest only one set of design standards could be uniformly applied throughout the country. At the same time, it is legitimate to suggest each community can measure its local practices against a set of objectives and principles that do have uniform applicability. Performance measured against these objectives and principles will identify the adequacy or inadequacy of current design and this will suggest modifications needed in local practice.

Chapter 23

RESIDENTIAL STORMWATER MANAGEMENT

1975
Objectives, Principles & Design Considerations
Published Jointly By: ULI; ASCE; NAHB

1.0 FOREWORD

Publication of this report under the joint auspices of the American Society of Civil Engineers (ASCE), the National Association of Home Builders (NAHB), and the Urban Land Institute (ULI) is a continuation of the unique cooperative effort that resulted in a previous jointly sponsored publication, *Residential Streets: Objectives, Principles, and Design Considerations*. This report is a second jointly sponsored and reviewed report that represents the attitudes and concerns of the three organizations on a subject of common interest and importance—the setting of objectives, principles and design considerations to be applied to the development of stormwater management systems to serve residential communities.

The content of this report evolved, as did its predecessor, from an assessment of current practices to a guide toward a more creative and thoughtful approach to stormwater runoff management. While not rejecting past practice, it clearly identifies and articulates a new underlying philosophy and approach that diverges significantly from the past.

It is hoped that this report will stimulate communities of all sizes, in all locales, to rethink current stormwater management practices and to adopt those that are responsive to local conditions and supportive of the basic objectives and principles presented herein.

2.0 INTRODUCTION

2.1 The Need for Change

The basic philosophy of stormwater management in residential, and for that matter, all kinds of development, is open to challenge and revision. Nationwide experience with the effects of narrow and inadequate philosophies on past practices indicates that stormwater has rarely been well managed, and has in fact often been mismanaged. To some extent, at least in residential developments, past approaches, development patterns and public policies inadvertently encouraged the very approaches this report seeks to modify. Simply stated, past philosophy sought maximum convenience at an individual site by the most rapid possible elimination of excess surface water after a rainfall and the containment and disposal of that water as quickly as possible through a closed system. The cumulative effects of such approaches have been a major cause of increased frequency of downstream flooding, often accompanied by diminishing groundwater supplies, as direct results of urbanization; or have necessitated development of massive downstream engineering works to prevent flood damage.

The downstream urban flooding problem has become acute during the past 30 years as communities have grown and as curbed roadways (paved channels) have been installed in both new suburban areas and throughout older areas that formerly provided runoff-retarding storage in roadside swales or ditches. Amelioration of the unfortunate results of past urbanization requires very large investments to construct additional flood control works. Where flood control is infeasible, the flooding hazard reduces property values and may lead to abandonment, which is unacceptable to community leaders.

The entire process of stormwater runoff management is currently undergoing a significant redirection, if not a revolution. This is evidenced by a new emphasis on the desirability of detaining or storing rainfall where it falls, on-site, which sometimes requires tradeoffs with short-term localized inconvenience. These kinds of solutions applied to individual sites or developments often have beneficial cumulative effects by attenuating both peak runoff and total short-term runoff. If fully applied throughout a drainage basin, they would reduce major facilities investments required to protect against flood hazards in the lower portion of the drainage basin.

2.2 The Problem

In an undeveloped area, the stormwater management system is provided by nature. The cycle begins with rainfall—the storm. Some of the water stands where it falls on leaf or plant, and evaporates; some is absorbed into the ground near the surface and feeds trees and plants, ultimately to be returned to the atmosphere by transpiration; some percolates deeply into the ground and replenishes the groundwater supply. The remainder gradually or quickly collects into rivulets, accumulating both in quantity and speed as it hurries down the watershed through drainageways and streams to its ultimate destination—the river and then the sea, to begin the cycle again (Figure 23-1).

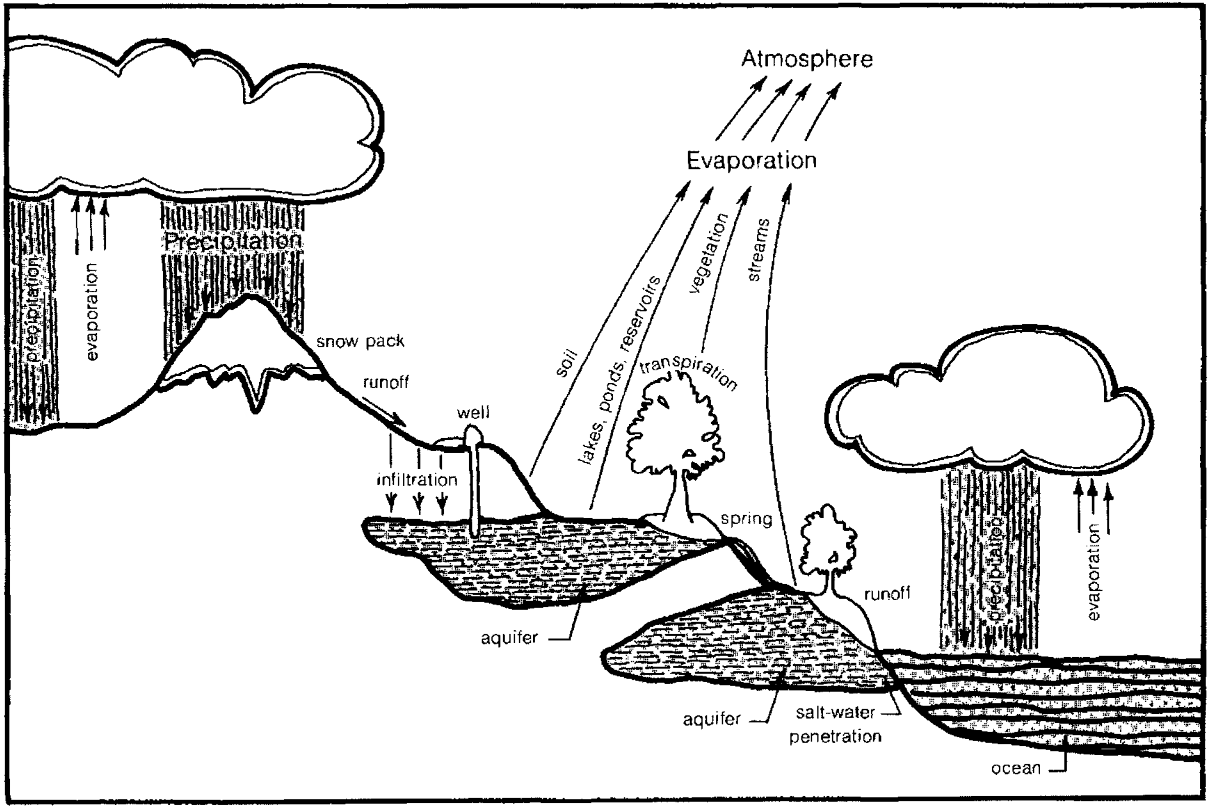


FIGURE 23-1. Hydrologic Cycle

All of us have been exposed to this simple explanation but its seeming simplicity belies its complexity. Nature's inability to accommodate severe storms without significant damage, even where urbanization has not occurred, is quite apparent. The natural drainage systems in an undeveloped area are not static in design, but are constantly changing. Streams change course, banks erode, vegetation and soil permeability change with seasons, lakes fill in with sediment and disappear. The stripping of ground and tree cover by fire may change an entire system, forcing new natural accommodations throughout the system. Urbanization has required new drainage systems because man was both unwilling to suffer inconvenience where it could be avoided and because he would not tolerate the loss of life or property. In an urbanizing area, those concerns often have been translated into stormwater management system requirements for convenience and safety, without recognition of other significant considerations. This has meant that no matter how large the rainfall or its duration, the drainage system was expected to remove runoff as quickly as possible, to restore maximum convenience in the shortest possible period of time. At the same time, fears of loss of life and of damage to possessions have encouraged a search for 100% protection against the worst storm that nature might generate.

It is the premise of this report that these two objectives are not mutually achievable without extremely high "cost." Where we have sought maximum convenience as our first choice in the upper and middle reaches of a watershed, we have created imbalanced systems, and increased hazard and risk of damage along the lower reaches. The need is obvious—to strike a realistic balance between elimination of inconvenience and protection against hazard. Past practice has not always achieved such a balance. In fact, it more often than not has encouraged acceleration of imbalance as areas urbanized.

2.3 New Development Patterns Provide New Opportunities

Every parcel of land is part of a larger watershed. Ideally, a stormwater runoff management solution for each development project should be based on, and supportive of, a plan for its entire drainage basin. This is not a revolutionary idea, but only recently have data collection and data handling technology made this economically possible in a meaningful way. Even in the absence of such basin-wide plans, new approaches to residential land planning, which have been evolving since about 1955, have made it possible to apply more creative approaches to stormwater management within a project.[1,2] With their application, the net effects of incremental urbanization can avoid most negative impacts and may produce benefits, enhancing opportunities for future implementation of an overall basin-wide drainage plan.

Before 1945, most residential development involved small parcels of land and often proceeded on a lot-by-lot basis. Development was easily accommodated using the then-prevalent philosophy of maximum convenience and rapid downstream disposal

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[1] Sevuk, A.S., B.C. Yen, and G.E. Peterson. October 1973. "Illinois Storm Sewer System Simulator Model: User's Manual." *Water Resources Center Research Report No. 73.* University of Illinois at Urbana, Champaign, Illinois.

[2] Tholin, A.L. and C.J. Keifer. 1960. "The Hydrology of Urban Runoff." *Transactions*, ASCE Volume 125.

of surface water. The pace of urbanization was slow and the cumulative impact of drainage decisions was difficult to assess, if it was even considered.

The major residential boom in the post-1945 era, relying on total subdivision of land into individual lots, often with complete stripping of natural site features and replacement by an "efficient" design, was the logical extension of that approach. Beginning in the late 1950s, however, some proposed residential developments clustered dwellings and created common open space, seeking to preserve and enhance natural site attributes. These various innovative concepts of land planning, which have now become grouped under the common title Planned Unit Development, present opportunities for stormwater management consistent with the emerging new philosophy advocated by this report. Traditional subdivision design practices will also benefit from the new stormwater management approaches, but not always to the same degree.

2.4 The Basic Concepts

It is almost impossible to summarize the development of this report into a concise series of statements of objectives, principles, and design considerations. The ideas are far reaching and deal with many levels of concern. Following, however, are at least some of the basic concepts upon which these principles and objectives are built.

- The water falling on a given site should, in an ideal design solution, be absorbed or retained on-site to the extent that after development the quantity and rate of water leaving the site would not be significantly different than if the site had remained undeveloped. This objective may conflict with present statutory and case law in some locales, which does not reduce its validity.
- Optimum design of stormwater collection, storage and treatment facilities should strike a balance among capital costs, operation and maintenance costs, public convenience, risk of significant water-related damage, environmental protection and enhancement, and other community objectives. The optimum balance among these factors is dynamic, changing over time with changing physical conditions and value perceptions.
- Just as the importance of water quality is being increasingly recognized, a major new emphasis needs to be placed on the identification and application of "natural" engineering techniques to preserve and enhance the natural features of a site, and to maximize economic-environmental benefit. "Natural" engineering techniques are those that capitalize on and are consistent with natural resources and processes. Engineering design can be used to improve the effectiveness of natural systems, rather than negate, replace or ignore them.
- Among the new trends in basic philosophy that should be pursued, are concurrent recognition of the convenience drainage and overflow or flood conveyance elements of drainage systems, the use of on-site detention storage

and "blue-green"[3] development, the increased use of storage to balance out handling or treatment of peak flows, the use of land treatment systems for handling and disposal of stormwater, and perhaps most important a recognition that temporary ponding at various points in the system, including on the individual lot, is a potential design solution rather than a problem in many situations.

- A continuing recognition that there is a balance of responsibilities and obligations for collection, storage and treatment of stormwater to be shared by individual property owners and the community as a whole.
- A new recognition that stormwater is a component of the total water resources of an area that should not be casually discarded but rather should be used to replenish that resource. Stormwater problems signal either misuse of a resource or unwise land occupancy.
- A growing emphasis on the recognition that every site or situation presents a unique array of physical resources, occupancy requirements, land use conditions, and environmental values. Variations of such factors within a community generally will require variations in design standards for optimal achievement of runoff management objectives.

The above key concerns, while not all-inclusive, embody a basic philosophy that should receive wide dissemination and due consideration. Although this report focuses primarily on residential design practices, these concepts should be considered and applied to entire drainage basins in which any development may proceed. Therefore, underlying the points made above is another basic idea that is already well perceived and that is being implemented:

- Re-evaluation of the approach to basin-wide runoff management is a universal need. It is the responsibility of, and should be an objective of, the public sector.

Responsible solutions for individual developments in the absence of basin-wide plans will be more difficult to achieve, particularly where current practices are based on traditional drainage concepts. For example, if current practices allow upstream development to use traditional drainage approaches that increase runoff, a development relying on new concepts might be unable to accommodate the amount of excess runoff thereby generated without additional significant costs. The approaches suggested herein should allow development to proceed on individual projects in the absence of a basin-wide plan since the strategy for retention and attenuation of peak runoff and total runoff to values not significantly different from pre-development levels would normally be compatible with any future plan that might evolve for a watershed. Unfortunately, this can probably only be achieved at an initially higher cost for the project. Therefore, development of basin-wide plans should be pursued.

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[3] Jones, D.E. 1967. "Urban Hydrology—A Redirection." *Civil Engineering.* ASCE Volume 37, No. 8.

2.5 A Note for Readers

An area of concern as complex as that of stormwater management defies translation into simple terms that would be easily and completely understandable by all the audiences to whom this is addressed. It would be misleading not to have included a significant amount of technical material, which will be of more specific value to engineers attempting to apply the concepts embodied herein, but that may confuse and cloud the understanding of the less initiated reader. To the greatest extent possible, Objectives and Principles and to a lesser extent Design Considerations are described in a manner that will achieve general comprehension without ignoring the complexities and technical concerns. At the same time, there are significant amounts of technical information and references that can be useful to the engineer seeking to apply this report in actual practice.

Like all documents seeking to serve a wide audience but dealing with technical terms that have, of necessity, acquired very precise meanings for use in design formulae, the participants in the preparation of this report are very concerned about the choice of words and the definition of terms. The obvious temptation when trying to espouse new thinking is to establish new and never before used nomenclature. Even the use of the word *storm* can be questioned because it does not clearly define the true nature and complexities of natural events, each of which is unique. The term "storm" is often applied to synthetic events that for convenience are used in derived equations that sometimes approximate real events.

This report prefers to avoid such precise definitions during general discussion and to accept them, of necessity, when they are part of a technical discussion. This dichotomy will be more apparent to the technically trained reader than to the non-technical audience.

2.6 A Definition of "Stormwater Runoff System"

The term "stormwater runoff system" is used frequently and will be used herein, as described below.

First, a stormwater runoff system is composed of both natural and man-made elements. In the past, designers have often failed to capitalize upon natural elements and have at times ignored them when a constructed element was installed.

Second, these components include not only those that contain and convey stormwater, but also those that absorb, store and otherwise use stormwater rather than dispose of it.

Third, the stormwater runoff system is a single system having two purposes: 1) the control of stormwater runoff to prevent or minimize damage to property and physical injury and loss of life that may occur during or after a very infrequent or unusual storm; and 2) the control of stormwater to eliminate or minimize inconvenience or disruption of activity as a result of runoff from more frequently occurring, less significant storms. Some individual components of the drainage system may operate only in fulfillment of the first purpose. This dual purpose is characterized in much of the technical literature as "major" and "minor" functions.[4] Many American and foreign cities have revised their drainage design approaches to embrace the dual function approach.

Fourth, within a single system, there are components that are designed primarily to obtain convenience at the smallest scale of the system at the individual site or intersection, during minor or frequent storms. During an infrequent or major storm, the capacities of many of the convenience-oriented components will be exceeded and flow capacity must be provided by other components designed to provide safety and minimize damage throughout the system, from the individual site to the discharge point of the drainage basin or watershed. It must be recognized and emphasized that a total stormwater runoff system, subjected to an infrequent major storm, cannot be expected to prevent inconvenience and minor property damage. A design that would eliminate all such stress would be fundamentally unreasonable and almost certainly infeasible. Expected damages from such a major runoff event would include minor erosion and scour, damage to lawns and vegetation, and damage to unwisely located structures, but flooding or undermining of buildings or essential facilities should not occur.

Most of our Nation's past urban drainage construction expenditures have been for small pipes and inlets that cannot intercept and transport the total runoff volume from infrequent storms. The designer's attention should focus upon controlling and safely routing all foreseeable runoff, especially that which cannot find its way into storm sewer pipes. Such a focus often will lead to wiser land occupancy practices, and minimize expensive drainage works construction, while mitigating potential damages from runoff.

2.7 Using This Report

In the next 25 years, more money may be spent for stormwater management than has been spent for drainage during the entire history of the Nation. This will amount to billions of dollars, not including funds that will be expended to maintain water quality, which also is an increasing national concern. Much, if not most, of this investment will be private funds expended during the development of land for urban uses. Ultimately, however, all costs (capital, debt and maintenance) are borne by all citizens. Public investments should be made wisely in furtherance of the quality of life so highly valued. It is the opinion of ASCE, NAHB, and ULI that the application of the Objectives, Principles, and Design Considerations contained in this report, with due regard for unique and particular circumstances and conditions found in various areas of the country, will be a significant step in the right direction.

For the concepts in the report to achieve wide application, there will be a need to induce institutional changes even beyond the design professions and the regulatory institutions of governments. Changes also are necessary in the financial institutions

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[4] National Association of Home Builders. 1974. *Land Development Manual.* Washington, DC.

that fund development based on their approval of a project's design, in the insuring institutions and their perceptions of the insurability of this approach, and in the legal professions in relationship to the pre-existing body of land use law regarding rights, responsibilities and liabilities. The implication of the above statement, that change may be precluded by institutional constraints, is real. The importance of encouraging application of worthwhile new approaches should provide the impetus to achieve necessary changes.

It is hoped that this report will motivate creative rethinking and updating of drainage design practices. Anyone disagreeing with any part of this report is encouraged to advise the publishers of that disagreement, including their detailed reasons and alternative recommendations. Such information will help guide future revisions and enhance the document's value to our Nation.

3.0 OBJECTIVES

- To provide a clear understanding of residential stormwater runoff systems, including their intended functions and the impacts of alternative philosophies or policies in the design and effectiveness of stormwater runoff systems.

Residential stormwater runoff systems must fulfill two objectives: 1) they must prevent significant loss of life and property due to runoff from any foreseeable rainfall event; and 2) they must provide an acceptable degree of convenient access to property during and following frequent rainfall events. Both of these objectives must be accommodated in the design process with the understanding that some protection components of the stormwater runoff system may have to operate only infrequently. It must also be understood that providing protection against a given event, e.g., against the worst stormwater runoff of record, does not guarantee that a greater runoff event will not occur during the useful life of the property. Similarly, the enlargement of stormwater runoff system components providing access convenience is generally an infeasible approach to fulfillment of property protection objectives.

- To promote a philosophy for the design of residential stormwater runoff systems that anticipates the impact of alternative design solutions within both the land subdivision and the overall drainage basin.

Residential stormwater runoff systems are not restricted in their design or impact to the immediate tract of land that they serve. Each is a part of a basin-wide drainage system and must, at a minimum, accommodate stormwater flowing into the tract from upstream sources and mitigate the impacts of the outflow on downstream properties.

- To provide a methodology for determining the type and degree of analyses necessary to derive the optimum design solution.
- To provide guidance toward identification and selection of alternative solutions to drainage problems that utilize and preserve, to the extent possible, desirable existing natural systems.

To a large degree, it is only in recent years that the cumulative impact of subdivision drainage in a basin has been assessed with a concern for basin-wide runoff management.

- To encourage the design of systems that will minimize potential erosion and sedimentation problems.
- To encourage the design of systems that respond to the need to maintain or enhance groundwater resources, including groundwater quality, except where land stability might be impaired.

This has led some local ordinances to require that stormwater runoff rates after development not exceed pre-development runoff peaks, both to aid in erosion control and to decrease the probability of downstream flooding. This goal rarely is fully attainable.

- To encourage the design of systems that will reduce capital and environmental costs to the community.
- To encourage the design of systems that will minimize potential pollution from residential surface drainage.
- To encourage system designs that conserve materials.
- To encourage continuing development of additional methods and practices designed to enhance stormwater management contributions to life and environmental quality.
- To encourage continuous improvement of regulatory practices and policies, at all levels of government.

Practices suggested herein should not increase localized capital costs for development of quality stormwater systems. They should reduce cumulative downstream drainage construction costs by comparison with the effects of past practices that forced rapid runoff from land developments. The suggested practices are not new but their optimum implementation often requires re-evaluation of the basic philosophy of stormwater runoff system design. Changes in philosophy may produce additional alternatives for design solutions that this document may not cover. It is hoped that additional alternatives will achieve the primary goals of safety, economy and the enhancement of residential environments.

- To summarize recent advances in practice for the benefit of professionals not routinely involved in drainage design.

The design of residential stormwater systems can utilize various methodologies to arrive at solutions or alternatives for the drainage plans. Differences in runoff management philosophies and in environmental conditions should be carefully evaluated in selecting methodologies to be used. The scale of development and the size and physical characteristics of the drainage area affect selection of methods, as do climatic characteristics. While certain methods may be desirable for assessing small sites with simple drainage patterns, they may be inappropriate for larger sites with more complex patterns. Conversely, the methods available for assessing large scale or basin-wide stormwater systems may be too complex, costly and/or inefficient to be useful for smaller sites, and often fail to consider important micro-scale details and alternatives. No one method or solution is possible for all areas. This document will suggest some logical methods of analysis for selection of optimum localized approaches to stormwater management. Inherent in the philosophy regarding those choices is the basic need for practical conservation of natural systems while providing for safety and convenience in land development.

4.0 PRINCIPLES

4.1 General

Future problems can be minimized or avoided if all political jurisdictions within a drainage basin collaborate to define and implement optimum analytical methodologies, standards and regulations pertaining to drainage systems and land development.[5]

- Local government should study and develop master plans for each drainage basin within its jurisdiction that may be impacted by development. Optimally, contiguous communities will collaborate to master-plan drainage basins that extend across jurisdictional boundaries.
- All individual land development proposals should include stormwater runoff system plans that will be compatible with any basin-wide master drainage plans, and that anticipate and provide for potential effects of upstream development on the proposal area and on downstream areas.

The design of a stormwater system must consider convenience and safety both at the subdivision level and at the drainage basin level. Cumulative basin-wide effects of proposed land development practices should provide the basis for decision-making, rather than the generally minimal incremental effects of individual land developments.

- Stormwater runoff systems should be designed to assure provision of both major and minor components that will serve specific access convenience and property protection objectives.

4.2 Storage

When provision of storage is being considered, the designer should verify that attenuation of the runoff peak will not aggravate any potential downstream peaking conditions.

- The design of permanent and temporary ponding and/or storage should be an integral part of the overall development planning process, and should consider

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[5] Wright-McLaughlin Engineers. 1969. *Urban Storm Drainage Criteria Manual.* Denver Regional Council of Governments, Denver, Colorado.

opportunities within the open space and landscaped areas for the creation of such facilities.

- Stormwater runoff systems should be designed to facilitate aquifer recharge when it may be advantageous to compensate for groundwater withdrawals. Conversely, designs should avoid recharge where groundwater effects might be harmful.

Storage should not be created by happenstance or strictly in response to aesthetics. Storage must be rationally planned to accomplish its intended functions. Improperly located storage may create its own flooding problems or aggravate others.

- Design of permanent storage facilities should consider safety, appearance, recreational use and effective, economical maintenance operations, in addition to the primary storage function.

4.3 Open Channels

Open channels and swales should harmonize with the natural features of the site. They should relate closely to individual lots so that occupants will not be tempted to use them for disposal of lawn clippings or other debris or wastes. Provisions must be made for maintenance on a routine basis to assure that open channels can function at or near design capacity. The utilization of open channels must be carefully evaluated. They may become depositories for debris if installed in neighborhoods not having a strong pride of ownership. The *blue-green* approach can help make a channel into an aesthetic focal point and discourage its abuse.

- Natural overland flows, and open channel and swale routings should be the preferred alignments for major components of a residential drainage system.
- Open channels and swales should be routed and designed to avoid or minimize safety hazards.

Generally, street and lot patterns and grades should be designed around natural drainage routings, if excessive land development expenses are to be avoided and environmental values preserved and enhanced.

- Alignment of open components of a drainage system must be coordinated with the design of lot and street patterns and grades.

4.4 Streets and Curbs

Residential streets may be broken down into five classifications that may be utilized in the determination of safety and convenience associated with storm drainage considerations.[6] Each of the five classes—place, lane, sub-collector, collector and arterial—has its own limitations on the depth-of-flow in gutters and spread of water across the pavement. Arterial streets should remain as free of water as is practical.

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[6] *Residential Streets: Objectives, Principles and Design Considerations.* 1974. ULI, ASCE and NAHB, Washington, DC.

The incidence of high traffic volumes and speeds and probable pedestrian traffic preclude any appreciable spread of water from the standpoint of both safety and convenience.

- Stormwater management systems, street layout, lotting patterns, and the horizontal and vertical locations of curbs, inlets and site drainage and overflow swales should be concurrently designed.

Depth and velocity of runoff flows in the gutters (if any) on places and lanes must be given careful consideration due to the likely presence of children shortly after the end of the rainfall. Prudence suggests that safe velocities for pedestrians, especially small children, will be substantially less than ten feet per second.

- The depth-of-flows in gutters and the allowable spread of water across the pavement should be consistent with the classification of the street based upon its anticipated use and traffic load.
- Appreciable amounts of runoff usually should not be permitted to flow across an intersection under normal rainfall conditions.
- The maximum velocity of flow, in the deepest part of the gutter under "convenience" design conditions, should not exceed ten feet per second.

4.5 Stormwater Inlets

Consideration should be given to:

1) Inlet hydraulic capacities as limited by gutter gradients and cross sections.
2) Peak flow and time of concentration.
3) Erosion, debris, and potential inlet blockage.
    a) The number and spacing of inlets should be carefully regulated.

Grated inlets will generally prove more hazardous and more subject to debris blockage than properly designed side-opening inlets.

- Inlet design should consider pedestrian and bicycle safety as well as hydraulic efficiency.
- Sediment traps should not be designed into the inlet box. Inlet boxes should be self-scouring, even under low-flow conditions.

4.6 Enclosed Systems

The use of enclosed components should be minimized to the extent consistent with: 1) the ability of the existing natural systems to accommodate storm runoff; and 2) the degree to which the local public will accept and act responsibly toward open channels.

- Enclosed components of a stormwater runoff system should help manage stormwater, not just dispose of it.
- Energy dissipaters and other outfall protection should be designed and installed where enclosed drains discharge onto erodable soils.
- Conduit sizes of enclosed components of a drainage system should be selected by use of computed hydrologic and hydraulic data.

4.7 Other

Siltation (sedimentation) ponds should be installed at the start of construction, but only where the soil's particle sizes and specific gravity are sufficiently large to assure entrapment of a significant proportion of eroded materials. Where sedimentation ponds foreseeably will have only limited effectiveness, erosion control at the sediment source should be emphasized. Direct flows from parking lots, streets, and roofs to natural water courses should be minimized. Where practical, the outflow from small parking lots and roofs should be sheeted across turf. Detention storage should be incorporated in their design, whenever feasible.

- Erosion from stormwater runoff should be minimized by appropriate design within the system, but erosion control or prevention generally should be achieved at the source if downstream sedimentation problems are to be avoided.
- Construction, amortization, maintenance and operating costs are integrally related over time and their initial *present value* should be minimized.

5.0 DESIGN CONSIDERATIONS; STORMWATER RUNOFF; ANALYSIS CONSIDERATIONS

Stormwater runoff is the water flowing over ground surfaces during and immediately following a rainfall. In specialized cases, stormwater runoff may be augmented by groundwater flow and melting ice and snow. The runoff passing a particular point is the total rainfall at that point less the amounts of infiltration, transpiration, surface storage and other losses.

The primary goal of stormwater runoff management is to assure the provision of facilities to control stormwater runoff in ways that will minimize hazards to life. In addition, such systems should reduce inconvenience and minimize hazards to property. Achievement of such a system requires careful estimation of hydrologic factors at controlling design points in the system and includes the siting of property improvements in relation to all potential hazards. The primary factors are discharge rates, runoff volume, flow velocities, stage-discharge characteristics, maintenance of downstream flow conditions and water quality.

Initial planning for a residential subdivision should begin with a study of the total drainage area. The major components of the system (that should be readily identifiable; e.g., streams, large depressions, existing lakes and ponds) should be located and their potentials for stormwater management assessed. Utilization of storage facilities should be given particular attention, as they can often significantly reduce the in-place cost of the system. Storage facilities are not inherently beneficial unless they are properly located and designed. Improperly designed or located ponds or recreational lakes may be desirable.

In this initial planning phase, existing plans for stormwater management—or the lack of such plans—should be assessed both as to the effect of the subdivision drainage on basin-wide drainage and vice-versa. Preliminary decisions should identify acceptable levels of temporary inconvenience to residents, such as ponding at inlets or limited storage in swales or depressions.

It may be desirable or necessary to estimate amounts of runoff at various design points within the system prior to planning street layouts and system details. Runoff estimates can range from a single point estimate—e.g., peak flow rate at the discharge point(s) from the proposed subdivision—to normal or integrated hydrographs, which account for varying flows, over time, at various points in the drainage system.

The selection of controlling design points depends upon the methodology employed to evaluate the rainfall and runoff rates, on the component of the system under consideration, and on the specific local terrain and its existing or planned development characteristics. The major components of the system are outfalls, storage ponds, reservoirs, large channels (open or closed), emergency overflow routes, natural streams and others. The minor components include inlets, drainage swales, storm sewers and feeder pipes, and minor drainageway crossings.

The degree of sophistication used in the runoff analyses depends on the size and complexity of the drainage area under consideration, the available flexibility in site improvements to avoid flooding losses, the nature of potential losses, and the types of drainage facilities to be incorporated in the design. It should be emphasized that sensitive planning in the development of a stormwater runoff system that maximizes the use of natural elements can dramatically reduce initial capital costs and future maintenance costs of the system. The stormwater runoff system design should also be correlated with the earliest design and layout of the street system. The street system is an integral component of the stormwater runoff system and its coordination with drainage design is essential to conserve costs, avoid problems, and enhance the neighborhood.

There are a wide range of analysis techniques available for guiding the design of stormwater runoff systems. The choice of technique must be suited to the size and complexity of the area, the degree of safety and convenience sought, and the cost factors involved. Regardless of the techniques selected to guide the design the following factors must be considered:[7]

1) Rainfall

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[7] Local government should develop basin-wide drainage studies to which individual sites can relate their specific designs.

   a) Historic
   b) Predictable future
   c) Bases for design

2) Drainage Area Characteristics
   a) At the site
   b) Downstream
   c) Upstream
   d) Basin-wide
3) Land Use Characteristics
   a) Present
   b) Future—short term
   c) Full development
4) Design Options
   a) On-site detention/storage
   b) Overland flow
   c) Channel capacity; volume/storage
   d) Storage, detention, routing
5) Risk Analysis
   a) To life
   b) To property
      - On the site
      - Downstream
      - Upstream
6) Costs
   a) Initial
   b) Amortization
   c) Operation
   d) Maintenance
   e) Replacement
   f) Inconvenience
   g) Flood damage

5.1 Review of Analysis Techniques

The quantification of expected rainfall generally is a pre-requisite to stormwater runoff analyses. Rainfall estimation permits approximation of runoff discharge rate, volume and stage.

A rainfall event is a random occurrence—no two events are alike. All analysis techniques must, therefore, begin with a description of historical rainfall events that will be used in the analysis. Ideally, an analysis would be able to accurately describe the management of a wide variety of natural rainfall events with all their differences and complexities. The degree of refinement of analysis techniques and data precludes this ideal.

Rainfall quantification is achieved through an analysis of historical records and translation into a synthesized description that approximates a natural event. This description has three parameters: intensity—how much water; duration—in how much time; and frequency—how often this situation occurs. More sophisticated analysis techniques will use many sets of parameters. The National Weather Service publishes reports on rainfall frequency, intensity and duration for many cities. Their local offices can provide up-to-date information and references.

The following section is a brief summary of the prime analysis and design techniques currently used in the design of urban stormwater runoff systems. Past designs of stormwater runoff systems have most commonly been based on the Rational Method for rainfall/runoff analysis, and steady rate flow equations for sewer flow analyses.[8,9] The limitation of these approaches in the design of relatively complex urban systems under dynamic flow conditions is well recognized.[10] A number of mathematical models have been developed recently to aid the design engineer in his task. The decision as to what level of sophistication is desirable and, indeed, achievable, must be made for each project. There are no hard and fast rules as to what the "optimum" technique might be in a general sense. The decision must be made in the light of the size of the drainage basin, type of development under consideration, terrain variables, natural stream systems, aesthetic goals, drainage codes, data availability and so on. The primary techniques of practical importance are reviewed below with their merits and problems briefly noted.

Quantitative estimates of runoff in a given drainage system or from a watershed or drainage area can be made by using one of four basic techniques, or combinations thereof.

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[8] Joint Committee of the Water Pollution Control Federation and the American Society of Civil Engineers. 1969. *Design and Construction of Sanitary and Storm Sewers*. WPCF Manual of Practice No. 9. Also available as ASCE Manual of Practice No. 37, New York, NY.

[9] Yen, B.C., W.H. Tang, and L.W. Mays. October 1974. *Designing Storm Sewers Using the Rational Method.* Water and Sewage Works.

[10] McPherson, M.B. January 22, 1969. "Some Notes on the Rational Method of Storm Drainage Design." ASCE Urban Water Resources Research Program Technical Memorandum No. 6. ASCE, New York, NY.

5.2 Hydrologic Simulation Models

For decision-makers, designers and operators, a comprehensive mathematical computer simulation program that models quantity (flows) and quality (concentrations) during the total urban rainfall runoff process can be an invaluable tool. Such a model can give a good representation of the physical system. It can also serve to evaluate the physical and cost effects of alternate schemes of stormwater management or pollution abatement procedures.

An urban runoff model in its elementary form is simply a group of mathematical expressions that simulate the processes of the conversion of rainfall to runoff. Additionally, it may realistically reflect dry weather flow, infiltration, treatment and storage, and water quality parameters. Models can range from the crude approximations of the Rational Method to the solution of many simultaneous differential equations. Nearly all applications of detailed models require a high-speed digital computer.

The physical characteristics of the tributary area and the drainage system (size, slope, land use, imperviousness, sewer characteristics) must be embodied to some degree in the input to all models. The extent of data and processing required varies with the model employed. Much of the data reduction is relatively straightforward (e.g., tabulation of diameters, slopes, lengths of a sewer system).

Many models are designed such that if all input parameters are reasonably accurate, the physics of the processes are simulated well enough to secure satisfactory results without calibration. However, neither the input data nor the numerical methods are accurate enough for most specific areas. Additionally, there are computational procedures within some models that have been developed from limited data. This is particularly true of the quality components of models. Consequently, it is normally essential that some local verification-calibration data be available for a specific application site to lend reliability to the predictions of any urban runoff model. Calibration of most models against measured rainfall amounts and associated runoff flows can be accomplished through adjustment of the input parameters. Quality measurements, to be of calibration value, require time-related flow measurements.

There are three categories of models: planning, design/analysis, and operation. Such models have somewhat different characteristics and various models overlap on objectives to some degree.

Planning models give an overall assessment of the urban runoff problem and may also provide estimates of the effectiveness and costs of alternative storm runoff management procedures. Relatively large time steps (hours) and long-term simulation periods (months and years) generally characterize these broad-objective models. Minimum data requirements and low mathematical complexity are typical. Long-term planning models may also generate initial conditions for input to design models. The effects of urbanization are readily computed.

Design models generally involve the simulation of selected storm events with short-time steps such as minutes (the shortest interval for which Weather Service data are generally available is five minutes) and short simulation periods (hours). Several of these can be used for a complete description of flow, storage and pollution routing from the point of rainfall through the total drainage system and into the receiving waters. As with planning models, design models can be used to arrive at least-cost abatement procedures for both quantity and quality problems. Data requirements can be moderate to very extensive depending on the particular model involved.

Operational models help resolve actual control decisions during a storm event. From telemetered rain and flow gauge signals as inputs to the model, estimated system responses are projected a short time into the future. In-system storage, regulator settings, or diversions, or combinations of these, may then be employed as control options. Informational needs for operations models are much greater than for either planning or design models. A number of models have been developed and variations and improvements are almost continuously evolving, making it difficult to characterize them in any lasting way. However, several comparisons of model characteristics were reported in 1974 and 1975, and the reader is referred to them for details.[11,12,13,14,15,16]

Quality aspects will become increasingly important with the growing emphasis on minimizing the ecological impact of all developments on their surroundings. In those models accommodating water quality considerations, components considered range from erosion rates and sediment loads to biochemical oxygen demand, nitrogen and phosphates.

Hydrologic modeling is as much an art as a science and, thus, can never be, and need not be, perfect or complete. Discussions of the current (1974-75) use of stormwater models are in Huber APWA 1975; and McPherson and Schneider WRR 1974; and each of these has a good coverage of related recent literature.

5.3 Unit Hydrograph Techniques

A unit hydrograph is the runoff hydrograph resulting from one inch of excess rainfall applied to a given drainage basin over some specified time interval.

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[11] Brandstetter, A. May 1975. "An Assessment of Mathematical Models for Storm and Combined Sewer Management." Draft of a U.S. EPA project final report by Battelle, Northwest, Richland, Washington.

[12] Brown, J.W., M.R. Walsh, R.W. McCarley, A.J. Green, and H.W. West. August 1974. "Models and Methods Applicable to Corps of Engineers Urban Studies." U.S. Army Engineer Waterways Experiment Station, Misc. Papger H-74-8. Vicksburg, Mississippi.

[13] Huber, W.C. May 1975. "Modeling for Storm Water Strategies." The *APWA Reporter*, Volume 42, No. 5, The American Public Works Association.

[14] McPherson, M.B. June 1975. "Urban Mathematical Modeling and Catchment Research in the U.S.A." ASCE Urban Water Resources Research Program Technical Memorandum No. 1HP-1. ASCE, New York, NY.

[15] McPherson, M.B., and W.J. Schneider. June 1974. "Problems in Modeling Urban Watersheds." Water Resources Research, Volume 10, No. 3, American Geophysical Union.

[16] Torno, H.C. 1975. "Storm Water Management Models." *Urban Runoff, Quantity and Quality*. ASCE, New York, NY.

Hydrographs for a given storm can be computed from the unit hydrographs through a simple scaling and lagging operation. The unit hydrograph itself is derived either from analysis of historic records or through a regression equation(s) based on watershed characteristics such as area, length and slope. Although unit hydrograph techniques have been widely used for rural watersheds, their use in urban areas is limited by data availability. Several regions in the country have performed the necessary studies, however, to enable acceptable synthetic unit hydrographs to be derived for given development patterns.[17,18,19,20] Reliability of these hydrographs reportedly is good in the regions for which they were developed.

Unit hydrograph techniques yield the total runoff hydrograph at various locations within the drainage system and can be combined with suitable reservoir simulation techniques to evaluate the operation of storage as a control measure in the system.

5.4 Regression Models

Regression models, of which many exist, seek to relate a causal factor such as rainfall and/or watershed characteristics with an effect such as peak discharge, runoff volume or annual mean flows, through statistical correlation. Their applicability to urban stormwater systems is minimal, mainly because of the lack of adequate historic data for regression analysis and because of the constantly changing watershed characteristics where urbanization is in process. These models do not in general predict the total hydrograph and are of limited use whenever storage in the system is being considered.

5.5 Empirical Formulas

A wide range of empirical formulas to relate runoff to rainfall have been developed over the years. The oldest of these is the Rational Method, which together with its derivatives, forms the basis for much of urban hydrology as currently practiced. The Rational Method is presented below in some detail, since it is the most popular method and may be useful in the detailed design of the minor components where a high degree of accuracy is not required.

In the Rational Method, the peak of runoff, Q, in cubic feet per second, is computed as:

$$Q = CiA$$

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[17] Hamm, D.W., C.W. Morgan and H.A. Reeder. October 1973. "Statistical Analysis of Hydrograph Characteristics for Small Urban Watersheds." Tracor, Inc., Report No. T73-AU-9559-U. Austin, Texas, (available from NTIS as PB 228-131).

[18] Linsley, R.K., Jr., M.A. Kohler, and J.L.H. Paulhus. 1958. *Hydrology for Engineers*. McGraw-Hill, New York, NY.

[19] Rantz, S.E. 1971. *Suggested Criteria for Hydrologic Design of Storm Drainage Facilities in the San Francisco Bay Region*. U.S. Geological Survey, Menlo Park, California.

[20] Schulz, E.F., and O.G. Lopez. December 1974. "Determination of Urban Watershed Response Time." Colorado University, Hydrology Paper No. 71. Fort Collins, Colorado.

In which C = runoff coefficient representing the characteristics of the drainage area.

i = average intensity of rainfall in inches per hour for a duration equal to the time of concentration, t, for a selected rainfall frequency.

t = time in minutes after the beginning of rainfall for runoff to peak at the point under consideration.

A = size of drainage area in acres.

Guidance for selection of coefficient C is provided by Table 23-1 that shows commonly used values in accordance with the type of development and local soil characteristics. A composite C value should be weighed in proportion to the acreage in each part of the sub-drainage area.

Common practice in determining the time of concentration, t, to a given design point has been to specify a fixed time for the flow to reach the first inlet and then to add the time of flow in the pipe system to that point. The time of concentration in overland flow can be estimated from Figure 2, which was extrapolated by Wright-McLaughlin Engineers. In determining the time of concentration for downstream locations, paved gutter, swale or channel velocities may be estimated by making a preliminary estimate of their discharges and using open-channel flow charts published by the Bureau of Public Roads.[21] Travel time is then computed using these velocities.

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[21] Bureau of Public Roads. 1961. *Design Charts for Open Channel Flow*. Hydraulic Design Series No. 3. Washington, DC.

TABLE 23-1

RUNOFF COEFFICIENTS

Description of Area	**Runoff Coefficients**
Business	
Downtown	0.70 to 9.95
Neighborhood	0.50 to 0.70
Residential	
Single Family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.10 to 0.25
Residential (suburban)	0.20 to 0.35
Apartment	0.10 to 0.30
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30
Character of Surface	Runoff Coefficients
Pavement	
Asphalt or concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.70 to 0.95
Lawns, sandy soil	
Flat, 2%	0.05 to 0.10
Average, 2% to 7%	0.10 to 0.15
Steep, 7% or more	0.15 to 0.20
Lawns, heavy soil	
Flat, 2%	0.13 to 0.17
Average, 2% to 7%	0.18 to 0.22
Steep, 7% or more	0.25 to 0.35

The coefficients in these two tabulations are only applicable for storms of 5- to 10-year return frequencies, and were originally developed when many streets were uncurbed and drainage was conveyed in roadside swales.

For recurrence intervals longer than 10 years, the indicated runoff coefficients should be increased assuming that nearly all of the rainfall in excess of that expected from the 10-year recurrence interval rainfall will become runoff and should be accommodated by an increased runoff coefficient.

The runoff coefficients indicated for different soil conditions reflect runoff behavior shortly after initial construction. With the passage of time, the runoff behavior of sandy soil areas will tend to approach that of heavy soil areas. If the designer's interest is long-term, the reduced response indicated for sandy soil areas should be disregarded.[22]

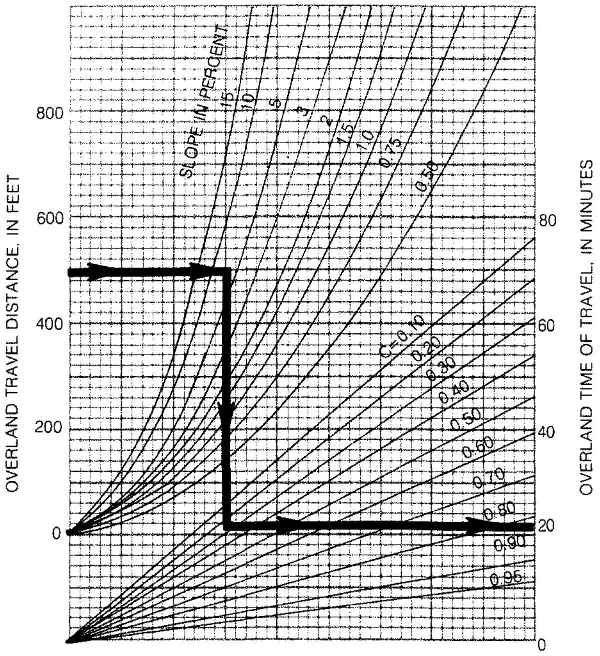

FIGURE 23-2. Relation of overland time of travel to overland travel distance, average overland slope, and coefficient C—for use in Rational Method.[23]

Appropriate values of rainfall intensity, i, may be available from local studies or obtained from the intensity/frequency/duration data.[24,25] Coefficients usually used in

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[22] *Design and Construction of Sanitary and Storm Sewers*. 1970. ASCE Manual of Practice No. 37. Notes revised by D.E. Jones, Jr.

[23] Wright-McLaughlin Engineers. 1965 "Airport Drainage." Federal Aviation Administration, Washington, DC.

[24] U.S. Department of Commerce, National Weather Service. 1961. *Rainfall Frequency Atlas for the United States,* for durations from 30-minutes to 24-hours and return periods from 1- to 100-years, Technical Paper No. 40, Washington, DC.

[25] U.S. Department of Commerce, National Weather Service. 1964. *Two to Ten Day Precipitation for Return Periods of 2- to 100-years in the Contiguous United States*. Technical Paper No. 49, Washington, DC.

the Rational Method must be revised if the method is used to forecast peak runoffs from very infrequent rainfall events.

For more definitive information regarding the Rational Method and its appropriate application, several works (some previously referenced herein) are available.[26]

6.0 STORAGE CONSIDERATIONS AND CRITERIA

One of the primary factors to consider in stormwater runoff management is storage. The availability or absence of facilities for temporary or permanent runoff storage is an important element in selecting the analysis methodology and establishing the underlying philosophy for design. As important as storage is, it should not be seen as the cure-all for storm drainage design. It is likely that in many instances, the storage capacity required to assure both maximum safety and convenience will not be economically feasible; but, this amount may still be desirable.

Provision of storage can reduce peak runoff rates; aid in the replenishment of the water supply; provide an attenuation mechanism if stormwater is to be treated; lessen the possibility of downstream flooding, stream erosion, and sedimentation; and can be used in the development of upstream areas to avoid increasing the runoff peaks that impact existing downstream facilities.

Storage occurs naturally on a small scale in most drainage areas. Natural storage is provided during overland flow in surface depressions and on wetted vegetation. Greater storage is possible where larger depressions and swales exist in the drainage area and where highly pervious recharge areas exist. Much natural storage is temporary, of small volume, and can be lost through development. This volume can be replaced by using swales, by revegetation and by utilizing special inlets that meter the outflow from planned ponding areas. Where detention storage is used, overflow routing must be provided with sufficient hydraulic capacity to assure freedom from significant downstream damages in the event of improbable runoff peaks. Large scale temporary retention storage can be used to replace storage loss due to the increase in impervious surfaces associated with development.

Rooftop and parking lot ponding are just two methods for temporarily storing and then slowly releasing this outflow of stormwater. In addition, the design of percolation ground storage facilities and dry ponds may be utilized to accommodate large amounts of stormwater.

Permanent storage (ponds, reservoirs and stream channels) provides maximum amounts of storage with the greatest amount of certainty. The "blue-green" approach to development, where practical, provides such storage in an economical manner consistent with environmental protection and enhancement.[27]

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[26] *Erosion and Sediment Control Handbook.* December 1974. Fairfax, Virginia.

[27] Poertner, H.G. 1973. *Practices in Detention of Urban Stormwater Runoff.* Department of the Interior. Washington, DC.

6.1 Degrees of Storage

Different degrees of storage should be considered in residential design. The lowest degree is the natural storage provided by surface depressions and by foliage and ground cover interception of rainfall. To take advantage of this storage, natural ground cover should be maintained. Temporary, usually small volume storage can be provided for in the design of swales, pipes, and channels upstream from embankments. Outlets from temporary storage can be choked or otherwise controlled to attenuate peak outflow, but safety and protection of adjacent and downstream properties should be assured by conscious design of overflow capacity and siting of damage-susceptible improvements. Facilities may be designed specifically for storage. Rooftop and parking lot ponding, recharge basin storage and normally dry ponds, may be employed in a stormwater runoff management system. Permanent storage, especially as provided by use of the "blue-green" approach, may be particularly useful. The comparative amounts of storage that may be achieved using different combinations of facilities will vary. The designer of storage should determine that the cost of storage provisions will not exceed benefits accrued and that the designs will be economical to maintain. The residential storage system should be coordinated with watershed and regional storage plans for flood control, water supply and recreation.

6.2 Factors to Consider

Permanent ponds and lakes have multiple benefits including short-term and long-term enhancement of property values and the landscape; possibilities for boating, ice skating, fishing and swimming; and habitat for resident and migratory wildlife. Proper maintenance and protection from health and safety hazards and positive control of visual appearance must be integral parts of storage design and planning. Permanent storage sites must be evaluated to assure their capability to retain water and to determine if an adequate natural or artificial supply of water is available year round to replace evaporation and infiltration losses. Eutrophication, or declining water quality, can be a very serious problem in shallow lakes.[28]

Excessive lowering of the water surface during dry spells can decrease the aesthetic and recreational value of storage. Siltation of permanent ponds may result in loss of storage capacity or undesirable weed growth. Control of erosion is a major consideration in the design, construction and maintenance of storage facilities. General information on residential lakes is available from the USGS.

Temporary storage in "dry ponds" can be used effectively in areas designed specifically for that purpose; water can accumulate in these areas during and for a short period after storms. Since these facilities are designed to completely drain after the storm, they can serve a dual purpose. Golf courses, recreation fields and parks are examples of compatible uses. Temporary storage can also be obtained in parking lots, on rooftops or in underground seepage pits. Footnote 27, sponsored by the

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[28] Rickert, D.A., and A.M. Spieker. 1971. *Real Estate Lakes.* Department of the Interior, USGS Circular 601-G.

Office of Water Resources Research and published (1974) by the American Public Works Association on "Practices in Detention of Urban Storm Water Runoff" can be consulted as a current investigation of some of the concepts, techniques, applications, costs, problems and legal aspects of urban stormwater storage.

Parking lot storage has been combined with percolation trenches filled with coarse gravel that intercept some runoff from the lot. Some experimental parking areas on non-cohesive sub-grades are being constructed with a porous pavement that allows direct recharge of groundwater. These types of storage may prove advantageous in some built-up areas where large amounts of open spaces are not economically available.[29]

6.3 Rooftop Ponding

The structural capability of the roof system must be considered when designing a temporary rooftop storage system. A 3-inch water depth is equivalent to a load of 15.6 lbs./sq. ft. that is less than most current building code requirements for live loads. Overflow mechanisms should be provided so that there is no danger of overloading during major storms. Special considerations of roof watertightness may be necessary since storage may be effective only if the water is detained for a significant period of time. Many flat roofs already pond significant amounts of water, although not by design, which should be considered when evaluating drainage conditions in established commercial areas.

6.4 Parking Lot Ponding

Parking lot ponding should be arranged so that pedestrians can reach their destinations without walking through ponded water. The ponding should be relegated to those portions farthest from the use served or to overflow parking areas, and should be a reasonable portion of the total area so that sufficient parking remains available for use. The maximum design depth of ponding can vary depending upon the location. A 7-inch design depth is not unreasonable where access to parked vehicles will not be impaired. An overflow outlet should be provided so that runoff from major storms will be limited to a 7-inch depth. Debris may accumulate at outlet drains, which may reduce the capacity of the drain and become unsightly, so provisions must be made for periodic cleaning. Thought should be given to the use of semi-paved/semi-grassed areas for overflow parking that will permit infiltration of rainfall and reduce the total runoff associated with parking lot pavements.

6.5 Percolation Storage

Under favorable conditions of a deep, permeable subsoil, runoff may be discharged into trenches back-filled with sands and gravels chosen and placed in accordance with sound graded filter principles. So long as the system does not become clogged by sediment, it will accomplish the dual purpose of disposing of at least part of the

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[29] Thalen, E. 1972. *Investigations of Porous Pavement for Urban Runoff Control.* Pennsylvania: Franklin Institute Research Laboratory.

stormwater and of recharging groundwater storage. Percolation tests must be run on the stratum at the bottom of the proposed trench. The rate of percolation will then control the outflow from the trench (exfiltration) provided the groundwater table is below the trench. An excess of inflow rate over percolation rate will result in temporary storage of water in the voids in the filter materials. Design of percolation storage must consider the potential effects of clogging of voids over a period of time and reduced load carrying capacity of pavement sub-base in a saturated condition. An overflow channel should be provided to carry off excess stormwater when the percolation trench capacity is exceeded. Under appropriate conditions, piped storm drains can be open-jointed or perforated to permit exfiltration.

6.6 Lakes and Ponds

The design criteria for stormwater runoff storage in lakes and ponds are numerous. Hydrographs of runoff from both frequent and unusual storms must be evaluated. For major installations, streamflow measurements in advance of the planning stage may be desirable.

The retaining structure, usually an earth dam, except where storage is created by further excavation of a low point in the site, must be designed and constructed according to the best-accepted practices for such structures. The outlet works:

- Must be designed to release the allowable flow in the downstream channel at the storage level established for the minor storm;
- Must include an overflow spillway to handle potential peak runoff from major storms so as not to cause serious damage to adjoining and downstream properties; and
- Must provide for draining of the lake or pond.

Adequate provision must be included for energy dissipation and erosion protection where outlet works discharge into the outfall channel. The outlet channel must be capable of handling the released flows without being damaged during a minor storm event and, where practical, within acceptable damage limits during a major storm. It is often completely impractical to avoid damages from a rare major storm.

The probable quantities of sediment coming off the watershed during the life of the facility should be estimated, taking into account the degree of erosion control likely to be achieved during and after construction. Temporary structures may be necessary to trap sediment before it enters the storage facilities during the construction phase. Planning of the facility must recognize loss of storage capacity from silting or anticipate occasional sediment removal. In the case of permanent lakes, the maintenance of a minimum level should be ensured by the inflow from the watershed (or by other augmentation) during prolonged dry periods and by the capability of the pool bottom to retain water.

The design of storage facilities to meet these criteria requires the services of engineers experienced in hydrology, hydraulics and earth structures. Aesthetic considerations suggest the services of a landscape architect. The engineering economy aspects of the design are especially important since considerations of alternative design possibilities as they affect the costs of construction and maintenance, reduced to average annual costs, can substantially lower the ultimate cost to the public.

6.7 "Blue-Green" Storage

Where streets must cross drainageways, there is an opportunity to utilize the roadway embankment as an effective dam for only moderate additional cost. Such dams are the heart of the "blue-green" development approach. Embankment quality and stability must be assured as for any dam, but the entire roadway and embankment may serve as the overflow spillway. This necessitates continuous erosion protection from the upstream point on the embankment face, where approach velocities become significant, to below the downstream toe of the embankment. Such dams may be used to provide a chain of lakes as neighborhood focal points that enhance long-term neighborhood values and stability. Potential overflow areas usually should be developed as green spaces. If left substantially open to public access and view, the blue-green spaces created can enhance entire neighborhoods rather than just immediately adjacent properties.

6.8 Other Storage Considerations

In creating urban ponds or lakes, certain special considerations are worthy of mention.

- Access to and along shorelines may be effectively limited to desired locations by planting thorny decorative shrubs.
- Lake bottoms within 10 feet of the shore should be so graded that water depth normally will not exceed 18 inches, to simplify immediate rescue of small children.
- Extensive areas of shallow water, especially in upper reaches of the lake, should be avoided to prevent undesirable weed growth.
- Dense plantings of shrubs that will act as barriers to automobiles are appropriate where vehicles might otherwise run into the lake, especially at night.
- Paved walkways roughly paralleling the shoreline, low-level night lighting, fixed benches, floored rain shelters and sensitive landscaping can add considerably to the charm of a lake or pond setting, and to the desirability of the surrounding neighborhood. Massive plantings of seasonally colorful shrubs, such as azaleas, redbud, dogwood or Japanese maple, can help publicize an area and create particular pride of ownership throughout the neighborhood.

7.0 STREETS AND CURBS

The primary purpose of residential streets is to provide vehicular access to homes and community facilities. Vehicles using the streets will vary from routine automobile traffic to larger delivery and service trucks and emergency police and fire vehicles. Streets also have several secondary functions. One is to provide routes for pedestrian and bicycle traffic; another, more relevant, is to collect and convey stormwater runoff.

Planning a drainage system should be done simultaneously with street layout and gradient planning, and careful consideration should be given to the following:

- The functions of streets as parts of the stormwater management system.
- Street slopes in relation to stormwater capacity and flow velocity in gutters and/or street swales.
- The location and sizing of street culverts. Culverts may be sized to create temporary upstream storage if there is proper consideration of earth bank stability and potential overflow effects during major flood conditions.
- Location of streets in relation to natural streams, storage ponds and open channel components of the system.
- Location and capacity of inlet points to pipes in relation to gutter slopes, the spread of water across streets and the flow of water across intersections.
- Coordination of street grades with lot drainage. Positive slope away from all sides of the house must be accomplished. Lot drainage becomes difficult when there is less than 1.5% to 2% (usually from 14- to 24-inches) fall from the earth grade at the center rear of the house to the street curb at the lowest front corner of the lot.

TABLE 23-2

Drainage Advantages And Disadvantages Of Street Cross Sections

Type of Cross Section	Advantages	Disadvantages
Normal crown with curb and gutter	Center lane clear during minor storms. Traffic barrier on both sides. Driveway ramps behind curb keep water confined.	Curb and gutter increases cost. Must have longitudinal grade to assure drainage. Concentrates water and increases downstream flooding.
Cross slope	Reduces number of inlets and manholes. Decreases earthwork. Fits better with natural topography. Traffic barrier on both sides.	Water from streets intersecting on high side must be picked up as it will overflow intersection rather than "run around corner." Maximum width of sheet flow. Hazardous if sheet flow from rain or snowmelt freezes. Flow capacity can be achieved in only one gutter.
Asymmetrical crown	No cross flow until crown is overtopped. Lessens hazard of icing. Fits better with natural topography. Traffic barrier on both sides.	Limited flow capacity on upper side. Allows less cross fall than section above. Rides "funny."
Drainage swales	Lowest cost where usable. Allows for infiltration of runoff in channels. No water confined on pavement, so freer movement of traffic during storms. Can be merged into the natural topography. Fewer underground storm drains. Slows down the runoff because of much lower velocities in the grass-lined channel and because considerable storage must be filled before overflow.	Not advisable where small lots require frequent driveway culverts. More of a maintenance problem on shoulders and channel. May require wider right-of-way to accommodate flat side slopes on drainage swales. Less adaptable to sidewalks, but compatible with off-street walk systems.

7.1 Street and Curb Cross Sections

There are typical street cross sections in common usage. The advantages and disadvantages of each from the drainage standpoint are summarized in Table 23-2.

The most commonly used cross section is a center crown sloping at a rate of one-quarter inch per foot toward a swale or curb and gutter, on each side of the street. Sidewalks, if present, can be placed against the curb or, as is more common, more desirable and necessary in wet or snowy areas, can be separated from the curb by a planting area.

On a side hill section, the street section can be designed with a straight crown or with a crown at the one-third point with a slope toward each curb. This section should not be used on collector streets where speeds are higher, because the ridge tends to make car control more difficult. The cross slope on roadway sections should not exceed 5% in any case.

A street cross section with drainage swales replacing curb and gutter, and often with no sidewalks, is currently being used for many residential streets. This approach is compatible with development concepts that utilize path and walkway systems and low traffic volume streets such as cul-de-sacs, loops and courts. Such practice is a return to urban practice, common to the early part of this century, which provided about 40 acre-feet (AF) of street side storage per square mile. Elimination of that storage and installation of curbs during the past 35 years has created significant urban flooding problems where none previously existed, by accelerating downstream runoff peaking.[30]

Since slip forming of curbs and of curbs and gutters often is generally used for construction economy, it is important that uniform curb and gutter sections be used as much as possible. The hydraulic capacities of the straight and battered curbs are about the same and they lend themselves to side-opening curb inlets. The rolled or mountable curb has a lower hydraulic capacity and requires a transition to a vertical face to accommodate a side-opening curb inlet; flexibility in accommodating field changes in driveway locations is a characteristic of rolled curbs that should be considered where their limited hydraulic capacity is not a significant issue.

7.2 Hydraulic Capacity

The hydraulic capacity of a street section to convey water can readily be calculated by the Manning Equation in the following form as developed by Izzard:

$$Q = 0.56\frac{Z}{n}d^{8/3}S^{1/2}$$

In this equation, the symbols are defined as follows:

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[30] Jones, D.E. February 1971. "Where is Urban Hydrology Practice Today"? *Journal of the Hydraulics Division*, ASCE Proceedings Paper 7917.

Q = discharge in cfs

Z = $1/S_x$ where S_x is the cross slope of the pavement

d = depth of water in feet at face of curb

S = longitudinal grade of street

n = Manning's roughness coefficient

Experiments have proved that this form of the equation is more accurate than would be obtainable by computing the hydraulic radius based on the wetted perimeter and the area of the cross section. The equation applies directly to a section having a straight cross slope.

The compound section, with the gutter having a Z-value of 12, is widely used for streets because the hydraulic capacity for a given spread of water on the pavement is substantially increased. (About 30% more for a spread of 10-feet and a 2-foot gutter width.) This is because the velocity at any point on the cross section is directly proportional to the two-thirds power of the depth, which is increased by steepening in the gutter cross slope.

Since more water is concentrated close to the curb, the compound section also increases the capacity of inlets to intercept flow. To facilitate computation of inlet capacity, the compound section can be converted to an equivalent straight cross slope having the same capacity as the compound section for a given depth "d" measured from the gutter flow line.

The equivalent straight-slope section can be computed by the following equation:

$$Z_3 = Z_1\left[1 + \left(Z_2/Z_1 - 1\right)\left(\frac{T-W}{T+W\left(Z_2/Z_{1-1}\right)}\right)^{8/3}\right]$$

in which

Z_3 = reciprocal of cross slope of equivalent section

Z_1 = reciprocal of cross slope of gutter

Z_2 = reciprocal of cross slope of pavement

W = width of gutter in feet (this often will be identical to the width of the depression of a curb opening inlet since it is impractical to extend the inlet depression appreciably beyond the gutter width)

T = top width of water surface

By using this value of Z_3 in the previous equation and the same value of "d" as for the compound section, the exact Z for the latter is found. The first equation given can be directly applied to a compound section by use of Nomographs.[31]

7.3 Estimating Runoff in Streets

The peak flow contributed to a gutter or swale is normally estimated by the Rational Method. Drainage areas are subdivided so that runoff contributed to each gutter or swale can be computed at the end of a block or at other points where an inlet or pick up point is required. Inlets are usually sized so that a portion of the flow is bypassed; the actual flow in the next reach of gutter includes this bypass flow. The flow reaching the second inlet is a portion of the flow contributed from its drainage sub-area plus the flow bypassing the first inlet. The low point or sump inlet catches the remaining flow from both directions and must be sized accordingly.

Locations and required capacities of inlets and swales are established by computing estimated flow rates, depth and velocity of flow, and spread across street. The design of sump inlet capacity may provide a degree of outflow control from the sump storage, safely obtained, through localized temporary ponding.

Under conditions prevailing during a major storm, the storm drain system will be surcharged and the rest of the flow will be carried on the lawns, the streets, etc. Inlet capacity in this case is indeterminate but is probably somewhat less than the inlet would handle under sump conditions because of debris blockage and surcharge back pressures. It is probably safe to assume that the flow on the street would be the difference between the total runoff and the capacity of the storm drain surcharged to the level of the gutter. Since debris blockage of inlets is most likely during extreme runoff events, emergency overflow routing and analyses for runoff extremes should assume that no more than 50% of pipe capacity is available under these circumstances.

7.4 Criteria for Spread of Water Across Street

The allowable spread of water across the street from the curb is limited by the criterion of maintaining two clear 10-foot moving lanes of traffic for collector streets during minor storms. One clear lane should be maintained on sub-collectors, and lanes and places may have a spread equal to one-half of their width. These criteria are fully justifiable in humid areas, but may be difficult to meet in arid areas where:

- Available slope is limited,
- Runoff may contain considerable suspended solids,
- Drainage essentially is by surface flow, and
- The public accepts the resulting inconvenience.

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[31] Federal Highway Administration. March 1969. *Drainage of Highway Pavements.* Hydraulic Engineering Circular No. 12, U.S. Department of Transportation ,Washington, DC.

When a steep cross slope is used (5% is suggested as a limit), a 10-foot spread would produce an excessive depth at the curb making it difficult to intercept the flow except with a very long curb opening inlet.

Consequently, with pavement cross sloped from 3% to 5%, the depth of flow at the curb should not exceed 6 inches.

7.5 Flow Across Intersections

The most critical situation exists where a street on a grade intersects with another street, especially a collector or sub-collector. Even when the flow on the grade is severely limited, great care must be taken to provide inlets that will intercept virtually all of the flow from a minor storm. Full interception may be impossible for a major storm.

A "T-intersection" requires special care because houses directly below a steep "T" stem are particularly subject to intersection overflow damage during major storms. Overflow of "T" intersections can be somewhat impeded by:

- Elevating the through-curb;
- Installing a higher-than-normal roadway crown on the through street; and,
- Using a straight-faced through-curb of 7½-inch or greater height. In severe situations, a decorative wall (low) placed behind the through-curb and landscaped will provide practical control, often more economically and assuredly than inlet construction.

Flow across collector streets should not be allowed during frequent storms. Controlled flow across sub-collectors is acceptable and there are no design limitations placed on flow across lanes and places. During major storms, permissible flow across intersections will be a function of the street's traffic-carrying importance and the availability of convenient alternative access.

7.6 Ponding at Low Points in Grade

Some ponding of water at low points in the grade is inevitable, even during a minor storm, and in some cases may be desirable. However, because of the driving hazard and splashing of water on pedestrians, ponding should be minimized. This is done best by intercepting most of the flow before it gets to the bottom of the grade. In picking up most of the flow prior to the sump locations, a large percentage of sediment will also be removed from the flow. This prevents sediment deposition in the gutter as velocity is checked on the decreasing grade. A curb-opening inlet at the low point in the grade is an efficient structure, but must be of ample size because this is the point where debris is most likely to accumulate. Effective performance during a major storm should be assured by considering:

- Damage from water overtopping curb and sidewalk

- Methods of minimizing damage
- Relative sizing of inlet and pipe
- Overflow mechanisms

7.7 Maximum Velocity in Gutters

Water flowing down steep grades can be dangerous. For example, water flowing at 10-feet per second (fps) can exert a force of 100-pounds against a flat, 1-foot-wide object placed across the flow. A 20-pound push at shoe level will sweep a grown man off his feet, so gutter flow can be very hazardous to a child. Aside from the safety hazard, such flows are difficult to intercept at inlets and can create difficulties, such as shooting across an intersecting street or overriding curbs and causing severe localized erosion and sometimes damage to downhill properties.

A recommended criterion is that the velocity in the deepest part of the gutter be limited to 10 fps. This velocity is readily computed by the Manning equation using the depth at .6-inches from the face of the curb as the hydraulic radius.[32] The mean velocity for the entire cross section is not a good measure. If the calculated velocity exceeds 10 fps, the allowable discharge in the gutter must be reduced until velocity is within the limit. The designer is then faced with the problem of where and how to reduce the runoff entering this gutter. Additional inlets could be installed upstream, but this is expensive. If possible, some way should be found to divert runoff to some path other than the steep street, preferably by a revised street layout. Future street resurfacing that will reduce capacity should be considered in the calculations.

8.0 NATURAL DRAINAGE

Natural drainage flow techniques serve very useful functions in the control and management of stormwater runoff. The primary function is to provide an opportunity for natural infiltration of stormwater to the groundwater supply system. Secondly, it helps to control the velocity of runoff flows, which is a necessary factor in the control of erosion and sedimentation. Thirdly, and perhaps most important, natural drainage techniques can extend the time of concentration of stormwater runoff, thereby contributing to the ultimate goal of maintaining the rate of runoff at or near the levels existing prior to development. As noted earlier, street swales can provide up to about 40 AF of runoff storage per square mile, thereby contributing significantly to runoff attenuation.

The achievement of the goals stated above is important. Urban and suburban areas have experienced a rise in the frequency of severe flooding and an increase in the amount of hazard and damage associated with floods at or near past levels. An important factor in this phenomenon is the rate at which stormwater runoff reaches the receiving streams from developments in the drainage basin. Natural stormwater runoff systems can help to control that rate and release the accumulated runoff over a

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[32] Searcy, J.K. May 1965. *Design of Roadside Drainage Channels.* Hydraulic Design Series No. 4, Bureau of Public Roads, Washington DC.

longer period, thus contributing to a reduction in the rate and volume at given points in the lower reaches of the receiving streams. This is especially important where older areas downstream of new developments either have inadequate storm drainage systems or a combined sanitary and storm sewer system. Specific reduction in the amount of runoff contributed from each new development in the upper and middle reaches of a drainage basin will reduce the cumulative runoff impact of development, thereby contributing to reduction of hazard and/or reduction in the need for costly supplemental systems in the downstream areas. Natural drainage systems must be properly maintained to assure their continued performance at the designed capacity. It must be noted that open channel flows in residential stormwater management systems are not completely "natural" systems, although they rely heavily on existing natural features and qualities of the development site. Virtually all development will increase the amount of stormwater that becomes runoff during and after an event because of the impervious surfaces used in development. Increases in runoff that change the dynamic equilibrium of natural areas used in the system mandate specific engineering solutions to conserve these natural systems and the pre-development characteristics of the area. The creation of swales, alteration of small channel capacity or direction, changing of ground cover and the lining of existing channels with other materials, natural or man-produced, are necessary in some parts of the system to achieve the desired objectives.

These alterations or improvements will have to be maintained if the total system is to function properly. The maintenance of swales and open channels in the interior areas of residential development is most critical during construction. Debris from this source, including plastic wrapping materials and other non-biodegradable substances, can diminish the capacity of the system or effect changes in the flow characteristics of the runoff that tend to cause erosion and sedimentation. After construction, internal maintenance problems within residential areas will not be significantly higher if property owners understand that their normal maintenance responsibilities are integrally related to regular public maintenance efforts. The primary problem areas for roadside swales or open channels are along major arterials bordering residential areas or areas where non-residential uses contribute large quantities of manmade debris.

An awareness of potential problems is the first step towards their prevention, but should not be an overriding cause for rejection of a system that would produce benefits that exceed potential maintenance disadvantages.

The characteristics, function and maintenance of open channel elements should be evaluated prior to the final design of the total network that forms a residential stormwater management system.

8.1 Overland Flow

In planning an open channel system, overland flow distances should be made as long as possible consistent with other constraints and requirements. Overland flow should be over and through turf or other flow retardants such as ground cover or forest litter.

This is one reason why natural woods should be preserved whenever possible. Slopes of overland flow areas should be as flat as possible, but maintaining natural topography and ground cover should take precedence over regrading to achieve flat slopes.

Overland sheet flow is a significant factor affecting the peak rate of runoff reaching the first collecting channel. Gently sloping turf areas shorter than 100 feet will probably not detain runoff significantly during intense rainfalls. The runoff rate from meaningful overland flow areas will be substantially less than the rate at which rain is falling. Minor surface depressions due to irregularities in grading add further to the storage potential. In addition, the retarded passage of water provides additional time for infiltration into the soil (if permeable), thus reducing the quantity of runoff. On paved surfaces, such as on parking areas, the storage in overland sheet flow is only about one-fifth as much as on turf, but is still a significant factor. On steeper slopes, whether pavement or turf, the velocity increases and storage decreases, so that less time is required for runoff rates to become equal to the rate at which the rain is falling. Most presently available mathematical models for computing runoff include the length and other characteristics of overland flow as essential inputs.

8.2 Hydraulics for Swales and Open Channels

When runoff reaches swales and open channels, principles similar to those for overland flow should be applied. If feasible, they should be wide and shallow with a rough surface and on as flat a grade as topography will permit. In this way, storage will further retard runoff and reduce the peak flow. Flat slopes may present the problem of marshy low areas after a storm. A minimum slope should be established, based on soil permeability and the capacity of the swale or channel.

The Manning Equation is almost universally used in calculating open channel flow. Numerous aids to facilitate computations are available including computer programs. A very useful publication for those not having access to a computer is *Design Charts for Open Channel Flow*. This publication includes consideration of flow in open channels and swales on mild and steep slopes. Velocity of flow in open channels and swales is dependent on rate of flow, slope, surface roughness, and cross section, and can be calculated using the Manning Equation. Surface roughness and consequent velocity will depend on the quality of maintenance. Well-mown grass will have a different surface roughness coefficient than tall grass.

8.3 Control of Erosion

In designing channels for erosion control, the velocity must be estimated and compared to the allowable velocity for the material on which the water is flowing. Table 23-3 indicates the allowable velocities for grass channels. It should be noted that the quantity of water that can be carried in well-established, dense turf swales without erosion is surprisingly large, even for steep slopes. For urban residential drainageways, flow velocities for erosion potential evaluations should be based upon the 10-year frequency runoff event, which generally is a practical break-point between initial cost and excessive maintenance cost.

TABLE 23-3

Permissible Velocities For Swales, Open Channels And Ditches With Uniform Stands Of Various Well-Maintained Grass Covers

Cover	Slope Range Percent	Permissible Velocity on:[a]	
		Erosion-Resistant Soils (fps)[b]	Easily Eroded Soils (fps)[c]
Bermudagrass	0-5	8	6
Buffalograss	0-5	7	5
Kentucky bluegrass	5-1	6	4
Smooth brome	Over 10	5	3
Grass mixture	0-5	5	4
	5-10	4	3
Lespedeza Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0-5	3.5	2.5
Common lespedeza[d] Sundangrass[d]	0-5	3.5	2.5

Original table from Soil Conservation Service. 1947. *Handbook of Channel Design for Soil and Water Conservation*, , U.S. Department of Agriculture Publication No. SCS-TP-61, revised June 1959.

[a] Velocities in excess of 5 fps to be used only where good cover and proper maintenance can be assured.

[b] Defined as CL, CH, OH, GM, GP, GC and GW (United Soil Classification System Designation).

[c] Defined as ML, SM, SC, MH and OL (United Soil Classification System Designation).

[d] Annuals, used on mild slopes or as temporary protection until permanent cover is established.

Source: National Academy of Sciences. 1969. *Slope Protection for Residential Developments*, Washington, DC.

However, when the allowable velocity for a turf channel is exceeded, there are a number of alternatives to consider as shown in Table 23-4. They include: lining channel with an impervious material; drop structures or other velocity and erosion control measures; gravel or rip-rap bottoms; and gabions (rock enclosed in wire baskets).

TABLE 23-4

Alternate Methods Of Controlling Erosion In Open Channels

Type of Control	Advantages	Disadvantages
1. P.C. or asphaltic concrete or soil cement paving.	Permanent if carefully constructed and maintained.	High cost. Speeds up runoff.
2. Drop structures across channel at intervals so that slope of channel between drops restricts allowable velocity for turf.[33]	Satisfactory if drops are well-designed and built. Does not speed up runoff.	Unsightly, if natural material is not used. May have maintenance problems.
3. Lining with crushed rock or gravel sized for requirements.[34]	Satisfactory if well-designed. Can harmonize with landscape. Can allow both infiltration into permeable soil and exfiltration of groundwater.	Can be costly if rock or gravel are not available locally.
4. Rip-rap.[35]	Permanent if carefully placed and maintained.	Can be costly. Can be unattractive.
5. Rock enclosed in galvanized wire baskets (gabions). This generally requires wide shallow channels so that drops will not be over-ridden and channels destroyed during unusual extreme flow events.	Permanent if carefully placed. Can allow infiltration on permeable soil. Aesthetically pleasing—becomes invisible. No limit to range of flow.	Can be costly.

The probable performance of the open channels and swales should be evaluated for major storm runoff with respect to the depth and spread of water and the erosion potential. Antecedent flow conditions resulting from previous storms are an important consideration. Open channels and swales may suffer damage during major storms, even if properly designed. The potential for incurring these infrequent

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[33] Searcy, J.K. May 1965. *Design of Roadside Drainage Channels.* Hydraulic Design Series No. 4. Bureau of Public Roads, Washington, DC.

[34] Anderson, A.G., A.S. Paintal, and J.T. Davenport. 1970. *Tentative Design Procedures for Rip-Rap Lined Channels.* Highway Research Board, National Academy of Sciences. National Cooperative Highway Research Program Report No. 108, Washington, DC.

[35] Searcy, J.K. *Use of Rip-Rap for Bank Protection.* Federal Highway Administration. Hydraulic Engineering Circular No. 11.

maintenance costs should be balanced against the initial cost of attempting to make them “flood proof” during the design process.

It is important that open channels be constructed in accordance with plans. When intermittent channels are sodded to the depth of the expected flow, they can immediately provide protection for minor storms. It is not practical to establish turf in a drainage channel by seeding and mulching unless jute mats, or similar protective materials, are placed over the seed bed.

8.4 Flow and Erosion in Natural Streams

Maintenance of streams in their natural condition is a desirable goal. A natural stream normally adjusts its cross section and slope so that they are in approximate equilibrium and flowing bank full at the average annual peak flow rate. For greater flows, the banks are overtopped and flow also occurs on the floodplain. If the floodplain is then constricted due to development, more flow will be concentrated in the channel, probably disturbing its equilibrium and resulting in more than normal erosion. Similarly, additional erosion may result if the peak discharge is increased, even if the floodplain is not constricted. Limiting the minor storm discharge from a residential or other development to pre-developed flow rates is a means of controlling the peak discharge. The two concepts, of storage and the use of natural open channel flow, assist in this objective.

9.0 UNDERGROUND PIPE SYSTEMS

9.1 Layout of the Storm Drain System

The layout of the storm drain system for residential areas should make maximum use of existing open channels and natural streams, before resorting to enclosure of runoff in underground pipes. Former practice tended toward enclosing small streams in conduits that was not only costly, but also concentrated the flow downstream and increased the peak rates of discharge. The preferred approach is to leave natural streams undisturbed and to limit peak runoff conditions. Erosion of the stream channel usually is not accelerated when this is accomplished. As recommended in this report, runoff will be collected in swales or open channels and curb and gutter sections and carried as far as practical before entering an underground pipe system. The underground pipe system consists of a series of inlets, pipes, and manholes. Output from computer simulation models of runoff or empirical techniques may be used in the detailed design of the system. Decisions made in the planning stages of the residential development, some of which are listed below, will have a great influence on the final form and cost of the underground pipe system.

- Planning of both minor and major components with appropriate design rainfall recurrence intervals.
- Limitation of peak runoff rates after development.
- Type and amount of storage.

- Use and incorporation of natural drainage such as overland flow, street swales, open channels and natural streams.
- Assurance that all contributory upstream areas will be similarly regulated or initially assessed and developed, so projected conditions may be relied upon.

9.2 Pipe Location and Alignment

The pipe system is usually in the street right-of-way, but portions may be in easements along lot lines when that route provides the best outlet path to a natural stream. One common location within the right-of-way is behind the curb. This method connects inlet boxes with the least amount of pipe and junctions.

There is no reasonable objective to the pipe being laid on horizontal and vertical curves conforming to the curvature of the street. The horizontal and vertical alignment of storm drain pipes must be coordinated with the location of all other utilities. In cases where joint trenching of electrical, telephone, CATV, gas and possibly water lines is allowed, it is much easier to coordinate the locations of utility lines during construction. Since storm drains are generally constructed early in a residential project, require gravity profiles, are located at shallow depths, and require trench backfill density control, the opportunity for joint trenching with other utilities rarely exists. As most localities now require electrical power and telephone lines to be underground, there no longer are possible conflicts with utility poles behind curbs.[36]

9.3 Culverts

With the emphasis on keeping surface runoff in street swales and open channels, culverts may be required for street crossings. Driveways crossing properly designed street swales should not automatically require culverts, as vehicles should be able to cross the swale. When required, culverts preferably should have flared end sections for good appearance and hydraulic characteristics.[37] Precast or pre-fabricated flared end sections for pipes are available.

Any culvert will cause an increase in water level in the upstream channel. This backwater can flood property and overflow into the streets. The frequency with which this is allowed to happen should depend on the amount of damage, including delays to traffic, caused by the backwater.

Where storage is required to control peak discharge, a road fill can provide a dam for temporary impounding of stormwater. Legal and physical requirements for such temporary ponding are constraints to be considered.

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[36] NAHB Research Foundation, Inc. 1974. “A Summary of a National Study and Survey of Existing Utility Design and Construction Practices for Residential Development.”, Rockville, Maryland.

[37] Federal Highway Administration. 1972. *Hydraulic Design of Improved Inlets for Culverts.* Hydraulic Engineering Circular No. 13. U.S. Department of Transportation , Washington, DC.

Culverts may also require energy dissipaters at the outlet if the streambed is erodable. These can range from simple placement of rip rap to elaborate concrete stilling basins. Similar structures may also be required on storm drain outlets.[38,39]

9.4 Materials

Storm drain pipes are most commonly plain or reinforced Precast concrete; however, other materials may be used. There has also been good experience using carefully controlled installations of un-reinforced cast-in-place concrete pipe, which may also provide initial construction cost savings. Corrugated metal pipe can be used for short runs, but its high resistance factor often requires a larger pipe size for equivalent flow capacity that weighs against its use for long lines, unless grades are very steep or unless the invert and sides are lined with asphalt to give a smooth interior. Where culvert hydraulics are governed by inlet control, concrete and corrugated metal pipes may be interchanged size-for-size.

Curb opening inlets are usually reinforced concrete, often with precast units for top slabs and other parts. Concrete block can also be used for walls if the inlet opening is short. On longer openings, the structural problem of carrying the top slab across the opening with no intermediate supports and very limited thickness of slab allowable, may require cantilevering the slab off the rear wall of the reinforced concrete inlet box.

Manholes can be brick masonry or reinforced concrete and are frequently built as precast concrete units. Fiberglass units are seeing limited use.

9.5 Hydraulics

The usual hydraulic practice is to select the pipe size for the accumulated runoff rate at each inlet or manhole assuming that the pipe is just barely flowing full. For the water to accelerate to the uniform flow velocity computed from the Manning Equation, water must back up into the inlet to a depth above the pipe equal to the outflow velocity head plus the inlet lateral entrance loss. Standard practice also requires that the hydraulic gradient, or pressure head line, be computed for the full length of the pipe system, usually starting at the downstream end. The pressure (head) losses at junctions must also be computed.[40] Guide vanes and other geometric improvements within the junction or manhole can be used to minimize pressure losses. The pressure losses become particularly significant for velocities in excess of 8 fps. The pressure line should be kept close to the crown of the pipe for the design discharge of the minor rainfall. Surcharging the pipe with the pressure line above the crown of the pipe but still remaining about 0.5-feet below ground level is acceptable.

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[38] Bureau of Reclamation. 1964. *Hydraulic Design of Silting Basins and Energy Dissipaters*. Engineering Monograph No. 25. Washington, DC.

[39] Simons, D.B., M.A. Stevens, and F.J. Watts. 1970. *Flood Protection at Culvert Outlets*. Colorado State University.

[40] Sangster, W.M., and others. 1958. "Pressure Change at Storm Drain Junctions." Engineering Series Bulletin No. 41. University of Missouri, Columbia, Missouri.

During a major rainfall, the underground system will be under pressure with the pressure line probably at the surface of the water in the gutter. Since the pipe slope is usually about the same as the street slope, the pipe will be carrying only the discharge for that slope, the remainder of the stormwater being on the surface of the street. When testing a tentative design for performance under major storm runoff conditions, it may be necessary to increase the size of one or more reaches of pipe to avoid incurring excessive damage costs resulting from overflowing the street. Other alternatives for disposing of the excess flow may be more economical.

The minimum pipe size required should be based on hydraulic considerations rather than arbitrary standards; it should be noted that if a less than 15-inch pipe is satisfactory, then the storm sewer is probably unnecessary, since a curbed 26-foot-wide roadway will handle more than three times the flow of a 15-inch pipe on the same grade. Consideration should be given to hydraulic capacity of a pipe given its size, slope and roughness characteristics, tendency to become clogged, self-scouring velocities and ability to clean the pipe and remove obstructions. The use of street swales to collect water, and criteria that allow more flow in gutters prior to initial pick-up, will mean that the initial minimum pipe size will generally be larger than under the old philosophy of "get it into pipes as quickly as possible."

The minimum allowable velocity to keep small particles of sediment moving in a pipe is about 2 fps. It is advisable to have a minimum pipe slope of about 0.1% in the approach to a junction box. While the thrust of the stormwater management program includes keeping sediment out of the system as much as possible, some sediment load may be unavoidable. With a heavy sediment load, coarser particles may settle out where there is a reduction in transport capacity because of decreasing grade.[41]

The use of an inlet box as a sediment trap is ill-advised. Experiments have shown that the turbulence created by the falling jet moves sediment beyond the box. In addition, catch basins are costly to clean out, and the pipe system usually has the capacity to carry away the sediment anyway. If trapping of sediment becomes necessary, it should be done by designing a sediment trap located at a point where the pipe capacity to transport sediment is reducing. The trap should be accessible for cleaning with mechanical equipment. Stormwater loaded with abrasive sediment particles can cause wear on a concrete pipe at high velocities, so the entrance of such sediments should be minimized. The duration of stormwater flow is relatively short and infrequent, so abrasion by particles that do get into the pipe may not be serious. There is little that can be done to slow down water on a steep grade, other than building a vertical drop structure to dissipate excess energy.

Difficulty with high velocity flow is more likely to occur when the energy gradient (the position of pressure head plus the velocity head) is allowed to rise well above the ground surface at a manhole where the pipe slope decreases abruptly. A hydraulic jump can occur that would check the velocity and result in conversion of velocity

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[41] Laursen, E.M. June 1956. *The Hydraulics of a Storm-Drain System for Sediment-Transporting Flow.* Bulletin No. 5. Iowa State Research Board.

head to pressure head sufficient to blow off the manhole cover or the whole top of the manhole. This is a particular problem where pipe alignments cause flow through inlet boxes, which are often shallow, as the intended inlets may instead function as outlets.

9.6 Manholes

The principal purpose of manholes is to provide access for cleaning and inspection. They usually should not be more than 500 feet apart on small pipes; spacing may be greater for larger pipes and unlimited for 66-inches and larger. Maintenance and safety requirements will dictate spacing on the big pipes. Inlet boxes should also have manhole openings to provide access. If storm drain pipes run through inlet boxes, they can be used for inspection and maintenance and can be counted when determining minimum spacing. This practice is only recommended where there is no likelihood of soil movements or external live loads on the inlets. Where pipes are routed through inlet boxes both pipes and boxes usually must be at least 1 foot below conventional depths to offset hydraulically adverse head recoveries, turbulence and entrance losses, to assure functioning of the inlets.

10.0 STORMWATER INLETS

Stormwater inlets are located at the transition between open surface flow and a closed conduit system. They are constructed as part of the street's curb and gutter system, located in street swales or used to drain open areas. The inlets should remove runoff from surfaces when the flows exceed the criteria for velocity, spread of water across streets, or flow across intersections. Inlets in street swales also remove flow when it exceeds swale capacity. Drainage of open areas is often picked up by an inlet in a depressed area.

In utilizing natural systems effectively, the employment of inlets should be delayed as long as possible because as soon as the runoff enters the pipe system, it is carried rapidly downstream.

10.1 Grate Inlets

Grate inlets consist of metal bars or a grid encased in a frame. When grate inlets are placed in the street, the bars are usually aligned with gutter flow for maximum hydraulic efficiency. This allows the intercepted water to fall between the bars. The bars of the grate may be placed at right angles to the curb for the safety of cyclists. When this is done, water hitting the bars may be projected upward, causing some of the flow to be deflected away from the inlet. The efficiency of both applications is improved by depressing the grate below the plane of the gutter within the transition area. This creates a sump condition and the small amount of ponding helps reduce approach velocity abating the tendency of the water to be deflected beyond the inlet.

Grate inlets are often placed in street swales and other overland flow areas, when it is necessary to intercept the flow due to velocity or the lack of a satisfactory route for continued flow. Grate inlets are also used to drain parking lots. The design of these inlets must account for any expected reduction in inlet interception capacity due to clogging by grass cuttings and water borne debris.

Periodic maintenance to assure the capacity of the inlet is necessary. Grate inlets in unpaved areas should be placed according to the most efficient design, as it is not likely that bicycle traffic will be a factor in this type of installation. Grate inlets on urban streets are not recommended.

10.2 Curb Opening Inlets

The capacity of an inlet to intercept water flowing down the street depends to a large degree on the distribution and velocity of water in the gutter cross section. On a continuous grade, an inlet will intercept only that flow within its hydraulic reach. In the case of the curb opening inlet, the width, length and depth of the depressed section of the gutter in front of the opening is very important. Steepening of the cross slope of the gutter enables gravity to begin turning the flow into the opening, but because of the inertia of the flow, considerable opening length is necessary.

The vertical height of the opening should be not greater than 6 inches in order to eliminate the possibility of a child being washed into the inlet basin. This places a restriction on hydraulic capacity for major storm conditions when most of the street may be flooded. The outlet pipe would probably be flowing full by that time and would limit the inflow anyway. The opening preferably should be clear for the entire length on continuous grades.

A sediment trap formed by lowering the floor of the inlet box below the elevation of the outlet pipe is unnecessary and undesirable since there is too much turbulence for effective trapping, and cleaning is costly.

10.3 Capacity of Curb Opening Inlets

The most authoritative data on capacity of curb opening inlets is contained in a circular published by the Federal Highway Administration (FHWA). That report includes charts for estimating the interception ratio Q_l/Q for inlets 5-, 10- and 15-feet long, and curb widths of 1, 2 and 3 feet when used on street sections with cross slopes ranging from 0.015 to 0.06 ft./ft. and roadway grades up to 4%. The charts, as published, are based on full scale tests by Colorado State University. Extension of the charts to steeper grades has been developed and is awaiting publication.

10.4 Curb Opening Inlets at Sump Locations

Curb opening inlets at a low point in the grade (sump) operate efficiently, since the water is trapped at the opening. Charts are available for estimating the capacity of inlets 5-, 10- and 15-feet long in terms of the depth of ponding over the inlet lip. There is also a procedure for determining when a given inlet will restrict flow to the point where backwater will occur in the approach gutters, thus increasing the width of ponding. For major storms, the restriction caused by the outflow pipe may govern.

10.5 Curb Opening Inlets—Deflector Type

The state-of-the-art literature contains references to the use of deflector vanes on the surface of the depressed gutter in front an inlet to force more water into the curb opening. The only experimental data available on deflector vanes were based on a steep crown slope $S_x = 0.055$ with a narrow range of water depth and with water spread only twice the width of the depression. These results are not applicable to flatter cross slopes, and are applicable least of all to large ratios of water spread to curb width. The Colorado experiments on which the charts in *FHWA's Drainage of Highway Pavements* were based demonstrated conclusively that the interception ratio is a function of water spread to curb width.

The use of deflector vanes is not recommended, since they complicate construction and in the past have been rendered inoperative when buried by street resurfacing.

11.0 OTHER DESIGN CONSIDERATIONS

11.1 Soil Erosion

Control of erosion *during residential construction* requires an examination of the entire site to pinpoint potential problem areas, such as steep slopes, highly erodable soils, soil areas that will be unprotected for long periods or during peak rainy seasons and natural drainageways. Steps should be taken to assure erosion control in these critical areas. After a heavy storm the effectiveness of erosion control measures should be evaluated. Periodic maintenance and cleaning of the facilities is also important.

Control of erosion *after construction* consists primarily of minimizing bottom and side scouring of natural drainageways. This can be accomplished by proper initial design that limits velocities and specifies correct drainageway linings and structures, and by proper routine maintenance and repair of the system.

Some of the basic concepts for controlling erosion during and after construction are:[42,43,44]

- **Earth Slopes**: erosion of cut or fill slopes is usually caused by water concentrations at the top of the slope flowing down an unprotected bank. Runoff should be diverted to safe outlets. Slopes should be protected from erosion by quick establishment of vegetative cover, benches, terraces, slope protection structures, mulches, or a combination of these practices as appropriate.

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[42] Becker, B.C., and T.H. Mills. 1973. *Guidelines for Erosion and Sediment Control: Planning and Implementation.* Maryland Department of Water Resources, Annapolis, Maryland.

[43] New Jersey State Soil Conservation Committee. 1972. *Standards for Soil Erosion and Sediment Control in New Jersey*. Trenton, New Jersey.

[44] Guy, H.P., and D.E. Jones, Jr. December 1972. "Urban Sedimentation—in Perspective." *Journal of the Hydraulics Division*, ASCE (Proceedings Paper 9420).

- **Waterways or Channels**: waterways should be designed to avoid serious erosion problems. Wide channels with flat side slopes lined with grass or other vegetation will usually be free from erosion. Where channel gradients are steep, linings or grade control structures may be required. Space limitations may make it necessary to use concrete or stone linings. Every effort should be made to preserve natural channels.
- **Structures for Erosion Control**: erosion may be controlled by the use of grade control structures, energy dissipaters, special culverts, and various types of pipe structures. Structures are expensive and should be used only after it has been determined that recommended vegetation, rock revetment or other measures will not provide adequate erosion control.
- **Existing Vegetation**: good stands of existing vegetation adequate to control erosion should be preserved wherever possible.
- **Soil Treatment, Seeding and Mulching**: the ability of the soil to sustain vegetation intended for erosion control must be ascertained. The admixture of fine textured topsoil may be warranted to assure success of more attractive, lower maintenance vegetation. Liming and fertilization should be done according to recommendations based upon soil test information. After the soil has been prepared, the correct seed mixture, sod, ground cover and mulch should be applied.
- **Outfall Design**: the outfall pipe should be designed and located so as to minimize erosion; especially if the outfall is to an overland flow area with a steep slope or is elevated above the base flow of the receiving streams. An energy dissipater may be necessary.

11.2 Siltation and Sediment Control

Proper control of soil erosion during and after construction is the most important element of siltation and sediment control. However, it is physically and economically impractical to entirely eliminate soil erosion. Secondly, erosion is a natural function and is required in certain portions of the drainage system in order to provide future stream capacity. Therefore, provisions should be made to trap eroded material at specified points. Some measures that can be implemented are:[45]

- Temporary ponds that store runoff and allow suspended solids to settle out can be used during construction and may be retained as part of the permanent storage system after construction.
- Protection of inlets to the underground pipe system can be accomplished during construction by placing hay bales around the structure.
- Egress points from construction sites should be controlled, so that sediment is not carried off-site by construction traffic.

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[45] U.S. Environmental Protection Administration. August 1972. *Guidelines for Erosion and Sediment Control Planning and Implementation.*

11.3 Stormwater Runoff Pollution

As stormwater runoff flows over surfaces, it picks up pollutants and carries them downstream. The magnitude of the pollution load has been the subject of recent investigations by the U.S. Environmental Protection Agency (USEPA) and others.[46] It is generally conceded that the magnitude is sufficient to warrant serious examination of alternate methods of controlling and treating stormwater runoff pollution.

Documentation of pollution loads in streams before and after construction is a necessary first step prior to embarking on extensive runoff treatment programs. Measurements might include suspended solids (TSS), biochemical oxygen demand (BOD), dissolved oxygen (DO) and the concentrations of toxic materials, bacteria and nutrients. Under ideal conditions the measurement of pollution should be coordinated with simultaneous collection of data on rainfall and runoff. Pollution loads are the result of:

- Soil erosion and dissolving of minerals in the natural ground cover;
- Overland flow that picks up fertilizer, animal droppings, and organic material, and
- Flow on parking lots, roofs and streets that carries petroleum products, trash, dust fall and debris from cars and trucks into the drainage system.

Three basic methods of treatment can be used:

- The first controls pollution loads at their source. For example, proper erosion control and sediment control will reduce the suspended solids levels. Also, periodic street cleaning will reduce pollution loads.
- Stormwater runoff can be treated at the source. Temporary storage of runoff to allow suspended solids to settle out is one example. Diversion of runoff to land treatment areas for spraying or controlled overland flow is another. The fact that most runoff pollution results from the "first flush" of runoff should be considered when planning source treatment facilities.
- Treatment of stormwater runoff at a centralized plant downstream is the third alternative. This is probably the most costly method because of the vast volume of water requiring treatment. Consideration may be given to storage facilities enabling stormwater to be released to treatment plants at a gradual rate after the runoff peak has passed.
- Treatment of runoff to reduce pollution loads is probably unnecessary for most low density residential development, but the availability of pertinent information is limited. It seems obvious that the cost of such treatment will be high, so it follows that treatment should not be considered unless there is

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[46] Sartor, J.D., and G.B. Boyd. 1972. *Water Pollution Aspects of Street Surface Contaminants.* Environmental Protection Agency, Washington, DC.

documentation of the need and a demonstration that the benefits from treatment will be consistent with its costs.

11.4 Maintenance

Adequate provision for short- and long-term maintenance of the residential stormwater system is an important design consideration. Maintenance and replacement needs and costs should be part of the economic analyses.

Maintenance requirements for the type of system suggested in this document may be different from those for a fully enclosed pipe system. Mowing, trash and sediment removal, replacement of sod and repair of eroded areas will become parts of the maintenance program. Conversely, less pipe inspections, repair and cleaning will be required.

When planning an on-site storage system, determination must be made about long-term ownership and/or maintenance and operation of the facility. The choices will generally be between public and private organizations and the final decision will be dependent on local conditions.

11.5 Encroachment into Potential Floodplain Areas

No consideration of residential storm drainage is complete without recognition of potential flood hazard exposure. These potential hazards are obvious along major rivers and streams, but can also occur along tributaries and drainageways and in headwater areas as will be discussed later.

Management of major drainage basins and rivers is usually controlled by other than local governments; however, local government should be concerned with controls for development in all other areas subject to flooding.

The Federal Insurance Administration (FIA) is currently involved in a two-staged effort to delineate and map some floodplains and their potential flood elevations for many communities as part of an effort to provide insurance for structures located in areas of potential hazard. The first stage provides a gross delineation of the floodplain, to permit subsidized insurance for properties already located within a hazard zone and a basis for planning future land uses in such areas. The second phase provides more accurate and detailed mapping of the floodplain to guide future land use practices and determine insurance rates for structures permitted to locate in a designated hazard zone. It will take years to complete this mapping effort, and it is uncertain whether it will be possible to identify and map all potential flood hazard areas. Thus, it is necessary for all involved in the process of land development to recognize and consider any potential flood hazards associated with the land being developed.

In a subsequent discussion of basic legal aspects of urban drainage and flood control, potential liabilities associated with land use and development are outlined. The direction of liability may be shifting, by placing increasing culpability on parties involved in the approval of land uses in a hazardous area or in the creation of hazards. The availability from FIA and others of hazard information and delineated potential hazards makes it easier to assess liability for improper location or land use.

As a result of FIA actions and federal regulations relating to flood hazards, local governments have been stimulated to provide new regulations for their jurisdictions. These regulations cover areas delineated by FIA and may cover tributary and headwater areas not currently mapped. The new regulations do not prohibit all development in potential flood hazard areas but rather require that development be consistent with wise floodplain management to qualify for insurance or loans from federally regulated or insured institutions.

Within flood hazard areas, there are often locations where occupancy is fully justifiable under certain conditions. The Minimum Property Standards (MPS) of the Department of Housing and Urban Development recognize this and permit floodplain occupancy provided the finished first floor of a dwelling is no lower than the 100-year frequency flood elevation and the building site grades adjacent to the dwelling are no lower than the 50-year frequency flood level.

In practice, the use of the 50-year frequency flood elevation as a limit to acceptable encroachment upon a floodplain may justifiably be varied. Where the difference in elevation between the 50-year and 100-year frequency flood levels is less than 8 inches, it often will be fully justifiable to occupy sites having elevations below the 50-year frequency flood elevation, sometimes as low as the 20-year frequency flood elevation, if dwellings will have their floors at or above the 100-year flood elevation and the dwellings will have no basements. This justification reflects the typical low-flow velocity of floodwaters in such locations and the minimal effect of buildings upon flooding depths upstream from such locations. Where the difference in elevation between the 50-year and 100-year frequency flood levels is more than 18 inches, residential encroachment upon the floodplain to the 50-year frequency flood elevation generally is unwise since potential flooding depths and fast-flow velocities could cause severe damage to properties. Where properties may potentially be flooded for more than a few hours, more stringent encroachment limits than the 50-year frequency flood elevation may be appropriate because the social displacements and flooding damages will usually be greater than from short-term flooding of similar depth. Of course, technical justification of a variation will not necessarily be acceptable to regulatory authorities.

The economic benefits that may be derived from occupancy of portions of a designated special flood hazard area, or other areas subject to flooding, may be appreciably greater than probable future flooding losses. There is a statutory provision for federal acceptance of alternative land use and control measures applicable to such locations, but the burden of developing alternative measures and demonstrating their assured fulfillment of federal loss mitigation objectives is upon the local community. The community must demonstrate conclusively that the economic and social benefits derived from occupancy will be greater than, and of overriding importance relative to, the potential flooding losses. For locally-suggested alternative measures to be given credence, proponents of the alternative measures should clearly explore whether immediate costs and adverse effects upon property values, tax revenues and public facilities (including utilities), attributable to adoption of generally promulgated land use and control measures, are greater than the probable future flooding losses discounted to present value. Locally suggested alternative measures should also demonstrate how their implementation would mitigate future flooding losses, and to what degree. Regardless of whether flooding is caused by inadequate drainage or by streamflow, the derivation and support of locally suggested alternatives optimally requires interaction and close cooperation among all local interests.

Selection of appropriate types of dwelling units for construction in areas exposed to flooding may significantly reduce potential flooding losses, by comparison with potential losses using other dwelling types. Where the occupancy will be in a community that is generally exposed to flooding, or in a substantially developed area in which it is not feasible or reasonable to forego building upon vacant sites, selection of the merchantable building type having the lowest flooding loss expectance is appropriate. In such situations, the probable market response to a change in building type should be carefully assessed during decision-making. Based upon the Federal Insurance Administrations' Depth-Damage curves,[47] the relative flooding loss characteristics of common types of residential buildings are as follows:

Building Type	**Ration of Damage***
Two-story dwelling without basement	1.0
Split level dwelling	1.15
Two-story dwelling with basement	1.2
One-story dwelling without basement	1.5
One-story dwelling with basement	1.65
Mobile home	2.3

* Base level is two-story dwelling without basement.

As shown, substitution of a building type with a lesser loss potential could reduce potential average annual flooding losses to levels comparable to loss expectancies outside of other designated special flood hazard areas. Where such an alternative relationship can be demonstrated, there would seem to be a persuasive argument for acceptance.

Additional loss mitigation can be effected by reducing land occupancy densities, thereby reducing the total value of exposed property per acre, and by flood proofing, to reduce the losses expected from any given level of potential flooding. Flood

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[47] Federal Insurance Administration. 1970. *Flood Hazard Factors, Depth-Damage Curves, Elevation-Frequency Curves, Standard Rate Tables.* Washington, DC.

proofing[48,49,50] involves a series of construction modifications either to exclude water from entering buildings or to reduce the potential for water damage if water does enter buildings. In some situations, flood proofing can reduce potential flood losses by as much as 60% by minor initial increases in construction costs.

The potential for flood hazards and the issues of encroachment into floodplain areas discussed above tend to presume a continuation of past practices in runoff management in upstream tributary areas. A major objective of this publication is to encourage new approaches to stormwater runoff management that would attenuate peak runoff, thus, reducing frequent flood hazard threats in the middle and lower reaches of a drainage basin. Thus, application of the objectives, principles, and design considerations set forth in this publication may in themselves provide further justification for cautious variations of limitations on land use in flood hazard areas.

11.6 Potential Flood Hazards in Tributary and Headwater Areas

The prior discussion has been concerned with flooding associated with overflow of channels, streams and rivers into adjacent, generally identifiable, flood hazard areas. There is considerable potential for water damage associated with unwise siting and drainage practices that are often overlooked and are often a source of residential flooding in tributary and headwater areas. These hazards generally are foreseeable, are usually the result of poor application of good design practices, and can be avoided without significant increase in development or construction costs.

Streets, highways and railroad crossing drainageways or streams are commonly elevated, on embankments, with culverts or small bridges passing beneath them to accommodate runoff flows. When the runoff flow is too great to pass through the culvert or bridge, or when the culvert or bridge is blocked by debris, the embankment will act as a dam causing runoff to accumulate upstream and possibly overflow the embankment. The depths of potential flows over roadway embankments are variable and should be computed, but most commonly are from 1 to 2 feet. The potential for residential flooding upstream from drainageway-crossing embankments can be eliminated if dwelling floors and openings into dwellings are higher than potential runoff overflow elevations at embankments. Failure to recognize such conditions is a widespread source of residential flooding. A proper application of the "blue green" concept discussed elsewhere should eliminate this hazard.

One common, but easily overlooked, source of residential flooding occurs where runoff from small areas will naturally follow a lot line swale between dwellings.

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[48] Jones, D.E. 1975. "Basis for Flood Plain Occupancy Decisions." *Urban Runoff Quantity and Quality.* American Society of Civil Engineers, New York, NY.

[49] Jones, D.E. and J.W. Davis. August 8, 1966. "Appendix F Site Preparation and Flood Proofing of Buildings, A Special Study by the Federal Housing Administration." *Insurance and Other Programs for Financial Assistance to Flood Victims.* A Report from the Secretary of the Department of Housing and Urban Development, to the President, as required by the Southwest Hurricane Disaster Relief Act of 1965. (Public Law 89-339) Washington, DC.

[50] Sheaffer, J.R. April 1967. *Introduction to Flood Proofing.* Center for Urban Studies, The University of Chicago, Chicago, IL.

Even though a drainage area may be very small, the quantity of foreseeable runoff will be frequency-related. The worst foreseeable flows should be anticipated during design. Appropriate designs for such locations will consider both the size of the swales and the elevations at which buildings are sited. There is always a potential for water-related damage, from stormwater runoff or groundwater, to structures improperly sited or improperly graded. Thus, detention ponding on individual sites, as suggested elsewhere in this publication, may be impractical or unwise because of such local problems as impermeable soils, expansive soils or seasonally high groundwater. Under such conditions, positive drainage of individual building sites may be essential.

Some relationships between basement construction and flood hazard exposure should be emphasized. Typical basement construction is incompatible with on-site detention ponding where site soils are more than slightly permeable and where detention ponding might contribute to the rise of groundwater to building footing elevations. The most commonly observed cause of residential basement flooding is entry or penetration of stormwater runoff due to the failure to drain runoff quickly and positively away from buildings. Where soils are essentially impermeable, protective slopes around a dwelling can be used to assure quick and positive drainage of runoff away from the dwellings either to off-site locations or to ponding storage areas on-site with controlled outfall.

Again, it should be emphasized that the flood or water damage hazards described here would be the result of improper site-specific application of recommended design approaches suggested earlier in this publication. Proper application of on-site detention, and proper use of swales and other engineering techniques, should avoid creation of residential flood hazards.

12.0 LEGAL IMPLICATIONS

Homebuilders and developers are familiar with zoning, subdivision regulations and building codes. Stormwater is another control that the public sector has placed on the use and development of land or that has arisen through liability imposed by courts when the acts of one landowner have adversely affected the property of another. Stormwater, like stormwater engineering, can be divided into two areas—floods and drainage—even though they obviously belong to the same system of surface water runoff.

12.1 Regulating the Floodplain

The floodplain is usually defined as that area bordering a watercourse that would be inundated by a flood of a certain magnitude. The magnitude used in establishing the federal flood hazard areas is the "100-year flood," that is, a flood that has a statistical 1% chance of occurring or being exceeded in any one year. Often this floodplain is further subdivided into a "floodway" and a peripheral area.

Billions of dollars have been spent on flood protection works. In spite of this, nationwide flood losses have continued to escalate. The response to this dilemma has been a change in philosophy in dealing with flooding. Instead of attempting to keep rivers away from people by damming and channelizing them, the trend is towards keeping people away from rivers by preventing further unwise encroachment onto the floodplains. This is not to say that no development should occur at all, but rather that development must be consistent with good floodplain management. The greatest impetus has come from the federal government through its National Flood Insurance Program, the Flood Disaster Protection Act of 1973 (PL 93-234, amending 24 U.S.C. Ch. 50).

Briefly, it works in the following manner. First, special flood hazard areas are identified and designated on maps by the federal government. If the community has become a "participating community" by adopting adequate land use measures and other controls for its floodplains, those buildings that already exist in the flood hazard areas are eligible for heavily-subsidized flood insurance. Flood insurance for new construction, however, will not be subsidized; instead, premiums will reflect the actual flooding risks to the property. The crux of the program is that flood insurance is *required* before the vast majority of lending institutions in the United States can make, increase, extend or renew any loan secured by improved real estate located or to be located in one of these special flood hazard areas. For the developer, this means he must investigate whether or not the property he proposes to develop is or is likely to be in a federally-designated special flood hazard area. If it is, and the community has failed to become a "participating community" by adopting acceptable land use controls, residential financing will probably be unavailable. If the land is within a "participating community," the developer must investigate what controls the community has placed on the land and what the flood insurance costs would be. A federal rate map identifies applicable insurance rates. Occupancy and insurance costs can be mitigated by taking certain precautions (such as raising the elevation of the building), but may still make construction in that location less feasible. Therefore, not only how the building is constructed, but also whether the building should be built at that site at all, is an initial consideration affected by the National Flood Insurance Program. Another site, outside of the hazard area, may be financially more advantageous for development.

Additionally, even if the federal government has not designated an area, both state and local laws should be consulted. Some states and communities have adopted floodplain regulations and maps on their own initiative, or areas in addition to the federally-designated areas might be locally controlled, such as on smaller tributaries of the main stem. Local land use controls may be in the form of building codes, subdivision regulations, or specific floodplain regulations. Since the floodway is supposed to be adequate for the safe passage of the floodwaters through the community, building restrictions within it are severe. In the peripheral area, sometimes called the low-hazard zone or flood storage area, development is usually permitted within certain less restrictive design parameters and precautions. Since the federal requirements are minimum, local floodplain controls can be more restrictive. It behooves a developer to find out what they are. Some regulations declare that a building that is not in compliance with the floodplain regulations is a public nuisance that can be enjoined or even abated. In addition, where such development is the proximate cause of injury to the person or property of another, and the non-compliance could constitute negligence, the owner or developer might be liable for damages in a tort action.

12.2 Drainage Law

While floodplain regulation is of fairly recent vintage, drainage law dates back to ancient times. Here we are looking at the respective rights and duties of the "upper" landowner versus the rights and duties of an adjoining "lower" landowner. The "upper" land lies at a higher elevation, and water drains down onto the "lower" land. This relationship is based on the lands in their natural, unaltered state.

There are basically two doctrines that have been adopted by various state courts: the "common enemy rule" and the "civil law rule." Under the "common enemy rule" the lower landowner may take any measures necessary to keep water off his land, even to the point of turning the water back onto the upper land. The upper owner can similarly protect his property from the "enemy" by diverting water around his property causing greater quantities at higher velocities to flow onto his neighbor's land. In its pure form, it would be a might-makes-right situation.

Therefore, courts have modified the rule to require that such acts be reasonable *vis-à-vis* each other. The "civil law rule" states that the upper landowner has an easement over the lower land for the natural drainage off of his land. The key word is "natural," meaning the same quantity and velocity as drained from the upper land in its undeveloped state. It was felt that, in its pure form, the law would substantially restrict development of the upper land, so again courts have modified it to accommodate reasonable use of the upper property. Finally, both of these doctrines, which are based on the property-law concepts of dominant versus servient lands, have been rejected by some courts. These courts focus on "reasonable use" alone, based on tort-oriented law. While these modifications tend towards the same results, the practical questions of predictability and proof requirements remain substantially different.

The developer will want to protect himself from possible exposure to a potential liability suit for damages, or from time-consuming and costly injunction action. Under any of the doctrines mentioned above, his best protection is to develop in such a manner as to keep the runoff as close as possible to runoff conditions in the natural state—in quality, velocity, and location. If he has obtained the hydrologic, soils, and other data recommended for good engineering design, and has developed his project accordingly, the same facts will protect him from liability because he can prove that he has not materially changed the natural drainage conditions and has acted in a reasonable, non-negligent manner.

Some communities have established special assessment districts or storm drainage fees for the purpose of constructing drainage improvements. The developer should also investigate how these might affect the property. The basis for the fee may be the difference between the amount of runoff that was generated from that property in its developed condition. Here again, if the same amount of runoff has been maintained, by on-site ponding or other techniques, the fee may be negligible. If, on the other hand, the natural permeability has been reduced by extensive paving, he may be committing the property to be subjected to high drainage fees; or the fee may reflect the cost of flood control works that are necessary to remove the property from a flood hazard zone. This may affect not only how he constructs, but whether he constructs there at all.

12.3 Conclusion

From a legal point of view, as from an engineering point of view, the developer must accept the fact that every piece of property involves stormwater runoff in either a major or minor way and as both a contributor and recipient. It is imperative, before purchase or development, to get the physical facts and to investigate the local, state and federal laws that could affect the property. The stormwater aspects of the property may be one of the controlling factors on how to develop or even whether to develop that site at all. However, after having done his homework, and developing the property in a responsible and reasonable manner, the developer can rest assured that he has good protection from liability.

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Chapter 24

COMPUTERIZED CITYWIDE CONTROL OF URBAN STORMWATER (NOTE: SUMMARY PROVIDED, NOT ENTIRE DOCUMENT)

February 1976
ASCE Urban Water Resources Research Program
Technical Memorandum No. 29
New York, New York

1.0 SUMMARY OF REPORT

Increasing political and economic pressures are causing today's urban water manager to place a greater emphasis on cost-effectiveness of urban services and more efficient spending of public funds. Accordingly, he or she has a great interest in searching for innovative solutions. It has been conclusively demonstrated that storm and combined sewer discharges are significant contributors to the total pollution reaching our receiving waters and the price tag to clean up these discharges has been estimated to be in the 150 to 170 billion dollar range. Since neither this country as a whole, nor individual cities can afford expenditures of this magnitude for the problem, better ways to manage existing systems and affordable new systems must be found.

Automatic computer control has been applied with success in industry for more than 15 years. It is only recently, however, that a few U.S. cities have implemented limited scale computer systems for controlling combined sewer overflows from portions of their urban complexes, with a number of other cities in various stages of planning for such systems. *Much of the control technology developed for combined sewer systems is expected to be useful in storm sewer system applications.*

If better urban stormwater management by computer control can reduce some of the expected capital and operational expenditures, a substantial market can be projected for computer control facilities. Assuming an approximate $10 billion annual urban water construction market, a possible total $100 million annual computer control market does not appear to be unreasonable.

The application of computer control to combined sewer and stormwater systems has challenges for three specific groups:

1) *Public works managers and engineers:* To carry out planning, design, and operational studies leading to the practical implementation of computer control for urban stormwater management.
2) *The control industry:* To develop the low cost, reliable, and efficient control devices necessary for providing automatic control of combined sewer and stormwater systems.
3) *The academic community:* To provide the fundamental research leading to practical techniques and procedures necessary for the ultimate success of the above efforts.

Computer control must be recognized as an innovation. The urban water manager must learn to approach the application of such innovations with a *healthy* degree of caution and skepticism. It should be made clear, however, that too much caution can be just as harmful as the other extreme.

For combined sewer systems, separation of dry and wet weather flows had been advanced as a possible solution to the pollution problem, though it was obvious that this *solution*, besides being extremely costly and disruptive, essentially ignored the possibly high pollution loadings in stormwater alone. It is clear that many cities with separated sewers need to take a serious look at controlling stormwater pollution.

Looking at other abatement measures, it is clear that finding an *optimal* stormwater management strategy may ultimately involve finding the proper tradeoff between the volume and distribution of in-system storage and the level and capacity of treatment, along with decisions regarding expansion of conduit capacity. The modes of in-system storage for stormwater or combined flow that are particularly suited to an existing highly urbanized environment are ambient or online storage and auxiliary or offline storage.

It is extremely important that detention storage be carefully planned on a scale covering the entire urban area. It has been shown that providing uncontrolled storage in a piecemeal and unplanned manner could actually result in more serious flooding than would otherwise occur.

A combined sewer system, as a particular kind of urban stormwater system, can be regarded as a working process. The inputs are storm rainfall and sewage waste flows. The process consists of collection, transport, treatment, and disposal operations. The output consists of the disposal component in the form of treatment plant effluents, overflows, and unaccommodated floodwaters. A properly operated system will maximize throughput, minimize overflows, and thereby minimize flooding and pollution.

Once a decision is reached for controlled storage, it makes sense to consider automatic control by digital computer, especially in light of the tremendous advances made in recent years in industrial computer control. The availability of attractive computer hardware, however, does not by itself guarantee success in automating a system. The cost of such hardware may be the least of the costs involved.

The normal computer control project should proceed cautiously through phases, beginning with the simple to the more complex. The four basic levels of computer control can be listed as follows:

1) Data logging and processing
2) Conventional remote supervisory control
3) Automation of parts of systems and computer assisted control
4) Closed loop automatic computer control

Proceeding through the above stages will allow the step-by-step development of data collection and reporting systems, local control capabilities (such as pumping stations and detention tanks), testing and calibration of necessary rainfall and runoff prediction models, and development of appropriate control logic to ensure the effectiveness of automation. In a project of this type, an excellent *management* capability is developed, regardless of when total computer control is eventually achieved.

Correct operation of the remotely controlled valves, pumps, gates, and various sensors is validated using the remote supervisory control level, along with verification of the communications network between the central location and the remote processors. Ideally, the control system should be operated at this level for only a few months (three to six months), though this time will vary considerably with the particular application.

Implementation of the next level of control, namely computer assisted operator control, would extend the previous level by implementation of the control optimization algorithms in the computers, but communicating the results of these algorithms to the control personnel. The control personnel could then override these decisions or could indicate that the proposed control settings are to be used. This level would be the most profitable in detecting logical flaws in the control system design and optimization procedures as implemented in the computer software. Experiments with totally automatic control for portions of the city could profitably be carried out at this level. Extensive testing and evaluation of the system would be performed in this level of control and would require several months of operation, perhaps as long as two years.

The third step in the evolutionary process is the implementation of the fourth level of control described above, namely, closed loop automatic control. As a result of modifications made to the control algorithms during the second stage of the evolutionary process, the transition from Stage 2 to Stage 3 should be relatively simple and straightforward. Even here, however, it is necessary to provide the capability for central control personnel to monitor the status of the system and override any decisions made that are obviously incorrect.

The proposed three stage evolutionary approach provides progressively greater automatic control with sufficient manual control and override to assure that major design flaws will not result in a catastrophe if something should go wrong. Each step in the process is built upon the previous step in which major portions of the control system have been thoroughly tested, thereby resulting in an increased confidence level that that portion of the system is operating correctly.

In addition to the importance of an automated real-time data acquisition system, the effectiveness of any automatic control system is directly related to the ability of mathematical models to represent adequately relevant aspects of the system. Such models may be used in computer studies leading up to real-time control (*offline* models), or actually programmed into the computer control system (*online* models).

Mathematical models are required for simulating the behavior of the system so that the control computer can find the best operational strategies based on these models.

There are at least five basic categories of models needed for implementing automatic control. These include:

1) Rainfall forecasting models;
2) Rainfall-runoff models;
3) Direct runoff forecasting models (as an alternative to 1.);
4) Models for routing flows through sewers and storage devices; and
5) Water quality prediction models.

In addition, there exist comprehensive models that incorporate together some of the more advanced forms of the models in categories 1, 4, and 5. Such comprehensive models tend to be computationally time consuming and unwieldy for real-time use, and are best employed for design purposes and offline operational studies. Comprehensive models that employ simplified versions of the basic models in categories 2, 4, and 5 may, however, have potential for real-time application. The challenge is to develop models that are sufficiently accurate, and yet feasible for operating within the computational limitations of real-time control systems.

There are many possible objectives for real-time control. Due to the unavailability of accurate water quality prediction models, we must probably settle for: 1) minimizing total volume of overflows reaching receiving waters; or 2) minimizing the maximum rate of overflow discharge. Through proper application of these objectives, it may be possible to satisfy indirectly more realistic objectives, such as minimizing pollution impacts on receiving waters.

The constraints on real-time control include:

1) Maximum available capacities of treatment plants, transport works, and storage devices;
2) Operating under dynamic physical laws governing transport of storm flows;
3) Limitations in the computer control system;
4) Limited *time* in real-time to render a control decision; and
5) Human error and equipment malfunction.

Control policies may be reactive (or set point control), adaptive, or stochastic-adaptive. The latter two are based on the inclusion of a *learning model* so that succeeding storm flow forecasts can be improved as real-time information on the storm in progress is incorporated. Stochastic adaptive control policies go one step further and incorporate the *risk* of a poor forecast into the making of control decisions. Results of recent research conducted at Colorado State University indicate that they are better than adaptive-only policies. Reactive policies appear to be adequate if the goal is simply to minimize total volume of overflows during a storm event, though they may allow high overflow rates. The rate of overflow discharge may be a more critical factor in assessing pollution impacts on receiving waters than total volume of overflow. Stochastic adaptive policies perform better than reactive policies in minimizing overflow rates.

The direct application of a stochastic adaptive control approach to the overall citywide control problem, in real-time, is probably infeasible. Attempts can be made, however, to decompose this large problem into smaller, more easily solvable control problems coordinated through a hierarchical control structure. This can be accomplished by dividing a city sewer system into tributary areas referred to as *sub-basins*.

The goal is to be able to develop optimal control strategies for each of the sub-basins independently, and recombine them together in such a way as to achieve an overall optimum strategy. Thus, the computationally infeasible large-scale problem is decomposed into several smaller problems that are more readily solved.

The advantages of this kind of decomposition or decentralization can be listed as follows:

1) Greater conceptual grasp of the complex problem is attainable through attempting to define its component parts and their interconnections.
2) Since the control strategies for each of the sub-basins are developed independently, special structure of the sub-basin problems can be exploited.
3) Application of optimization techniques to the control problem is facilitated since it is generally more efficient to replace a large optimization problem with several smaller problems that are solved a number of times. As stated previously, direct solution (without decomposition) of the large-scale stochastic problem would be computationally infeasible.

Since the sub-basins are effectively independent, except for their common input to the interceptor sewer, the following hierarchical control strategy can, therefore, be envisioned. An overall *master controller* is set up whose task it is to establish maximum levels of discharge from each of the sub-basins in such a way that interceptor capacity and treatment plant capacity are fully utilized. Information on these proposed maximum discharge levels is then passed to the lower level sub-basin controllers. The problem associated with each sub-basin controller is to attempt to find the best way of manipulating the control devices under its jurisdiction such that expected overflows from its tributary areas will be minimized, subject to limitations on discharge into the interceptor that were imposed by the master controller.

This kind of hierarchical strategy is particularly beneficial for online use. One of the most important benefits is the possibility of the online sub-basin control policy computations being carried out simultaneously. For example, a minicomputer could be allocated to one or more sub-basins, with one or more additional minicomputers serving to carry out the master control functions. If one large computer was used for all the computations, the sub-basin control policies would have to be found in a serial fashion, thereby greatly increasing the total computer time. The idea is to use several small central processors, rather than one large one, in order to carry out the computation more efficiently.

In addition to computational efficiency, there are definite advantages in system reliability with this approach. If one large computer was utilized, the whole system would break down if this computer failed. Duplexing the computer, or providing a back-up computer, would be expensive. With systems of minicomputers, on the other hand, if one minicomputer breaks down only a small portion of the system fails. The minicomputer engaged in master control is critical, of course, and would probably have to be duplexed. This, however, would be much less expensive than duplexing a large computer.

Although this kind of hierarchical computer control is new to the field of urban water management, it is currently being implemented in certain industrial process control applications. The terms usually associated with it in the industrial setting are *distributed* or *multi-computer processing.*

Many difficulties are implicit in the application of computer control to urban stormwater systems. In particular:

1) Poor or non-existent instrumentation;
2) Hostile environment for electronic equipment;
3) Large and complex collection networks;
4) Shortage of experienced personnel familiar with control and computer practices; and
5) Lack of prototype installations.

Certain process characteristics are common to most industrial processes where computer control has been successful. These characteristics include: 1) large size; 2) frequent disturbances; 3) high complexity; 4) adequate instrumentation; 5) continuing future importance; 6) favorable labor and management attitudes; and 7) availability of technical people. To make a computer control economical for supervisory control applications, a plant should usually represent a product value of $4 million or more. Whether these conditions can be made to exist for combined sewer systems must still be proven.

The minicomputer market will continue to grow at an ever-increasing rate in the future years. Technological advances and volume production will cause further reduction in prices and/or increased capability. With decreased size and cost, it will be necessary to define a new category of computers: the *mini minicomputer* or the *microcomputer* selling for $100 to $2,000 will be utilized increasingly. The continual decrease in cost of these computers and their use as components in dedicated equipment will cause the minicomputer to look less like a portion of the general purpose computer industry and more like a portion of the electronic equipment market. Eventually, these computers will find their way into the consumer product market.

Minicomputers are most frequently used for single purpose computer control functions. Whenever a large integrated control task is present, several minicomputers operating in an interconnected or *hierarchical* environment can be used. These multi-computer or *distributed processing* systems are becoming more common in practice because of their increased reliability. If one computer fails, the entire control system is not disabled. On the other hand, these distributed processing systems do present some inherent problems such as intercomputer communication and intercomputer task assignment not found in stand-alone applications.

Hardware is available from several minicomputer manufacturers that is capable of supporting the control system requirements. In addition to the memory and speed requirements defined in the next part of this section, the sub-basin processors must have extensive input-output (I/O) capability for process control-type devices. This includes analog to digital converters, digital to analog converters, and digital I/O for such functions as controlling relays and sensing contact closures. Minicomputers have supported this type of I/O equipment for many years and in general their capabilities are superior to large machines.

A second major requirement on the sub-basin processors is the availability of communications interfaces to permit communications with the central facility. This is another area in which minicomputers have long been applied, having been used as communication concentrators, front-end communications processors to large-scale equipment, and stand-alone communications control processors. A wide variety of software is also available from the manufacturers of minicomputers to support communications applications as well as supporting process control type applications.

The central processor in which the master control program is executed requires not only the capability to communicate with the remote sub-basin processors, but also requires more extensive secondary storage in the form of magnetic tape and magnetic disk storage for the large historical files of data that are utilized. Minicomputers have a wide variety of disk cartridge, disk pack, fixed head disk, and drum equipment available, as well as a variety of speeds of magnetic tape drives and line printers. In general, from a hardware viewpoint, several minicomputers are capable of supporting the control system requirements.

Based on the definitions of functions to be performed by the master processor, the general specifications for the hardware and software that are necessary for the master processor can be established. The functions to be performed are:

1) Provide communications with sub-basin processors and read the current state variables.
2) Accept weather inputs via the console and edit these inputs.
3) Perform storm and inflow predictions.
4) Match predicted storm with historical storm data and modify offline solution, when possible.

5) Solve master control problem and transmit master control settings to sub-basin processors.
6) Maintain historical files of storm data.
7) Provide operator communications.

The major computational task performed by the sub-basin processor would be local optimization for the sub-basin. The local sub-basin optimization would be performed within the constraints established by the control settings that would have been determined by the master processor. When the optimum solution had been obtained through dialogue between master processor and the sub-basin processors, the programs that actually control the valves and pumps would be utilized to implement the solution.

The following list summarizes the functions that would be performed by the sub-basin processor:

1) Communications with master processor.
2) Inputing sensor data.
3) Outputing control data to pumps and valves.
4) Performing sub-basin optimization calculations.

In the organization of a control project, a number of specialties must be involved. Once approach to organization might be as follows:

Project Management - Local government involved

Vendor/Contractor - An appropriate computer hardware/ software firm

Modeling Contractor - A specialized consultant to produce the system simulation and/or control models needed

The skills required, regardless of their source, must include management, urban water modeling, and hardware/software interfacing. In addition, *a reputable firm must guarantee the performance of the equipment used.*

The experience gained in industrial process control applications should tend to ease the difficulty of *pioneering* the computer control application for combined sewer systems. Although in many ways the state-of-the-art in combined sewer system control resembles the situation over 15 years ago in certain process industries, the operating climate for computer control has substantially improved since then. From the ultimate success of these early applications and the routine nature of current process industry applications, the computer control engineer faced with implementing a combined sewer control system can take considerable consolation. In addition, the state-of-the-art in computer and control technology has substantially advanced since those early beginnings, with the result that the tasks are made easier and the mistakes less costly.

Chapter 25

URBAN HYDROLOGICAL MODELING AND CATCHMENT RESEARCH: INTERNATIONAL SUMMARY

November 1977
ASCE Urban Water Resources Research Program
Technical Memorandum No. IHP-13
New York, New York

INTRODUCTION AND SUMMARY

Abstract

Twelve national reports for the International Hydrological Program (IHP) have been compiled on the state-of-the-art in urban catchment research and hydrological modeling, with particular attention given to underground conduit systems. Summarized in this report are their principal commonalities, together with particularly noteworthy observations or advances reported for individual countries. The reference reports are for: the United States;[1] Australia;[2] Canada;[3] the United Kingdom;[4] the Union of Soviet Socialist Republics;[5] the Federal Republic of Germany;[6] Sweden;[7] France;[8] Norway;[9] the Netherlands;[10] Poland;[11] and India.[12]

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[1] McPherson, M.B. November 1975. Urban Hydrological Modeling and Catchment Research in the United States ASCE UWRR Program Technical Memorandum No. IHP-1, ASCE. New York, New York, 20 pp (NTIS No. PB 260 685).

[2] Aitken, A.P. May 1976. Urban Hydrological Modeling and Catchment Research in Australia. ASCE UWRR Program Technical Memorandum No. IHP-2, ASCE. New York, New York, 20 pp (NTIS No. PB 260 686).

[3] Marsalek, J. June 1976. Urban Hydrological Modeling and Catchment Research in Canada. ASCE UWRR Program Technical Memorandum No. IHP-3, ASCE. New York, New York, 20 pp (NTIS No. PB 262 068).

[4] Lowing, M.J. July 1976. Urban Hydrological Modeling and Catchment Research in the United Kingdom. ASCE UWRR Program Technical Memorandum No. IHP-4, ASCE. New York, New York, 23 pp (NTIS No. PB 262 069).

[5] Kuprianov, V.V. August, 1976. Methods for Calculating Maximum Flood Discharges for Natural Watercourses and Urban Areas. ASCE UWRR Program Technical Memorandum No. IHP-5, ASCE. New York, New York, 15 pp (NTIS No. PB 262 070).

[6] Massing, H. September 1976. Urban Hydrological Studies and Mathematical Modeling in the Federal Republic of Germany. ASCE UWRR Program Technical Memorandum No. IHP-6, ASCE. New York, New York, 50 pp (NTIS No. PB 267 364).

[7] Lindh, G. October 1976. Urban Hydrological Modeling and Catchment Research in Sweden. ASCE UWRR Program Technical Memorandum No. IHP-7, ASCE. New York, New York, 33 pp (NTIS No. PB 267 523).

[8] Desbordes, M., and D. Normand. November 1976. Urban Hydrological Modeling and Catchment Research in France. ASCE UWRR Program Technical Memorandum No. IHP-8, ASCE. New York, New York, 58 pp (NTIS No. PB 267 524).

[9] Selthun, N.R. December 1976. Urban Hydrological Modeling and Catchment Research in Norway. ASCE UWRR Program Technical Memorandum No. IHP-9, ASCE. New York, New York, 18 pp (NTIS No. PB 267 365).

[10] Zuidema, F.C. January 1977. Urban Hydrological Modeling and Catchment Research in the Netherlands. ASCE UWRR Program Technical Memorandum No. IHP-10, ASCE. New York, New York, 18 pp (NTIS No. PB 267 587).

[11] Blaszczyk, P. February 1977. Urban Runoff Research in Poland. ASCE UWRR Program Technical Memorandum No. IHP-11, ASCE. New York, New York, 6 pp (NTIS No. PB 267 871).

Most of the substantial progress that has been made has taken place in the 1970s. However, to be consistent with the economic and social importance of urban drainage, much more needs to be done everywhere. Three fundamental research objectives are identified with regard to urban runoff: determination of the hydrological effects of urbanization; development of measures that would offset the adverse effects and enhance the assets of urban runoff; and resolution of improved tools of analysis for the planning, design and operation of urban drainage systems. However, insufficient knowledge has been acquired on crucial characteristics, such as on the processes involved in the accumulation, distribution and transport of pollutants. This summary report is addressed principally to practitioners in urban hydrology.

Background

This endeavor originated from activities and aspirations of the Unesco Subgroup on the Hydrological Effects of Urbanization of the International Hydrological Decade (IHD) that concluded in 1974. Members of the subgroup represented the Federal Republic of Germany, France, Japan, Netherlands, Sweden, United Kingdom, Union of Soviet Socialist Republics and the United States. The first writer served as U.S. representative and chairman of the subgroup and the second writer served as the Netherlands representative. The subgroup final report[13] is divided as follows: Part I, International Summary, Part II, Case Studies of Hydrological Effects of Urbanization in Selected Countries, and Part III, Illustrative Special Topic Studies. The final draft of Part I was resolved by representatives of over 30 nations who participated in an International Workshop at Warsaw, Poland, November 1973.[14] A very important output from the workshop was the identification of ten crucial international research projects proposed for inclusion in the Unesco component of the IHP that commenced in 1975 as the successor to the IHD.

Because of pessimism over the prospects of Unesco being able to support all of the proposed research, the American Society of Civil Engineers (ASCE) took early supportive action by applying for a U.S. National Science Foundation (USNSF) grant to assist in two of the ten recommended projects:

R1. **Catchment Studies Report**. Prepare a state-of-the-art report on research executed in urban catchment areas, which would include instrumentation, data acquired, analysis performed and applications.

R3. **Mathematical Models Report**. Prepare a state-of-the-art report on mathematical models applied to urban catchment areas and

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[12] Ramaseshan, S., and P.B.S. Sarma. May 1977. Urban Hydrological Modeling and Catchment Research in India. ASCE UWRR Program Technical Memorandum No. IHP-12, ASCE. New York, New York, 21 pp (NTIS No. PB 271 300).

[13] Unesco. 1974. *Hydrological Effects of Urbanization.* Studies and Reports in Hydrology 18, The Unesco Press, Paris, 280 pp.

[14] ASCE. January 1974. Report, International Workshop on the Hydrological Effects of Urbanization, Warsaw, 1973, to the U.S. NSF, New York, New York, 61 pp.

> dealing with, for instance, rainfall-runoff relationships and water balances, both with respect to water quantity and quality.

At its first session in April 1975, the Intergovernmental Council for the IHP adopted IHP Project 7, Effects of Urbanization on the Hydrological Regime and on Quality of Water, which includes the two subjects in question. This action made possible close coordination and cooperation with Unesco by the ASCE Program on the state-of-the-art reports.

Of particular significance was the very strong emphasis of the Warsaw Workshop and the subgroup on the urgency of addressing all such reports to users of research findings. That is, an accentuation of user participation and user orientation of the IHP urban products clearly indicated that facilitation of the translation of research findings into implementation practice should be a central goal.

In most countries, economic growth, population growth, non-agricultural water use and pollution are intertwined. Water in its many manifestations plays a vital role in the extremely complex processes of urbanization, and thus affects a nation's health and growth. The most significant conclusion reached by the IHD/Unesco subgroup is that most urban hydrological problems and effects are similar in technologically and economically advanced countries. Further, many problems confronting the developing nations have, at one time or another, already been encountered by many developed nations. This strongly suggests that great benefits would result from exchanging of information and increased international cooperation in research and development.

Procedure

Two of the five subprojects under IHP Project 7 adopted by the Intergovernmental Council for the IHP in April 1975, were: Subproject 7.1, Research on Urban Hydrology; and Subproject 7.2, Development of Mathematical Models Applied to Urban Areas, Considering Both Water Quality and Quantity Aspects. In the meanwhile, ASCE had already received a grant from the USNSF for assembling national reports, but with an emphasis on urban drainage systems. This emphasis was in keeping with the findings of the IHD subgroup and the Warsaw Workshop, but had a narrower scope than elected later by the Intergovernmental Council for the IHP. Thus, the national reports, and their summary that follows, are contributions to Subprojects 7.1 and 7.2 for the period 1975-1977, and subsequent IHP activities on these subprojects are expected to focus on subjects other than drainage. An initial emphasis on urban drainage systems was consistent with the fact that this subject had repeatedly been singled out as having the largest gaps in knowledge in urban hydrology.

The report for the United States served as the prototype. Its initial draft was completed in June 1975, and distributed by Unesco. In a letter to all National Committees for the IHP in September 1975, the Director of the Unesco Division of Water Sciences requested the contribution of other national reports, of which 11 were received. The letter noted that Messrs. M.B. McPherson and F.C. Zuidema had been designated by the Bureau of the Intergovernmental Council for the IHP as judges, respectively, for Subprojects 7.1 and 7.2.

Processing, duplication and distribution of the national reports as Technical Memoranda of the ASCE Urban Water Resources Research Program was supported entirely by U.S. National Science Foundation Grant Number ENG74-20326, awarded to ASCE by the Division of Engineering, Civil and Environmental Technology Program, for 1974-1977. Technical liaison representative for the USNSF was Dr. Arthur A. Ezra. The texts of the 12 reports total almost four hundred pages.

Preparation, duplication and distribution of the present Technical Memorandum, the last in a special IHP series, was supported entirely by USNSF Grant Number INT77-15021, awarded to ASCE by the Division of International Programs, for the second half of 1977. Technical liaison representative for the USNSF is Mr. Charles T. Owens. Any opinions, findings, and conclusions or recommendations expressed in this report are those of the writers, and do not necessarily reflect the views of the USNSF.

We are greatly indebted to the authors of the other national reports and others for their constructive review of this report.

Final typing and distribution of all 12 national reports, and this summary report, was done by Mrs. Richard P. Symmes, Marblehead, Massachusetts. A number of the illustrations were drafted or redrafted by Mr. William G. Minty.

Summary

For assembly of most of the national reports, a large number of organizations and individuals were contacted in each nation and reference was made to a multitude of technical reports and publications. Because of limitations in time and funds, none of the national reports can be regarded as exhaustive. Further, because they are status reports it must be accepted that there is a good prospect that new advances have been made or that additional information has accumulated since they were written. Hopefully, there will be opportunities later in the IHP, to update the national reports. This is not to suggest that the national reports are not comprehensive. As a group, they are the most informative and revealing collection of information on the subject that has ever been made available. Moreover, while 12 nations are a small sample of the world's total, the many aspects of urban drainage that they share in common very strongly suggest that they are much more representative than their small number would indicate.

Also to be considered is the important background developed during the IHD on the hydrological effects of urbanization. The 12 national reports have built upon that foundation. Reference to the broader considerations is exemplified in an introductory portion of the report for the Federal Republic of Germany:

> *The social process of urbanization, as far as it affects hydrology, manifests itself mainly through the following symptoms: high local population density; intensified industrial and trade activity; changes in ground surface; heavily increasing water demand; increased energy consumption; physical and chemical changes in the quality of surface and subsurface waters; air pollution; great need for protection from natural phenomena (e.g., floods); need for the disposal of increasing quantities of waste of all kinds; and recreational requirements to be met by surface waters.*
>
> *Urbanization affects all phases of the water cycle in settlement areas, with far-reaching changes taking place in precipitation, evaporation, evapotranspiration, infiltration, and runoff. Urban hydrology is that part of the comprehensive field of hydrology that deals with effects and phenomena in human settlements.*

The purpose of the national reports is stated particularly well in the report for the Federal Republic of Germany:

> *As expressly requested by the IHD subgroup and emphasized by the Warsaw Workshop, this report is intended for practitioners in urban hydrology, to give them a concentrated survey of available research results. It is hoped that this will facilitate translation of scientific results into actual practice and communicate unsolved practical questions and problems to researchers.*

On the whole, the status of urban hydrological research in the Netherlands appears to be quite typical of an economically developed nation:

> *In spite of a continuous growth in the population of the Netherlands (13 million in 1970 with 15.6 million expected in 2000) and an increasing population density (384 inhabitants per square km in 1970, 403 in 1975, 434 in 1985, and 463 in 2000), development in urban water resources research has been rather tardy. However, there is an enormous diversity among urban hydrological problems, which are solved adequately. For example, while only a few urban catchment studies are going on, mathematical models are used for different goals and at different levels: models for rainfall-runoff, water quality, comprehensive urban water and water resources management.*

Summary Section 2—Urban Water Resources

Illustrated are the interrelations, interdependencies and interconnections among the elements of the water resources of a metropolis. Such complexity calls for a systems approach in analysis. A hierarchical view of the water management system is disaggregated into the physical urban water resources system, from which the stormwater and wastewater portion is further segregated. Recognized is that although drainage, regarded as a subsystem, is currently the least connected and least dependent of the urban water subsystems, expected more extensive practice of the management of runoff for beneficial uses will increase its dependence. As a result, management of stormwater, such as for pollution abatement or water reuse, will inevitably become more complex.

This section commences with a discussion of water balances. Very few water inventories have been made for entire, separate, urban areas, yet this is the initial step for embarking on a total system management approach.

A major objective of this section is to develop a broad base or background to place drainage in its proper context as a part of the total resource. In this way, an opportunity is provided to define the scope of coverage of the remainder of the report.

Summary Section 3—Urban Runoff

Distinctions are drawn between the convenience provided by underground systems of drainage and the threats to public safety that are offset by flood mitigation measures in natural floodplains. While modern research on urban runoff can be traced as far back as a quarter of a century, present knowledge is heavily associated with advances made over only about the past ten years. A primary impetus was the surge of a general public interest in the abatement of water pollution in recent years.

Identified in the national reports are three fundamental research objectives with regard to urban runoff: determination of the hydrological effects of urbanization; development of measures that would offset the adverse effects and enhance the assets of urban runoff; and resolution of improved tools of analysis for the planning, design and operation of urban drainage systems. These objectives are interrelated and rely for their attainment on the effectiveness and reliability of tools of analysis. In turn, the acceptability and credibility of the tools hinge upon the extent and representativeness of the field data available for their validation. In sum, solutions to problems, tools of analysis and their supporting field data are part of a complicated circle of interdependencies.

None of the national reports even implied the existence of an adequate national database for validating tools of analysis. An ideal minimum national data acquisition program is postulated for the purpose of indicating worldwide deficiencies in a generic sense. Concluded is that despite substantial advances in recent years, attainment of the three goals described above appears to be a long way off in most, if not all, of the dozen nations for which status reports have been prepared, despite substantial advances in recent years. However, because recognition of the importance of runoff aspects of urban hydrology is quite recent, we optimistically expect that more comprehensive national programs will eventually evolve.

Despite the social importance of urban water resources and the substantial investments in associated facilities, we find national, territorial and local governments involved in a tangle of interests. This fragmentation is one of the reasons why urban water resources research around the world commonly has suffered from inadequate attention and support and from discontinuous and erratic efforts. Urban drainage has suffered the most research neglect and technical knowledge of it is the most primitive among the various aspects of urban water resources. On the premise that the manner in which various nations have attempted to overcome the enigma of fractionalized responsibility can be instructive, we have summarized such activities as have appeared in the national reports. Central coordination efforts have been spearheaded by groups ranging between national interagency coordinating groups to voluntary professional groups.

All the national reports but one are for nations that are regarded as economically developed. The sole report from an economically developing nation was for India. Because of the very great importance in the IHP of assisting economically developing nations by transferring information to them on the positive and negative experiences and progress of more economically developed nations, selected excerpts from the Indian report form the closure of this section. The different perspective is both entirely lucid and pointedly direct.

Summary Section 4—Urban Catchment Research

Because little would be accomplished by reciting all the urban catchment research detailed in the dozen national reports, we have used examples from among the reports to illustrate the major considerations in such research. Because the national program in Norway appears to be one of the most integrated and comprehensive in meeting nationwide needs, we have summarized the field research there as an example. We take advantage of this example by pointing out possible shortcomings, only because they are essentially shared everywhere else and we have found no instance of an example that meets minimum ideal criteria. The United States is the other example, because it is a much larger nation and, thus, has engaged in more extensive research activities. This latter example gives us an opportunity to remind our readers of the well known axiom that a collection of data has no inherent value, only an unknown potential one, until it has been subjected to thorough analysis and used in some beneficial way. This example also gives us an opportunity to mention a point raised in several reports, that many research stations were installed before the more modern tools of analysis were developed, and, hence, more recent data needs could not be anticipated, and as a result the data from these older stations is often of limited value.

A general dissatisfaction has been expressed on available instrumentation, particularly for in-sewer flow measurement and for water quality sampling. Recommended in the report from France is the conduct of an international symposium for the comparison of experiences.

One of the most challenging aspects of data collection is the synchronization of readings. Automatic data assembly-reduction installations, which overcome the synchronization difficulty, have been described in four of the national reports. Telemetry to a data receiving and processing center are described in two of the reports. We reiterate the truism that data has no inherent value until it is analyzed, and it cannot be analyzed until it has been reduced to hyetographs, hydrographs and displays of water quality parameter concentrations versus time.

Special studies are cited in various reports: on roadway drainage, groundwater hydrology and receiving waters. We note that conjunctive analysis of flow and water quality of urban catchments and receiving waters, while highly desirable, is seldom performed, despite a widespread concern over reducing stormwater quantities and pollutants entering receiving waters. Interest in the incorporation of retarding or detention storage in urban drainage is noted. Lastly, mention is made of research on the snowmelt fraction of urban runoff.

Summary Section 5—Urban Hydrological Modeling

Discussion centers on the simulation of performance of urban underground conduit drainage systems. Receiving water modeling is mentioned only where it is conjunctively or conjointly involved with drainage modeling.

This section opens with reasons for using simulation, among which is primarily the eventual abandonment of established wholly empirical methods. Models are characterized in terms of their applications, and the scarcity of suitable data for the regional validation of such tools of analysis is necessarily emphasized. Comparisons of various models are mostly inconclusive and, hence, are controversial. As an alternative to an innocent entrapment among the scores of models cited in the national reports, we confine our attention to the most comprehensive and flexible tools. These contain capability for routing of rainfall excess over the ground surface to street inlets, routing of runoff (and wastewater in combined systems) through the underground transport conduits, and routing through relevant reaches of receiving waters. Also, conveyance of both water quantities and pollutants are simulated. With greater comprehensiveness comes increased complexity, and three models described in some detail in the reports with capabilities just described are cited as examples that have enjoyed international application for design and planning purposes. Also cited is a special design model that is in even wider use.

Advantages in the use of models are drawn from the reports and a recent reference. For example, noted in the report for France is that the introduction of mathematical models made it possible to reduce substantially the degree of empiricism inherent in traditional methods of analysis. Several national reports emphasize the continually escalating costs for new or modified drainage systems, and spiraling investment requirements are particularly evident where urban runoff pollution abatement is an emerging objective. The keen interest in pollution simulation is quite recent in most countries. That a group of models rather than a single model is needed for important projects is well argued in the report for Canada.

Among model limitations, the one that particularly stands out is the primitive state of knowledge on the underlying processes in the accumulation and transport of pollutants. A more subtle liability is the fact that just about all catchment observations have been at a single point, obviating the possibility of validating the spatial performance of models and forcing a reliance on total catchment response as a measure of capability. The report for the United Kingdom observes that pragmatic practitioners are more impressed with tools that can be applied expeditiously than in an assurance that an alternative has a more scientific foundation. Further, the predictive precision of even the most widely used models may not be much greater than for simpler empirical approaches under certain circumstances.

Control of flooding and water pollution must be based on probabilities of occurrence because of the random nature of precipitation. Little research has been conducted on the temporal and spatial characteristics of storms that is applicable to urban drainage systems. Practically all of the national reports cited studies in which a variety of attempts have been made to synthesize suitable storm rainfall for planning and design applications of various tools of analysis. It is concluded that rainfall, the input to runoff models, may become the last element of outright empiricism to be placed on a more scientific footing.

A recurrent opinion expressed in the reports is that advances in modeling capability have surpassed the availability of suitable field data for their validation. Particularly feared is indiscriminant use of complex models without recourse to local field data for their calibration, a practice that subverts the purpose of such tools and earns them an undeserved reputation for impracticality.

This report closes with some important conclusions and recommendations from the report for India. In particular, a need is expressed for identifying tools of analysis suited to urban areas in economically developing nations, and for determining the type of data needed for the use of such tools in design applications.

Summary Section 6—References

The primary entries are the 12 national reports. These are cited for attribution rather than the original references credited in the reports, to avoid duplication and for the sake of brevity. Unesco is publishing the 12 national reports and this summary in an articulated form.

Most of the other entries are references that have become available since the relevant national reports were written.

Several entries carry an NTIS identification number. For these, copies of the reference can be obtained for a cost-recovery charge from the National Technical Information Service, U.S. Department of Commerce, Springfield, Virginia 22151.

Concluding Remarks and Recommendations

Unesco has published the first five of the special IHP series of ASCE Program technical memoranda as Volume 1 of Research in Urban Hydrology.[15] The other seven national reports and this international summary are expected to be published in succeeding issues. We are also pleased to note that two of the technical memoranda on which this summary report is based have been reprinted, for national distribution

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[15] Unesco. 1977. *Research on Urban Hydrology*. Volume 1, Technical Papers in Hydrology 15, Imprimerie Beugnet, Paris, 185 pp.

in Canada[16] and in the Netherlands.[17] Other indications of heightened interest are: the recent publication in the Union of Soviet Socialist Republics of a book on hydrological aspects of urbanization[18] by the author of the technical memorandum for that nation; and announcement of an international conference on Urban Storm Drainage to be held in Southampton, United Kingdom, 11-14 April 1978. In many respects, the numerous papers to be presented at the latter conference will represent an updating of aspects of several of the national reports, and extension of related information to include experiences of some other nations.

Related activities completed under IHP Project 7 have included: a Workshop on the Socio-Economic Aspects of Urban Hydrology held at Lund, Sweden, 15-19 November 1976;[19] a Symposium on Effects of Urbanization and Industrialization on the Hydrological Regime and on Water Quality held at Amsterdam, Netherlands, 3-7 October 1977;[20] and a Workshop on Impact of Urbanization and Industrialization on Regional and National Water Planning and Management held at Zandvoort, Netherlands, 10-14 October 1977.[21]

We are of the opinion that the work on urban hydrology under the IHD and the 1975-1977 phase of the IHP has adequately stipulated priority research needs. On the whole, it remains for individual nations to satisfy these needs, now that they have been internationally documented and recognized. Therefore, we urgently recommend that the next phase of IHP Project 7 should emphasize applications, with Unesco simultaneously continuing to play a vital role in the international exchange of information on research, but with this role extended to include applications.

Development of urban runoff tools of analysis has gone beyond the field measurement base that supports their validity. There is a need everywhere for more field observations from representative and experimental sewered catchments to improve the reliability of tools of analysis used in planning, design and operations. This is particularly true with respect to water quality considerations. The urgent need for a better understanding of the underlying, fundamental processes involved in the accumulation and transport of pollutants in urban catchments was emphasized at the concluding plenary session of the 1977 Amsterdam Symposium and was recognized among the recommendations of the 1977 Zandvoort Workshop participants. These expressions of concern reflect the great interest in most nations in recent years on pollution analysis, for the definition of sources of pollutants and their environmental impacts, and resolution of means for effective pollution abatement, all of which

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[16] Marsalek, J. 1976. *Urban Hydrological Modeling and Catchment Research in Canada.* Technical Bulletin No. 98, Canada Centre for Inland Waters, Burlington, Ontario, Canada, 52 pp.

[17] Zuidema, F.C. 1977. *Urban Hydrological Modeling and Catchment Research in the Netherlands.* Report IHP-77-01, Netherlands National Committee for the IHP, The Hague, 42 pp.

[18] Kuprianov, V.V. 1977. *Hydrological Aspects of Urbanization* (in Russian with foreword and conclusion in English), Hydrometeorology Press, Leningrad, U.S.S.R, 181 pp.

[19] Lindh, G. 1978 (in press). *Socio-Economic Aspects of Urban Hydrology.* The Unesco Press, Paris.

[20] International Association of Hydrological Sciences. October 1977. Effects of Urbanization and Industrialization on the Hydrological Regime and on Water Quality. *Proceedings.* Amsterdam Symposium, IAHS-AISH Publication No. 123.

[21] Zuidema, F.C. 1978 (in preparation). *Impact of Urbanization and Industrialization on Water Resources Planning and Management.* The Unesco Press, Paris.

require more reliable tools for their analysis than are presently available. Moreover, more reliable tools for water quality analysis are needed to help resolve the controversy over the merits of separate storm sewer systems *vis-à-vis* combined sewer systems. Still another reason is the growing interest in the potential use of stormwater as a supplemental source of water supply.

Among the recommendations of the Zandvoort Workshop was that Unesco should give serious consideration to the establishment of demonstration training projects on water resources management problems in urban areas situated in different climatic regions, and that these projects should have as one of their long-term objectives the preparation of manuals dealing with local problems of water resources planning and engineering design. In support of this overall objective, and echoing a viewpoint expressed in several of the national reports summarized here, we recommend that an international workshop be convened by Unesco on the collection, analysis and use of urban stormwater data, including an exchange of experiences with field instrumentation devices.

In summary, we envision the need for several important forms of Unesco activity over the next few years. To reiterate, these include: the establishment and promotion of demonstration training-projects centering on applications; the promotion of the acquisition of new knowledge, particularly with regard to basic processes underlying urban runoff pollution; the promotion of advances in technological capability via manuals of practice for various climatic regions; and facilitation of international exchange of information on both research results and applications. While the IHP may be the most suitable vehicle for continuing and initiating such activities, we earnestly hope that the IHP together with other branches of Unesco will accord still more attention to urban hydrology issues than in the past because of the great socio-economic importance of urban areas, their dependence on water resources for their survival, and the numerous water-related amenities affecting the quality of life in human settlements. As noted by a leading participant at the Amsterdam Symposium, urban hydrology appears to be a particularly suitable vehicle for interdisciplinary interaction. The effectiveness of Unesco/IHD and Unesco/IHP in stimulating the international and internal-national interest and activity that has been demonstrated thus far holds at least an equal promise for future projects and programs on urban hydrology.

URBAN WATER RESOURCES

Introduction

Before embarking on any detailed discussion of urban drainage, we will look at the water cycle of an urban area in its conceptual totality. A means for conducting an input-output inventory of water resources is the assembly of water budgets or water balances for a specific geographic entity. We commence our discussion with a collective national annual water balance for all urban areas of Sweden. Illustrated is the comparative isolation of drainage facilities from the remainder of the urban water infrastructure, except where combined sewers are included. Examples of water balances for an entire nation or a large portion of a country are noted, but very few metropolitan water balances have been reported. In order to account for the interrelation, interdependence and interconnection of the water resources of a metropolis it is necessary to adopt some form of a total resource systems-approach. Having illustrated the use of such an approach for a province of the Netherlands, we point out that water balances are the initial stage of a comprehensive systems analysis. Retaining the total system concept, we next present a schematic representation of the physical aspects of the urban water resources system, and then segregate what may be regarded as the stormwater and wastewater portion, the focus of this report.

Water Balances

Water balances (or water budgets, or water inventories) describe the quantity and quality aspects of the destiny of water from its appearance as precipitation through its departure from a metropolis as runoff and evapotranspiration. While very few balances have been made for metropolitan areas, they would provide a basis for better recognition of the interrelation, interdependence, and interconnection of the elements of the water resources of a metropolis.

The collective average annual water quantity budget for urban areas in Sweden is shown in Figure 25-1. The volumes indicated are initial rough approximations that are undergoing continuous refinement. The upper right portion of Figure 25-1 involves natural processes that occur in urban and non-urban areas alike. However, the remaining portions represent the complex infrastructure introduced to support urban areas. Particularly noteworthy is the limited number of connections between the natural processes and the infrastructure: via combined and stormwater sewers; via groundwater; and at receiving waters. Thus, urban runoff can be seen to be a rather special part of the total resource.

A preliminary collective average annual water quality budget has also been made for urban areas in Sweden. Initial estimates include the following total pollution loads from urban areas entering the receiving waters of Sweden, in tons per year:

TABLE 25-1

Pollutant	Wastewater	Combined Sewer Overflows	Stormwater Discharges	Sums
BOD_7	33,000	3,000	12,000	48,000
P	3,000	100	100	3,200
N	17,000	400	1,200	18,600

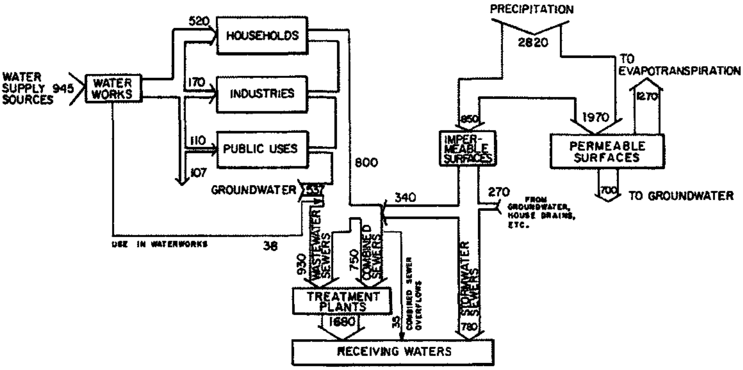

FIGURE 25-1. General Urban Area Water Budget for Sweden (values in millions of cubic meters per year)

Water is constantly in motion, a fact that can be obscured by averaging quantities over a year. Comprehensive study of water resources would include development of seasonal balances as well. Water originating solely in an urban area that can be captured is seldom adequate for its water supply. The impact of pollution burdens is often felt in receiving waters well beyond their points of entry. Thus, to close the water balance of an urban area usually requires accounting for causes and effects that occur some distance away.

National, seasonal water balances have been achieved for each of two parts of the Netherlands. Projected water balances for numerous sectors are obtainable from a digital computer devised for that purpose. Also included are capabilities for modeling salt balances.

Another example of nationwide water balances is found in Israel, where such inventories are a part of the development of a group of simulation models for optimizing its total water resource.[22]

From the preceding discussion, it is evident that a water balance is a complete inventory at a given time. A general accounting for the overall movement of water and pollutants can be ascertained by comparing such complete inventories over successive time intervals. (The remainder of this subsection is from the report for the United States.)

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[22] Shamir, U. June 1972. Optimizing the Operation of Israel's Water System. *Technology Review*. Volume 74, No. 7, pp. 41-48.

Satisfactory evaluation of hydrological effects of urbanization, and related development of strategies for resource management and environmental protection, have been hampered around the world because of minuscule research investments despite the economic and environmental importance of urban water resources. Serious obstacles have impeded advances, but progress is being made in a few notable instances.

An ASCE task committee on the hydrological effects of urbanization highlighted in the conclusions of its final report a need for more comprehensive and more highly systematized investigations of hydrological changes in urban areas. The committee found that available information is of severely limited transfer value, partially as a result of the web of complexities imposed when open land becomes urbanized. While concluding that useful results will be obtained only via coordinated efforts on a metropolitan scale, the unsolved central problem is the absence of suitable means for achieving the needed coordination.

The IHD/Unesco Subgroup on the Effects of Urbanization on the Hydrological Environment arrived at very similar conclusions:

> *More metropolitan-scale water-balance inventories and their analysis should be undertaken as a means for improving overall water resources planning and management, and follow-on inventories should be made periodically to document change and to provide a better understanding of the hydrological effects of progressive urbanization.*
>
> *The interrelation and interdependence of water and wastewater, and the competition and conflict between multiple jurisdictions have intensified with the growth of metropolitan areas. The variety of uses for water in metropolitan areas are continually enlarging, particularly for recreational purposes and for esthetic enhancement. Thus, hydrological surveys of urban areas should be updated frequently and regularly.*

Both the ASCE and IHD groups struggled with the quantification of generic hydrological effects of urbanization on national scales. Despite the fact that most problems and effects are very similar in technologically and economically advanced countries, very few generalities can be drawn. To cite one of the few successful examples, it has been demonstrated in a number of countries that urbanization increases the local contribution of direct runoff volume and that systems of storm drainage conduits result in greater direct runoff peaks with shorter rise times than for pre-urban conditions. A source of impotence in generalization is the fact that, worldwide, the field of urban hydrology is almost devoid of modern research investment and that there has been relatively little study to date of the effect of human settlements upon natural hydrological conditions.

These calls for water-balance inventories are the direct result of a clear recognition of the interrelation, interdependence, and interconnection of the elements of the water resources of a metropolis. That is, a total resource systems approach is necessary if subsystem phenomena truly are to be identified, because of the complex linkages involved. Also, metropolitan land-use constantly changes, an occurrence that can only be accommodated for a complex system by using a total systems approach.

Systems Analysis

Since 1971 a committee in the Province of Gelderland, the Netherlands, has been developing a scientific base for optimal planning and management of available surface water and groundwater, from the standpoint of both quantity and quality. An approach embodying systems theory is being used, with the water resources management system divided into three types of elements (social, natural and artificial) and into a number of water subsystems. These levels and elements are being investigated at varying degrees of detail with the aid of mathematical models, mostly by interdisciplinary teams. The water resource system is regarded as a hierarchy of four strata of elements, Figure 25-2.[23]

All relevant artificial, natural and social elements are taken into account in the first stratum of Figure 25-2. Econometric models have been developed for extrapolation of water demands, taking into account various factors that affect such demands.

At the second stratum in Figure 25-2, the coherence or interrelation between the natural elements is being studied to establish the interactions among the water subsystems. Models have been developed to simulate the flow of groundwater and surface water, and the oxygen dynamics of streams. Quantity and quality aspects have been partially integrated by coupling these models. Moreover, in another model, the natural elements of the first stratum are included in a generalized network format adaptable to numerous surface water and groundwater applications.

By 1976 the usefulness of the modeling efforts undertaken in conjunction with the first two strata had been forcefully demonstrated. More complete coupling of quantity and quality models, and the incorporation of social elements, will complete the third stratum and lead to the fourth stratum, the ultimate goal, an integral water resources management model.

From the standpoint of an individual metropolitan area, it is important to note that the inventory aspects of the artificial and natural elements of the first stratum in Figure 25-2 are essentially a water balance, described in the preceding subsection. Putting it another way, the initial stage of a comprehensive systems analysis of the water resources of a metropolitan area is essentially nothing more than the attainment of a suitable metropolitan water-balance inventory.[24]

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[23] van de Nes. 1977. The Structures of the Decision-Making Process Within the Water Resources Management System. *Proceedings, Amsterdam Symposium, October 1977, International Association of Hydrological Sciences*. IAHS-AISH Publication No. 123, pp. 529-542.

[24] McPherson, M.B. April 1975. Need for Metropolitan Water Balance Inventories. *Journal Hyd. Div.* ASCE Proc., Volume 99, No. HY10, pp. 1837-1848, October 1973. Author's closure, Volume 101, No. HY4, p. 409.

The Urban Water Resources System

If we reduce the considerations of the water resources management system of Figure 25-2 to the physical water-handling aspects, alone, the result is what might be called the urban water resources system, depicted broadly and schematically in Figure 25-3.[25] Capability has advanced satisfactorily for the analysis of water supply-treatment-distribution and wastewater collection-treatment-disposal. In Figure 25-3 it should be noted that storm sewers are almost an isolatable part of the urban water resources system. Not shown explicitly in the pictorial representation of Figure 25-3 is the interconnection between wastewater collection-treatment and storm sewers, via combined sewers, elaborated upon below.

Capability for analysis of urban runoff, particularly drainage, has lagged substantially behind that for other components. In recognition of this fact, the emphasis for the initial phase of the IHP (1975-1977) on urban hydrology was primarily on urban drainage research and development. Because simulation models are the basic tools of analysis, they comprised one of the two themes reviewed. Because field data is essential for the validation of analytical tools, the other theme was catchment research.

Figure 25-4[26] is a schematic representation that includes the storm and combined sewer subsystem of Figure 25-3, together with treatment of dry-weather flows carried by combined sewers. The modeling and catchment research summarized in the present report is mostly in connection with this subsystem, which is a specific portion of the urban water resources system.

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[25] Water Resources Engineers, Inc. September 1968. Comprehensive System Engineering Analysis of All Aspects of Urban Water, A Prefeasibility Study. Appendix H, *Urban Water Resources Research*. ASCE, New York, New York (available from ASCE).

[26] Sonnen, M.B., L.A. Roesner, and R.P. Shubinski. 1975. Urban Water Management Models. *Urban Runoff, Quantity and Quality*. ASCE, New York, New York, pp. 89-97 (available from ASCE).

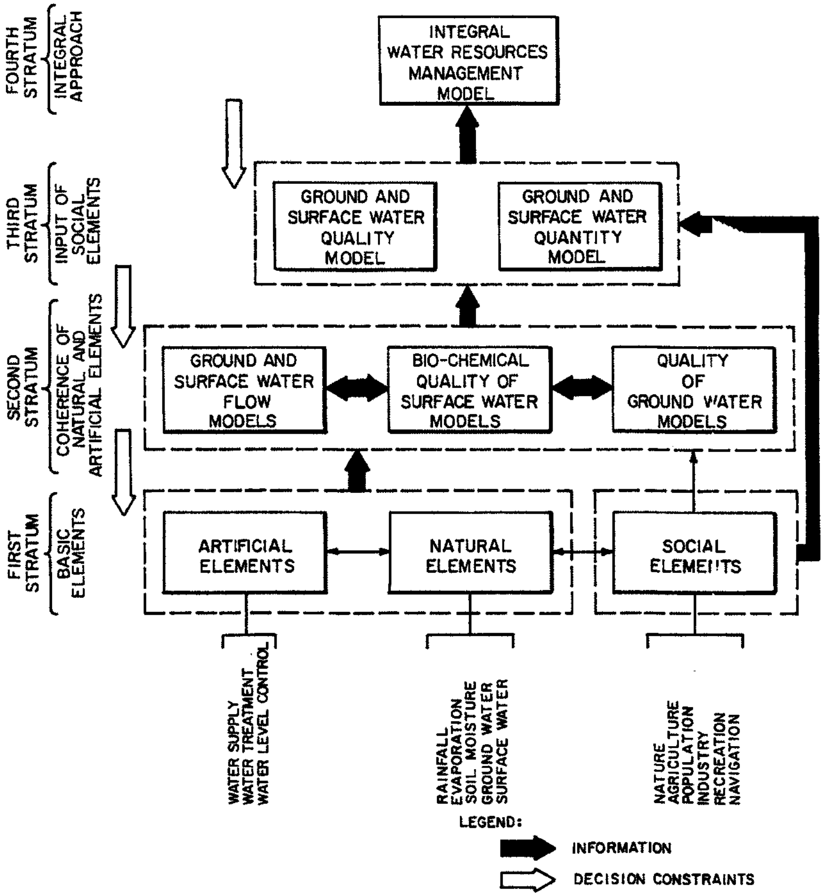

FIGURE 25-2. Hierarchy of the Water Resources Management System

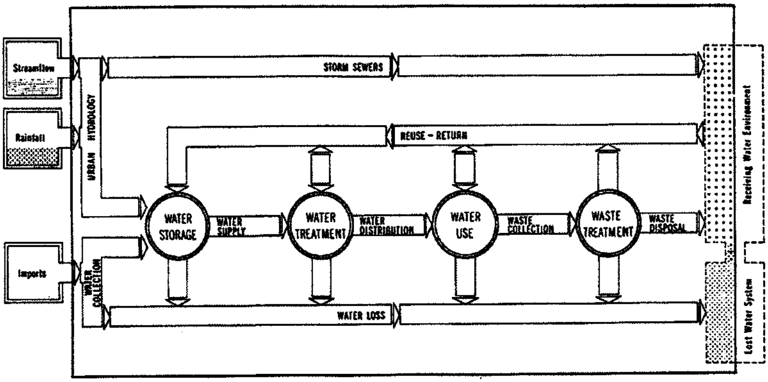

FIGURE 25-3. The Physical Urban Water Resources System

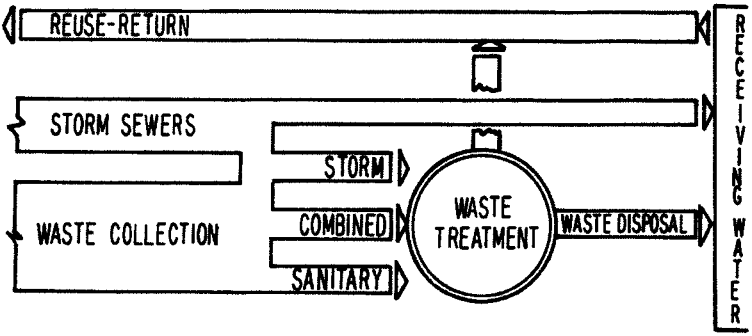

FIGURE 25-4. Stormwater and Wastewater Portion of Urban Water Resources System

URBAN RUNOFF, QUANTITY AND QUALITY

Introduction

Whereas there is a continuum between the subsystems of water supply, water use and wastewater reclamation (Figure 25-3), stormwater has been historically regarded as purely a negative good or nuisance and its subsystem (Figure 25-4) has seldom been deliberately connected to the other urban water subsystems.

The preceding section closed with a description of the storm and combined sewer subsystem of the overall urban water resources system. The function of underground drainage conduits is to remove stormwater from urban surfaces (except combined sewers, which in addition convey wastewater on a perennial basis). The smallest catchment area (on the order of a hectare 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 also serves as a collector of surface runoff all along its route.

Human life is seldom threatened by the flooding of underground drainage facilities, except as a health hazard. 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 that will be overtaxed infrequently. However, because of the marginal level of protection afforded, storm drainage flooding damages are of considerable magnitude. 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.

Flood control, drainage and the quality of receiving waters are all closely related. Increased volumes of direct runoff from underground drainage conduits clearly can aggravate flooding of urban floodplains. 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. Moreover, 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.

Most forms of pollution enter receiving waters continuously or at least seasonally. Because the entrance of stormwater is spasmodic but comparatively violent, the effects of its pollution are both comparatively dramatic and transient, and its impact depends on the season of occurrence and ambient levels of non-storm associated pollutants in receiving waters. According to Figure 25-1, Section 2, the estimated annual volume of stormwater runoff for urban areas of Sweden is slightly greater than the annual volume of wastewater, and according to similar estimates it is about half as large for the Federal Republic of Germany. The impact of such large quantities entering receiving waters over very brief periods is only beginning to be appreciated. There seems to be a consensus of opinion supporting the contention that the abatement of pollution from combined sewer overflows and storm sewer discharges could well be more cost-effective than the adoption of increased levels of treatment of wastewaters in certain circumstances. Recent papers on research in the United Kingdom,[27] Australia,[28] and the United States[29] tend to support this contention. One of the difficulties in evaluation of such trade-offs is that little is currently known about either long-term or short-term toxic effects of urban runoff in a variety of receiving waters and ecosystems.

Research Status

Among the earliest comprehensive field data research programs were those of the Road Research Laboratory in the United Kingdom and The Johns Hopkins University in the United States, initiated over a quarter of a century ago. These programs were conducted as means for improving knowledge on underground drainage system rainfall-runoff relationships. Generally speaking, a broad interest in the water pollution aspects of runoff did not emerge until the 1960s, in connection with suspected significant pollution from combined sewer overflows. One thing led to another, and by the early 1970s there was a broad interest also on pollution from discharges from separate stormwater sewers.

Australia, which has no combined sewers, illustrates the recently emerged interest noted immediately above. It could be said in 1976 that:

> *...water quality measurement programs in urban areas were not integrated with the rainfall-runoff data collection programs until the last few years. This was probably due to the dispersed nature of responsibilities for measurement of rainfall, runoff and water quality. It was also in part due to a lack of awareness of the magnitude of pollutants in stormwater runoff and their effects on receiving waters. With the move towards the removal of nutrients from sewage effluents in some Australian cities, more attention has been devoted to the contribution of urban stormwater runoff to problems caused by*

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[27] Ellis, J.B. December 1976. Sediments and Water Quality of Urban Storm Water. *Water Services*, Volume 80, No. 970, pp. 730-734.

[28] Cordery, I. February 1977. Quality Characteristics of Urban Storm Water in Sydney, Australia. *Water Resources Research*, Volume 13, No. 1, pp. 197-202.

[29] Pitt, R. and R. Field. August 1977. Water-Quality Effects from Urban Runoff. *Journal American Water Works Association*, Volume 69, No. 8, pp. 432-436.

excessive amounts of nutrients or other pollutants reaching rivers, lakes, bays and estuaries.

Research Objectives

Identified in the national reports are three fundamental research objectives with regard to urban runoff:

- Determination of the hydrological effects of urbanization;
- Development of measures that would offset the adverse effects and enhance the assets of urban runoff; and
- Resolution of improved tools of analysis for the planning, design and operation of altogether new drainage systems and for the improvement and extension of existing drainage systems.

These three objectives are intimately related, and all rely on the effectiveness and reliability of the tools of analysis. In turn, the acceptability, credibility and reliability of the tools hinge upon the extent and representativeness of the field data available for their validation.

None of the national reports even implied the existence of an adequate national database for validating tools of analysis. Little, if anything, would be gained by probing the shortcomings of these dozen cases. Rather, an ideal minimum national data acquisition program will be suggested, and reported advances in knowledge will be reviewed in terms of that ideal.

An Ideal National Program

Schematically portrayed in Figure 25-5 is an hypothesized ideal minimum national field data network for sewered catchments. Assumed is the existence of four major national climatic zones, realizing that there are countries with only one recognizable zone and others where there would be more. Field data should be obtained in a sample metropolitan area of each major climatic zone. In each sample metropolitan area, about a half-dozen catchments should be selected for rainfall-runoff-quality observation, each representing a different major type of land-use. Land-use of a given catchment should be fairly uniform because that catchment is considered to represent that particular type of land use in its metropolis. In order to validate the multiple components of modern tools of analysis, flow measurements should be made, and water quality samples should be taken, not only at the outlet from the catchment (terminus of outfall sewer) but also within conduit branches below major tributary junctions and within some street inlets and at the roof leaders of some individual buildings in a sub-catchment. (Also, at least one rain gauge should be installed in each catchment.)

A specialist in urban hydrology would be justified in criticizing the Figure 25-5 representation as being extremely sparse, or worse. However, the minimum data network depicted in Figure 25-5, using the individual catchment shown as an example of the average case, calls for 192 flow-measurement and water quality sampling stations, ranging from a flume installation at an outfall sewer exit to a small device in a roof leader. In the hope that zonal differences might be small, the national program could be started with half as many stations, for the most disparate climatic zones (e.g., two of the four in Figure 25-5). Even so, none of the national reports reviewed here described anything that came anywhere near the ideal minimum. It is not just that not as many catchments were involved. Practically all observations have been at catchment outlets and water quality samples have been collected at only a minority of stations. (More about this later.)

Figure 25-6 contains the essential elements of an ideal national urban runoff program. The uppermost element is the sewered catchment field data network described in Figure 25-5. Attempts to generalize results of analyses of mixed urban catchments (*viz.*, small streams fed by sewered sectors) have been inconclusive because too many variables are involved, which is a pity because there appears to be more of this kind of station than for sewered catchments. Thus, urban stream data are segregated in Figure 25-6, but it should be understood that what is intended is stream stations where upstream sewered sectors would also be observed. A crucial consideration is the concurrent collection and analysis of field data, activities that could be essentially completed in as short a time as a three-year period of concentrated effort.

The sole purposes of the data collection and analysis phase are to derive national guidelines for planning and design and national indicators of effects of urbanization, by major climatic zones. Prerequisite to the attainment of these goals is: the elucidation of water quality processes (cause and effect relations); determination of zonal parameter values for tools of analysis; and resolution of linkages with receiving water tools of analysis. An intermediate phase would include: the simulation and economic analysis of land-water management strategies (such as diffused detention storage in sewered catchments, onsite management practices for reducing stormwater pollution, etc.); and evaluation of significant historical storm data, by major climatic zones, to place the provision of protection from flooding and abatement of runoff pollution on a probabilistic footing.

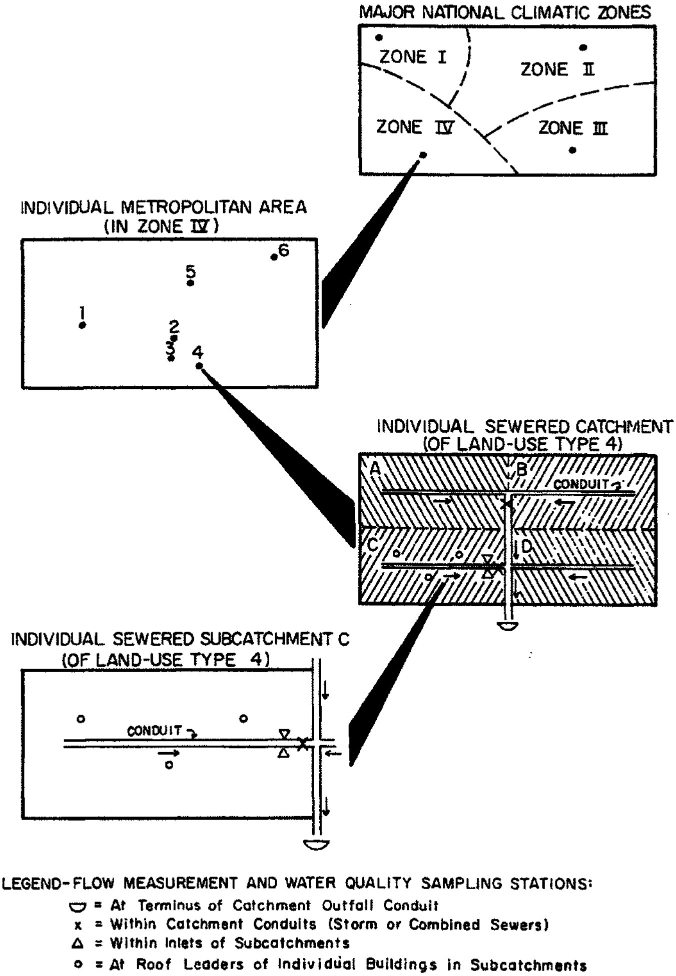

FIGURE 25-5. Ideal Minimum National Field Data Network for Sewered Catchments

We regret to say that, despite substantial advances in recent years, attainment of the goals described appears to be a long ways away in most if not all of the dozen nations for which status reports have been prepared. On the other hand, the ideal national program outlined in Figure 25-6 would require a substantial, concentrated, coordinated, well-managed effort spanning perhaps a minimum of a total of five years for its completion. Because recognition of the importance of the runoff aspects of urban hydrology is quite recent, we optimistically expect that more comprehensive national programs will eventually evolve.

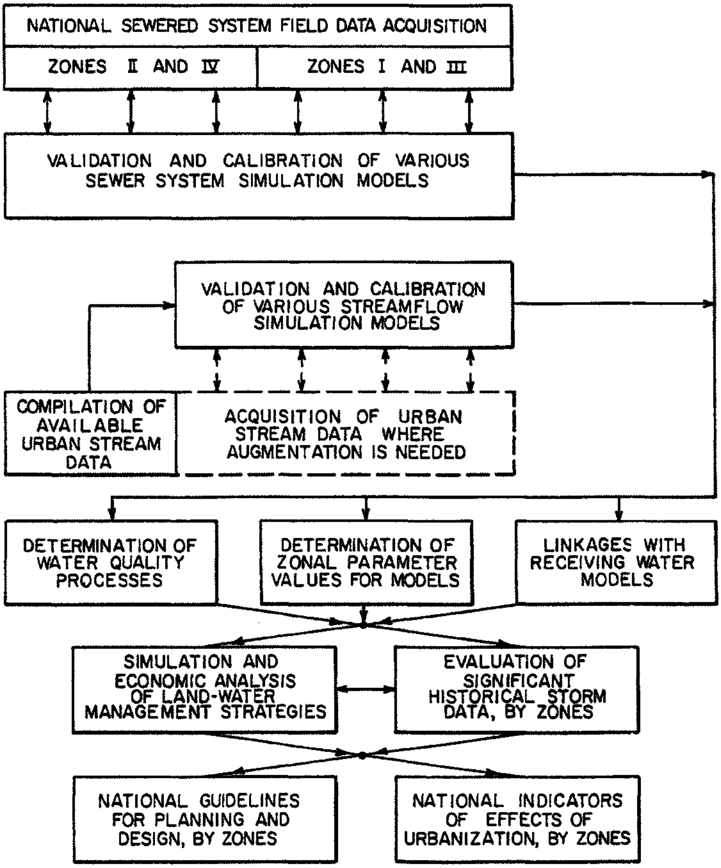

FIGURE 25-6. Ideal National Urban Runoff Program

Role of National Governments

We have noted that around the world the sizing of storm and combined sewers had long been, and mostly still is, determined by wholly empirical methods. Various versions of what is sometimes called the rational method are in common use. This simplistic procedure yields only a peak flow rate unless the empiricism is further extended to synthesize a hydrograph. The persistence of such a nebulous tool can be credited to a lack of adequate field data for validating more realistic procedures. However, as the next sections of this report will indicate, catchment research over the past several years, coupled with development of new tools of analysis, is gradually leading to the adoption of more realistic methods. The impetus for change in some countries has been a new concern over water quality coupled with an interest in more widespread use of detention storage; and both issues require employment of full hydrographs for their explication.

In nearly all great metropolises around the world, every important level of government participates in each major public service category; the proliferation of single-purpose agencies dealing with specialized public services has been continuous; and the most common type of metropolitan institution throughout the world, by far, is the independent district, corporation, or authority.[30] Thus, despite the social importance of urban water resources and the substantial investments in associated facilities, we find national, territorial and local governments involved in a tangle of interests. This fragmentation is one of the reasons why urban water resources research around the world commonly has suffered from inadequate attention and support and from discontinuous and erratic efforts. In addition, few local agencies can support hydrologic research that will yield results transferable to other metropolitan areas, or even from one jurisdiction to another in the same metropolis. Noted earlier was that urban drainage has suffered the most research neglect and technical knowledge on it is the most primitive among the various aspects of urban water resources. An important contributing factor is undoubtedly the conventional isolation in the past of the urban drainage subsystem from the total urban water system, discussed in Section 2. New interest in urban drainage performance has arisen at least partly because of a desire to integrate the urban drainage subsystem with other subsystems, such as for water pollution abatement or water reuse.

The manner in which various nations have attempted to overcome the enigma of fractionalized responsibility can be instructive. The remainder of this subsection is devoted to a summary of such activities as have been described in the national reports, and the order of their citation follows the numerical order of their release.

1) **United States.** A national manual of urban drainage design practice was prepared in the 1960s by two professional organizations, the American Society of Civil Engineers and the Water Pollution Control Federation. Since then, advances in the field have been supported primarily by over a half of a

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[30] Walsh, A.H. 1969. *The Urban Challenge to Government: An International Comparison of Thirteen Cities.* Frederick A. Praeger, New York-Washington-London, 294 pp.

dozen national government agencies augmented by state and local government activities. The result can be characterized as a fairly chaotic and uneven mixture of advances in knowledge. Concluded was that 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.

2) **Australia**. Gauging of urban catchments is under the authority of five territorial agencies, five local governments and two consultant firms. At least two national government agencies have a hand in research. Guidelines for flood estimation in urban catchments are included in a revised national report prepared by the Institution of Engineers, a professional organization.

3) **Canada**. A substantial research effort was initiated in recent years, centered in the research program of the Urban Drainage Subcommittee (UDS) for the Canada-Ontario Agreement on Great Lakes Water Quality. Other Provinces are informally involved. A survey of urban catchment research carried out in 1973 indicated that there were only a few instrumented urban catchments in Canada, of which only two had produced data suitable for urban runoff modeling. The UDS reacted to this situation by establishing several new urban test catchments in the Province of Ontario. A similar action has been taken recently by the Province of Quebec. On a nationwide basis, however, urban catchment research still has not reached a level consistent with the large expenditures in urban drainage facilities. Urban catchment research in Canada seems to be plagued by a number of problems, some of which are briefly discussed below:

   a) *Lack of coordination*. There is no national network of urban test catchments and no nationwide coordination of catchment research. As a result, the current network of test catchments does not extend to all climatic regions, and the type of information and data formats vary from case-to-case, thus inhibiting comparative studies on various catchments.

   b) *Lack of suitable instruments*. Several studies of instrumentation for catchment research have been carried out recently. All of these indicate a lack of reliable instruments for the measurement and sampling of sewer flows. Even well-designed instrumentation systems may require up to six months for the elimination of malfunctions before they become fully operational.

   c) *High costs*. Urban runoff data collection projects are rather costly and their success is not guaranteed.

The UDS is in the process of developing a manual of practice on storm drainage.[31]

1) **United Kingdom**. ...as elsewhere, urban hydrology is only now beginning to receive the attention it deserves. ...Although the total research effort is

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[31] Urban Drainage Subcommittee. March 1977. Manual of Practice on Urban Drainage. Draft No. 3, Environment Canada, Ottawa, Ontario, 398 pp.

increasing, it is still small by comparison, for example, with the United States. However, it is possible that work is more closely coordinated in the United Kingdom and that benefit derives from the frequent informal meetings organized by the active researchers in the field. The Department of the Environment/National Water Council Working Party on the Hydraulic Design of Storm Sewers was formed in 1974 and now acts as a focal point for research coordination and information exchange. Previously a national colloquium at Bristol University had helped to crystallize the growing dissatisfaction with existing design methods and with the inadequate body of knowledge relating to urban runoff, quantity and quality. Current research at central research stations and universities is concerned with the development of new methods but most results are, as yet, only tentative. There is also a growing body of opinion that is interested in total planning rather than subdivision planning. ...

2) **Union of Soviet Socialists Republic**. National agencies have developed regional parameters for application of a form of the rational method for sewered and small stream catchments. In recent years, national agency involvement has continued through development of new simulation methods for calculating runoff hydrographs.

3) **Federal Republic of Germany**. Extensive urban hydrology research has been undertaken in recent years. An indication of these activities is illustrated by the conduct of two important national meetings on subjects related to the Effects of Urbanization on the Water Cycle: German Hydrological Conference, September 1972, at Duisburg (Ruhr industrial region); and Convention of the German Water Management Association September 1975, at Wiesbaden (agglomeration area at the confluence of the Rivers Rhine and Main). While a profusion of national, territorial and local agencies appear to be involved in research, as in other countries, the role of private consultants (in particular) and universities is remarkably prominent.

4) **Sweden**. Research is pursued primarily at the four technical universities, but also by private consultants and local governments. Research support is from two national agencies with disparate missions: one is concerned with projects that will yield results useful in engineering practice; and the other has more scientific interests, such as fundamental effects of urbanization. Sweden has a population of about eight million persons, larger than only Norway (see below) of the dozen nations that have provided reports.

5) **France**. As a result of the constantly growing complexity of urban sewerage problems, the National Ministries of Equipment and Interior supported research which had as its 1970-1975 goals: to bring up-to-date the official regulations on urban sewerage; and to improve knowledge on rainfall-runoff transformations for urban watersheds by making runoff mathematical models adapted to the design of complex sewer networks. ...Attention to research on water quality aspects of drainage is very recent in France. Much of the research reported was conducted by a university and a consulting firm.

6) **Norway**. Until recent years, hydrological research on urban catchments virtually did not exist, partly due to a general orientation of the resources of Norwegian hydrology towards satisfying the pressing needs for hydrological data for hydropower development. This has focused interest on large and medium-sized catchments. During the last decade, the rapidly increasing investments in drainage systems have brought attention to the adverse economics of poor design methods. A broad six-year research program was funded beginning in 1971 by the Ministry of Environmental Affairs that included six subprojects (among a total of 40) related to urban hydrology. These six subprojects have been administered by four other national agencies. Norway has the smallest population (about four million persons) of the dozen nations that have provided reports. Yet even this example involves five national agencies in its urban drainage research, not to mention the local governments and others that are also participants.

7) **Netherlands**. Participants in the support and conduct of urban drainage research are at least as heterogeneous as in the other nations from whom reports were obtained. Cited are the projects of all three levels of government together with universities. In order to develop a national report about 35 experts and organizations were queried, and a still larger number is expected to be involved.

8) **Poland**. Since 1972, the Research Program of Urban Sewerage and Drainage Schemes has been conducted by the Research Institute on Environmental Development in cooperation with the Institute for Water Supply and Water Constructions at the Technical University in Warsaw. The main objective of the program is the verification of assumptions and methods for planning and designing stormwater and combined sewer schemes in large urban areas. The program is financed by the Ministry of Administration, Land Economy and Environmental Protection. Consideration of earlier preliminary findings emphasized their great importance in the design of economical drainage schemes adapted to urban requirements that would not unduly pollute receiving waters.

9) **India**. The preceding 11 cases are for nations that are regarded as economically developed. The sole report from an economically developing nation was for India. Because of the very great importance in the IHP of assisting economically developing nations by transferring information to them on the positive and negative experiences and progress of more economically developed nations, the perspective in the report from India deserves special amplification, the next subsection.

A Developing Nation Perspective

According to the 1971 census, 13% of the total national population of 547 million people in India lives in urban areas. Of the urban population of about 110 million, three-fourths are served by water supply facilities and two-fifths are served by wastewater sewerage. Urban areas in India have a small proportion of built-up area to total area and, hence, urban drainage problems in India as in most other parts of the developing world are quite different from the 'concrete jungles' of the developed nations.

In a developing country, the priorities for economic development and investment are for food, shelter, clothes, health and education. Urban drainage is generally not taken into consideration except when it affects significantly any of the above factors, particularly as a part of the more general problem of flooding of urban areas. As almost all important cities of India are on the banks of rivers and are subject to flooding, drainage of urban areas and riverine flood control are generally interlinked. Because of financial limitations and because urban drainage problems constitute 'negative goods', very little attention has been paid in India to urban drainage. ...

Low-lying areas prone to frequent flooding are often encroached upon by the poorest section of the population, and are covered with sprawling slum areas with a high density of population and meager civil amenities. Failure to provide an adequate urban drainage system seriously affects the life of these people and exposes them to potential health hazards. Thus, urban drainage systems are also linked with the overall problem of slum abolition, resettlement, and urban redevelopment.

Urban hydrologic problems of India, as in other developing countries, differ from those of developed nations in several important respects. They include:

- Lateral rather than vertical development;
- Limited amounts of paved areas;
- Intimate interaction between urban drainage and flood control;
- Preference for open drains over closed ones;
- Limited availability of continuous records of precipitation, streamflow and water quality;
- Low fiscal priority for drainage investment;
- Limited numbers of sewer connections and hence silting of combined sewers;
- High cost for construction and modification of combined sewer systems; and
- Limited capacity for financial investment.

As in most countries, all three levels of government are parties to urban drainage research: national, territorial (provincial), and local. However, involvement by universities and consultants appears to be smaller.

URBAN CATCHMENT RESEARCH

Introduction

Figure 25-4 (Section 2) depicts the elements of the stormwater and wastewater portion of the urban water resources system shown in Figure 25-3 (Section 2). Figure

25-7 may be regarded as incorporating the stormwater portion of Figure 25-4 together with other aspects of urban stormwater disposal. Supported to some extent by findings from catchment measurements, urban runoff and runoff quality simulation capability developments have ranged from the incorporation of all elements mentioned in Figure 25-7 to only a few.

Sewered catchment research has increased considerably in extent and intensity in the 1970s. However, little would be accomplished by reciting all the urban catchment research detailed in the dozen national reports. Rather, we will use examples from among the reports to illustrate the major considerations in such research.

A National Program Example

Viewed from outside, the research program in Norway appears to be one of the most integrated and comprehensive in meeting nationwide needs, if we have interpreted the dozen national reports correctly. Thus, it is appropriate to reproduce the research catchment characteristics, Table 25-2. The first ten catchments are locationally paired basins, one rural and one undergoing urbanization, gauged primarily to deduce hydrological effects of urbanization such as via catchment water balances. Analyses were delayed because development of the five urbanizing basins was more protracted than originally anticipated.

Urban development of the other 12 catchments described in Table 25-2 was stable. The primary purposes of gauging these catchments was to provide data for validation of tools of analysis and, for most of them, to obtain indications of the extent of pollution carried by underground drainage systems. Surface water loads in combined systems have been compared with those of separate systems and have been found to be generally higher in combined systems, probably due to washout of sedimented sewage from pipes during storms. This observation is in agreement with findings elsewhere.

A researcher studying Table 25-2 would wonder to what extent the last 12 catchments have a near-uniform land-use. Only the existence, not the extent, of 2-4 types of land use are indicated for all but one catchment. Presumably each has a predominant land use. These comments are not intended to be critical, because those of us who have attempted to locate catchments with locations suitable for gauging-sampling stations that also drain catchments with near-uniform land use (as suggested in Figures 30-5 and 30-6, Section 3) have frequently had to accept conditions well short of the ideal.

We also take advantage of this example to point out that flow has been gauged and water quality samples have been collected in the Norwegian catchments at only one point in a system. This is universally typical, but is nevertheless an important departure from the ideal program depicted in Figure 25-5. On the other hand, some internal but not connected sectors of some research catchments in Sweden and the Netherlands, for example, have or have had auxiliary gauging-sampling stations, but even these fall somewhat short of the ideal. In sum, we have found no instance of an example that meets minimum ideal criteria.

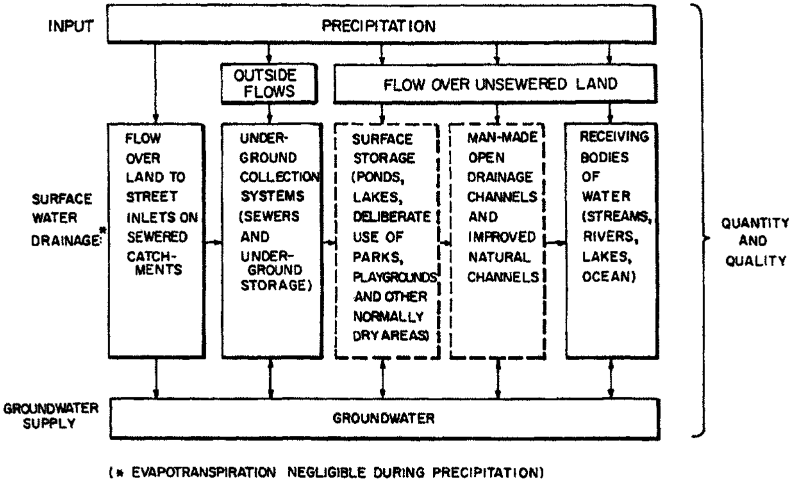

FIGURE 25-7. Urban Stormwater Disposal Physical Subsystem

TABLE 25-2

Catchments Investigated in Norway

ID #	Area, hectares	Impervious, Percent	Mean Precipitation, millimeters	Type of Sewers	Water Quality Sampling	Planned or Existing Lane Use					
						Rural	Undergoing Urbanization	Mult. Family Residential	Sing Family Residential	Commercial, Industrial	Industrial
1	195.3	?	829	Separate	No		X	X	X	X	X
2	84.0	0	829	None	No	X					
3	33.7	?	604	Separate	No		X	X	X		
4	36.9	0	604	None	No	X					
5	36.5	?	999	Separate	No		X	X	X	X	
6	9.9	0	999	None	No	X					
7	64.2	?	1,015	Separate	No		X	X	X	X	
8	66.6	0	1,015	None	No	X					
9	22.5	?	1,121	Separate	No		X	X	X	X	
10	18.5	0	1,121	None	No	X					
11	8.4	36	829	Separate	No			X	X		
12	23.6	20	1,417	Separate	No			X	X	X	
13	24.4	25	785	Separate	No			X	X	X	
14	9.9	97	740	Separate	Yes			X		X	
15	37.2	43	748	Separate	Yes			X	X	X	
16	36.6	33	789	Separate	Yes			X		X	
17	19.7	18	857	Separate	Yes			X	X		
18	21.3	37	857	Combined	Yes			X	X	X	
19	33.0	20-30	700	Separate	Yes				X		
20	175.0	10	700	Combined	Yes				X		X
21	219.3	69	740	Combined	Yes			X		X	X
22	380.0	10-15	999	Combined	Yes				X	X	

Another National Example

Having just discussed the program of the reporting nation with the smallest population, we turn now to the report of one with a population over 50 times as large. It is a well-known axiom that a collection of data has no inherent value, only an unknown potential value, until it has been subjected to thorough analysis, such as in comparison with simulated flow and water quality performance. Thus, for the report for the United States, mention was made of only those catchments where data had been collected that had been employed in the testing of tools of analysis. (To be consistent, only those models that had been tested against actual field data were given mention). Further, catchments and tools cited were confined to sewered and partially sewered cases.

Although the numbers have increased since, in late 1975 it could be reported that 16 different tools of analysis had been tested on from as few as only one to as many as 28 catchments, according to publicly available documentation. While the number of catchments involved, 64, was an impressive total, there are important liabilities to be considered: in almost every instance field flow measurements had been made only at one location; flow measurements for 44 of the catchments had been made indirectly via stage gauges, only some of which had been related to the characteristics of downstream hydraulic controls, with the remainder depending on assumed conduit friction coefficients; and water quality data had been collected on less than half of the catchments, and for only a fraction of these had such data been used in tests of analytical tools. Moreover, while 42 gauging stations had been located at outfalls or within sewer systems, 22 had been in streams and, therefore, were for only partially sewered catchments.

Despite the large number of catchments in the United States from which data had been used to validate newer sewered system tools of analysis, few had produced data truly suitable for that purpose. The main reason was that the gauging at many stations commenced or occurred some time before the more advanced tools of analysis were developed. The Norwegian situation was quite different, because the national data network was initiated in the 1970s, a principal purpose was the validation of the newest tools of analysis, and the network was much more centrally administered. In contrast, the situation in the United States can be described as highly random and minimally coordinated. However, to be fair, this criticism applies almost everywhere, and no satisfactory national prototype can be offered that adequately approaches the ideal minimum national network suggested in Figure 25-5, Section 3.

In the absence of a coordinated national field data and analysis program, much of what can be done is primarily at the initiative of local governments. The crucial need for gauging and sampling of representative catchments with highly uniform or homogeneous land uses has only recently been fully appreciated. Late in 1976 the City of Philadelphia, Pennsylvania, completed the instrumentation of an auxiliary research network of nine catchments, characterized in Table 25-3.[32] Note that uniformity of land use is deliberately high for the catchments selected, to insure a representation of land-use types. Because there are about 300 separate storm sewer and combined sewer catchments within the 330-km^2 of the City, nine catchments constitute a small numerical sample. Further, the purpose of the investigation is to calibrate tools of analysis for their application to the much larger local metropolitan area. While this network is among the first deliberately selected for uniformity of land use, it is nevertheless at least partially flawed by the fact that only one gauging and sampling station was included in each catchment. However, this expedient is consistent with the major purpose of the network, which was to develop metropolitan indicators quickly within a fixed budget. That is, it was necessary to favor extensiveness of catchments over intensiveness of stations within individual catchments. Parenthetically, it must be mentioned that these nine catchments are only a small part of the Philadelphia Water Department's urban hydrology network in the metropolitan area.

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[32] Radziul, J.V. and P.R. Cairo. November 28-December 3, 1976. Philadelphia Water Department Case Study of Philadelphia, Pennsylvania. Paper presented at *Engineering Foundation Conference on Instrumentation and Analysis of Urban Stormwater Data, Quantity and Quality.* Easton, Maryland, 39 pp.

TABLE 25-3

New Catchment Investigation Program
Philadelphia, Pennsylvania, United States

ID #	Area, hectares	Impervious, Percent	Type of Sewers	Water Quality Sampling	Existing Land Use						Uniformity of Lane Use, percent
					Residential			Commercial	Industrial		
					Single Houses	Semi-detached Houses	Multiple Attached Houses	Shopping Center	Light Industry	Heavy Industry	
1	58.7	43	Separate	Yes	X						86
2	13.0	31	Combined	Yes	X						88
3	33.2	63	Separate	Yes		X					100
4	48.6	68	Combined	Yes		X					81
5	30.0	84	Combined	Yes			X				98
6	32.4	98	Combined	Yes			X				90
7	3.6	100	Separate	Yes				X			100
8	17.4	75	Separate	Yes					X		100
9	40.5	76	Combined	Yes						X	100

Instrumentation

Already noted in Section 3 is the observation from Canada that there is a lack of reliable instruments for the measurement and sampling of sewer flows.... Even well-designed instrumentation systems may require up to six months for the elimination of malfunctions before they become fully operational. Also noted was the high cost of installations: At the present time, the data collection programs appear to be too expensive for all but the largest municipalities. The government funding of these projects also lacks continuity.

In the report for France a suggestion is made that despite the numerous efforts of researchers, technological studies seem to be needed that would lead to the design of new instruments and recorders fully adapted to problems encountered in urban hydrology. Only an international symposium comparing various experiences would lead to a quick resolution of this problem.

In the meanwhile, recommendations on the time-resolutions of rainfall data, sampling intervals, and flow measurement techniques, are available in an appendix of the report for Canada, abstracted from a comprehensive Canadian report. More recently, guidelines for North America have been assembled under the technical supervision of the ASCE Urban Water Resources Research Council that also include interim recommendations on instrumentation.[33]

For reasons of data reliability, the need for using some form of flow-constriction device for direct measurement of runoff from or in conduits is emphasized in several of the reports. Special arrangements for measurement of flows in road and street inlets have been cited and illustrated. Usually the most difficult kind of flow measurement is at points within conduit systems, and an insertable constrictive device developed for that purpose has been described in one of the reports.

Data Collection

While we have emphasized earlier that an assemblage of data has no inherent value until it is analyzed, an obvious precursor reservation is that data cannot be analyzed until it has been assembled in a format suitable for calculation. That is, raw data must first be reduced to hyetographs, hydrographs and displays of water quality parameter concentrations versus time. While there are devices for automatic collection of water quality samples, noted in several of the reports, the results of laboratory analysis can be obtained only some days after the event because of the long time required for determination of certain characteristics such as biochemical oxygen demand. However, signals from rainfall and runoff devices can be converted directly into usable forms almost immediately by means of automatic data processing equipment. Not only is the data thereby made available in the shortest time possible, but also the tedious and time-consuming task of manual reduction is mostly eliminated. But this convenience is purchased at the expense of a higher equipment cost.

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[33] Alley, W.M. 1977. *Guide for the Collection, Analysis and Use of Urban Stormwater Data.* ASCE, New York, New York, 115 pp. 155 (available from ASCE).

Automatic data assembly reduction installations were reported for the United States, Australia, Canada, Sweden, and the Netherlands. Before discussing this capability any further, we must caution the reader that there may be no more than perhaps a score of such installations in the world. We mention them here only to indicate the extent of advances in data collection systems and because they help to illustrate the steps that must be taken one way or the other.

The most comprehensive type of automatic data logging system incorporates telemetry of field sensor signals to a central data recording and analysis location served with digital computer facilities. Data logging systems incorporating telemetry were reported only for the United States and Sweden, and components of the one in Sweden are shown schematically in Figure 25-8. Telemetry occurs between the transmitter and the receiver. Notable is that this system was developed, installed and operated by a university, whereas most of the others were by agencies of local governments.

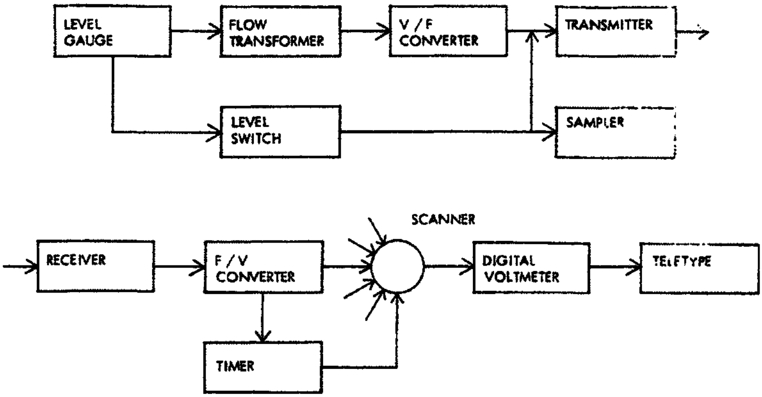

FIGURE 25-8. Data Transmitting System

One of the most vexing problems of data collection is the synchronization of readings. A principal reason cited frequently for using automatic data logging is much better synchronization.

Of more than passing interest is the adaptation of central data logging facilitated by telemetry as part of urban stream flooding warning systems in Australia.[34] This is

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[34] Earl, C.E., and others. June 1976. Urban Flood Warning and Watershed Management Advances in Metropolitan Melbourne. Melbourne and Metropolitan Board of Works and Dandenong Valley Authority. *ASCE*

only one facet of data needs for urban flooding mitigation, however, and there appears to be a dearth of hard urban flood damage data almost everywhere. The national study under way in the United Kingdom is delving deeply into this question.

Special Studies

Roadway drainage has received detailed study. Roadway surface runoff and subsurface drainage have been investigated in a water balance context in an elaborate project in the Netherlands. Water quality aspects of roadway runoff have been reported for the United Kingdom and the Federal Republic of Germany. Probably the earliest coordinated field data program with development of tools of analysis as its primary goal was by what was then called the Road Research Laboratory in the United Kingdom, with data collection initiated in the 1950s and resultant techniques of analysis reported in 1962. The most comprehensive data collection program in the United States was by The Johns Hopkins University, over roughly the same period, supported by highway agencies. It is of historical interest to note that some of the earliest important work on urban runoff was supported by roadway or highway agencies concerned mostly with transportation aspects of drainage.

Groundwater hydrology was discussed at some length in several reports. Studies ranged from analyses for conjunctive management of groundwater and stormwater in France through metropolitan groundwater simulation in Sweden, to the extensive investigations on groundwater regeneration and recharge that have been undertaken in the Federal Republic of Germany for quite some time and are now augmented with related studies of groundwater pollution including the effects of solid wastes.

Impacts of surface runoff from urban catchments in the form of increased rates, volumes and pollutant burdens, are experienced mostly in receiving open waterways, including streams, estuaries, lakes and coastal areas. Virtually all of the reports mentioned specialized studies of indirect indications of quantity and quality characteristics of such receiving waters as affected by urbanization. Noted in the Canadian report was that the impact of urban effluents on receiving waters is rarely studied in the field, a situation that is rather universal but appears to be slowly reversing. Runoff pollution adds to wastewater nuisances, coming as it does from the washing of pavements, streets and roofs after a time without rain. While we will mention in the next section the complex conjunctive analysis of flow and water quality of urban catchments and receiving waters, the reader is cautioned that such a linkage is too infrequently analyzed, despite a widespread concern over reducing stormwater quantities and pollutants entering receiving waters.

Retarding or detention basins have been a feature of stormwater drainage systems in Australia over a number of years, and this practice is on the increase there. Because of the complexities introduced when a number of such basins are included within a catchment, the best solution for this type of system appears to be through mathematical models which, in addition to estimating hydrographs at various

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Urban Water Resources Research Program Technical Memorandum No. 30, ASCE, New York, New York, 72 pp. (NTIS No. PB 259 549).

locations within the catchment, can also route hydrographs along creeks and channels and through retarding basins. Verification of models used to design complex systems including retarding basins is difficult due to the limitations of available data. While interest in detention storage in Australia was initially in terms of controlling runoff volumes, interest is growing in a number of countries in the use of the same or similar types of facilities for reducing the entrance of pollutants from urban stormwater runoff into receiving waters, such as in Poland.

The snowmelt fraction of urban runoff is receiving research attention in Canada and Sweden.

URBAN DRAINAGE MODELING

Introduction

In keeping with the thrust of the 12 reports being reviewed here, discussion will center on the simulation of performance of urban underground conduit drainage systems. Receiving water modeling will be mentioned only where it is conjunctively or conjointly involved with drainage modeling. The extensive number of receiving water studies that has been undertaken around the world, usually independently of drainage system modeling, has been noted in the preceding section.

Models are characterized in this section in terms of their application. The sparcity of suitable data for the regional validation of such tools of analysis is necessarily emphasized.

Why Simulation?[35]

All but a small fraction of storm and combined sewers around 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 gauged; (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 are 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

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[35] McPherson, M.B. June 1977. Urban Runoff Control Planning. ASCE, New York, New York, 118 pp. (NTIS No. PB 271 548)

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 using tools of analysis such as mathematical models.

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, design and operations. Some particular models have been employed in both planning and design, and a few models have been applied in design and operations applications, making it difficult to allocate them to a single category. Additionally, the reader is cautioned that on no account should the models to be mentioned be regarded as typical tools. Rather, common practice still favors rudimentary techniques, although the use of new tools of analysis seems to be growing rather rapidly everywhere.

Planning applications are at a macro-scale, such as for comprehensive metropolitan or municipal plans. Model requirements for planning are less rigorous and require and permit less detail than for 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 less than in design.

Design applications generally require more sophisticated, more detailed, tools, for the analysis of individual catchments and sub-catchments where the simulation of detailed performance of discrete elements within a sub-catchment must be achieved.

Operations applications are likely to be more use-specific because of wide diversities in management practices, operating problems and individual service-system configurations.

Components of Urban Runoff Models

A good overview of model components is given in the report for Sweden: We may of course regard urban storm runoff as a closed problem and analyze it as a whole. Because of the complexity of the problem this is not an approach especially well adapted to its purpose. The generally applied method of analysis is to divide the problem into at least three parts. These parts are clearly distinguishable from a phenomenological point of view. The first part may be described as an entirely hydrological process, where runoff from pervious or impervious surfaces is calculated with due regard to the hydrological water balance equation. The second part of the runoff process may be regarded as starting when storm runoff enters gutter inlets and the like, a hydraulic process that may be analyzed by routing inlet hydrographs through the network system conveying water beneath the ground. Due consideration must be given to storage effects and other factors. The last part of the runoff process may be considered as the treatment of polluted stormwater before discharging it to a receiving water body. This points out the essential problem included in stormwater runoff, namely to regard this process not only as one depending on quantity but in fact still more on quality.

The structural characteristics of urban runoff models can be segregated into two broad categories, lumped and distributed. In a lumped model, rainfall is transformed into the runoff at a given point without any hydraulic routing through the tributary area. An example is the conventional unit hydrograph, a tool in widespread use in river basin hydrological analysis and applied occasionally to urban drainage. A distributed model is characterized by a capability for the hydraulic routing of flows in addition to the hydrologic transformation of rainfall into runoff, such as through all or part of the underground conduit system within the tributary area being modeled. Because many more catchment details are accounted for, distributed models are considerably more complex than lumped models.

Space limitations do not permit review, or even mention, of the scores of models cited in the dozen national reports. As an alternative, we will confine our attention to the most comprehensive and flexible tools. The principal functional components of the most comprehensive, distributed models reported are depicted in Figure 25-9. Because pollutants are physically dissolved within and suspended by the flow of water, the runoff behaves as a pollutant carrier. Thus, pollutant routing (the two lower steps in the column to the right) is performed as an adjunct to hydraulic routing of flow (the two lower steps in the column to the left). Surface Runoff refers to the above ground flow of water from the time rainfall lands until it enters the underground conduit system; and the latter is termed Sewerage Transport in Figure 25-9. Routing in Receiving Water can accept the outflow from one or more sources of contributary sewerage.

Three models deserve mention that are described in some detail in the reports, because they have the capabilities indicated in Figure 25-9. All three of these models, in one variant or another, are programmed for routing flows using fundamental hydrodynamic equations of motion (after Barre de Saint-Venant). Late in 1975, it could be reported that publicly available documentation existed on the testing of variants of the Stormwater Management Model (SWMM) using data from 28 catchments (with water quality included for 18) in the United States, its country of origin; and the SWMM model has also been tested and applied elsewhere. The QQS model has been tested and applied in the Federal Republic of Germany its country of origin, and elsewhere. The CAREDAS Program (perhaps better known as the SOGREAH model) has been tested and applied in France, its country of origin, and elsewhere. All three models have been used in both planning and design applications.

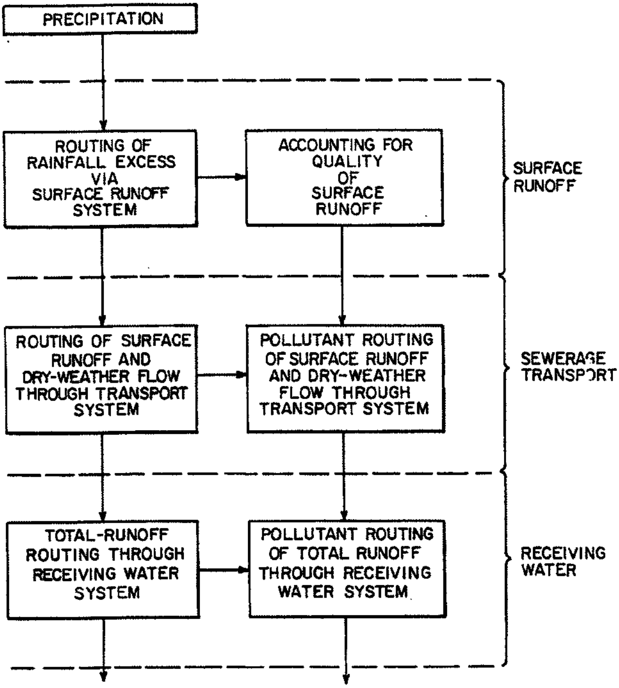

FIGURE 25-9. Components of Urban Runoff Models

A model developed expressly for the sizing of storm and combined sewers deserves mention because its use for design, in various modifications, has been reported in the United States, Australia, Canada, the United Kingdom, Norway, and India. Best known as the Road Research Laboratory method, its development in the United Kingdom was reported in 1962, making it one of the earliest distributed-type design models on the scene. All but a very few versions have been restricted to the surface runoff and transport system routing components of Figure 25-9, without water quality or receiving water routing capabilities. The RRL method has been validated in several countries. The method has been criticized because it is founded on empirical relations rather than a dependence on equations of motion, but its simplicity and orientation to design have been recognized as important attributes.

Comparisons of the virtues and capabilities of various models abound, according to several of the national reports. Because of wide latitudes in application criteria, the results of such comparisons are mostly inconclusive and hence are controversial. Thus, we see no point in entering into the widespread debate on the comparative virtues of various specific models. Instead, we will review some of the advantages and liabilities in the use of these tools of analysis in a general perspective.

Advantages in the Use of Models

A special session at the 1976 annual meeting of the American Geophysical Union attempted to define appropriate rationales and incentives for the more extensive use of urban runoff mathematical models for planning, design and operations. Several nations were represented. Among the advantages cited for the use of such models for planning were that:[36] tests can be made of alternative future levels of development and their impact on facilities needed in the future; several models well-suited to area wide planning are in the public domain and are regularly upgraded and made readily available by the national 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.

Noted in the report for France is that the introduction of mathematical models made it possible to reduce substantially the degree of empiricism inherent in traditional methods of analysis. Until then, no procedures existed for objectively verifying the validity of drainage schemes, while at the same time urban drainage authorities were facing increasingly complex and urgent problems.

Several national reports emphasize the continually escalating costs for new or modified drainage systems, and spiraling investment requirements are particularly evident where urban runoff pollution abatement is an objective. A statement in the report for the United States appears to be applicable in several nations: There is widespread interest in multiple-purpose drainage facilities that exploit opportunities for water-based recreation, provide more effective protection of buildings from flooding, and allow for the use and reuse of stormwater for water supply. All these considerations involve more complex situations than encountered in the past, and a concurrent withdrawal from traditional empirical approaches and adoption of the newer simulation techniques that are based on more scientific principles may be expected to grow.

Illustrative of the complexity of urban runoff is the finding from observations at residential catchments in Sweden that: Roughly speaking, one may say that the first third of the runoff volume contains 44% of the total pollution amount, whereas the last third contains only about 23% of the total pollution amount. The report for

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[36] McPherson, M.B, (Editor). July 1976. Utility of Urban Runoff Modeling. *ASCE Urban Water Resources Research Program Technical Memorandum No. 31.* ASCE, New York, New York, 126 pp. (NTIS No. PB 261 460.)

Australia notes that: Until recently it had not been generally appreciated that an urban catchment has a very complicated water quantity and water pollutant balance. Because significant interaction exists between water distribution networks, wastewater sewerage and storm drainage systems: Complete and meaningful balances cannot, therefore, be achieved without thoroughly considering the origin and movement of all water and pollutants in an urban catchment. Comprehensive models, or combinations of models including the type described in this section, are the only realistic tools available for the analysis of such complex questions. Or, as noted in the Canadian report, there is a need for an entire hierarchy of urban runoff models. Various applications require different models having certain features and belonging to various levels in the model hierarchy.

Also noted in the Canadian report was that, for examples cited, models not only contributed to a more rational design but, in many instances, led to significant savings in drainage costs.

Model Limitations

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. Perhaps more obvious advances have been made in water supply and receiving water hydrology simply because these are essentially supra-metropolitan. Another factor is undoubtedly the fact that they have benefited from advances in more traditional aspects of hydrology.

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.

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. Because of the very limited amount and kind of field data available, just about all sewer application 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 is not nearly enough. Credibility requires at least token calibration using some local rainfall-runoff-quality data.

Calibration and validation is further confused in some nations by the fact that much of the field data available are for partially sewered catchments, where flow is measured in receiving watercourses, than for totally sewered catchments. (That water quality samples may have been taken for only a fraction of these gauging 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.

The need for model validation and calibration using local field data would be vastly reduced, and in some applications practically eliminated, if the type of national indicators by zones noted in Figure 25-5, Section 3, was available for specific models. We emphasize the need for local calibration because a national indicator capability does not exist in any of the 12 countries whose reports we are reviewing, and because we are extremely doubtful that this capability exists elsewhere. Supporting our contention 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 that 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.[37] All this will take some time.

Progress in hydrological modeling inevitably appears to involve more complicated procedures for the designer to implement and more information to be gathered. It is vital for the researcher to be aware of this and to ensure that recommended improvements are truly beneficial. For example, the present use of the United Kingdom RRL method is probabilistically unsound and too simple in terms of scientific hydrology. But unless a new method can be shown to give more accurately sized pipes and less costly protection against surface flooding, no amount of technical elegance will persuade the engineering profession to adopt it. It is this reluctance to accept anything that appears more complicated than is considered necessary that is sometimes responsible for recommendations that we return to simpler techniques. Urban hydrological modeling in the United Kingdom continues to be geared primarily to the improvement of sewer design methods. The common aim is to seek a compromise between the mainly old, established, easily applied but theoretically unattractive methods, and the highly complex analytical models based on physical laws.

Buttressed by several of the national reports, once more we are impelled to reiterate that relatively few runoff-quality field gaugings 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.

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[37] Overton, D.E. and M.E. Meadows. 1976. *Stormwater Modeling.* Academic Press, New York, 358 pp.

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 database 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.

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 North American 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.[38]

A limited comparative study of models in Canada is cited in that national report, and although the results are hardly universal, they do give some indication of levels of reliability currently achievable: On the average, about 70% of the simulated runoff volumes and peak flows, and 85% of the times to peak, were within ±20% of the observed values. The tests were on data from catchments with a single gauging station.

Storm Characterization

Control of flooding and water pollution must be based on probabilities of occurrence because of the randomness of precipitation. Noted in the French report is the small amount of research that has been carried out anywhere in the world on the temporal and spatial characteristics of storms that are relevant to urban drainage systems. Research on storm movement and quantification using radar-augmented rain gauge networks has been cited in the reports for the United States and the United Kingdom. Arguments have been presented for using actual records rather than synthesized storms for large-investment projects in the United States. An apparently reasonable compromise has been made in France, by studying urban runoff model transformation sensitivity to rainfall parameters, in order to estimate their relative importance in the transformation and to retain the most important ones for a design rainfall definition.

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[38] Proctor and Redfern, Ltd., and J.F. MacLaren, Ltd. September 1976. *Storm Water Management Model Study*. Volume II, Research Program for the Abatement of Municipal Pollution under Provisions of the Canada-Ontario Agreement on Great Lakes Water Quality, Research Report No. 48, Environment Canada, Ottawa, 148 pp.

The design rainfall question would be resolved with a high degree of reliability on a national scale if a nationwide program incorporated the features of Figure 25-6, Section 3, including evaluation of significant historical storm data, by zones. There being no nation having such a program, expedients must be sought by default. As succinctly stated in the French report: Without design storm models there would be limited interest in the study of drainage projects for ungauged watersheds (using newer runoff simulation tools).

Practically all of the national reports cited studies in which a variety of attempts have been made to synthesize suitable storm rainfall for planning and design applications of various tools of analysis. As carefully elaborated in the report for India, among several, selection of the frequency of drainage system overloading in terms of a design rainfall is largely an economic question. Human life is seldom threatened by the flooding of urban drainage facilities. Such facilities are designed so they will be overtaxed infrequently and provision for near-complete protection from flooding can only rarely be justified. In terms of actual objective functions, the mean frequencies of occurrence of flow peaks and volumes and water quality constituent amounts is the issue, not frequencies, actual or synthesized, of the input rainfall. Widely recognized is that because there are inherent non-linearities in most methods for processing inputs for linear models, and dynamic models are non-linear by definition, the statistics of the rainfall input may differ appreciably from those of some or all of the arrays for runoff and quality characteristics. Thus, the input to runoff models, rainfall, may become the last element of outright empiricism to be placed on a more scientific footing.

Concluding Remarks

In Canada: The state of the art in urban hydrological modeling seems to surpass the available calibration/verification base. The ultimate goal of the creation of a good urban water resources database remains, therefore, worthwhile and necessary. The lack of urban runoff data seems to impair progress in the development, testing, verification and calibration of runoff models. Tendencies to substitute non-calibrated model results for actual field data, without any verification attempts, are showing up in engineering studies. Such a trend is undesirable and detrimental.

Commenting on the recent emergence of interest in urban hydrology in Australia, we are cautioned that a careful watch must be maintained to ensure that the quality of data is adequate for future analysis purposes.

A statement in the report for France is echoed throughout the others: Very quickly it became evident that the main problem in the advance of urban hydrology was the absence of good data. Theoretical research on modeling of hydrological phenomena has very quickly exceeded the data usually available.

Complete automatic control for abatement of pollution from combined sewer systems is under intensive development in the United States. A wide variety of models are involved.

Mentioned in the subsection above on Components of Urban Runoff Models was the Stormwater Management Model (SWMM). To the transport portion of the version 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 includes listings of the new computer program subroutines that have been added for solids transport characterization.[39]

We close this report with some important conclusions and recommendations from the report for India:

> *Design of urban drainage systems in India is based on the 'rational formula' using arbitrary assumptions concerning the duration and frequency of rainfall and the coefficient of runoff. The use of the rational formula may be justified by the lack of adequate continuous records of precipitation and streamflow. Yet, there is a vital need for rationalization and standardization of design procedures based on engineering and economic considerations.*
>
> *Simulation models are useful in the analysis of complex drainage systems where storage, pumping, silting and quality control are involved, and hence in the economic design of complex drainage systems. The type of drainage system simulation model that is suited to urban areas in developing nations needs to be identified, and computer programs suitable for applications need to be developed. Other mathematical models may also need investigation.*
>
> *...There seems to be a need for determining the type of data needed for urban drainage design in India and other developing countries, for designing and operating supportive short-term and long-term data networks.*

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[39] Sonnen, M.B. June 1, 1977. Abatement of Deposition and Scour in Sewers. Final Report to U.S. EPA Storm and Combined Sewer Section, Water Resources Engineers, Inc. 710 South Broadway, Walnut Creek, California 94596, 114 pp.

Chapter 26

EROSION AND SEDIMENT CONTROL

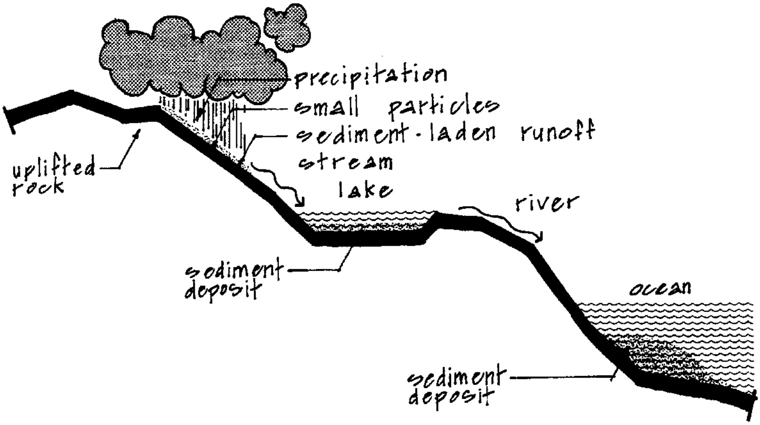
1978
Objectives, Principles & Design Considerations
Published Jointly by: ULI; ASCE; NAHB

FOREWORD

Publication of this report under the joint auspices of the American Society of Civil Engineers (ASCE), the National Association of Home Builders (NAHB), and the Urban Land Institute (ULI) is a continuation of the unique cooperative effort that resulted in a previous jointly sponsored publication, *Residential Streets: Objectives, Principles, and Design Considerations*. This report is a second jointly sponsored and reviewed report that represents the attitudes and concerns of the three organizations on a subject of common interest and importance—the setting of objectives, principles, and design considerations to be applied to the development of stormwater management systems to serve residential communities.

The content of this report evolved, as did its predecessor, from an assessment of current practices to a guide toward a more creative and thoughtful approach to stormwater runoff management. While not rejecting past practice, it clearly identifies and articulates a new underlying philosophy and approach that diverges significantly from the past.

It is hoped that this report will stimulate communities of all sizes, in all locales, to rethink current stormwater management practices and to adopt those that are responsive to local conditions and supportive of the basic objectives and principles presented herein.



Erosion-Sedimentation Cycle

1.0 INTRODUCTION

1.1 The Problem

Erosion and sediment movement and deposition are parts of a natural cycle in that landforms are built up, worn down, and built up again.

The process begins when rock is raised by geological forces. The rock is broken down into smaller particles through the action of rain, frost, plants, and animals. These particles are carried away by runoff from rainstorms, by the wind, and to some extent by glaciers and landslides. Settling out in the lower areas, the particles eventually may be reconsolidated into rock and uplifted to begin the cycle again.

Storms, earthquakes, avalanches, and tidal waves may cause rapid rates of sediment movement, even in places where man is not involved. Most of the time the cycle is slow, thereby providing enough time for nature and special segments of the ecosystem to adjust to the changing landscape. Man is a participant in these adjustments.

Increased erosion and sediment movement rates caused by man are superimposed on the natural rates of the cycle. Often the impact of these superimposed erosion and sediment rates causes nature's ability to harmoniously readjust the cycle to be overtaxed. The overtaxing of nature due to urban erosion and sediment movements is the problem.

Farming and construction often cause a large increase in the rate of erosion. Grazing, plowing, seeding, and harvesting change the types of vegetation found on the land; stripping, grading, paving, and storm drainage construction similarly change and speed up the natural erosion and sediment cycle. The erosion-sediment movement rate may accelerate or diminish over time, but the environmental systems that are affected will adapt to the changed rates. The erosion-sediment movement rate should be managed so that the resulting environmental adaptations will be beneficial and will not overtax the ecosystem.

High, localized erosion rates can result in the loss of soil and its many elements required to support vegetation from areas where it is needed for embankments, construction areas, or for farming. Hand in hand with high erosion rates are high rates of sediment transport and deposition in areas where it is not wanted-on roads, and in ponds, reservoirs, streams, rivers, and harbors. Excessive sedimentation in lakes and streams can reduce or destroy their aesthetic and practical values for recreation, flood control, and water supply; it can obstruct navigation by large and small vessels; and it can cause the loss of both commercial and recreational fishing activities by covering and destroying food sources.

When man's activities increase the rate of erosion and sedimentation, the effect of this change must be evaluated. If the changes have adverse impacts, steps must be taken to limit erosion, usually at the source.

Sometimes the changes caused by erosion and sedimentation can be modified by reconstruction or regrading, re-fertilization, or dredging. Sometimes modification is not a reasonable remedy.

In the United States it is believed that about three-fourths of man-caused erosion is from agricultural and mining activities, and less than one-tenth is from construction. About one-half of the sediment from construction activities is caused by the public sector in its work on highways, utilities, and other public facilities projects. While the principles and practices of erosion and sediment control as given in this report may apply to public works, the report is concerned only with problems that may arise as a result of residential housing construction.

The premise of this report is that erosion and sediment problems usually can be reduced to an acceptable level without placing undue burdens on the homeowner, the homebuilder, or the community. The best solution may involve alternative methods and timing for use of land.

1.2 New Opportunities

Every parcel of land, regardless of size or location, is part of a larger hydrologic unit or catchment. Ideally, the erosion and sediment control plan for each development should be based on and integrated with a plan for the entire catchment in that it lies. Only in recent years have data collection and the state of the art even begun to make this possible. In the absence of catchment plans, new approaches to residential land planning and development have nonetheless made it possible to find more creative and effective solutions to the problems of erosion and sediment control. With these approaches, the effects of urbanization on erosion and sedimentation problems can be minimized, and opportunities for developing and implementing overall plans improved.

Before 1945, most residential construction involved relatively small areas of land. Urbanization of an area generally took place on a lot-by-lot basis, with few large-scale subdivisions. The pace of urbanization was slow, and the cumulative impacts of erosion and sedimentation from residential development were small and negligible relative to those from other land uses.

After 1945, the pace and patterns of development changed radically. Large areas of land began to be subdivided into lots, with construction taking place on many or all of the lots at once, often with complete stripping of vegetation and obliteration of natural site features, resulting in dramatic changes in drainage behavior.

Starting in the late 1950s, some residential developments that clustered dwelling units made it possible to retain undisturbed common open space. Such land planning concepts have provided many opportunities for improved erosion and sediment control.

Following the construction phase, the landscaping normally undertaken stabilized the area further and reduced the erosion and sedimentation rates, sometimes to below the previous levels of the natural cycle.

Land use planning and acceptable methods of conservation, construction, and maintenance must be practical to achieve maximum erosion and sediment control benefits. Each site has its own characteristics, constraints, and critical areas. The challenge to the residential construction team of planners, developers, engineers, and public officials is to preserve the desirable features of each site and protect off-site values to a reasonable degree, while maintaining the flexibility that will permit reasonable construction of an attractive end product.

1.3 Using This Report

In the next 25 years the United States could spend more money for erosion and sediment control than has been spent during its entire history. On the other hand, if the development community works with nature, it may be possible to achieve more and spend much less.

Attempting to reshape nature can be costly. To work efficiently with nature, man must better understand natural regimes, the impact of changes, and what changes are environmentally acceptable in order to maintain an acceptable quality of life.

For the concepts in this report to achieve wide application, institutional changes are needed through and beyond the design professions and regulatory institutions of government. These include changes in the perceptions of financial institutions, changes in the legal profession as it relates to insurability concepts, and changes in the pre-existing body of land use law regarding rights, responsibilities, authorities, and liabilities. Existing constraints upon worthwhile actions are real and must be relieved. The importance of encouraging application of new approaches should provide the impetus to achieve necessary changes, and it is hoped that this document will be a step in those directions.

1.4 Basic Concepts

This report covers some *of* the basic concepts for protecting land and water resources against the detrimental impacts *of* erosion and sedimentation during residential construction. The objectives, principles, and design considerations are complex individually and collectively on a variety *of* levels; many detailed considerations have *of* necessity been omitted. The following are some *of* the basic concepts generally found to be useful:

- Erosion and sediment movement and deposition have both beneficial and detrimental effects.
- Sediment movements should not be permitted at rates or in quantities that will cause significant residual damage. Under ideal conditions any change in the nature or amount *of* sediment leaving a site as a result *of* construction should maintain or improve environmental quality when compared to pre-construction conditions. While these ideal conditions cannot always be achieved, the importance *of* environmental quality to man's long-term welfare and survival means that sound judgment must be exercised in establishing allowable rates *of* change from those expected in the natural cycle.
- Strong emphasis needs to be placed on "natural" engineering and land planning techniques that will not only preserve and enhance natural features *of* the land, both on and *off* the site, but also protect them. There are techniques that use and improve the natural processes taking place at a construction site, during and after the actual construction period, rather than ignoring or replacing them with artificial systems.
- There must be increasing recognition that each site has its own set of natural resources, land use limitations, environmental conditions, and occupancy requirements. These factors and their interrelationships vary from site to site within a community, and variations in design standards will be required for achievement of optimum off-site protection.
- There must be continuing recognition of a balance of responsibilities and obligations between individual landowners and the public for the protection of the environment from the adverse impacts of excessive erosion and sedimentation. It must be understood that significant immediate and long-term expenditures for the construction and maintenance of this protection will be incurred by individual homeowners and the community. A balance must be struck in determining the acceptable ranges of damage, in order to avoid restricting housing availability and choice.

2.0 OBJECTIVES

Based upon those concepts, the following objectives are apparent:

- To provide a clear understanding of erosion and sediment control processes and philosophies.
- To demonstrate the impact of these philosophies on an environment for human habitation that maintains a level of quality that stimulates the reaction that life is a rewarding experience.
- To promote realistic achievements consistent with alternative design solutions, environmental quality, and sound judgment.
- To provide a method of determining the type and degree of investigation required for finding acceptable design solutions that maximize environmental quality throughout the anticipated life of the capital investment.
- To encourage design methods and approaches that will result in effective facilities requiring minimum maintenance.
- To encourage the development of new and better understanding of the long-term results of erosion and sediment control practices.
- To encourage rules, regulations, and laws at all levels of government that will be sensitive to the particular environmental conditions and values and human needs associated with each specific location.
- To summarize recent developments for the benefit of professionals not involved in the actual design of erosion and sediment control measures.

3.0 PRINCIPLES

Principles essential for effective soil and water conservation in urban development areas include the following:

- There shall be a minimization of changes in the rate of existing erosion and the amount of sediment movement throughout the life of a project as part of the residential development design process, leading to the preservation of environmental quality.
- There should be a balance between the use of on-site and off-site techniques; legislative and regulatory modifications are needed to achieve this balance.
- Flexibility and creativity are needed in the emerging field of erosion and sediment control.
- Control measures selected should be based on evaluations of costs, benefits, and other needs.
- A well-conceived residential development can result in a reduction or an elimination of erosion and sediment problems that exist prior to construction.

- Overall catchment area plans and objectives are desirable and often help provide a uniform basis for evolving site-specific measures.
- Specific requirements to prevent erosion and sedimentation should recognize the issue of risk, most particularly the probable frequency of events for that protective measures are provided. (This is especially true for temporary measures.)
- Measures used will vary in their effectiveness at different scales. The suitability of measures for specific applications should be evaluated.
- Long-term maintenance is an integral aspect of erosion and sediment control design.
- The timing and location of construction affects the degree of risk and the effectiveness of measures required to control erosion and sediment deposition.
- The fundamental consideration in erosion and sediment control is the protection, maintenance, or establishment of ground cover, except in arid areas where permanent vegetative covers are impractical.
- There should be a balance between the measures required of private developers, public and private utilities, public works, and agricultural and extractive activities, in relationship to their proportionate share in causing erosion and sediment problems.

4.0 DESIGN CONSIDERATIONS

4.1 Risks and Costs

Before an erosion and sediment control program can be planned for a residential development project, two basic questions must be answered. What types of effects, if any, are likely to occur on and off the site? If changes are to be expected, what levels of change-both temporary and permanent-can be accepted without causing adverse environmental impacts?

The first step in this process is to determine whether or not off-site environmental damage is probable. This will determine the extent of public agency involvement. The developer's self-interest will determine his use of preventive or remedial measures to limit adverse on-site effects.

If on-site and off-site damage is unlikely, it is wasteful to expend design and construction effort on control measures because there would be no demonstrable benefit. If damage is foreseeable, it is important to determine its probable magnitude, character, and cause so that the most effective methods may be selected to avoid or minimize the impacts.

The determination of what constitutes an acceptable level of off-site environmental change is subjective. Laws, regulations, advice, and liabilities all affect such decisions, as do the attitudes of adjacent landowners.

The level of acceptable adverse environmental effects will vary, depending on the site, the climatic conditions, and the uses to that the affected land and its environs are or will be put. As an example, a very thin layer of fine sediment deposited on grass may have no adverse effects, but a similar deposit can cause hazardous driving conditions on a roadway.

Although deposited sediments can cause damage, they may also be beneficial. Some stream channel nourishment by sediments is essential for channel stability. If the sediment supply is cut off, channel bank and bottom erosion will be aggravated. Some areas require regular sediment replenishment for other reasons.

Caution: It is wise to avoid excessive reduction of erosion and sedimentation that can have disastrous effects upon stable biotic regimes. Although the effects of under-control of erosion are often visible immediately (as sediment deposits), over-control results in more subtle damage, sometimes visible only much later (as long-term channel erosion or loss of floodplain productivity).

The many variables by that subjective decisions are made should be considered in determining acceptable levels of environmental change. Precise evaluation is not possible. For this reason, experienced professional and scientific judgment is required to set reasonable limits for allowable sediment movement, appropriate for the areas involved.

4.2 Determination of Risk

It is important to quantify the degree *of* each risk involved in any construction project-not just the financial risks-but the probability that a damage-producing event *of* some specific type, such as a storm or a flood, may occur during the construction period. Risk must be determined before a proper erosion and sediment control plan can be prepared to identify the kind, amount, and extent *of* measures necessary to reduce or eliminate foreseeable damage. As mentioned previously, the level *of* protection required need not be set so high as to reduce the sediment leaving the site to levels lower than under natural conditions.

Erosion rates are directly related to the intensity, duration, and seasonality *of* rainfall and wind. It is usually possible to establish the probability that an event *of* a given magnitude may occur once during the period *of* construction. This may be expressed in two ways, either as a probability or as a recurrence interval. The probability *of* a given event states the odds that it will occur in a given year. The recurrence interval indicates the average worst severity *of* a kind *of* event expected during a stated period *of* years. The longer the construction period, the greater the probability becomes that some given event will *oc*cur. For example, there is a very *high* probability (96.9 percent) that one or more 2-year rainstorms (50 percent annual probability) will occur during a 5-year construction period. In fact, such storms could be expected to occur on an average *of* every other year. Similarly, there is approximately one chance in 20 that the much more damaging 1 DO-year storm will occur within that period. There is a very *low* probability that a 1 DO-year rainstorm (1 percent annual probability or one chance in 100) would take place during a 1-year construction period. Similar exposure risk determinations can be made for wind velocities and directions, droughts, and other phenomena.

It clearly is possible to determine the level of risk that a given magnitude and type of event will occur during a specific construction period. Consideration of possible types of off-site effects will then determine the acceptable level of risk. Specific measures that will protect off-site areas can thus be designed and installed.

Without such an analysis, even a brief or cursory one, there can be no quantifiable basis for action decisions, uniformity of action requirements, or evaluation of the balances between control costs and risks for the desired level of erosion and sediment movement control. There must be a sound and justifiable basis for trade-offs.

Determining the level of risk requires consideration of local and regional variations in climate, soils, topography, vegetation, and many other factors. A plan that is reasonable and cost-effective for one project may be arbitrary, expensive, and ineffective at another.

For example, let us compare two housing projects, one in Southern California and the other in the Maryland Piedmont. Both are assumed to have the same area and to be constructed between May and October.

For the California project, the risk of water erosion is almost nil because of extremely low rainfall during the construction period; for the Maryland project, however, there is a substantial risk that a 2-year storm (approximately 3.2 inches of rain in a 24-hour period) will occur. Obviously, a sediment basin or another water-related erosion control measure would not be needed for the California project but *should* be used for the Maryland project.

It has been suggested that the "design" rainstorm (or winds) be selected as an event that can be expected (statistically speaking) to occur once in a period of time equal to some multiple of the planned construction period (for temporary measures), or in the intended life of the final land use (for permanent measures). This approach provides a starting point, but other factors such as environmentally sensitive areas or downstream life safety may also have to be considered.

In addition to regional and climatic variations, conditions specific to a particular site influence the degree of risk involved. To cite another example, there is little risk of damage from erosion on a site where there is only exposed rock with very little surface soil.

Sandy soils generally have high water and wind erosion potential as compared to clay soils, but the resulting sediment is composed of large particles that are relatively easily removed from the runoff; the actual risk of off-site damage is therefore low if just minimal precautions are taken, such as diversion of runoff away from steep slopes and the installation of properly designed temporary sediment traps.

Clay particles suspended in water are visible far out of proportion to the actual amount of material present, and their removal is almost impossible.

4.3 Analysis of Cost

From the foregoing discussion it is evident that the determination of acceptable levels of risk involves a number of factors that must be balanced against each other to arrive at realistic final decisions.

When an acceptable level of risk has been determined, a preliminary erosion and sediment control plan should be prepared. The cost of implementing the plan should be estimated, and the cost of each measure suggested examined to compare the degree of protection required with the degree that probably will be obtained. This comparison should consider losses that cannot be measured in dollars. Although construction of a large sediment basin may trap a certain amount of sediment, the total cost of the basin will include maintenance, the loss of vegetation, scenic value, and other values at the basin site.

If the cost of necessary protection exceeds the anticipated damage or potential loss (in either monetary or environmental terms), the basis for the plan should be reexamined and a more cost-effective solution should be sought. Reexamination should not be limited to the proposed control measures; it may also be proper to reevaluate the required level of protection, or the ability to repair damage after construction is completed.

4.4 Relationship Between Erosion Control, Sediment Control, and Stormwater Management

Erosion control, sediment control, and stormwater management are different elements of one problem.

Erosion control attempts to reduce the loss of soil from a given site. A good site-grading plan will tend to reduce erosion losses by minimizing disturbed areas, and it should be prepared with that specific intent.

Sediment control attempts to prevent or reduce deposition damage both on the site and off the site. It seeks to trap eroded soil particles that are being transported. Sediment control measures must be integrated with stormwater management measures, and they often become part of the permanent stormwater management facilities.

Stormwater management serves to prevent or reduce damage to property, streams, lakes, and rivers. It involves erosion control; properly designed and constructed channels and streams receiving managed runoff are less subject to damage than would otherwise be the case. Unpaved grassy areas around catch basins (inlets) can be made to function as sediment control basins. After construction at a site is completed, some erosion will continue to occur and a portion of the resulting sediment will be deposited in the stormwater management structures or downstream.

[See Urban Land Institute, American Society of Civil Engineers, and National Association of Home Builders, *Residential Stormwater Management: Objectives, Principles & Design Considerations* (ULI, ASCE, NAHB: Washington, 1975).]

4.5 Temporary Versus Permanent Measures

Significant differences exist between temporary and permanent erosion and sediment control measures. Although they have similar functions, there are differences in design approaches, methods of construction and maintenance, and cost benefits.

Temporary measures are designed to have a short life-typically, as implied, for the duration of the construction period. They may be only "field expedients" used for a matter of days. Because of their short life, they need not be designed to last for many years with minimal maintenance, nor need they be built of highly durable materials. Nonetheless, they must receive regular maintenance during their period of use to remain effective. Such measures may have a low initial cost but may have relatively high maintenance costs if frequent or intense storms occur during the construction period.

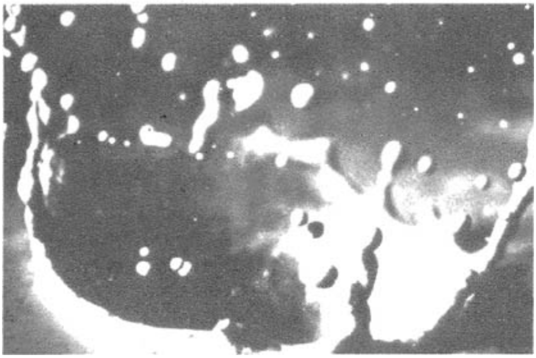
Permanent measures are intended to remain in place for 50 years or more with a minimum of maintenance, so they must be designed and constructed of durable materials with lifespan in mind.

4.6 The Nature of Erosion Processes

The most widespread, persistent, and perhaps most severe type of erosion is that caused by water. Wind erosion tends to be drought-related and of greatest significance in arid regions. There are five types of erosion by water: raindrop or splash erosion, sheet erosion, rill erosion, gully erosion, and streambank erosion. Each type may be aggravated when the natural landscape is disturbed.

Raindrop or splash erosion is the initial phase of water erosion. The impact of raindrops supplies the initial kinetic energy that starts soil erosion. Raindrop impact has a high capacity for detaching soil particles but only a low capacity for transporting them. The amount of soil detached increases with intensity, velocity, and drop size. Raindrop impact splashes small amounts of soil but tends to compact the soil mass, reducing its ability to absorb water and, in some cases, increasing its resistance to erosion by tractive forces.

The second type of water erosion is sheet erosion that is characterized by the general removal of a fairly uniform thin layer of soil from the land surface. This type of erosion is associated with runoff that often is referred to as sheet flow, due to its characteristic of flowing like a sheet over the ground. In contrast to raindrop impact, sheet flow usually has a low detachment capacity and a higher transport capacity. Much damaging erosion and sediment movement occur this way.



Splash Erosion

Rill Erosion

Rill erosion occurs when sheet flow moves down fairly steep slopes, forming small channels with depths up to 1 foot, fairly evenly spaced across a slope. When slopes are of loessial soils (unstratified loams chiefly deposited by wind) and are improperly finished, rill erosion can be unusually serious.

Gullies are an advanced form of soil erosion resulting from concentrated stormwater flow. Uncontrolled runoff in rill channels can continue to remove soil, often rapidly, and may turn into gullies up to 100 feet or more in depth. Gully erosion often moves more soil than sheet erosion. Gully erosion rates are highest for silty soils.

Streambank or channel erosion is the removal of soil from streambanks and stream bottoms. Clearing protective vegetative cover from banks, straightening and realigning channels, and construction projects that substantially increase the rates and volumes of runoff within the watershed can result in degradation and channel enlargement, and an increase in sediments transported far downstream from the disturbed area. For this reason, channel alterations should be avoided whenever possible; they should only be made when it becomes necessary to safely transport expected flood flows.

Wind erosion and its related transport and deposition of sediment should be considered at some locations at certain times of the year. Under certain conditions, soil particles are removed, transported, deposited, and mixed by wind, in a manner similar to the action of water erosion. Extensive areas of loess soils have been formed by wind erosion and subsequent deposition.

Soils newly deposited by wind, especially in stream channels, may be particularly susceptible to later erosion by water. Soil and other materials eroded by wind may be directly blown into streams, lakes, reservoirs, and other areas where they may cause environmental problems. The amounts of soil locally removed by wind usually are small compared to those eroded and transported by water.

Wind erosion is a serious problem in much of the Great Plains, the Columbia River Basin, some parts of the Pacific Southwest, the Colorado River Basin, the Gulf Coast, and the Atlantic Seaboard. Wind erosion is primarily of concern in arid and semi-arid regions, but it can be a problem elsewhere.

Strong turbulent surface winds that have sufficient energy to move or lift the particles cause wind erosion. Initial movement of the most erodable soil particles begins with a minimum wind velocity of as little as 10 mi les per hour; the soil particles are then transported by saltation (bouncing), surface creep (rolling), and suspension (borne by air).

The intensity of wind erosion will increase with time and distance across a large area. The rate of soil movement in surface creep and saltation varies with the velocity of the wind.

Deposition of airborne soil particles takes place when the wind subsides, usually far from their origin.

While all of the described wind-related erosion and sediment processes may occur simultaneously at a given construction site, they usually do not occur at the same time at any particular point on the site. The processes generally are mutually exclusive at any particular point, as a given soil particle cannot be undergoing both erosion and deposition at the same time. Erosion control measures are usually selected to reduce the relative rate of occurrence for each process.

4.7 Factors Affecting Water Erosion

Climatic factors, soil erodibility, slope length, slope gradient, and vegetation are the primary factors involved in the water erosion process.

4.7.1 Rainfall

The climatic factors important in determining soil loss include the amount, intensity, and frequency of rainfall, and especially its seasonal distribution. The amount and intensity of rainfall relate to the rate of runoff, that detaches and transports soil particles downhill and downstream. The frequency with that some given amount of rainfall occurs determines how often the effects associated with it will be felt.

4.7.2 Soils

The main soil properties that determine erodibility include particle size distribution, clay and organic content, pore water chemistry, soil structure, permeability, specific gravity, and root structure; some of these characteristics will be discussed briefly below. The erodibility of bare disturbed soil or subsoil can be estimated if these properties are known.

Particle size distribution refers to the relative proportion by weight of the various sizes of soil particles found in a sample of the soil. Soil texture has a direct effect upon the water infiltration rate and permeability of a given soil.

Soil erodibility decreases as the organic matter content increases. The organic matter decomposes, and the resulting soil humus is important in producing organic clods that result in a less erodable soil structure.

Porosity, capillarity, and water content also affect soil erodibility. Granular non-cohesive soils that contain large amounts of fine sands and silts with little clay and organic matter are usually more erodable than soils with blocky or massive structures.

Soil permeability is the ability of soil to transmit water, horizontally or vertically. Soil permeabilities vary from a high of 60 inches per hour to virtually zero. Soils with higher clay contents are generally less erodable than those with lower clay contents, even though the former are less permeable. Other properties not discussed here are responsible for these differences in erodibility.

4.7.3 Slope Gradient and Length

The rate of erosion occurring on a slope during a rainstorm increases roughly in proportion to the square of increasing slope steepness because of the proportionality to the velocity of runoff flowing down slope. The tractive force of flowing water determines the size and number of soil particles detached and transported.

Total soil erosion also increases with the length of down slope distance, due to the relative cumulative increase in runoff volume and hence in runoff flow velocity. As an increase in velocity causes faster erosion, the increased volume also results in a higher sediment transport capacity, so that more soil particles are carried away.

4.7.4 Vegetation

Well-established and well-maintained vegetation is a major deterrent to soil erosion because it shields the soil from raindrop impact and decreases flow velocity by increasing flow friction (resistance). Root systems may increase soil porosity, permitting greater water infiltration and reinforcing the soil mass. Stems, stalks, leaves, and roots break up flow patterns, increase flow friction, and cause deposition of some soil particles. Plants also remove water from the soil by transpiration, so the soil can absorb more water, potentially decreasing the amount of runoff. It is important to recognize that planting and maintenance of vegetation is practical only on slopes flatter than three horizontal to one vertical, and then only where there will be regularly distributed rainfall.

4.8 Factors Affecting Wind Erosion

The biggest factor affecting wind erosion is soil cohesion, as non-cohesive soils are most susceptible to it. Climate, other soil characteristics, surface roughness, unsheltered length of exposed surface, vegetative cover, and surface traffic also are related factors. As previously mentioned, wind erosion is an arid or semi-arid area phenomenon.

Precipitation, temperature, humidity, and wind are the main climatic factors. Wind erosion generally decreases as soil moisture increases, because the water film surrounding the particles tends to increase cohesion. Evaporation and transpiration rates, of course, are affected by temperature, humidity, and wind.

Alternating dry and wet periods, as well as freezing, and thawing, may increase soil susceptibility to erosion by loosening the soil and breaking it down into smaller particles.

The speed, turbulence, direction, and duration of wind affect movements of loosened soil particles. Soils containing significant portions of large non-erodable fractions by weight are generally the most resistant to wind erosion; small, loose soil particles are the most erodable, as low wind velocities often move them.

Vegetative cover, both as ground litter and standing plants, reduces wind erosion by breaking the force of the wind against the ground and by serving as an armor to keep moving air from setting soil grains in motion. Vegetative cover also tends to trap particles already in motion.

4.9 Evaluation of Water Erosion

The various methods useful for erosion and sediment control should be evaluated numerically to estimate their probable effectiveness. Quantified performance estimates must be largely based upon local experience and judgment; erosion prevention largely is a matter of avoiding the development of those conditions known to accelerate erosion.

A numerical evaluation process is often referred to as a model. Among the erosion quantification models proposed over the years are the Musgrave Equation that was proposed in 1946 and the Universal Soil Loss Equation (USLE), proposed in 1961 and still being developed. Of these, the most generally accepted is the USLE. None of these models are reliable for estimating general erosion behavior on construction sites, but the USLE is discussed here to illustrate the principles involved.

The USLE applies only to sheet, rill, and inter-rill erosion (it cannot be used to predict gully or streambed erosion), and it applies to large areas of loose soil, bare and exposed for 2 or more years. Frequently, applications of the USLE assume that all soil lost due to erosion will appear as downstream sediments. This assumption ignores the fact that substantial amounts of eroded soil will be deposited where slope gradients decrease or where runoff flow velocities are reduced for any other reason. The USLE is generally too conservative to use in estimating construction site behavior and may lead to over-design.

Quantification of the factors in the USLE is mostly a matter of very coarse judgment. Data are unavailable for reliable determinations, so the results obtained should be used with great caution, with recognition of the uncertainties involved. If they are used to estimate erosion at construction sites, USLE results should only be viewed as relative erosion rates, not absolute or reliable quantities.

The basic form of the equation is:

$$E = RLKSCP$$

Where:

E = soil loss, in tons per acre per year

R = rainfall factor

K = soil erodibility factor

LS = slope length gradient factor

C = vegetative cover factor

P = conservation practice factor

From its form, it is apparent that the USLE applies only to a single, large, homogeneous area, so it is necessary to evaluate separately the soil loss for areas having differing conditions. The number of calculations required for a construction site can be very large, as conditions change not only from preconstruction to post-construction phases, but almost daily.

The Universal Soil Loss Equation was developed for agricultural activities, where the setting normally involves relatively long, regular, gentle slopes, and where changes during the season are relatively predictable. Serious doubt exists about its applicability to short-term construction activities, where steep slopes (caused by grading), small areas, and mixed soil types are involved. Urban construction often includes excavations (such as basements) that produce steep, internally drained areas that trap much localized sediment and may act as sediment basins for surrounding areas. Unwisely, they will often provide support later for foundations or floors. The amount of sediment leaving the site at these areas may be less than that leaving tilled agricultural land.

Frequent reworking of the land in a construction site results in variable slopes with changing gradients and lengths. Much effort would be required to determine properly the best composite length-steepness (LS) factor for a given area, as the weighted changes must be considered separately for various segments of the exposed area.

Subsoil is likely to be more erodable than surface soil, because of natural differences in soil structure and organic matter content. Usually, soil texture is not uniform from the top to the bottom of a soil horizon (layer). It should not be removed from a construction site in its entirety, leaving only the deeper horizons exposed to erosion, although it is common practice to stockpile topsoil on the site for later use in re-vegetating individual lots. Therefore, when determining a K factor (soil erodibility) for the "average" soil at a construction site, much of the topsoil should be considered to be in place. Knowledge of comparative K factors can be useful during development planning, but it is comparative erodibility that really should guide planning and design decisions-not USLE factor values. Shapes of slopes and distribution of topsoil over exposed subsoil should be planned considering comparative soil erodibility, with the most erodable subsoils given' priority for stabilization.

USLE's failure to consider seasonal differences is a serious weakness that detracts from its applicability to construction sites. USLE uses a yearly rainfall erosion index and yearly distribution curves based on long-term averages. The short time period during that the soil is exposed by residential construction activities makes it more likely that either higher or lower rainfall will occur, often leading to large prediction errors.

Erosion can be aggravated by frost action that expands and loosens soil particles, increasing their susceptibility to loss during runoff after thaws. Large quantities of soil may be lost during the spring if runoff from snowmelt acts upon frost-loosened soil, a factor that should be considered in areas subject to heavy snowfalls. Thus, for short-term projects constructed during winter or spring months, the USLE estimates can be significantly wrong.

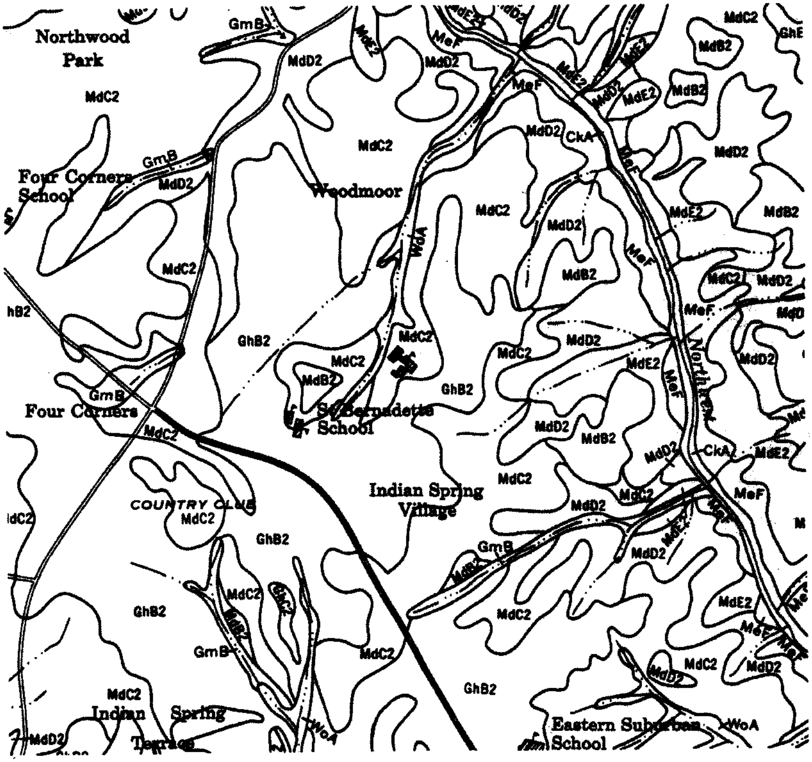
Because every storm has its own sediment movement characteristics, it is difficult or impossible to include this effect in applications of the USLE. Further, most studies conducted using USLE have been undertaken for total watersheds, and data for calculating sediment delivery ratios for small, rapidly urbanizing basins are practically non-existent.

From the foregoing, it is evident that caution is necessary in applying the USLE to construction sites to determine potential sediment movements to streams. Precise results are unobtainable, but the USLE can be used for a rough evaluation of erosion control alternatives. It can provide some insight into various design alternatives at a given site, but it is questionable whether such insights will be an improvement upon basic considerations of comparative soil erodibility.

4.10 Evaluation of Wind Erosion

Evaluation techniques to determine the potential for significant wind erosion are even more imprecise than USLE methods, even where both experience and judgment are available to evaluate the problem; precision should not be expected.

The probable combined influence of variables affecting wind erosion should be considered. A Wind Erosion Equation and instructions for its use are given in U.S. Department of Agriculture Handbook No. 346 (see Bibliography), but the methodology is only somewhat applicable to typical urban construction sites, and its general use is not recommended.



Soil Classification Map

4.11 Erosion and Sediment Control

As previously mentioned, erosion controls are temporary or permanent; most construction sites will employ both types of measures. Although their functions are similar, they differ in design approaches and construction materials. It should be noted that nearly all permanent structural control measures must be built before or during site grading operations, but that vegetation as an erosion or sediment movement control measure is useful only after grading is completed, and even then only during the early part of the growing season.

In any event, construction site grading and erosion control designs should be carefully integrated and reviewed, with attention directed to construction scheduling, minimal grading, phased construction, and the cost-effectiveness of alternate measures for erosion prevention and sediment collection.

Careful site planning can reduce or prevent many erosion and sediment control problems. Various planning and design decisions are discussed below in terms of the major factors that affect erosion.

4.11.1 Rain and Wind

Since rainfall varies seasonally, construction should obviously be scheduled to avoid seasons when heavy rains are expected. Major reductions in potential erosion often can be achieved by such scheduling. Because construction and rainy seasons often coincide in the United States, seasonal scheduling may be impractical, and other erosion control measures may be necessary.

Winds also are seasonal in direction and force in many areas, but low wind periods generally coincide with high rainfall periods. Fortunately, the adverse effects of wind erosion are comparatively negligible at the construction site in contrast with potential water erosion problems.

4.11.2 Soil Erodibility

Disturbance of erodable soils will, under any given set of conditions, increase erosion rates. This fact has two design implications. (1) When possible, less erodable soils should be exposed. (2) The heaviest soil particles may be trapped more easily. Both implications should be considered in site planning, design, and construction.

Sand particles are relatively large and therefore easily trapped. Little sand will leave a construction site if minimal precautions, such as diverting runoff away from slopes and installation of sediment traps, are taken. Clay soils, on the other hand, are usually more resistant to erosion, but fine clay particles that have eroded are extremely difficult to trap and remove from water or air; the finest particles may travel for many miles.

Vegetative Erosion Control

4.11.3 Slope Length and Steepness

The length and steepness of a slope affect the rate at that soil is lost from it through water erosion. The effect of slope length may be reduced by terracing (benching), that encourages deposition of eroded particles at the head of the lower terrace, preventing the transport of particles off the site. This practice is effective, but it consumes land for construction and maintenance of the bench-typically, a strip from 10 to 25 feet wide for each bench-and often leads to disturbance of larger areas. Benching may be essential for slope stability, however, despite land and construction costs. Terraces must be designed to minimize maintenance requirements after construction is complete.

4.11.4 Cover

Cover includes vegetative covers such as grass, shrubs, and trees and other covers such as mulches.

A good, healthy stand of vegetation will greatly reduce both water and wind erosion. Mature sod cover will almost completely prevent water erosion in many circumstances. Mature forests reduce water erosion to as little as one-thousandth of the rate on unprotected disturbed soils. Accordingly, every effort should be made to disturb as little existing vegetation as possible, and to reestablish good cover as soon as possible after grading.

Since it takes time for a live cover to become established, various mulches can be used as a temporary cover. While not as effective as live covers, mulches can provide substantial protection. Straw mulch applied at the rate of 2 tons per acre may reduce soil losses by as much as 98 percent on mild slopes. In addition to providing temporary cover, mulches help retain moisture and soil fertility, encouraging rapid germination and the growth of live covers.

Many non-vegetative mulches are available. Their relative effectiveness has not been fully qualified, but experience indicates their utility and value. Mulches may be asphalt emulsions, paper products, jute cloth, straw, wood chips, sawdust, nettings of various natural and manmade fibers, and in some cases, gravel. If substantial clearing must be done, wood chip mulches can often be manufactured and stored on the site.

4.11.5 Soil Surface

The condition of the exposed soil surface is disregarded in the USLE (that is based on most adverse surface disturbance), but it has a significant effect upon wind erosion and water erosion rates. The greatest reduction of soil loss is obtained from ridges at right angles to water flow or prevailing winds, but ridging is generally impractical on a construction site. A loose soil surface usually will have a high infiltration capacity for a limited depth, resulting in low erosion rates for moderate storms and higher rates for greater storms when the capacity of the soil to absorb water is exceeded and the resulting runoff dislodges the loose soil.

4.11.6 Exposed Area

It seems obvious that the total soil loss will be greater from a large area than from a small one, all other factors being equal.

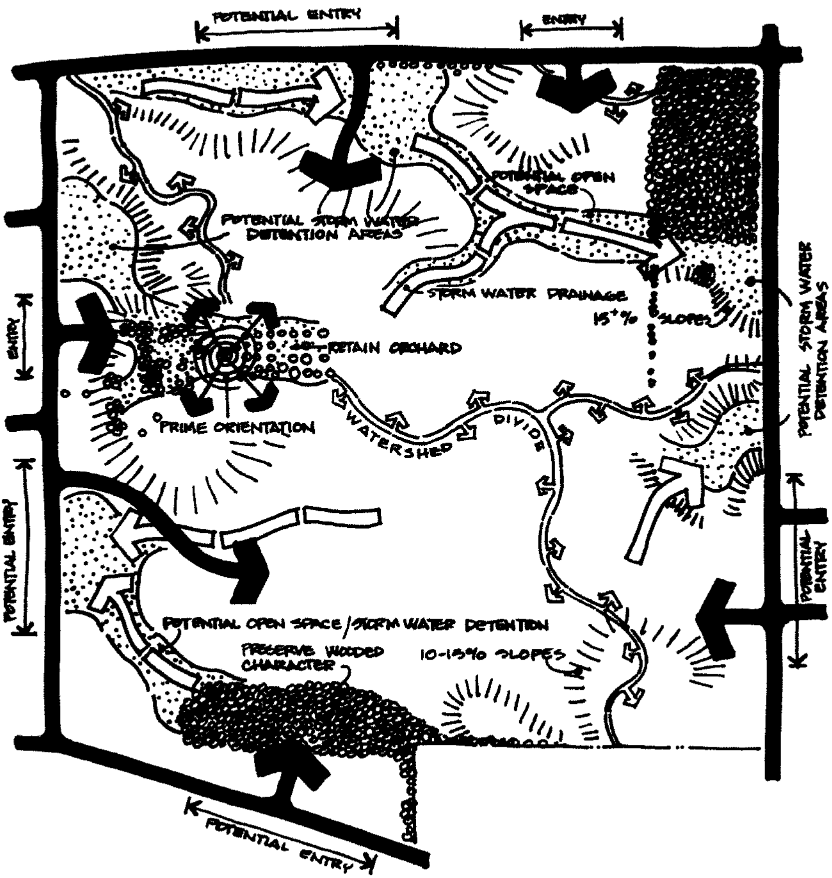
If graded areas are kept as small as possible, soil loss will be reduced. There will be an added degree of protection from the intervening undisturbed areas that will act as both filters and windbreaks. Graded areas that are laid out at or near right angles to the prevailing winds will experience less erosion if the prevailing winds are directly up or down the slope.

The urbanization of farmland may greatly reduce average annual erosion rates. This is especially true of poorly managed farmland, where the soil loss after urbanization occurs may be only a tiny fraction of the agricultural soil loss.

4.11.7 Other Planning and Design Considerations

The most effective method of controlling erosion and sedimentation is undoubtedly careful site planning. Good planning will balance site constraints and characteristics and incorporate the results into a plan that is environmentally, functionally, and economically sound.

A good site plan will preserve the natural site vegetation to the maximum extent possible, and it will limit clearing and grading to the minimum necessary to permit construction of homes, roads, and utilities. Street alignments will follow the natural terrain; buildings will be clustered in fairly compact groups; utility lines will be located within areas that will be graded or distributed for other purposes. Site plans of this type permit the most efficient use of the land and tend to minimize the percentage of the site that will be exposed to erosion.



Site Analysis

4.12 Phasing

The planning of a development may be divided into four phases:

- **Preliminary site investigation** is advisable before land is purchased for development, as the data collected bear directly upon site value. The economic importance of terrain and soil characteristics cannot be reflected in an adjusted land purchase price after the transaction is completed. Even when land is priced on a "take it or leave it" basis, the land developer should prudently determine what special costs he may face. Needed data include such items as availability of water and sewer connections, storm drainage requirements, topography, soil information, flood plain delineation, site geology, and other basic information required for preliminary design, as well as information regarding zoning, applicable laws, ordinances, and regulations. All of this information should be readily available from local planning agencies.

- **Preliminary design** includes the preparation of tentative development design based on the data gathered during the preliminary site investigation, and it should proceed with counsel from various technical specialists. The preliminary design can usually be adjusted to minimize probable erosion and to maximize deposition of eroded particles in ways previously mentioned.
- **Evaluation of the preliminary plan** may result in modification of the preliminary design to emphasize aesthetically pleasing features, to reduce the impact of less desirable features, and to provide protection for off-site areas as necessary. For small projects, phases one, two, and three are often combined.
- **Final design** includes preparation of all final plans and construction specifications; only minor changes should then be made during construction. Final design should be consistent with the combined viewpoints and opinions of all parties concerned-the developer, planner, architect, engineer, landscape architect, and public officials to evolve the best acceptable method of constructing a particular project at a given site, while protecting all interests to the maximum extent possible. Local development review agencies should incorporate the review of erosion and sediment control plans directly into the development review process to eliminate delays, overlaps, and conflicts with local requirements.

In many cases, published data in U.S. Department of Agriculture and Soil Conservation Service surveys will provide useful information for identifying erosion and sediment control alternatives; such generalized information should be verified in the field, however, and often must be supplemented to become a basis for sound decisions.

The soil and geological investigation of the site should be performed in detail appropriate to the size and nature of the project. Gathering information on the soils at the site is one of the first steps. Some common soil-related limitations, in addition to erodibility potential, are poor soil drainage, high shrink/ swell potential, shallow bedrock, low strength, corrosiveness, and compressibility. Information from soil surveys does not eliminate the need for on-site studies of detailed soil conditions and characteristics, and of such potential problems as landslides, flood hazard exposure, or exposure to a variety of other geologic hazards.

Soil erodibility is a consideration in the selection of the planting layer for finishing cut and fill slopes.

4.13 Summary

The most important steps in controlling erosion and sedimentation are briefly summarized below. These steps should be carried out to an extent appropriate for the site involved:

- Study the site and surrounding area and assess soil limitations and suitability of the site in view of the topography, geology, natural drainage, hydrology, prevailing winds, and other factors.
- Identify potential problem soils, if any.
- Select a method of development that will be compatible with site conditions.
- Examine existing and proposed drainage patterns.
- Examine the lengths and gradients of existing slopes.
- Evaluate watershed problems, upstream erosion conditions, and sediment conditions downstream from the construction site.
- Minimize through proper site planning the amount of site grading needed for development and utility construction.
- Avoid removal of existing vegetation insofar as possible.

The following general principles should be recognized through the site design and construction processes:

- Integrate clearing and grading with layout design.
- Keep clearing to a minimum and preserve as much of the existing vegetation as possible.
- Limit grading to those areas involved in current construction activities.
- Limit the time during that unprotected graded areas are exposed to rain and wind.
- Protect disturbed areas by using stabilization measures as soon as possible.
- Plan structural and vegetative measures to control the velocity and volume of runoff, and to provide windbreaks where needed.
- Divert and convey surface runoff safely through the area with structural measures such as diversions, storm drains, channels, or waterways. Ensure runoff velocities high enough to prevent unwanted deposition and low enough to prevent erosion (although this is almost universally impossible, it is the objective of practical design compromises).
- Construct sediment traps and basins to trap sediment on site when necessary.
- Stabilize exposed soils by adhering to time limits set out in the schedule for site grading, seeding, and mulching.

- Assure adequate maintenance of structural measures and of all plantings.

4.14 Water Erosion Control Methods

If analysis indicates control is necessary or desirable, water erosion control methods will include soil stabilization methods (vegetative and other), runoff control, and structural controls. To achieve the best results, these types of measures should complement each other. Erosion control measures reduce the duration of soil exposure and perform one or both of the following two functions: protect the soil by shielding it, and hold the soil in place. These functions may improve soil capacity to absorb stormwater runoff and thereby reduce the amount of overland runoff and its power to erode soil materials. The staging of grading operations and immediate re-vegetation will help minimize the exposed soil area at anyone time. The control of surface runoff may be accomplished by interception, diversion, and safe disposal of runoff, in coordination with staged construction activities, designed grading methods, and the preservation of natural vegetation.

Protection of exposed soil from raindrop impact and subsequent erosion is obtained by applications of organic mulches, rock, chemical additives, sheets of jute netting, planting, and paving. The choice of which material should be based on economy, the future use of the area to be protected, and the degree of protection required. Details of some measures used for erosion control may be found in U.S. Environmental Protection Agency guidelines and various standards and specifications prepared by the U.S. Soil Conservation Service, Soil and Water Conservation Districts (in cooperation with local government agencies), and other sources, some of which are listed in the Bibliography (see page 62).

In some instances it may be possible to achieve an acceptable area wide average erosion rate by stabilizing existing off-site areas to reduce their erosion potential during a short on-site construction period of high sediment yield.

4.14.1 Stabilization

Temporary measures typically should be used if the soil is to remain exposed for more than 30 days. Permanent structural measures must be installed prior to or during active construction, not after.

4.15 Vegetative

Fast-growing annual and perennial grasses may be used on partially completed construction projects to protect them from erosion for short periods of time. Completion of final grading during seasons unfavorable for permanent vegetative stabilization may necessitate temporary structural surface stabilization. Certain areas such as drainageways, cut and fill slopes, borrow pit areas, excavations, and soil stockpiles often require immediate structural surface stabilization, although this need may be temporary.

The need for temporary stabilization generally should be avoided, as it is costly and rarely can be salvaged or incorporated into final protective measures.

Permanent vegetative stabilization should be long-lived and require minimal care or maintenance. Grasses and legumes are generally superior to shrubs and ground covers, because of their more complex root systems that encourage formation of a water-stable soil structure. In addition, their leaves and stems protect the ground against erosion from wind and water. The selection of plant material should be based upon specific site growth expectancies, the purpose of the planting, and foreseeable assured level of maintenance activities. Any representation that a particular plant material is proper for a given slope, soil condition, and maintenance expectancy should be viewed skeptically unless the performance of comparable installations in the general area provides certainty.

The more fertile surface layer of the soil, if present, is usually removed and stockpiled during grading activities. Typically, exposed subsurface layers are less fertile, have lower organic matter content, and are more susceptible to erosion than surface soil horizons. For this reason, the physical and chemical properties of newly exposed soils should be considered. The principal chemical factors are nutritive elements such as nitrogen, phosphorus, magnesium, potassium, and occasionally certain trace elements. Systematic soil analyses of various horizons performed during the site investigation can be helpful in estimating the plant requirements and the proper application of fertilizers and other conditioning materials. For plant growth, factors such as soil texture, soil drainage, porosity, degree of aeration, structure, degree of compaction, soil temperature, slope gradient, pH, available nutrients, and exposure to the sun and wind must be carefully considered.

The steeper the slope, the more drought-resistant plantings should be. South-facing slopes will usually be drier than others. Two or three fertilizer applications may be necessary to ensure establishment of good stands of grass and legumes. Deep fertilization before the addition of topsoil on a slope may increase the potential for long-term growth.

Cut Slope Erosion Control

Manmade cut and fill slopes in construction projects steeper than three to one a0re often impractical to stabilize in order to prevent excessive long-term erosion. Maintenance equipment can be safely operated on slopes with a maximum gradient no steeper than 5:1, and it is hard to perform difficult maintenance even on a slope no steeper than three to one. If steeper slopes are incorporated in the grading plan, there should be positive assurance that plantings will flourish over the long term without maintenance. Plants should be selected accordingly. Except under unusual circumstances, vegetative and slope stability factors, as well as maintenance and other requirements, should preclude slopes steeper than three to one. Exceptions include rock slopes that may be safe and stable on faces as steep as one to nine, and slopes of loess or similar soils that should be finished as nearly vertical as possible but no more than 10 feet in height. Rock faces more than about 8 to 10 feet high should be benched, and drained soil pockets should be created to permit landscaping of the rock face. Slopes of loess soils should not have drainage passing over them from above. If vertical faces are impractical in loess soils, controllable slopes generally cannot be assured with slopes steeper than 5:1.

Seeding can be used for both temporary and permanent soil stabilization. A common method is hydroseeding; the seed is applied in a spray that also includes various soil surface stabilizers. As required, fertilizers and sometimes a fiber mulch or chemical soil stabilizer may be mixed with the spray.

After seed and fertilizer are applied to slopes, a mulch is usually needed for temporary protection. This may be applied as an asphalt emulsion, which is also sprayed on, or straw mulch distributed by a blower can be used. Often, fertilizer, seed, and mulch are applied in a single operation. Straw mulches must often be held in place. This is accomplished by machine "cleating" (a tracked vehicle such as a bulldozer is run over the mulch), by spraying asphalt emulsion, or by staking plastic netting down over the straw.

Seed may be applied by machine drilling in furrows, a method most applicable to large areas having gentle slopes. The more expensive hydro-seeding method is best adapted to long, relatively narrow areas having steeper slopes. Drilling is unsuitable for areas with moderate or steeper slopes.

The cost of seeding, fertilizing, and mulching varies greatly, depending primarily on the size and shape of the area and the season and level of treatment required, but the range is typically between $800 and $1,000 per acre (1978 prices), which makes this process the least expensive approach to stabilization. Dependent on the area and season, the cost of regular watering until a strong cover is established may also be necessary. On erosion-resistant soils; seeding may be adequate for intermittent waterways when the design flow velocity is less than 2 feet per second. Sodding is recommended for waterways with design flow velocities between 2 and 4 feet per second. For velocities above 4 feet per second, structural stabilization of some kind (concrete, treated timber, or riprap) is usually required if excessive erosion and swale or channel maintenance is to be avoided.

Protection against waterway erosion can be achieved if proper consideration is given to the erodibility of designed slopes, flow velocity, flow resistance of the selected vegetation, and method of establishing the vegetation, provided foreseeable extreme flows will not be appreciably faster than erosion prevention design velocities. It is important to recognize that hydraulically efficient channel sections, which require minimum widths, are inconsistent with the low flow velocities needed to avoid damaging channel erosion.

Sodding is used for the immediate establishment of a permanent ground cover, but it will not adhere well without several weeks of growth after it is placed. It should be used on critical areas such as steep slopes, channels, and areas adjacent to paved land and buildings (where splash from walls may cause erosion during storms). Where sod is laid on slopes steeper than five to one it should be pegged to prevent it from washing away. Caution should be exercised in selecting pegs; long-lasting ones can remain as a hazard to foot traffic and machinery, whereas pegs made of such materials as soft wood will eventually decay. While sodding is fairly expensive (usually in the range of $1.00 to $2.00 per square yard in 1978), it may be economical if it can obviate the need for structural measures. Sodding also requires fertilization, watering, and initial maintenance.

In the proper seasons seed-bearing hay is occasionally used to establish a temporary cover, especially when further grading will be deferred. In this method, the hay forms the mulch. Costs for this type of cover and mulch vary considerably, and materials of suitable quality are often unavailable or inappropriate during non-growing seasons. Where it can be used, its effect is similar to temporary seeding and mulching.

Sprigging is sometimes used to establish Bermuda grass and other plants that are easily propagated. Sprigging is propagating by cuttings that may be spaded or cleated into the ground. Costs vary but are usually between transplanting and seeding where labor is inexpensive. If sprigs can be cleated in place, the resulting roughened ground may be more resistant to wind erosion than if they are set with a sprigging tool.

Transplanting, which also includes plugging, is another method of establishing vegetative cover from live plants. It is used for propagation of grasses such as Zoysia, and to establish shrubs and trees. Relatively mature plants that are transplanted can significantly reduce wind erosion and enhance the aesthetic qualities of a site. This method of establishing cover is more costly than seeding or sprigging, but it is usually less expensive than sodding. The cost of transplanting trees and shrubs varies greatly, depending on the species of plant; labor rates; feeding, watering, and maintenance expenses; and site conditions. When transplanting is used to establish cover, special stabilization of intervening exposed soil generally is necessary to prevent it's washing away, even on fairly flat areas.

4.16 Non-Vegetative

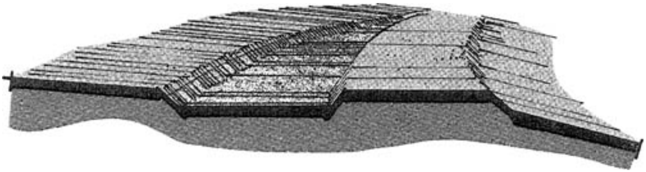
Non-vegetative soil stabilization also includes temporary and permanent measures. As well as giving temporary protection until permanent vegetative covers are established, temporary non-vegetative stabilization can protect during grading delays. Mulches, nettings, and chemical binders are typical temporary practices.

During periods of extreme drought, cold, or other conditions unfavorable for plant growth, a protective layer of mulch should be applied over exposed soil. Mulching is an important erosion control measure even when no vegetation is used, because it protects the soil against erosion; it is also important when establishing vegetation, particularly grasses and legumes, because it prevents seeds, fertilizer, and other soil additives from washing away, improves capacity for rainfall infiltration into the soil, prevents wide variations in soil temperature, encourages retention of moisture by reducing surface evaporation, and shields delicate young plants. The most common mulch materials are hay, small grain straw, wood chips, jute matting, glass fiber netting, plastic and asphalt emulsions, and various paper products. Most fiber mulches require immediate anchoring to prevent dispersal. Using plastic sheeting as mulch is unwise because direct sunlight may cause it to kill seeds and plants.

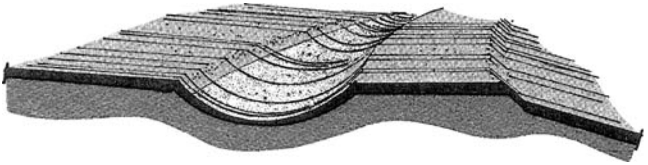
Permanent non-vegetative stabilization is used where conditions preclude the use of vegetation. Structural treatment may be required for excessively steep slopes, areas of groundwater seepage, droughty soils (soils which for one reason or another do not absorb or retain moisture well), or waterways subject to high flow velocities. Coarse crushed rock and gravel are commonly used materials where slopes are gentler. Since non-vegetative measures do not regenerate as live plants do, their use is limited to relatively level areas with maximum gradients no steeper than about five to one. Costs vary widely, depending on availability of materials and ease of application. This type of protection can be integrated with a permanent landscaping plan, permitting some degree of cost recovery. Except in unusual circumstances, rock and gravel should not be used for temporary stabilization, as they will interfere with establishment of permanent vegetative covers.

4.16.1 Structural Measures

Structural measures are designed and built to fulfill a specific function. The most common structures are those which intercept surface runoff and convey it to a safe disposal area to keep runoff away from erodable soil or to prevent gully erosion. Sometimes runoff is intercepted to trap moving sediment.



trapezoidal cross-section

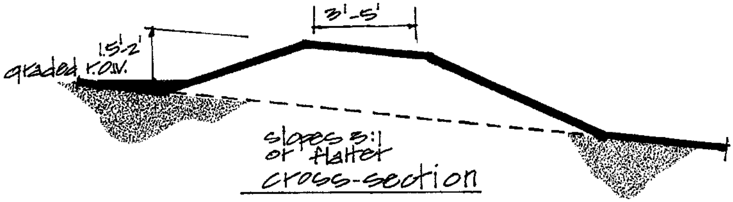


parabolic cross-section

Diversion Channels

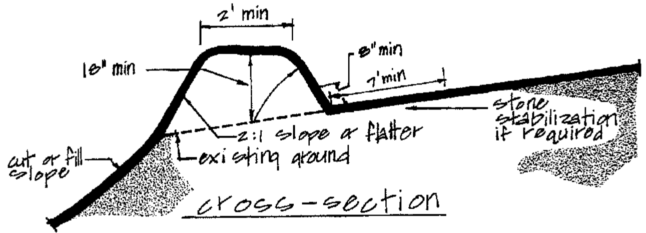
Diversion structures include soil or stone dikes, ditches or channels, and terraces or benches. Water is collected behind the dikes, in the ditches, or on the benches, and it flows along them at non-erosive velocities to an outfall where it can be released without causing excessive erosion. If a structure built along sloping land has a combination of a channel and a ridge, it is called a diversion. Channel slope design should take into account preventing scour without sacrificing the gradient necessary to prevent deposition of eroded materials, although it is impossible to fully achieve both objectives in one installation.

Temporary berms are relatively long, low mounds of earth used to shorten vertical runoff flow distances on cut and fill slopes. They may be installed on the contour or on a slight grade to control runoff water movements. Temporary berms are usually much larger than diversion structures though they serve the same purpose. Runoff diverted from berms may fall on grass or sad, on wooded areas, or on gentle slopes; gutters, paved ditches, or pipes may be used to divert runoff to stable areas below berms. Similarly, terraces or benches may be placed on or near the horizontal to reduce runoff velocities and erosion. Temporary berms are useful below soil stockpiles and slopes that will be further graded before they are stabilized. Sometimes a temporary berm can be modified to become a permanent terrace or bench.



Filter Berm

Diversion dikes are temporary ridges of compacted soil, constructed at the top of slopes or on the down slope side of cross slope channels. **Interceptor dikes** are temporary ridges of compacted soil constructed across graded roads. The purpose of interceptor dikes is to intercept storm runoff and direct it into vegetated areas or disposal structures (such as sediment basins). Interceptor dikes may be built of compacted soil, crushed stone, or gravel; the latter often are used to permit through-flow while filtering out fine soil particles, but crushed stone or gravel can be expensive. The cost of such dikes is about $1.50 per linear foot (1978), including both construction and removal provided there is a nearby place for waste disposal. Maintenance of interceptor dikes may be difficult and require constant vigilance, especially in areas of heavy vehicular traffic, but both diversion and interceptor dikes can be built easily and inexpensively.

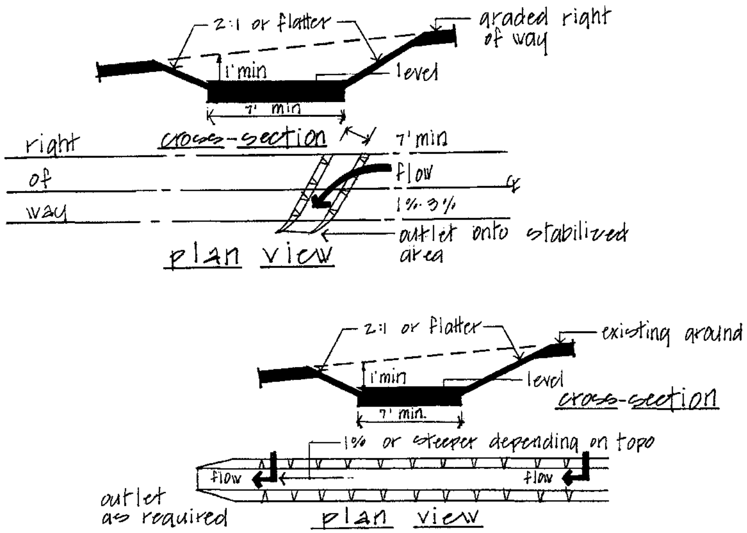


Diversion Dike

Waterways (as the term is used here) are natural drainage depressions or swales or constructed channels that will convey runoff to a stable (non-erodable) outlet. Flow velocities determine the type, if any, of waterway lining used-vegetative, concrete, riprap, or asphalt; a lining may be superfluous if the waterway discharges to a sedimentation basin. Waterways may be used to dispose of water from diversions, berms, benches, or other areas. They are normally designed for a specific probable rate of runoff selected in consideration of their duration of use, and they are located in accordance with normal storm drainage practices. Channels and waterways may need to be protected against erosion by bottom and bank protection structures, such as riprap or other armoring materials placed directly on the banks, or by in-channel structures that deflect or dissipate flow energy. Check dams with energy dissipaters are small structures constructed in gullies or small watercourses to reduce usual flow velocities and dissipate the energy of the flowing water, to promote upstream deposition of sediment, and to stabilize channel grades. Check dams and other grade stabilization structures in waterways may be made of stone, concrete, interlocking blocks, gabions (stone-filled wire baskets), paving, or wood. Wattling is often useful and economical for side slope protection in arid areas.

Disposal structures are used to convey concentrated runoff collected by diversion structures to a safe outlet, and to convey runoff safely down slopes that otherwise would be subject to significant gully erosion. The location of disposal structures should be chosen in anticipation of ultimate removal of those that are temporary. Locations where equipment must be maneuvered in small areas, such as between buildings or midway on a long slope where the removal of a temporary structure can cause considerable soil disturbance, should be avoided.

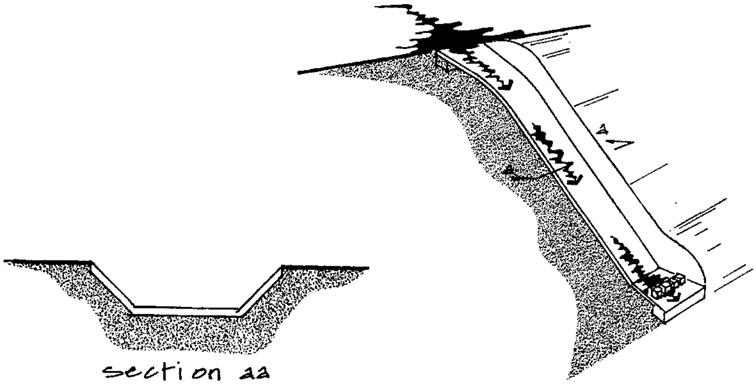
Flexible downdrains, sectional downdrains, flumes, and level spreaders are other means for conducting concentrated flow down a graded slope to a disposal point. Flexible downdrains are conduits of heavy-duty fabric or other material, usually installed temporarily. Sectional downdrains are usually half-round pipes made of bituminized fiber, concrete, or metal, shingled, and permanently staked into place. Flumes are channels made of concrete, metal, or (infrequently) asphalt. level spreaders are outlets constructed at zero grade across a slope, to spread concentrated runoff so it may overflow at non-erosive velocities in the form of sheet flow over lower undisturbed areas stabilized by existing vegetation.



Swales

Some considerations involved in design of the described structures include the following:

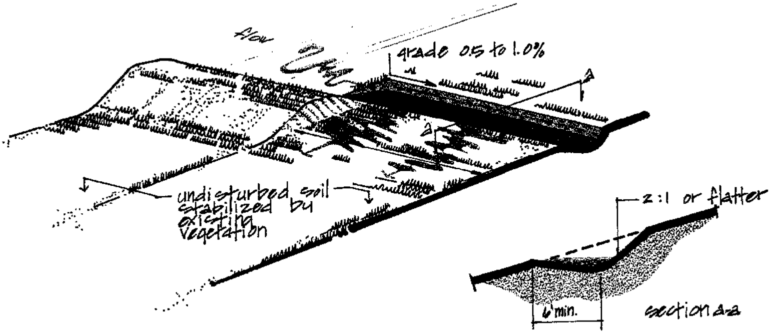
- **Flow**. What flow should be accommodated, and why?
- **Diversion structures**. The surface runoff (design storm runoff) accumulated behind the dikes should never spillover the dike. Therefore grading should provide positive drainage to the outlet. The diverted runoff should be discharged directly onto a stable area, if possible, or through an energy-dissipating structure designed to accommodate the amount of flow being discharged. Gravel or crushed stone should be used for interceptor dikes across graded roads.
- **Downdrains**. The design of the structure should accommodate the greatest amount of runoff expected to avoid washout. Inlet and outlet conditions should be checked regularly to insure proper functioning. Soil around the inlet should be compacted or otherwise protected to prevent piping failure. Disposal areas should either be already stable or stabilized. The installation of an energy dissipater designed to reduce flow velocity may be necessary if the outlet cannot discharge onto a stable area.



Downdrain

- **Level spreader.** The length of the spreader depends on the amount of water to be disposed of. The top must be exactly level along the entire length of the spreader in order to spread overflow water evenly. Construction of level spreaders requires a high degree of skill, and their maintenance may be difficult. Level spreaders should not be used on highly erodable or expansive soils because their maintenance will be almost impossible. Their use is recommended only where other control means are impractical.

To be effective, temporary measures must be relocated or removed as construction progresses. A construction site is subject to changes almost daily, and the erosion control program must permit the flexibility needed to accommodate these. In the final analysis, a successful erosion control program is one that provides proper protection



Level Spreader

at all times during construction at the most reasonable cost possible. Practical day-to-day decisions require common sense; they should not be hampered by inflexible rules or regulations.

For control measures, maintenance is the paramount need for continued effectiveness, Most of the measures described above require regular repair and maintenance, usually after rainfalls. Vegetative cover will almost always require watering and perhaps fertilizing until permanent maintenance responsibility is delegated; protection from traffic also may be necessary. Berms may occasionally require rebuilding to restore their original height. Regular preventive maintenance should be included in the construction program, and the purchaser as well as the community should be made aware of permanent measures requiring future maintenance. A brochure describing all maintenance actions needed for continuing protection and maximum enjoyment of the new property should be made available to future owners. It should be noted that some interim erosion and sediment control measures serving more than one residence may be converted to other uses after they have served their original function; they may be more properly maintained by the immediate or overall community.

4.17 Sediment Control

4.17.1 The Sedimentation Process

Sedimentation is the process by which suspended particles are removed from water. The rate at which these particles are removed is influenced by the size and specific gravity of the particles, the temperature of the water in which they are suspended, and motion of the water. In moving waters, particles may roll along the channel bed or be suspended within and carried along as the water flows. Turbulent flow can lift and mix particles that would otherwise settle out, keeping them in suspension; turbulent water may cause an increase in solids content by eroding additional materials from channels.

4.17.2 Sediment Control Design Principles

The design objective is to insure conditions most conducive to deposition and least likely to hold particles in suspension, at locations where deposition is desired.

Particles larger than a certain size having a specific gravity greater than water will eventually settle out. For given suspended particle characteristics, the time necessary to achieve settlement is a function of water depth and flow characteristics.

If all other factors are constant, larger particles will settle out more quickly than small ones, and lighter particles (lower specific gravity) will settle more slowly than heavier ones. (Particles will settle just a bit faster in warm water than in cold because cold-water viscosity and density are slightly greater.)

If all other factors are constant, particles will settle more quickly in quiet waters than in turbulent waters.

Particles smaller than a certain size will not settle within a practical time period, as other factors such as electrical charges keep them in suspension. Some clay particles exhibit this behavior. Accordingly, sediment-trapping devices such as basins are often ineffective for many clay soils. On sites having these soils, it is more practical to increase efforts to reduce the rate of erosion rather than to make futile attempts to trap suspended sediments.

In general, the amount of soil material eroded and transported to streams will be proportional to runoff flow velocity and the duration of flow, although soil surface self-armoring and some other conditions may tend to reduce erosion rates over time. Erosion rates increase with increasing runoff flow quantity and velocity. Flow quantity and flow velocity are therefore essential factors to be managed if erosion is to be controlled.

If the flow of water or air is slowed, reduced in volume, or its flow turbulence is reduced, less sediment will be transported. Sediment deposition can be stimulated by reducing runoff flow quantity and velocity. Such reductions are the basis for all sedimentation trapping measures.

Sediment removal also can be achieved by filtering-the removal of sediment by straining it out of the water with grass, filter cloth, straw bales, or other materials that will either trap or absorb the soil particles but permit clean water to pass. Filtration can most often be effective where contributory runoff areas are small. Filtration is not universally useful, however, and it should be used with caution. Excessive flows over a vegetative or other filter strip may smother or destroy the filter strip itself or erode and carry away material previously deposited. The trapping efficiency of filter strips is difficult to quantify, and filters such as gravel barriers tend to clog and then lose their effectiveness. Nonetheless, when dealing with small area problems or small parts of larger areas, filters can be practical. When properly used, filter strips and filters often offer high sediment removal efficiency coupled with minimal maintenance.

Trapping is a term describing removal of suspended particles by reducing the forward velocity of flow, reducing the turbulence of the suspending water, and preventing them from becoming suspended again. Because trapping is relatively expensive, it is practical only for larger (non-clay) soil particles. Consideration should be given to combining sediment trapping and stormwater runoff management whenever possible to reduce their combined costs.

Trapping devices generally consist of a structure designed to retard the velocity of sediment-laden runoff flowing to an outlet, allowing time for sediments to accumulate and fall at a desired location to allow the runoff to flow safely out of the structure without causing further erosion or other hazards, and to permit periodic maintenance including removal of deposited materials. Such structures may be arbitrarily divided into two classes, traps and basins, which function almost identically.

4.17.3 Planning and Design of Sediment Control Methods

How much sediment control is needed is determined by specific conditions at specific sites that govern the effectiveness of alternative sediment control measures. Because good planning and design reduce construction and operating costs, they are essential for functionally effective sediment and erosion control.

To assure that a given potential sediment impact on downstream areas will not be exceeded, the necessary complexity of an erosion and sediment control program should consider the size of the project, the intensity of the proposed development, and all physical characteristics of the site and its environs especially topography, soils, drainage features, geology, and climate.

A workable program will be kept as simple and flexible as possible. A highly complex program may be neither desirable nor useful on a small project such as a single residence on an isolated lot. A major construction project affecting several hundred acres of land may require much more sophisticated approaches. An erosion and sediment control program should be tailored to the specific project.

The program should include the staging of the development, with a description of the sequence. Since construction schedules are subject to change, program actions should be triggered by specific construction progress instead of actual calendar dates. Plans should include a requirement that sediment control structures and hardened drainage channels in development areas be installed prior to clearing, grubbing, or grading of upstream areas, when such timing is feasible. Plans should also show locations of sediment retention structures and the sequence of their construction.

Critical areas along streams should be clearly delineated and methods for their protection or stabilization indicated. The plan should define schedules for temporary and permanent stabilization. The location and timing of sodding and temporary or permanent seeding, as well as ground preparation, sod quality, seed type and quality, fertilization, mulching and irrigation, should be specified and detailed in the plan. Provision should be made for repairs and additional stabilization after the completion of grading, during grading shutdowns, during winter months, and during unforeseen delays.

Sediment control practices are intended by design to slow the flow of surface runoff by spreading, ponding in traps or basins, or filtering. The sediments settle out because the carrying capacity of the runoff is reduced. The effectiveness of sediment retention by filtering depends largely upon the nature of flow, the concentration and size of the sediment particles, and the nature and location of the filter. The amounts of sediment removed by methods that involve ponding are a function of flow velocity and the detention time of runoff passing through the structure. Flow velocity may be decreased by diverting runoff flow through a filter (which requires increased runoff storage capacity), and detention time may be increased by temporary storage of runoff in a trap or basin. In either case, only some percentage of the sediment carried by the flow will be deposited; in the case of filters, some amount of very fine material will adhere to the vegetation, fibers, or stones of which the filter is composed.

4.17.4 Filtering

Sediment control measures based on filtering may be divided into two types: vegetative and structural. The choice of type depends primarily on economics and space limitations; while vegetative filters and buffers are usually economical and. when properly used, require little maintenance, they require relatively large land areas for effectiveness. The more expensive structural filters conserve land area by concentrating filtering action in localized structures. The more commonly used filtering measures are briefly described below.

4.17.5 Vegetative Control Practices

Vegetative control practices include vegetative buffers and sod inlet filters. Vegetative buffers may be natural or manmade; they detain, absorb, and filter surface runoff, thus reducing sediment load, but they have only limited filtering capacity per unit of their area.

Natural vegetative buffers, sometimes called **filter strips**, are one of the more effective and economical methods of controlling entrained sediment. They consist of strips of natural vegetation such as grasses or woody plants. When they are already in existence, their economy is obvious. When mature and well established, they are able to handle a greater load than immature stands, and their filtering capacity is greater because of increased density.

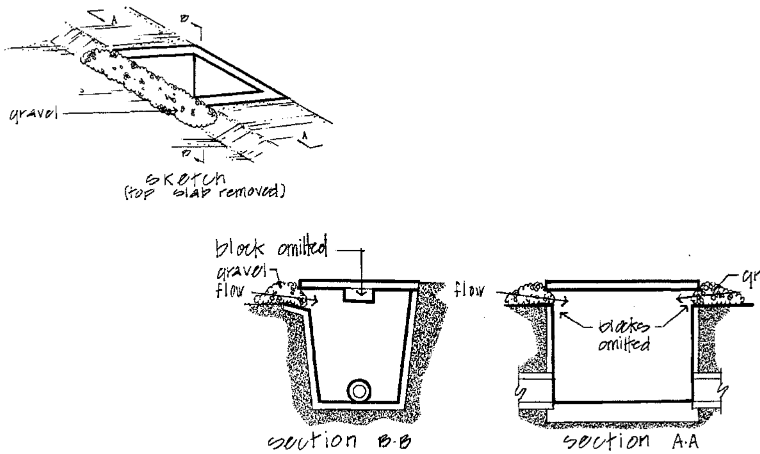
Filter strips are quite helpful; they can be developed or preserved along perimeters of graded areas, generally parallel to natural drainageways or waterways. Grass is often the best vegetative filter because of its low dense growth and well-developed root system. Filter strips developed by mixing white and red clover with grass are also effective. The need to preserve such buffers must be recognized during the planning and design stage. No construction or other traffic should be allowed in filter strip areas, as traffic will interfere with normal growth and filtering effectiveness.

Manmade vegetative buffers may be installed if the following three conditions are apparent.

1) The existing vegetation is unsuitable or cannot be preserved.
2) Poorly adapted or developed vegetation cannot reliably form a satisfactory buffer.
3) A new open drainageway or waterway is to be constructed.

The effectiveness of a buffer area increases with its length in the direction of flow, as longer flow paths result in the trapping of more particles. Flattening the slopes of the buffer will increase its effectiveness, mainly because a controlled spreading of runoff is more easily achieved, but this will destroy existing vegetation. If necessary, vegetative strips along waterways should be graded and fully re-vegetated prior to the beginning of land disturbing activities in the upland area.

Vegetative contour strips, usually sod, are installed at intervals paralleling contours of graded slopes to reduce and filter storm runoff as well as to check erosion. Their function is similar to that of buffers; they are not often used because they break up control areas that might otherwise be seeded at relatively low cost into sections that may be uneconomical to work on, and because installation of sod is expensive and may be avoided if sound alternatives can be defined.



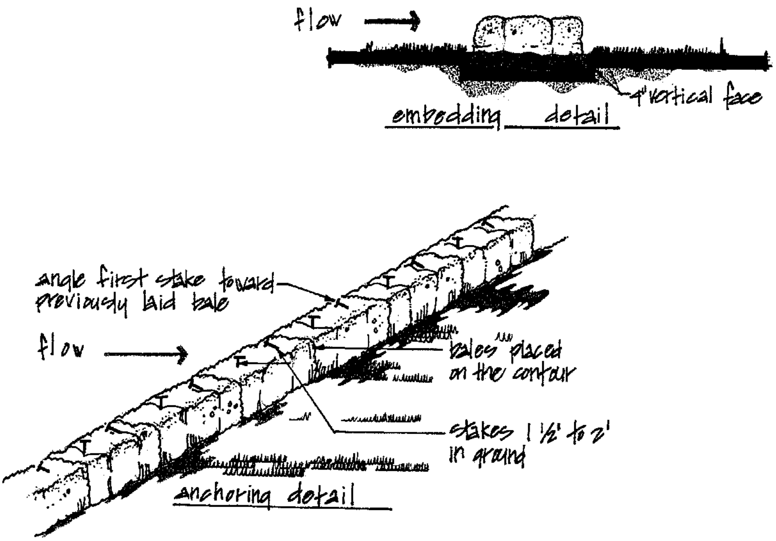
Gravel Inlet Filter

Sod inlet filters are pads of sod placed around storm drain inlets. The amount of runoff expected at the inlet usually determines the size of the sodded area. This practice is used after final grading is complete, and during the establishment of general area vegetative cover when the concentrations of sediment in runoff are relatively low.

4.17.6 Structural Control Practices

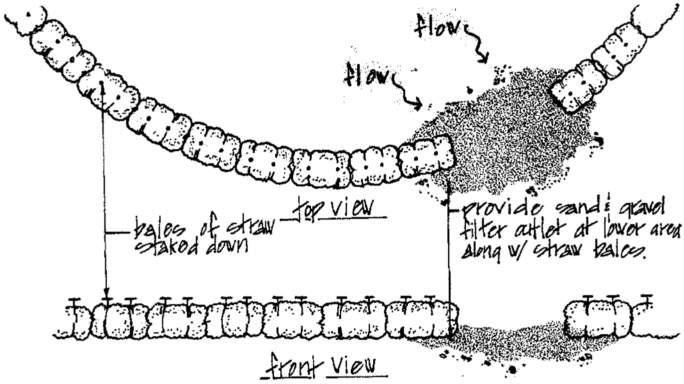
These practices include the use of barriers made of gravel, straw bales, and other materials. To provide for drainage during construction, necessary storm drains should be installed before final grading takes place. It is important that storm drain inlets and' pipes be protected by trapping more sediments before they reach the constructed storm drainage system.

Gravel inlet barriers are temporary barriers made of crushed stone or coarse gravel placed around or in front of an inlet. Storm runoff is retarded and ponded by the stones before entering the storm drain. One design provides for placement of hollow-core concrete blocks in inlet openings to prevent stone from entering the storm drain. Another method is to place a board across the opening, leaving unblocked at least one-half inch at the top and at the bottom for water flow. Such barriers must be inspected after each storm and cleaned regularly when necessary to maintain their effectiveness; excessive gravel entering inlets may cause problems downstream. Gravel inlet barriers ultimately must be removed and replaced with permanent controls, so their attractively low initial cost may be misleading.



Straw Bale Dike

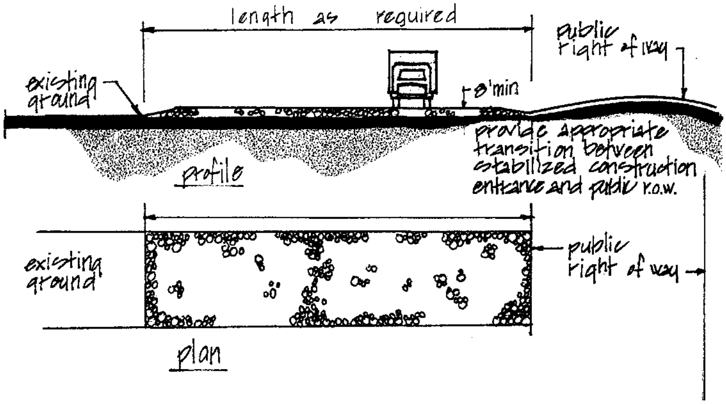
Loose rock berms are large stone or gravel dikes placed across graded roadways to retain sediment without drastic interference with construction traffic. These berms are also used in drainage ditches prior to paving of roadways and prior to establishing permanent ground cover. As in the case of any measure placed across a traffic area, their maintenance can be difficult; in addition to the sediment trapped by the berm, mud and soil from vehicle tires and tracks will tend to clog berm openings, with a consequent loss of effectiveness. Considering their usual small sediment entrapment capability as compared to their placement and removal cost and their temporary nature, they usually are only considered as a last resort.



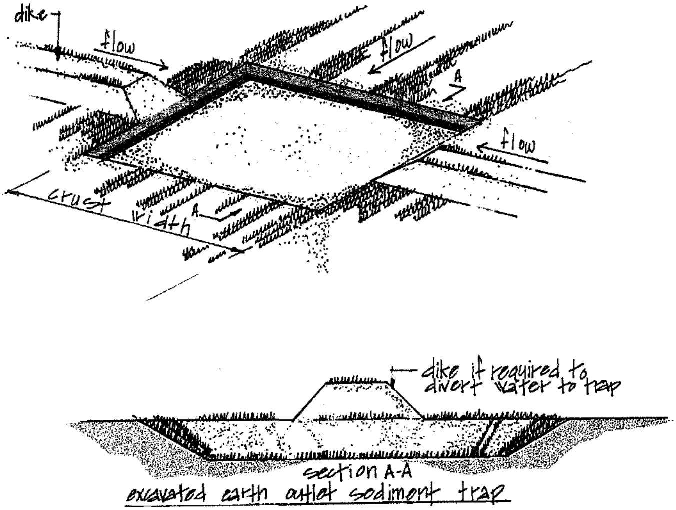
Barrier With Gravel Outlet

Straw bale sediment barriers are temporary berms, diversions, or other barriers constructed of baled straw or hay; they retain sediment by retarding and filtering surface runoff. They are used at storm drain inlets, across swales and ditches, as dikes and berms along property lines, and in other temporary situations. Straw bale barriers can be combined with sand or gravel barriers. Bales should be placed parallel to contours, and they should be firmly anchored and tied to prevent their displacement. Wood anchor stakes may be the most desirable, as wood will decay, possibly eliminating or simplifying the need for stake removal when construction is complete. Tying bales together with twine or cord is preferable to using wire or plastic, for similar reasons. To prevent piping where runoff flows between bales or between bales and the ground, straw bales may be placed in a shallow trench with their ends tightly abutting or chinked with straw; anchor stakes should be driven through each bale into the ground. This type of barrier should not be relied on for long-term effectiveness as straw bales can deteriorate rapidly. They have the advantage of relatively low initial, replacement, and removal costs.

Stabilized construction entrances are pads of crushed stone located at points of access to a construction site. The purpose of the stabilized entrance is to reduce or eliminate the tracking of sediment onto roads or streets. However, as with loose rock berms, their high first cost and relatively low sediment capacity make their use primarily one of public relations in construction area neighborhoods.



Stabilized Construction Entrance



Earth Outlet Sediment Trap

4.17.7 Trapping

Sediment trapping measures generally are used where channelized flows contain sediment in greater than acceptable amounts. Sediment traps and sedimentation basins are differentiated by their size and method of design and construction.

Traps are relatively small installations used for small drainage areas; they can be inexpensive to construct and comparatively simple to maintain. They are useful in areas unsuited to larger sedimentation basins. Several traps can often be substituted for a single, larger sediment basin if an area is divisible into small sub-watersheds.

Sediment basins are relatively large and frequently expensive to design, construct, and maintain. They can be relatively efficient sediment removal devices, but that efficiency often is obtained at high cost. Accordingly, sediment basins should be considered a last resort and used when other approaches to site planning, erosion prevention, or trapping are inadequate to reduce off-site sedimentation to acceptable levels.

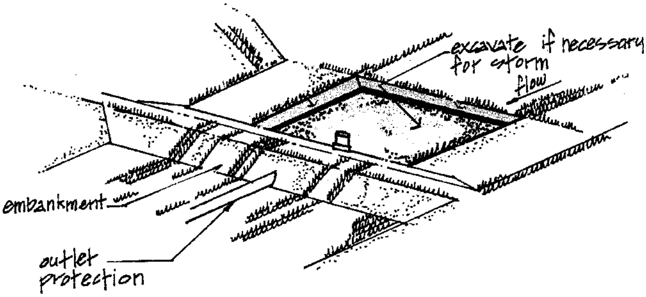
4.17.8 Sediment Traps

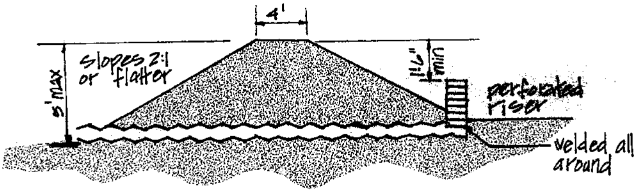
Sediment traps are small temporary detention structures used to intercept runoff and trap sediment. They rarely are practical for drainage areas larger than about 5 acres. They are usually installed by excavation and/or embankment; embankments usually should not exceed 5 feet in height. Depending on climate sediment trap volume customarily is about 1,800 cubic feet per acre of drainage area, which seems to provide adequate trapping efficiency and sediment storage. The volume of storage required depends on the amount and intensity of expected rainfall and on estimated quantity of sediment. Traps should be cleaned when accumulated sediments equal about one-half of trap storage capacity.

Sediment traps may be constructed with earth, pipe, or stone outlets, or they may be installed at storm drain inlets. Outlet selection is based on construction costs.

Earth outlet sediment traps discharge over or onto natural ground.

Pipe outlet sediment traps have outlets consisting of a piped riser which functions as a skimming weir, and they discharge through the embankment. The diameter of the riser should be larger than that of the discharge pipe





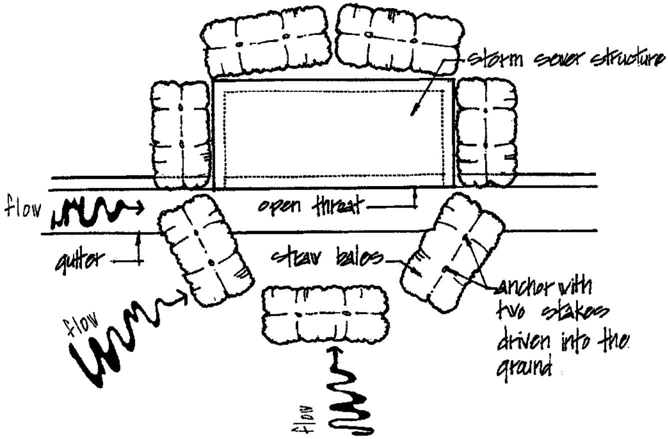
Embankment Section Through Riser

Storm inlet sediment traps consist of a basin formed either by excavation or a natural depression in the ground adjacent to and upstream from storm sewer inlets. Water may be discharged (and often filtered) through an opening into a storm drain inlet structure. The opening may be either the inlet opening or a temporary opening made by omitting bricks or blocks in the inlet.

Sandbag sediment barriers consist of bags filled with soil or stone, stacked at regular intervals along the ditch upstream of a storm drain inlet or culvert, for trapping coarse sediment particles.

4.17.9 Sediment Basins

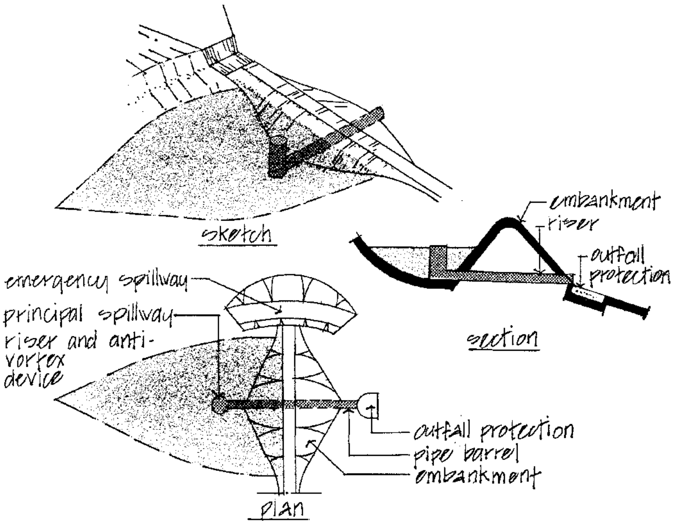
Sediment basins are the most effective approach to trapping sediment, in terms of the percentage of sediment removed from runoff. They are useful on relatively large construction sites and consist of temporary or permanent impoundments, usually constructed by damming a waterway. Sediment basins are the best approach to controlling off-site sediment movement. Dry sediment basins are constructed on waterways that flow only during storms, while wet basins are constructed on intermittent or perennial streams and may impound a permanent pond. Permanent shallow ponds should typically be avoided, however, as they have many characteristics which make them undesirable in urban settings.



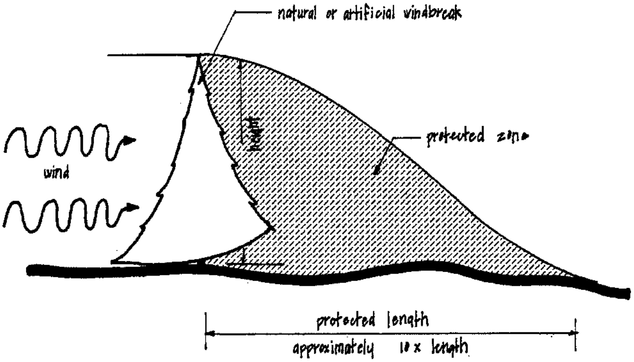
Storm Inlet Sediment Trap

Careful design is required to properly size and locate a sediment basin. Regardless of their location, sediment basins should be designed and constructed to provide a high level of safety from a hydraulic and a structural point of view. Provisions should be made for the safe passage of runoff from heavy storms, and embankments should be constructed to support water and trapped sediment loads. Required design criteria vary among jurisdictions, so designers should be familiar with applicable state and local requirements and standard design theories. General information can be obtained from local Soil and Water Conservation Districts, county or city public works departments, or appropriate state agencies.

While effective, permanent sediment basins are the most expensive structural sediment control measures. Their costs vary widely, depending on size, terrain, soil character, and safety. Trapped sediments must be removed from the basins, often using expensive equipment. Costs of cleaning can be high, and a permanent disposal site for waste material must be provided. The costs of temporary sediment basin removal and site salvage may also be high. A permanent sediment basin may function both as a stormwater management device and a key element in blue and green open space. Although a basin per se may be an expensive item, its sediment control function may be a low-cost by-product of stormwater management and open space functions. Where large stormwater management basins or lakes are to be built, it may be possible to design them for permanent storage of the entire quantity of sediment that may be generated during the construction phase and the facility's life. Where this is done sediment need not be removed, and only minimal costs may be incurred during basin conversion from sediment control to stormwater management functions.



Sediment Basin



Windbreak

4.18 Wind Erosion Control

Few techniques are available for dealing with wind erosion at construction sites, other than reducing exposure to high velocity winds at ground level. Special precautions are justified only in areas where excessive wind erosion is probable. Precautions may include avoiding grading during high prevailing wind seasons; incorporating strip development with temporary retention of natural vegetation; making sure that tree lines and hedgerows are perpendicular to prevailing wind directions; maintaining a rough graded surface with large clods insofar as possible; avoiding unnecessary heavy traffic over disturbed areas (to prevent destruction of soil aggregates); erecting artificial barriers if necessary; and watering traffic areas and dusty soil surfaces during dry periods to reduce soil and dust blowing.

The above precautions to control wind erosion involve one or more of the following basics: (1) increasing soil stability and surface roughness; (2) establishing and maintaining vegetation, mulches, and other types of cover; and (3) placing barriers perpendicular to the direction of the prevailing wind.

In arid and semi-arid areas the most easily installed (and probably the most effective) measure is the erection of temporary or permanent wind barriers such as snow fencing and hedgerows. In more humid areas, watering may be effective. As in the case of water erosion, good planning and construction techniques can sharply reduce wind erosion rates. In addition, as with water erosion, the risk of damage should be carefully evaluated; the plan or construction schedule should be based on an economic comparison of the benefit, risk, and the cost of control measures.

5.0 LEGAL IMPLICATIONS

Soil erosion and sedimentation can have legal implications for land development. Two aspects of the law are of concern. Developers, engineers, and public officials are exposed to litigation seeking redress for erosion and sediment related damages to surrounding property or to enjoinder where such damages are foreseen. In many states, statutes and regulations to control pollution address sediments originating at construction sites. Lack of awareness of such controls may lead to avoidable and unexpected construction delays or expense.

Liability occurs when the off-site effects of erosion or sedimentation include damage to surrounding property. Damage occurs when drainage from a developing tract becomes concentrated with a high-suspended sediment content and affects soil, crops, livestock, watercourses, or even structures on adjacent tracts. A sufficient deposit of sediment in a stream may also alter or obstruct its flow, causing the water to leave its natural course and damage surrounding property.

Liability usually is determined by those common law rules regarding drainage that apply in each specific locale. There is no single rule or standard for determining liability that is applicable everywhere. Local statutes and case law precedents have developed independently in each state and therefore have varying significance and import in each of literally hundreds of political jurisdictions.

Nevertheless, nearly all states follow one of two basic rules. Under either rule, if damage occurs while the land is in its natural state, there is no liability, as liability arises only where natural conditions have been altered. The basic legal concepts are known as the "common enemy" and the "civil law" rules. The "common enemy" rule, strictly interpreted, permits a land owner to alter the shape of his property or the drainage of storm runoff from it as he sees fit, without liability for damage caused to "lower" properties. In contrast, the "civil law" rule, strictly interpreted, imposes liability on the "upper" landowner for any damage caused by an increased downstream drainage burden. In many jurisdictions, modifications and exceptions have tended to narrow differences between the two rules so that liability usually will not be found where the landowner has made reasonable use of his property and has taken reasonable precautions to prevent undue adverse effects on other properties.

Although many states have gravitated toward such a middle position, the only generalized advice that is reliable is that reasonable means should be used to control drainage, erosion, and sediment movement on construction sites so that downstream flows and sedimentation will be as little changed from natural or pre-construction conditions as is reasonably possible.

When erosion and movements of fertile topsoil became a catastrophic Midwest problem in the 1930s, the federal government established a Soil Conservation Service within the U.S. Department of Agriculture. Thereafter Soil Conservation Districts were created in each state. The chief concern then was the conservation of soil in agricultural areas, so urban areas often were excluded from a district's jurisdiction. Districts usually lacked authority to enforce conservation standards and depended on voluntary compliance. The current trend is to expand the districts to include non-agricultural areas to provide authority for districts or others to enforce compliance with conservation standards. This often is done as an extension of the police power of municipalities.

National concern is also focusing upon water quality conservation. The federal government and most states have enacted laws to correct and prevent air and water pollution. One of the more recent enactments is Public Law 92-500, an amendment to the Federal Water Pollution Control Act Amendments of 1972, which encourages states to establish regulatory systems to identify and control nonpoint sources of water pollutants. One such source is sediment eroded from construction sites. P.L. 92-500 accelerated introduction of soil erosion and sediment control legislation in nearly every state; such legislation has already been enacted in many states, and it seems likely that similar actions in other states will materialize.

Although the National Association of Conservation Districts, the Environmental Protection Agency, and the U.S. Department of Agriculture jointly prepared a model act for soil erosion and sediment control for enactment by the states, the statutes already enacted cannot be considered uniform. They vary in such details as sanctions, procedures, and the delegation of enforcement responsibilities.

Maryland was the first state to enact regulatory legislation covering erosion and sediment control on construction sites. The Maryland law states that local governments cannot issue grading or building permits until the developer: (1) submits a grading or sediment control plan approved by the appropriate Soil Conservation District; and (2) certifies that all land clearing, construction, and development will be done in accordance with the plan. The local government must then make on-site inspections for compliance and forward a final report to the appropriate Soil Conservation District.

The Maryland law also mandates that before any person can disturb land for any purpose, the local Soil Conservation District must approve the proposed land change. A violation of either of these provisions can result in a misdemeanor conviction or a suit for an injunction, brought by either a government agency or any interested person. Not all states have taken as strict an approach as Maryland has.

In summary, developers, public officials, builders, engineers, and architects must be aware of the erosion and sediment control laws applicable to each project location. The first step a developer and his engineer should take is a check with local regulatory authorities; consultation with an attorney may also be proper.

6.0 BIBLIOGRAPHY

The publications listed represent a selection of various sources of information. This bibliography does not purport to be comprehensive; such a listing would contain literally thousands of titles in several languages. Readers interested in obtaining information pertinent to a specific area should contact the Office of the Area Soil Conservation District or the State Office of the Soil Conservation Service.

6.1 U.S. Department of Agriculture Publications

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Chapter 27

RESEARCH ON THE DESIGN STORM CONCEPT

September 1978
ASCE Urban Water Resources Research Program
Technical Memorandum No. 33
New York, New York

1.0 PREFACE

1.1 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 area-wide 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 that follows is also directed to area-wide agencies and their agents, it is expected that it will be of interest and use to many others, particularly local governments.

1.2 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. Administrative support is provided by ASCE Headquarters in New York City.

1.3 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 1970s.

1.4 Acknowledgments

This report draws on and is an extension of findings from papers presented at four international conferences. Mr. Marsaiek 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. Marsaiek 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. Marsaiek 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. Marsaiek or this writer and do not necessarily reflect the views of the National Science Foundation.

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1.6 Footnote

Area-wide 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, DC 20036, 64 pp., March, 1978).

ADDENDUM 4 OF THE ASCE PROGRAM REPORT "URBAN RUNOFF CONTROL PLANNING," JUNE, 1977 BY JIRI MARSAIEK

September 1978
Hydraulics Research Division
National Water Research Institute 867 Lakeshore Road
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1.0 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 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.

2.0 SYNTHETIC DESIGN STORMS

2.1 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.

2.2 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 27-1(a) is an example, for a "Two-Year Storm". Figure 27-1(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 27-1(c) were for about a two-year frequency). Parameter values used for the Chicago method are listed in Table 27-1.

Table 27-1

Value 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

2.3 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.

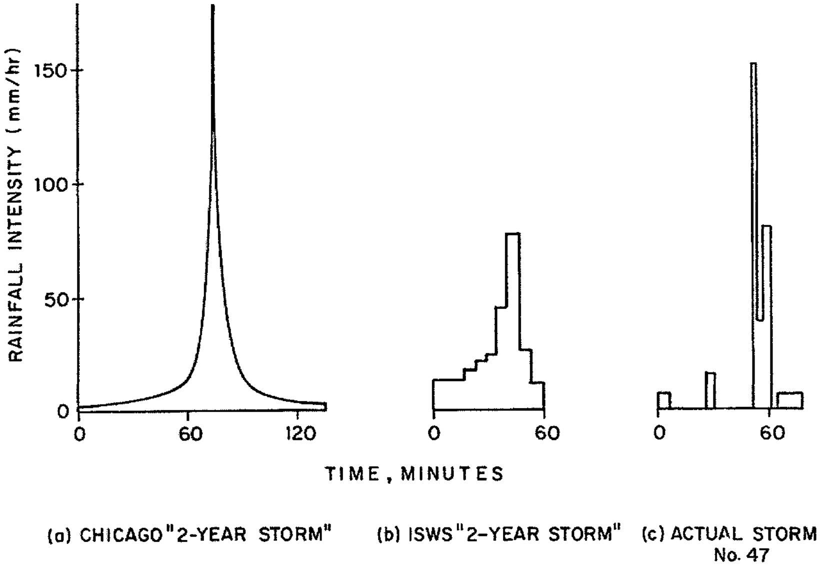


Figure 27-1. Synthetic and Actual Storm Hyetographs

As for the Chicago method, the data used to develop ISWS design storm 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 27-2.

TABLE 27-2

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 percent of rainfall, t_{cp} is the cumulative percent 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 27-3 and an example hyetograph (two-year return period) is shown in Figure 27-1(b). Again, there are important departures from actual storm variations, such as in the example of an actual storm in Figure 27-1(c).

TABLE 27-3

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

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

2.4 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 those 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 selected storms are summarized in Table 27-4.

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 27-4.

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.

Table 27-4

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

2.5 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 Sections. However, long-term antecedent conditions indicated by the raingage record from Hamilton in Table 27-4 may well be unusual. Also, for small rural or very sparsely 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 were 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.

3.0 SIMULATED PEAK FLOWS

3.1 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(15). Its development was first reported in 1971 for the U.S. Environmental Protection Agency(17), it is continuously upgraded by the USEPA(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.

3.2 *Actual Catchment Simulations*

The test catchment, known as the Malvern catchment(20), is a modem residential area of 23-ha with single-family housing units. The catchment is fully developed, its imperviousness is 30 percent, 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 27-4 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 27-2. 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.

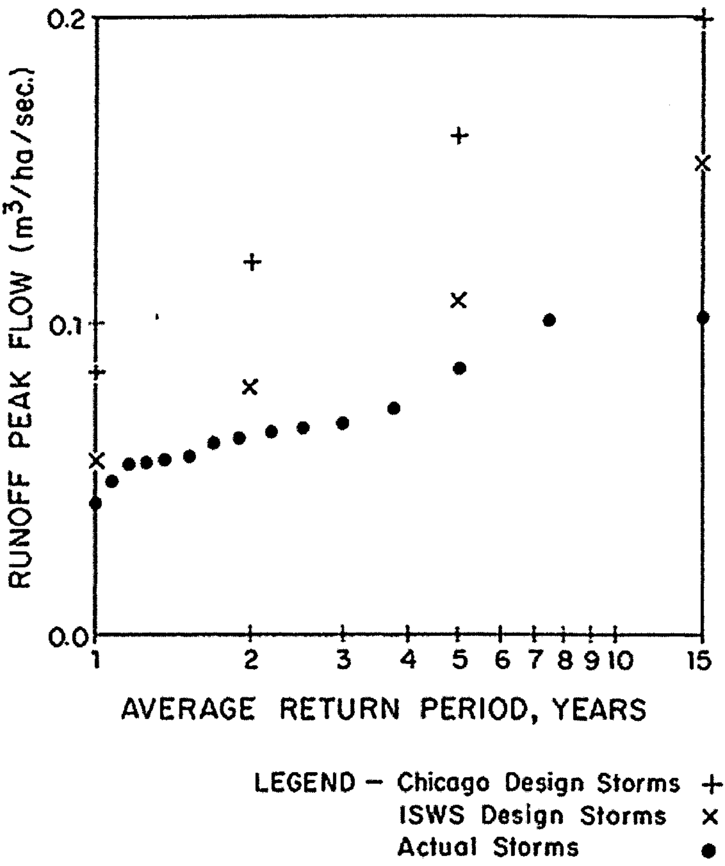


Figure 27-2. Recurrence Intervals of Runoff Peaks Simulated, Malvern Catchment, Actual and Design Storms

3.3 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.

3.4 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 modem 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 27-3 for the smallest hypothetical catchment (26-ha), separately for each of the three levels of imperviousness. Figure 27-4 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 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.

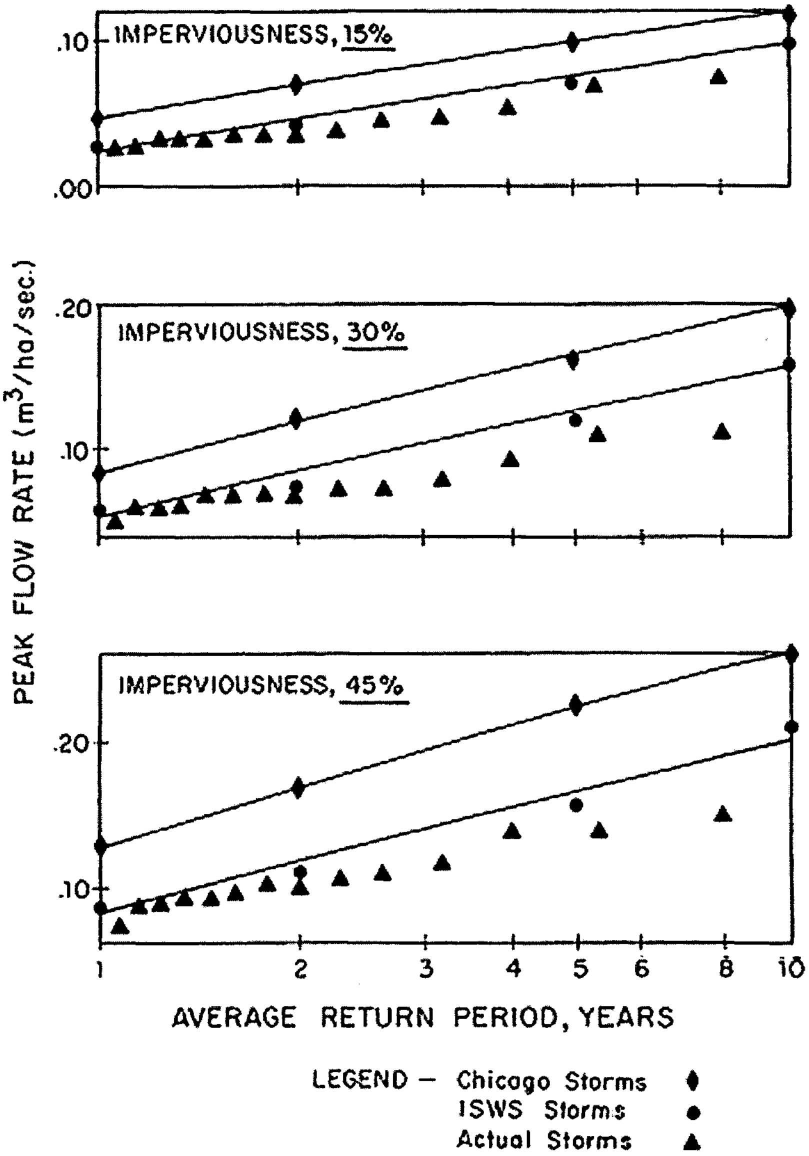


Figure 27-3: Comparison of Simulated Peak Flows, Hypothetical 26-ha Catchment

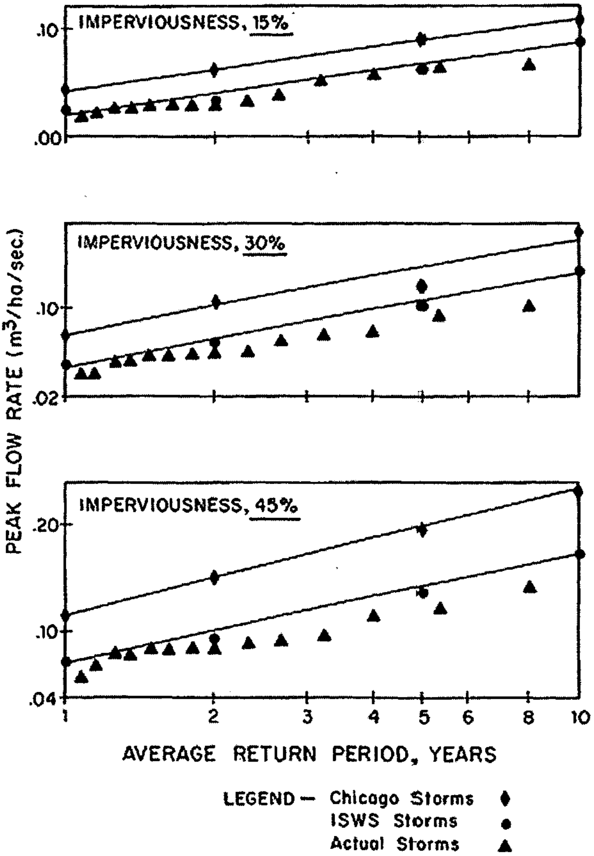


Figure 27-4: Comparison of Simulated Peak Flows, Hypothetical 130-ha Catchment

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.

3.5 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 27-3 and 27-4 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 27-4) 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 27-3 and 27-4, 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.

3.6 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.

4.0 EFFECTS OF DETENTION STORAGE

4.1 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).

4.2 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 27-5. In all three cases, the reservoir was assumed to be located in the vicinity of the catchment outlet.

TABLE 27-5

Detention Reservoir Characteristics

Reservoir Designation	Volume, cu. m.	Depth, m.	Outflow, cu. m/sec
R-l	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 was accomplished by using the modified Puls method(24). They were then used to establish new peak flow versus recurrence interval values, plotted in Figure 27-5. Included also in Figure 27-5 are the values from Figure 27-2, designated in Figure 27-5 as "urbanized, no control". For reference, results for "non-urbanized" conditions are also given in Figure 27-5.

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 27-5 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.

4.3 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 27-5).

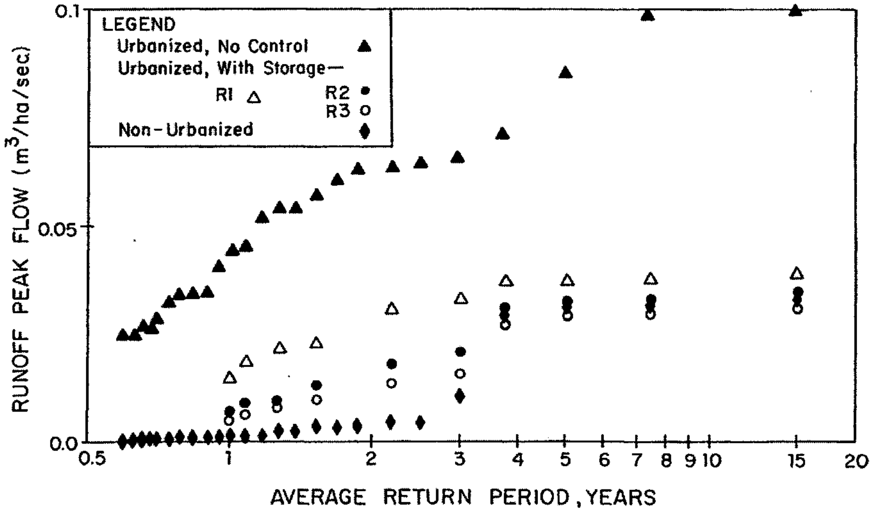


Figure 27-5: Simulated Effects of Storage, Hypothetical Catchment (26-ha, 30% Imperviousness)

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 tr-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 27-5 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 27-6, which is a plot of frequency of flow volumes without storage for the related simulations (middle graph of Figure 27-3). In Figure 27-6, the 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.

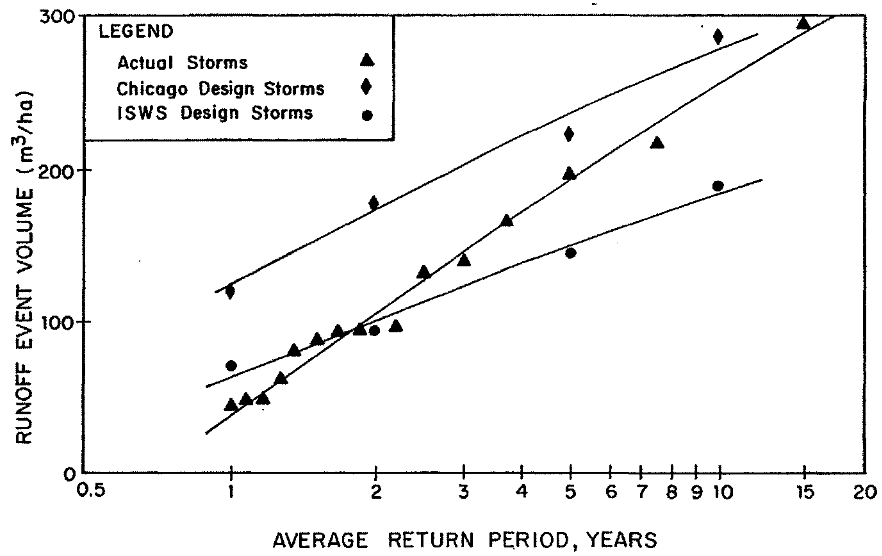


Figure 27-6: Runoff Volumes Produced by Selected Actual and Design Storm (26-ha, 30% Imperviousness)

Peak flow rates and magnitude of flow volumes are not highly correlated, as indicated in Table 27-6, 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 27-7.

TABLE 27-6

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 27-7

Examples of Transformation of Frequencies by Storage

Storm Number (Table 27-4)	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.

5.0 WATER QUALITY SIMULATION CONSIDERATIONS

5.1 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).

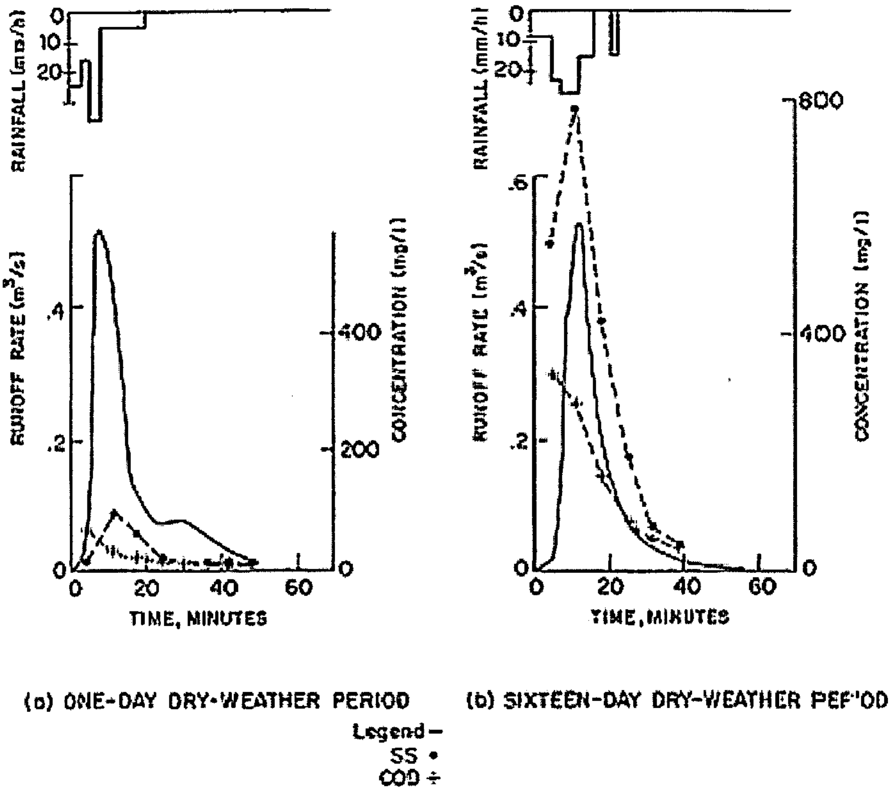
Some water quality considerations have to be recognized early in the planning of urban drainage. Such considerations are related to pollution control at the source and include the planning of land use, type of development and extent of natural drainage.

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.

5.2 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 27-7, which contains runoff hydrographs and pollutographs from the Malvern catchment for two storms of similar volume and intensity of rainfall. The two hydrographs agree quite well but the storm with the longer antecedent dry-weather period, Figure 27-7(b), produced pollution loads about five times higher than those conveyed by the other storm, Figure 27-7(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.



(a) ONE-DAY DRY-WEATHER PERIOD

(b) SIXTEEN-DAY DRY-WEATHER PERIOD

FIGURE 27-7: Effects of Dry Weather Period, Malvern Catchment

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 27-8. 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 27-8 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.

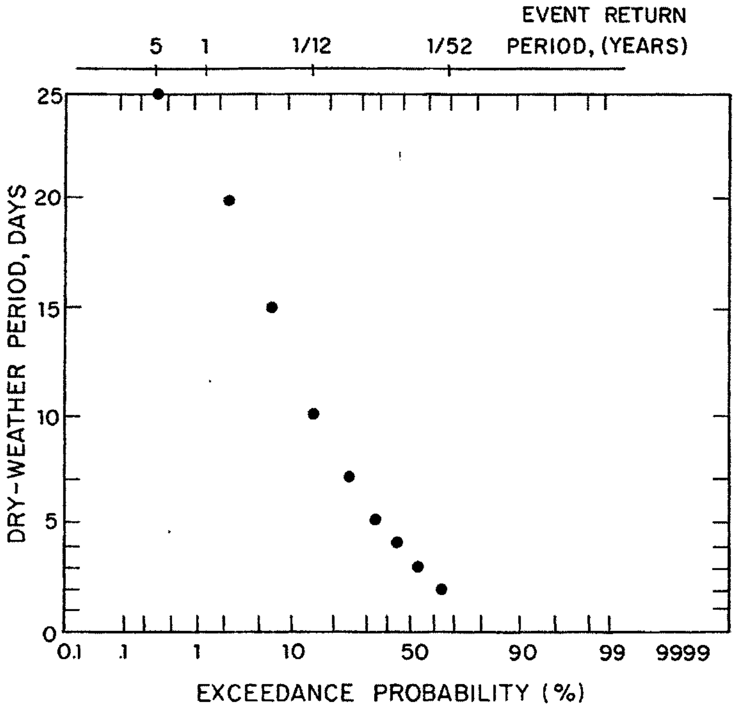


Figure 27-8: Probability of Dry-Weather Periods, Burlington, Ontario

5.3 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 27-7. Typically, historical rainfall events are used in quality design. Additional data, such as rainfall chemistry and atmospheric fallout rates may be useful.

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Chapter 28

INTERNATIONAL SYMPOSIUM ON URBAN HYDROLOGY (PROCEEDINGS OF TWO SPECIAL SESSIONS, SPRING ANNUAL MEETING, AMERICAN GEOPHYSICAL UNION, WASHINGTON, DC, 28 MAY 1979; ONLY MEETING SUMMARY PROVIDED)

September 1979
ASCE Urban Water Resources Research Program
Technical Memorandum No. 38
New York, New York

1.0 SUMMARY OF SYMPOSIUM DISCUSSION

Some parenthetical remarks made during the floor discussion were too faint to be recorded. The editor has paraphrased, condensed and moderately rearranged some discussion comments to preserve clarity, continuity or content. Discussants are identified by last name and country or organization, only. Full names and affiliations are included in other sections of this report, except for the following: Professor Miguel Medina, Duke University, Durham, N.C.; Mr. Michael L. Terstriep and Dr. Krishan P. Singh of the Illinois State Water Survey, Urbana, Illinois; Mr. Marshall Jennings, USGS, Bay St. Louis, Mississippi; Mr. William H. Sammons, Soil Conservation Service, Lanham, Maryland; Mr. Harry C. Torno, US Environmental Protection Agency (USEPA), Washington, DC; and Professor Jacques W. Delleur, Purdue University, West Lafayette, Indiana.

Laurenson—Australia

> *Two of the speakers referred to the very small areas (sub- areas) that they used in their models, one mentioned inlet areas as small as 100 m^2, and both referred to the small size of areas that drain to any given gully pit. Some urban catchments are quite large, and I wonder whether the amount of detail in defining the characteristics of such minute sub-areas, starting from 100 m^2, does not become a bit burdensome when you are trying, say, to estimate the flood from an area of several square kilometers. Does that same amount of detail have to be included all through larger catchments?*

Lowing—United Kingdom

> *The smallest of the areas that we were gauging was about 100 m^2; in size, but this is still bigger than some of the areas that we used for calibrating the relationship between the model parameters and catchment characteristics because we used laboratory catchment data also to pin down the small end of the range. We were interested to explain area-dependent variation in the values of parameters in a model of runoff from very small inlet areas. I believe that the largest size we would have draining to a gully would be around 500 m^2 or 600 m^2. In my presentation I showed a set of nine standard hydrographs, and in design use you would just specify each sub-area as being*

one of these several appropriate drainage densities. A certain group of gullies draining to a manhole would have an average density of perhaps 100 m^2 *per gully or 200* m^2 *or 400* m^2*, which, I believe, were the three size ranges that we used. So you would perhaps describe a whole area as having this approximate drainage density. It is not necessary in design application of the model to measure individual areas drained to a gully.*

Falk—Sweden

Only some ten days ago we had a seminar at Lund, Sweden, on runoff from urban surfaces. At the seminar the question was raised about the use of knowing the runoff from such a small area. There must be an upper limit in catchments when attenuation of the hydrograph is not very important any more. I think that for catchments at least up to a size of something like 2 km^2*, it may be of some importance to know what is happening in detail. We have found in a residential area in Lund built around 1963, that the catchments draining each gully range in size between, say, 80* m^2 *and 700* m^2*, and that must be considerably smaller than in North America. For example, last week I was watching a very heavy rainfall in Montreal and there were reaches of several hundred meters before the water entered a gully.*

Zuidema—Netherlands

It is not a disadvantage that a catchment area is as small as 700 m^2 *to determine the physical characteristics of a catchment and to be assured that the results can be applicable for other areas in common and will be reliable.*

Marsalek—Canada

I would like to comment on the measuring of inflows into sewer inlets. I would be quite concerned about doing so in North America because many inlets here are designed to intercept only part of their tributary inflow. Runoff entering the inlet does not necessarily equal the contribution of the drainage area. This is done quite intentionally to exploit the maximum diversion capacity of inlets. We recently concluded a study for the Ministry of Transportation and Communications dealing with highway drainage. In some cases, the flow bypassing an inlet could be as much as 90% of the gutter flow. Obviously, the European design of inlets differs.

Lowing—United Kingdom

I have been asked if we have had any problems with the measurement of flows into gullies. We had to choose these installations with care. In some cases at Southampton, we had to make miniature berms on the road with tarmac in order to direct bypassing water into the gullies, but we also tended to monitor them in nested groups so that we would have two or three gullies monitored collectively. Hopefully, the sum total was right even if there might have been some bypassing of individual inlets. However, the main thrust of that data *collection program was directed towards the calibration of the routing part of the model, the storage routing constant.*

Falk—Sweden

On three of the catchments in Lund we had devices like that to avoid bypassing.

Lowing—United Kingdom

While we are on the subject of gullies, I would like to raise the question of spacing. As we both said, we have in our countries very small areas draining to gullies. We are wondering now whether this is so clever because there is more storage available above the ground than had been allowed for in previous methods. Previously used times of entry (2 or 3 minutes) have now been shown to be rather large under-estimates of what the real delay times are. If there is more storage above ground, why do we not use even more of it by having fewer inlets, thereby having fewer lengths of first-order pipes and lesser maintenance problems? As somebody pointed out in a recent committee meeting, what if the water is flowing down the street for 10 or 15 minutes after the storm? Nobody is going to be out on the street anyway, so you might as well use the roadway as a primary drainage system. Only recently has it been realized at home that the gully pots themselves act as concentrators of pollution during dry periods, and the more gully pots you have the more oil and rubber you have sitting there waiting to be flushed out the first time it rains. This has been shown to be quite a significant factor in affecting the quality of the first flush runoff. So there are one or two arguments suggesting that we might in future try to have fewer inlets.

Medina—Duke University

I would like to ask Mr. Skretteberg about the assumption of no surface runoff from pervious areas. In some urban catchments in the United States, there can be significant surface runoff from pervious areas such as parkland. Would you please comment on the NIVA time-area method versus SWMM in this regard?

Skretteberg—Norway

I mentioned that there is a problem, especially along the coast, with runoff from permeable areas, particularly in the winter. This problem is going to be investigated under a new research project.

Falk—Sweden

During the summer, it can take quite a heavy rainfall to create any runoff at all from permeable areas. But there are certain places in Sweden where the *ground surface is very hard, and you can have a considerable contribution from such surfaces even during low intensity rainfalls.*

Lowing—United Kingdom

In the Road Research Laboratory method, they certainly made the assumption of no runoff from pervious areas, and I know that Mr. Terstriep of Illinois found it did not work too well in the United States, and introduced a model component for the pervious area runoff. The Road Research Laboratory itself introduced a model for pervious area runoff when it came to apply the method in tropical Africa, but it was not so happy when we wanted to do the same thing in the United Kingdom because, looking at typical summer events, it does not appear as though you get any runoff from pervious areas. Yet when you do regressions of runoff volume on soil properties and antecedent conditions (which you think would have more to do with the wetness of the soil rather than the wetness of paved areas), these variables are significant (in the United Kingdom). So one does not really know, but if they are significant variables in a regression study they should be included and so we allow pervious area runoff to contribute in theory. But having said that, the average runoff that we observe is certainly less than 100% from the impervious areas only. So there might have been two compensating errors in the original assumptions, but the one that assumed 100% from the impervious areas was the dominant error.

Terstriep—Illinois State Water Survey

Mention has been made of the fact that we added a pervious area component to the RRL. The primary reason was the difference in rainfall regime between most of the United Kingdom and at least Illinois. I do not remember the numbers exactly, but for the 5-year, 1-hour storm, for example, I believe we have nearly three times the expected rainfall in Illinois than you see in most of the United Kingdom. Also, in relating this to quality of runoff of urban areas, in the quality modeling that we have done we have gone back to considering the loadings from paved areas only. We did not consider any loadings from pervious areas. Now we feel that we are probably ignoring certain nutrients in doing this, but also we are dealing with a much smaller rainfall. For the most part, a half-inch of rain in an hour is about the maximum rainfall that we have dealt with in looking for high concentrations of pollutants in urban runoff. In the range of a half-inch or less the contribution from pervious areas tends to be negligible.

Singh—Illinois State Water Survey

There may be considerably more discharge from non-paved areas on steep slopes than from paved areas on flat slopes. As far as pollution from a pervious area is concerned, it may not show up as a peak but maybe sustained *for some time, and might have a serious impact on the quality of receiving waters.*

Marsalek—Canada

I would like to return to the question of directly connected impervious areas. Because certain properties may have roof leaders that are connected to sewer systems while others may not, it becomes practically impossible to determine the directly connected impervious area just from plans. I would like to refer to experiences we have had in Burlington, Ontario, where we have been collecting data for some time. If we use the total impervious area, which is something like 110% of the connected impervious area, we simply have too little runoff or we cannot explain the variation from one event to another. Once we use only the directly connected impervious areas, and we introduce surface storage more or less as a constant value, we have extremely good explanations of runoff volumes from events of various magnitudes. That is, if one takes an incorrect value for the directly connected impervious area then another source has to be found to explain where any residual measurement error is coming from. I presume if I were to do a regression analysis, the answer would be that it is coming from the pervious areas. So I think this is one of the dangers. Staying on the same point, when looking at the regression equation showing the possible contributions. from the pervious areas via soil moisture conditions, etc., Dr. Lowing mentioned that the magnitude of the storm or the depths of precipitation did not matter at all. I would say that this would probably contradict the assumption of a contribution from pervious areas, because I would expect that for minor storms there would be no contribution, but for rather severe storms there would be a sizeable contribution.

Lowing—United Kingdom

Yes, I share that intuitive belief. I suppose we should not start into this because we can argue intuition all afternoon. We have tried to stick with analysis of actual measured data that is why we are unhappy about talking about runoff from pervious areas because we cannot separate it by measurement. But I do not want to be misunderstood on the other issue. We certainly were considering only directly connected impervious areas and we always do know where the roof leaders discharge. That was not the problem so much as accidental mistakes in individual cases. It has been suggested that those should be investigated in detailed surveys and that it made a great deal of difference whether the surface was old or recently resurfaced, and so on, and one should make allowance for this, but we thought that was impractical.

Falk—Sweden

For one of 13 small asphalt catchments in Lund, we found that something like 8% of the rainfall volume minus depression storage was a continuous loss and *we explained that as infiltration. But that was the worst maintained surface of all, with a large number of cracks. For the rest of the catchments that loss was only something like 2% on the average.*

McPherson—United States

I wonder if micro-catchment research might not be useful that would include not just a single inlet area but also maybe multiple inlet areas in series, and such things as house leaders, not for quantity so much as for quality, to try to track the accumulation and transport of pollutants through urban areas. My feeling is that until we start going into this kind of detail we end up heavily in conjecture. Any opinion on that point?

Terstriep—Illinois State Water Survey

I would like to support that point. We propose one microbasin in our upcoming project in the National Urban Runoff Program where we will measure quantity and quality at an inlet. I had to argue strenuously to keep this in our proposal. In fact; I had two in there originally and we ended up with one. I feel, for example, that the washoff function that is used in quality models now is one of the weakest portions of these models and there is virtually no data to test the washoff, particularly on a particle size analysis, and I think this is a severe data need right now.

Falk—Sweden

The inlet runoff gauge used in Lund is now at the Chalmers University of Technology and the purpose there is to look at washoff. They will make measurements on catchments of the same size but will especially be looking at pollution.

Jennings—U.S. Geological Survey

I wonder if the question of micro-catchment gauging perhaps bears on the original question that Professor Laurenson asked. When the ultimate objective is to study a 50-mi^2 *watershed, we need information from some small catchments on basic processes.*

McPherson—United States

At the design storm seminar at Ecole Polytechnique in Montreal last week, we discovered that we were arguing with each other about different kinds of problems. Someone was talking about metropolitan-wide planning and someone else was talking about designing a particular neighborhood collection system, and someone else was talking about automatic control of a system. Still another was talking about research where they were trying to learn more about basic fundamentals and processes. Therefore, I think that today we must define what we are talking about. I was really talking about *fundamental processes, because that was supposed to be one of the objectives of the National Urban Runoff Program in the United States. It depends on what we are looking for. If you want a demonstration, that is something else again, where you are talking about an area of a square mile or something like that. I guess it all boils down to: what is a representative sample?*

Jennings—U.S. Geological Survey

We may have a possible cooperative study in Rochester, New York, on an area of 100 mi^2. All the point sources have been taken care of yet water quality problems remain, so urban runoff is thought to be the culprit. The question is: do we invest in very small catchments studies or do we choose a relatively large catchment that becomes a sub-area in the total watershed model and study the processes on, say, 100 acres using very detailed gauging? A 50 mi^2 to 75-mi^2 basin will have several hundred sub-areas in any distributed rainfall-runoff-quality model. We all know what happens when you put that many sub-areas into a model. Things kind of blend together in unusual ways, and we found ourselves getting reduced accuracy of prediction when we had more elements than if we had simplified to a smaller number of sub-areas. I agree that with the micro-catchment we can really learn about processes, but where does that leave us when we have to do management-type studies on practical large basins?

McPherson—United States

Another aspect of this seems to be the struggle between art and science. One side is the science of the processes and the other is the art of engineering. Right now, the emphasis is on the art.

Lowing—United Kingdom

I am sure there are many different ways that you can look at this question, but the one that occurs to me straightaway on the size of the catchment you should be gauging is that we were looking at small source areas because that is where it all happens. The runoff gets into the drainage system by sitting around for awhile above the ground, and by measuring the very first point you can, you learn more about the proportions of storage that operate above and below ground. As I was saying earlier on, this is beginning to affect our thinking about the number of gullies you should have and this has implications on the costs of construction and maintenance of the smallest pipe size and connections, and street-sweeping effectiveness, and whether people are happy seeing water in the streets or not. I think it is important to do both; and you have to investigate the small end as well as the large end.

Sammons—Soil Conservation Service

What seems to be missing completely is whether or not the source areas you are working with have any transfer value. In the wide range of climatology *that we have here in the United States, we can have adjacent watersheds where you cannot transfer the information from one to the other although you are gauging both of them, because one may have a thermal spring on it or something else that is really characteristic of the geomorphology. Everybody wants to know rationally what goes on, but as Mr. Jennings brought out, we are forever putting these things together in model and yet we do not know how to take the sub-areas into account or even how to take to areas and bring them together. We do not really know what happens downstream from two areas coming together unless we have some gauging. I have seen estimates for areas of reasonable size being 5 and 10 feet off using different methods of routing. We have all those kinds of problems, as well as your carry-over effect, etc. And then here in the United States, and especially in urban areas, they believe in onsite storage. In other words, so that you do not harm your neighbor, you have to store runoff onsite. You get into all kinds of events, in two categories, what some people refer to as design concepts and actual events. One of them is the real world and the other is merely a concept and there is no realistic relationship between them.*

McPherson—United States

I believe that this discussion points out the frustrations engineers have about the state-of-the-art versus the scientific content of urban hydrology, while at the same time urban America continues to grow and to be rebuilt and the work must go on. We are faced with this unfortunate dichotomy. On the other hand, a sarcastic view holds that lack of data has never particularly bothered us before, so why reform suddenly?

Laurenson—Australia

The problem of the size of areal discretization that we have been discussing is quite analogous to the problem of time discretization that all of us have thought about a long time. I think that in the time domain it has come to be realized that processes have to be studied on a time scale of as small as 1-minute, or 5-minutes, or so if one is to gain a complete understanding of the processes that have been taking place, but if, on the other hand, you are dealing with a design problem in which the relevant storms have durations of 6-, or 12-, or 24-, or 36-hours, then there is no way in the world that in the design problem you are going to use a time increment of 1-minute or 5-minutes. You average things out over a much longer period and I suppose the same thing applies also in the areal discretization problem. If we are to research and understand the processes that are really taking place on the catchment, then we need to work in terms of areas as small as 100 m^2 if this is the inlet area, but that in design problems for large areas of 10 km^2, then clearly one has to lump these things together to a scale of sub-area that is just adequate to represent the areal variability of the important parameters in the particular design problem which is of concern. That is, I think there is a very strong analogy between the areal discretization problem and the time *discretization problem. To change the subject, I would like to pose a question about the frequency of street sweeping. This has come up in some of the comments that have just been made and I would like information on just what is the order of frequency of street sweeping that people are thinking about when they talk about this subject. Where I live, the streets outside my house get swept about once a week, and in the city areas they get swept about once a day. Yet, last week in Canada I heard someone comment about frequency of sweeping of once a year to remove the gravel and other larger particles that build up during the winter. Just what frequency are we talking about in North America when we talk about frequency of street sweeping?*

Marsalek—Canada

Street-sweeping practices vary, not only in Canada but I guess all over the world. In fact, we had a project in which some of these practices were surveyed in a number of countries. For example, in downtown Zurich, Switzerland, people sweep about five times a day. In Canada, the situation with which I am familiar is the city of Hamilton, which has about half a million people if its suburbs are included, and Burlington, which has about one hundred thousand people. In Burlington, the commercial or downtown districts are swept daily. However, in outlying residential areas this frequency is reduced dramatically, and in many areas streets are swept only once a year, typically in the spring, to remove the sand and grit that remain on the catchment after the winter. Having monitored water quality on one of these catchments that is swept once a year, with removal of leaves in the fall, that is about all there is to it. Water quality is not as bad as one would get by multiplying standard daily loading rates by 365 days. Referring to the situation in downtown Hamilton, commercial areas are swept daily, whereas residential areas are swept perhaps once every two weeks to once every four weeks.

McPherson—United States

> *Some U.S. communities have only an annual spring cleanup. At the other extreme is a city in the west that gets only 7 inches of rainfall a year that both sweeps and washes its downtown streets every day because its economy is based on gambling casinos.*

Desbordes—France

Almost two weeks ago I asked the Director of Sewage Service of the City of Montpellier about street cleaning and he said that with 6,000 street catch basins and with the equipment and personnel available, each catch basin is going to be cleaned each six years! Street sweeping is not the only practice that affects urban runoff pollution control.

Torno—U.S. Environmental Protection Agency

> *I am not sure that frequency is the entire question. I live in the Washington, DC, area and they run trucks down the middle of the street, fire a little water off to the side, and I suspect they are probably running at an efficiency rate of only a few percent. I do think that frequency and efficiency are related. One of the objectives of the National Urban Runoff Program is to test the efficiency of what we choose to call in this country "Best Management Practices," which is interpreted to mean remedial measures that do not involve substantial capital construction works, and hopefully, we will be able to shed more light on that question. I think the answer to your question is that practices are highly variable, as you have heard here.*

Lowing—United Kingdom

> *I was in southern Brazil a few months ago, and I doubt that they even sweep their streets at all in the town I was in, because I was looking at the road gullies and they were totally choked up. There was no way, I thought, that any water could get in them because they were totally choked with, rubbish. Then I saw how the street cleaning was achieved because it rained. It had all gone—the storm just cleared the streets and the gullies were emptied by the pressure of water. Maybe this is the best way of doing it.*

Marsalek—Canada

> *I personally think that if we are interested in street sweeping from the pollution control point of view, that not only is the frequency of street sweeping important, but also the frequency of rain. If it rains every third day and we sweep once every two weeks, or once every month, the catchment has been cleaned by rain before the sweeping machinery is put on the streets.*

McPherson—United States

> *What goes up eventually comes down, and what we have been overlooking in many cases is taking measurements of precipitation quality especially in some of our larger urban centers. It is amazing that this is often overlooked.*

Falk—Sweden

> *In four of the catchments situated in central parts of Gothenburg and towards the outskirts of that city, it was possible to relate much of the runoff pollution to the atmospheric fallout that tended to decrease with the distance from the towns. Also, they could explain some pollution from traffic volume, especially lead.*

Delleur—Purdue University

In making correlations of some models in the United States, I have found that the slope of the urban catchment is usually not a variable, which I have attributed to the fact that the sewers in this country are usually designed with limitations in velocity, usually on the order of about 2½ feet per second for the minimum and about 10 feet per second for the maximum. However, this is not the case for many European catchments where the slope is a significant variable, such as in the United Kingdom. I would like to ask what the respective experience has been and if there is a physical reason why slope may happen to be a statistically more important variable in some European places than it seems to be in the United States.

Lowing—United Kingdom

The statistical dependence of a response-time parameter on slope was indeed observed in the United Kingdom but, in the urban context, we have demonstrated it only on the aboveground sub-area draining to road gullies rather than in pipes. I am, however, confident that response times of complete pipe network in the United Kingdom would also show a marked dependence on slope even though our range of permitted velocities is broadly similar to yours. I think the difference can be explained in terms of the size of catchment being studied, the extent of within-catchment variation of ***pipe*** *slopes, and the particular method of quantifying the catchment slope.*

Laurenson—Australia

In a recent study of the time lags on some urban catchments in Australia, I looked at two catchments, one of which had an area of 0.7 km^2 and the other 5 km^2. The lag time for the smaller catchment was one hour and the lag time for the larger one was half an hour. The relevant slopes of the main stream were something like approximately 0.51 for the smaller one, and 2% and 3% for the larger one, and it seemed very clear that the reason for the longer lag time of this much smaller catchment was the very much greater slope of the larger catchment. As for the reason, I think that Australian engineers tend not to worry about extremely high velocities in the pipes, and they prepare to lay concrete pipes on any slope whatsoever without worrying about the velocity in them.

Chapter 29

STORMWATER DETENTION OUTLET CONTROL STRUCTURES

1985
A Report of The Task Committee on The Design
of Outlet Control Structures of The
Committee on Hydraulic Structures of The
Hydraulics Division of The
American Society of Civil Engineers
New York, New York

ABSTRACT

The Hydraulics Division's Task Committee for the Design of Stormwater Detention Outlet Control Structures was formed to review and report on the state-of-the-art of stormwater detention pond outlet controls. This paper addresses the Committee's findings regarding: 1) hydraulic function, 2) water quality, 3) public safety, 4) maintenance, and 5) aesthetic aspects of outlet controls, and includes results of an extensive literature search and survey of stormwater management professionals. A checklist of design considerations should serve as a useful design aid to practitioners in stormwater management. A discussion on the accuracy of stormwater detention design and the differences between single and multi-frequency control provides the practitioner with a basis for understanding the state-of-the-art limits in accuracy of design. In addition, the interrelations between the appearance of outlet structures and the considerations related to economics, public safety and hydraulic function are discussed. Finally, recognizing the youthful stage of the state of the art of detention basin design, research needed to advance the state of the art is outlined.

1.0 INTRODUCTION

1.1 Background

Traditionally, the prevention of flooding and other drainage related problems in urban areas has been accomplished by a "conveyance approach," which relied on a system of swales, curbs and gutters, inlets, storm sewers and channels to quickly carry water away. In contrast, the last 15 years has seen an introduction of the "storage detention approach" in urban stormwater management. The approach is to capture and temporarily detain all or part of the runoff during and immediately after a rainfall or snowmelt event, and then release it at controlled rates downstream. In so doing, nuisance and flooding damage can sometimes be reduced or prevented, thereby allowing downstream conveyance facilities to be smaller and less costly. In recent years, the idea of using detention facilities for water quality enhancement has also been gaining wider acceptance. Figure 29-1 provides a schematic comparison between the conveyance approach and the detention approach.

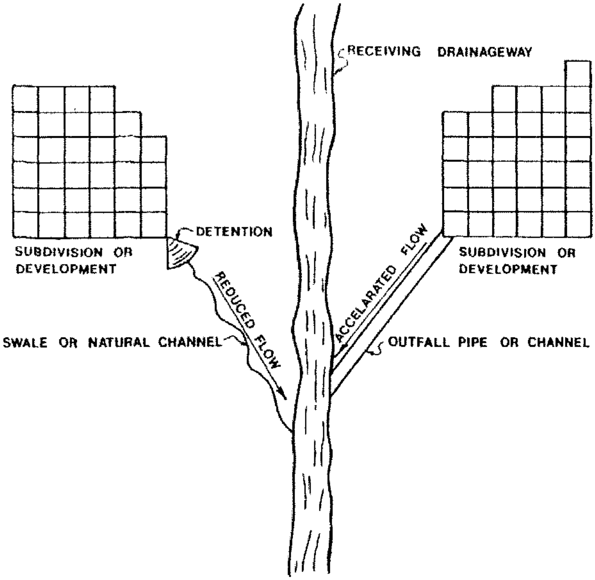

FIGURE 29-1. Conveyance vs. Detention Approach

Outlet control structures are an important component of stormwater detention facilities since they control both the rates of release from the pond and the water depth and storage volume in the pond. Figure 29-2 schematically illustrates a detention pond embankment cross-section and associated outlet structure. Outlet control structures are often called upon to perform several functions in an urban area, some of which may conflict. In urban setting, the complexity of the task faced by a designer of detention outlet controls also increases. Although practicing engineers have developed a variety of outlet control works, very little can be found in technical literature about this topic.

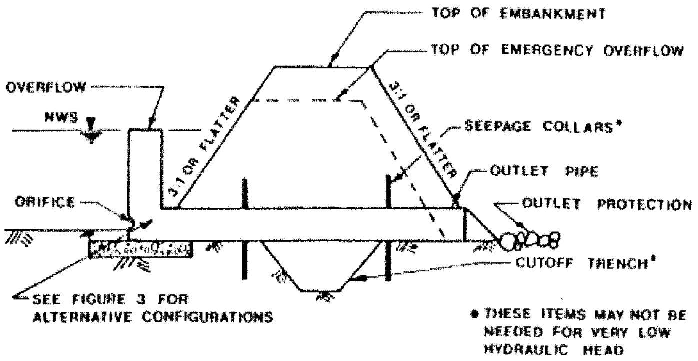

FIGURE 29-2. Basic Outlet Through Detention Pond Embankment

The growing use of detention facilities and the proliferation of independently developed approaches to their outlet structures suggested a need for a Task Committee to review systematically and report on the state-of-the-art of outlet controls; consequently, the Task Committee for the Design of Detention Outlet Control Structures was formed. The Committee consisted of four control members: Richard McCuen, Ronald Rossmiller, Ben Urbonas (Chairman), and Kenneth Wright. In addition, the following seven corresponding members contributed extensively their time and energies to this task: D. Earl Jones, Jr., David Kibler, Arthur Miller, Samuel Marcey, Peter Stahre, Richard Kropp, James Stubchaer, and Jonathan Jones.

The charge of the Task Committee was comprehensive in that it was asked to address hydraulic function, public safety, maintenance, and aesthetic aspects of outlet controls. These four topics, along with water quality as a fifth major subject, are addressed in this paper primarily for the smaller, individual development, on-site ponds. Major regional detention reservoirs are often controlled by various state dam and reservoir criteria and, because they are larger and more expensive to build, more comprehensive engineering investigation and analysis effort usually is focused on them than on smaller on-site ponds. Much of the information presented herein also will be applicable to larger facilities and both "on-stream" and "side channel" detention facilities. Furthermore, "detention ponds" as defined herein, encompasses both "wet" ponds (those that maintain permanent pools) and "dry" ponds (those which store water only during runoff events).

The Committee was tempted to expand its scope to address stormwater detention in its entirety because it is difficult to address outlet controls autonomously since they are a part of the detention system as a whole and interrelate with tributary watershed hydrology and storage basin geometry. To avoid an excessively narrow focus, the Committee decided to include in its report a limited discussion of general stormwater detention concepts, which will, hopefully, clarify where and how outlet controls interact with the overall system.

The Committee limited its considerations to surface storage ponds. Although much of what is discussed here also is applicable to detention storage on parking lots, on rooftops, in grassed swales, and in underground storage vaults or tanks, such facilities were not individually addressed. It is noteworthy, however, that such types of localized on-site detention facilities are extensively used and each type require their own special considerations that may not be addressed here.

1.2 Organization of Paper

This paper begins by summarizing the results of a literature search and survey of professionals conducted by the Committee. Following this background information, a discussion regarding general design objectives and criteria for ponds is provided, including a series of questions which merit consideration during design. Following the discussion of design objectives and criteria, five topics are addressed including: 1) hydraulics, 2) water quality, 3) safety, 4) maintenance, and 5) aesthetics. The paper concludes with a discussion of research needs. References are provided covering nearly all aspects of detention for the reader seeking additional information.

2.0 LITERATURE SEARCH

A search revealed little about urban detention pond outlet control structures in the English language literature. Practically all available literature addressed structures associated with larger water resources projects and flood control reservoirs. The primary references for design of small structures were found to have been developed by the Soil Conservation Service (SCS). Although the SCS guidelines are comprehensive, their emphasis is on non-urban settings. SCS treatment of factors such as safety, aesthetics, and maintenance are not always reflective of the special constraints and needs imposed by urban settings. Nevertheless, the SCS publications[1,2] offer the urban stormwater detention outlets designer considerable guidance in hydraulic and hydrologic design techniques and should be a part of every designer's library. Comprehensive references regarding stormwater detention are available,[3,4] but even these publications contain only limited information on outlet structures. Other references that provide background information on hydrology, hydraulics and hydraulic structures, dam construction, and other factors inextricably related to detention ponding are provided[5,6,7,8] for the reader seeking general

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[1] *Earth Dams and Reservoirs.* Revised 1981. Technical Release No. 60 Soil Conservation Service, U.S. Department of Agriculture.

[2] *U.S. Soil Conservation Service Hydrology Handbook.* U.S. Soil Conservation Service, Department of Agriculture.

[3] *Proceedings of the Conference on Stormwater Detention Facilities.* 1982. Co-sponsored by Engineering Foundation and ASCE Urban Water Resources Research Council, ASCE.

[4] Sheaffer, J.R., and K.R. Wright. 1982. *Urban Stormwater Drainage Management.* Marcel Dekker, Inc.

[5] Daugherty, R.L., and J.B. Franzini. 1975. *Fluid Mechanics with Engineering Applications.* McGraw-Hill Book Company 1965 (and updated editions).

[6] *Design of Urban Highway Drainage; The State of the Art.* August 1979. U.S. Department of Transportation, Federal Highway Administration, Offices of Research and Development, Implementation Division (HDV-21).

information. In summary, the Committee found a lack of helpful technical literature and had to draw upon experiences of its members and their colleagues in the urban stormwater management field.

3.0 SURVEY OF PROFESSIONALS

In the fall of 1982, the Committee prepared a questionnaire concerning stormwater detention practices, observations and opinions, which was mailed to approximately 150 stormwater management professionals including consulting engineers, government officials and university professors. Sixty-five responses (43% response rate) were received, of which 45% were from consulting engineers, 37% from government officials, 8% from universities, and 10% from unidentified respondents. The 43% questionnaire return rate was excellent but easily could reflect a response bias from the more sophisticated practitioners.

The responses to these questions are summarized in Table 26.1. All respondents believed that detention plays an important role in stormwater management. A majority of the respondents evidently adopt a somewhat sophisticated approach to design, as 60% of all respondents stated that they designed for multi-frequency control and 65% have considered water quality enhancement at some time in the past. Most respondents believe the detention ponds not only reduce peak flows, but also help to enhance water quality through settling out sediments.

It is difficult to summarize the responses to the essay type questions; nevertheless, the following summary provides the general sense gleaned from the 65 responses:

1) Hydraulic function was considered to be the most important design consideration in design of outlet structures.
2) Safety, other than assuring prudent design, was not considered to be of great importance by the majority of respondents.
3) Maintenance and aesthetics were considered of little importance by the majority, except for the local government respondents who overwhelmingly stated that safety and aesthetics deserve much greater attention than is the prevailing practice in design of outlet structures.
4) Water quality design considerations were deemed relatively unimportant by most respondents, but most felt that ponds improve water quality.
5) The majority expressed the opinion that multi-frequency outlet controls were especially important in overall stormwater management; at the same time, the overwhelming majority did not consider it important to convert existing single frequency outlet controls into multi-frequency facilities.

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[7] *Hydraulic Charts for the Selection of Highway Culverts* and *Capacity Charts for the Design of Highway Culverts.* U.S. Department of Transportation, Federal Highway Administration as Hydraulic Engineering Circulars 5 and 10 respectively.

[8] Linsley, R.K., M.A. Kohler, and J.L.H. Paulhus. *Hydrology for Engineers.* Second edition, McGraw-Hill Series in Water Resources and Environmental Engineering.

TABLE 29-1

Summary of Responses to Questionnaire

Question	Percent Responding Yes by Professional Category			
	Consultants	Government	Academic	Unidentified
Percent of all responses by group	45	37	8	10
Have you designed or analyzed a detention pond?	93	72	75	100
Did you design to control a range of runoff frequencies?	58	59	33	100
Did you consider water quality enhancement?	75	47	67	80
Does detention play an important role in stormwater management?	100	100	100	100
Have you reviewed or inspected detention ponds?	89	88	50	100
Did you feel they functioned properly to reduce peak flows?	84	95	100	75
Did you feel they improved quality by settling solids?	80	76	100	100

The Committee gathered from the responses that there is inconsistency and some confusion among practitioners concerning the design of outlets and ponds intended to control discharges for more than one recurrence frequency. This indicates a need for expanded ASCE training opportunities in multi-frequency design. Most detention pond design for water quality enhancement is reportedly undertaken to satisfy local government requirements and is not otherwise a general practice among designers, despite the fact that water quality can be enhanced by stormwater detention.

Finally, the Committee took special note of the responses from local government officials regarding the importance of safety, maintenance and aesthetic considerations in the design of outlet controls. These individuals represent organizations that are ultimately responsible for operation and maintenance of these facilities (i.e., owners of the facilities) and their opinion deserves special recognition.

4.0 DESIGN OBJECTIVES AND CRITERIA

Instead of treating the design of runoff detention facilities as an isolated design problem, the overall drainage system goals, objectives, and design criteria need to be clearly understood by the designer before starting the design. Next, the role that detention storage plays in helping to achieve these goals and objectives needs to be defined. Only then can the objectives and design criteria for the outlet works be selected, tabulated, and checked for consistency with the overall system objectives.

The Committee is of the opinion, reinforced by the survey, that detention ponds and outlets too often are designed to meet minimum criteria established by a local regulatory agency, often without regard to the goals and objectives of the overall urban drainage system. Without a full understanding of the fundamental planning strategy for the urban drainage system, which interrelates competing objectives such as economic considerations, regional development, transportation, water supply and environmental quality, the design of the detention ponds and their outlets is a haphazard exercise.

4.1 Design Objective Categories

This report is centered on the following five categories of specific design objectives:

1) Hydraulic Function—This deals with the inflow-outflow hydrographs and their return frequencies, storage volumes, depths required, and outlet features necessary to achieve the design performance objectives.
2) Water Quality—Urban runoff often contains water pollutants, such as solids, heavy metals, bacteria, pesticides, nutrients, and other substances. Detention ponds can, under certain circumstances, remove such pollutants to varying extents through different mechanisms. Enhancing pollutant removal should be a design objective for most urban ponds.
3) Safety—Safety of the detention pond and outlet works, both in a passive condition and when functioning to control upstream stages and downstream discharges must be addressed in design. In addition, embankment stability and the consequences of embankment failure must be addressed. A hazardous detention pond may be worse than none at all.
4) Maintenance—As an objective, a detention pond and its outlet works should be relatively maintenance free. Required maintenance is facilitated by good equipment access for inspection, cleaning, repair and reconstruction.
5) Aesthetics—A visual amenity consistent with the neighborhood and/or coupled with multi-use potentials related to recreation can create benefits well beyond those related specifically to its hydraulic function. Evidence suggests that ponds that become neighborhood amenities receive superior on-going maintenance.

4.2 Checklist of Questions

With respect to the previous five design objectives, the Committee recommends that a project engineer approach the matter of defining design objectives for outlet works by addressing a set of questions pertinent to the detention facility and its outlet requirements. The following kinds of questions merit consideration:

4.2.1 Hydraulic Function

1) What are the hydrologic characteristics of the tributary basin?
2) What is the future development potential of the basin and resulting runoff peak flows, volumes, and hydrograph characteristics?
3) What are the design inflows for various frequencies of occurrence?
4) What are the design outflow objectives?
5) How does and how should the detention facility interact with the overall basin drainage and flood control system?
6) Should the detention pond outlet control only one design flow, or a full range of frequencies?
7) How precise do outflow controls need to be in terms of matching the desired outflow hydrograph?

4.2.2 Water Quality

1) Is it important to detain a large portion of the volume of runoff, or is it satisfactory to merely reduce the peak of the runoff hydrograph?
2) To what extent should water quality be improved and for which pollutants?

4.2.3 Safety

1) Is the proposed detention facility located in a residential, commercial. industrial, or agricultural setting and what are the implications of the particular setting?
2) Is the pond readily accessible to the public? Is it safe?
3) Can the outlet works be simplified in any way to increase reliability and safety?
4) What is the risk if an event larger than the design flood overtops the embankment?

4.2.4 Maintenance

1) How will trash, debris, and sediment be controlled?
2) Who will maintain the detention facility? Are resources available for maintenance?

3) What will be the rate of sediment accumulation in the basin? How will sediments be removed? Where will they be disposed of?

4.2.5 Aesthetics and Multiple Use

1) Should the detention facility have multi-purposes such as recreation and water quality improvement? Should the basin be wet or dry?
2) Is appearance a factor? Is the pond readily visible to the public?
3) Can it serve as an amenity to the neighborhood?

4.2.6 General

1) Will benefits from the pond exceed the total costs of land, construction, maintenance, added infrastructure extensions, investment mortization and reduced tax revenues?
2) Why is the detention pond being constructed in the first place?

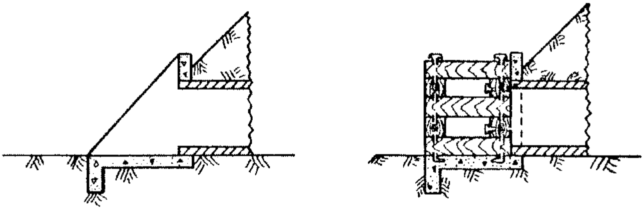
These questions may need to be supplemented with other questions that are applicable to the specific locality and proposed storage facility in question.

4.3 Summary

The design process should include a clear statement of the derived design objectives and criteria for review by the regulatory authorities. A tabulation of such goals and criteria also provides the engineer with a checklist to ensure that the design is adequate. Goals, criteria, and design considerations for the five major subjects addressed herein are now explored in greater detail.

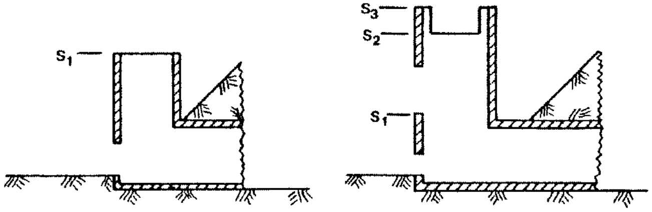
5.0 HYDRAULIC FUNCTION

Hydraulic function is considered by the majority of engineers to be the most important element of detention outlet control structure design. The state-of-the-art in hydraulic design is to take individual hydraulic control elements such as orifices, weirs, and outlet pipes, and combine their hydraulic functions to produce integrated stage-discharge behavior. Four examples of inlets for outlet structures are presented in Figure 29-3. The concept of combining orifices and weirs to produce a composite discharge curves is illustrated by Figure 29-4. This approach assumes that the individual hydraulic control elements for each stage will function independently, and the total discharge for each stage is the sum of the discharges of each control element at that stage. However, the assumption of hydraulic independence between several control elements in an outlet appears to have never been tested in the field or the laboratory.



SIMPLE HEADWALL

MULTIPLE STAGE TIMBER INLET



SINGLE STAGE RISER

MULTIPLE STAGE RISER

FIGURE 29-3. Examples of inlets for outlet structures

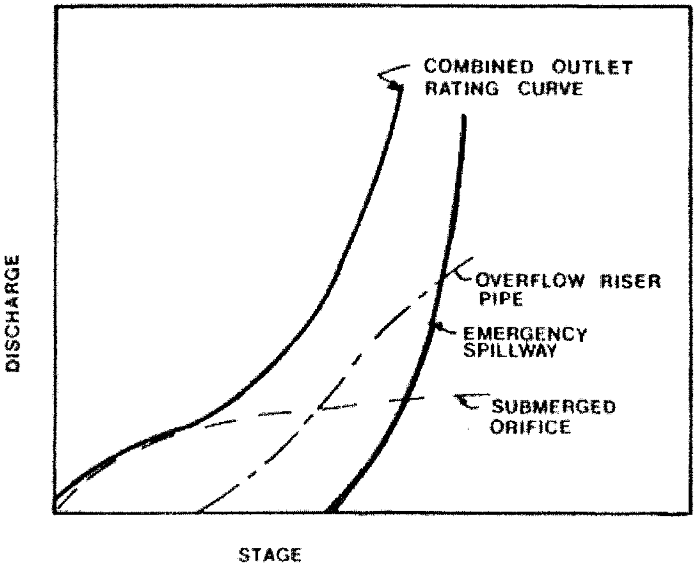


FIGURE 29-4. Development of outlet rating curve

Much attention may be focused on "optimizing" the outlet size and the pond volume. This too often is done with little understanding of how the individual pond fits in the overall stormwater management system and without understanding of the accuracy or reliability of the information being "optimized." The Committee's efforts included the evaluation of accuracy in design and the merits of controlling a single design versus controlling several events of different recurrence frequencies.

5.1 Accuracy of Detention Basin Outlet Design

An understanding of accuracy in detention basin design is an important determinant in establishing stormwater management policy and in assessing the associated benefits and costs. Design accuracy is a function of the stormwater management policy as well as the hydrologic and hydraulic models used in design. These factors may either introduce a bias or cause a lack of precision, with bias and precision being components of net accuracy. Bias is defined as a systematic variation from the true value, and precision is defined as nonsystematic, random, or chance error.

A common practice is to measure the effectiveness of a detention basin design by how closely it will match the post-urbanization peak rate of runoff to the pre-urbanization peak rate of runoff for all recurrence storm frequencies. As evident from the work of Kamedulski and McCuen,[9] a stormwater management policy that requires control of a peak discharge for a single exceedance probability will lead to a bias in the computed design discharges for other exceedance probabilities. For a case study in Maryland, McCuen showed that a policy statement requiring control of single frequency could introduce bias in peak discharge estimates ranging between –65% and +67% for other frequencies. This clearly also introduces a bias in the volume of detention storage and the design characteristics of the outlet facility.

In addition to the policy statement, the following three items have an effect on the design accuracy: 1) the hydrologic model used to compute the peak discharges, 2) the hydrologic/hydraulic model used to determine volumes of detention storage, and 3) the hydraulic model used to size the outlet facility. In comparison with the first two models, design inaccuracies introduced by the hydraulic model used to size the outlet facility are considered to be relatively small.

The greatest source of design inaccuracy is probably the hydrologic model that is used to estimate the before and after development peak discharges. Inaccuracy is the result of inaccuracies in the input assumptions and parameters (i.e., rainfall, land use, drainage area, time of concentration, etc.) and the inability of the input parameters to reflect hydrologic responses of the watershed. While the Task Committee does not advocate the use of regression equations in urban hydrology, it notes that regression equations developed for each state by the U.S. Geological Survey often report standard errors of estimate from 30% to 50% and in one known case,[10] reported a

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[9] Kamedulski, G.E., and R.H. McCuen. September 1979. "Evaluation of Alternative Stormwater Detention Policies." *Journal of the Water Resources Planning and Management Division*, ASCE Volume 105, pp. 171-186.

[10] *Techniques for Estimating Flood Peaks, Volumes and Hydrographs for Small Streams in South Dakota.* September 1980. Water Resources Investigations 80-80, U.S. Geological Survey.

maximum range of +152% and –60% for the peak flow estimates and +136% and –58% for the volume estimates.

Other methods typically can have similar or even larger standard errors.[11]

According to the Water Resources Council report, the coefficient of variation for the time of concentration is considerably larger than the coefficients for the other input parameters. McCuen[12] showed in his analysis that the errors in the input parameters may not cause a significant bias in the volumes of detention storage; however, the precision of the control outlet diameter and the high-stage weir length are highly dependent on the accuracy of the high-stage peak discharge. Similarly, the precision of the low-stage orifice area is directly dependent on the low-stage peak discharge. Both depend on the accuracy in estimating the before development peak runoff rates.

The accuracy of any model used to estimate the volume of detention storage is an important element in the accuracy of design. Most storage volume models involve a relationship between the ratio of the volumes of required storage and runoff and the ratio of the allowable and peak inflow rates. In comparing five frequently used methods, McCuen showed that the precision of the storage/runoff volume ratio was approximately 12%. This variation can produce an error in the storage volume of almost 6%. Extreme deviations from the use of the five models evaluated can produce errors in the storage volumes as large as 150%. It is evident that the choice of a model for estimating the volume of detention storage has a significant effect on the required volume and the size of the outlet facility.

In addition to the hydrologic and hydraulic models and the local stormwater runoff management policy, design accuracy is affected by the choice of a design storm. Biases of approximately 25% in the peak discharge may typically result from the choice of the design storm. While the bias introduced by the choice of design storm is not expected to introduce a significant bias in the volume of detention storage, the bias in peak discharge can introduce a bias in the riser diameter, low-stage orifice area, and the high-stage weir length. As another example, Kurtyka[13] showed that routine precipitation measurements typically are more than 15% too low and that measurements of extreme intensities often are more than 40% too low, which illustrates an important major bias in basic hydrologic data.

Unlike many other fields of engineering, the statistics of hydrologic data and modeling techniques can have very wide bounds of design confidence. The uncertainties discussed above, if applied to structural analysis, would be considered intolerable and would be dealt with by the use of very large safety factors. On the other hand, drainage and flood control engineers work with such uncertainties every day whether they realize it or not. Thus, the Committee cautions the professionals in

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[11] *Estimating Peak Flow Frequencies for Natural Ungaged Watersheds.* 1981. Water Resources Council, Hydrology Committee, Washington, DC.

[12] McCuen, R.H. 1983. "Design Accuracy of Stormwater Detention Basins." Technical Report, Department of Civil Engineering, University of Maryland.

[13] Kurtyka, J.C. February 1953. "Precipitation Measurements Study." State Water Survey Division, Urbana, Illinois.

stormwater management against over-optimizing hydraulic design of outlet structures. It is important to recognize the earlier bias and precision discussion when designing the hydraulic function of the outlet control structures. More often than not, two significant figures in calculations are sufficient for hydraulic design and any refinement beyond this is unjustified and definitely beyond the state-of-the-art.

5.2 Control of Single Recurrence Frequency

While two-stage and multi-stage control are becoming more widely used, the single stage control is still the most widely used outlet control design. The recurrence frequency of the runoff to be controlled is either dictated by a regulatory agency or is selected by a design engineer or his client. In practice, the control is often anywhere between the 2-year and 100-year runoff event. The hydraulic control of the outlet is provided either by an outlet pipe an orifice or a weir. An emergency spillway is often provided to safeguard against catastrophic failure.

Some type of a routing procedure is used to size both the outlet and the pond volume. The Committee recommends the "Modified Puls" or storage-indication method for routing inflow hydrographs through reservoirs. The design and routing process is straightforward, but it does require the use of an iterative trial and error procedure to size the basin and outlets. In practice, a variety of orifice and weir configurations are used by engineers. Suitable discharge coefficients for common orifice and weir configurations may be found in most hydraulic texts and in handbooks such as King and Brater.[14] When the water surface elevation just downstream from a weir is higher than the crest elevation of the weir, it is an indication that a submerged flow condition may exist. Formulas have been developed for submerged weirs and are available in most hydraulic handbooks.[15] It is important that engineers recognize that the simple, non-submerged flow, weir equations need to be modified whenever a submerged weir condition is encountered.

In some outlet configurations, the use of streamlined elbows and transitions between the inlet or riser and the outlet pipe will minimize the potential for excessive or less predictable losses and provide for smoother transition of flow. Flow separations at inlets and bends under high velocity conditions can cause damaging cavitation and/or excessive hydraulic losses that are difficult to calculate with reliability and accuracy. Vortex potential at the inlet to the outlet should also be studied. Thorough hydraulic analysis must account for major and minor head losses, and standard fluid mechanics and hydraulics texts provide loss coefficients for bends, elbows, inlets, etc.

The susceptibility of various types of inlets to clogging by debris and trash needs to be considered when estimating the hydraulic capacities of inlets to the outlet. In many instances, trash and debris potential will dictate a need for trash racks. Trash racks need to be sufficiently large so that their partial plugging will not adversely restrict flows reaching the control outlet. No universal guidelines exist for the design of trash racks to protect urban stormwater detention control outlets, although a commonly

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[14] King, H.W., and E.F. Brater. *Handbook of Hydraulics*. McGraw Hill, New York, New York.

[15] U.S. Bureau of Reclamation. 1977. *Water Measurement Manual*. U.S.G.P.O., Washington, DC 20402.

used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice. Regardless of how the trash rack is sized, its effects on the hydraulic function must be calculated under the "no blockage" and "practical maximum blockage" conditions to assess how the outlet will perform. The debris problem usually is greatest during the most infrequent runoff events.

The effectiveness of a single frequency control has been addressed by Kamedulski and McCuen for a single detention facility and by Urbonas and Glidden[16] for a system of random on-site detention facilities. Both papers concluded that a single frequency control will not effectively control recurrence frequencies other than the design frequency. Furthermore, Urbonas and Glidden showed in the Denver area that the smaller and more frequent runoff events cannot be effectively limited to historical peak flows along major drainageways whenever these drainageways receive stormwater discharges from many detention ponds. These ponds were designed using the policy of limiting the "after development peak discharge" to the "before development peak discharge."

5.3 Control of Multiple Recurrence Interval Events

Recognizing that single stage outlet facilities do not control flood discharges for more than a single recurrence interval, the recent trend in stormwater management policy is towards the use of two-stage outlet facilities, which control two pre-selected points on the stage/frequency curve to the "before development peak discharge" rates. Urbonas and Glidden showed that a two-stage design to control the 10- and 100-year peaks along major drainageways receiving flows from a number of on-site ponds was effective in controlling the 10- and 100-year peaks along the study major drainageway in the Denver area. In addition, Brulo et al.[17] have described the advantages of two-stage outlets in controlling multiple flood frequencies.

Methods for designing two-stage risers are often based on hydraulic equations for weir and orifice flow in much the same way as single-stage design. In two-stage riser design, the lower-stage orifice controls the more frequent event while the larger, less frequent event is controlled jointly by both the high-stage weir and the low-stage orifice. Schematics of two types of multi-stage outlets are shown in Figure 29-3. The outlet conduit should be sufficiently large to carry the high-stage peak with the water surface in the riser being below the crest of the high-stage weir. Optimally, physical separation of multi-stage outlets will be sufficient to prevent hydraulic interference at all foreseeable operating stages.

A recent design development by the Soil Conservation Service[18] provides a method for designing two-stage outlet facilities. The system considered by SCS is a drop-inlet structure consisting of a barrel (i.e., a riser), a rectangular orifice at the lower

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[16] Urbonas, B.R., and M.W. Glidden. November 1983. "Potential Effects of Detention Policies." *Proceedings of the Second Southwest Regional Symposium on Urban Stormwater Management.* Texas A&M University.

[17] Brulo, A.T., D.F. Kibler, and A.C. Miller. August 1984. "Evaluation of Two-Stage Outlet Hydraulics." *Proceedings of the ASCE Hydraulics Conference on Water for Resource Development.* Cour d'Alene, Idaho.

[18] Woodward, D.E. April 1983. "Hydraulic Design of Two-Stage Risers by Shortcut Method." USDA-SCS Engineering Technical Note. No. 28, Northeast Technical Service Center, Broomall, Pennsylvania.

part of the riser and a weir at the top. The operational feature peculiar to this type of structure is that the type of hydraulic control at a given time depends upon the relative position of the water surface inside and outside the riser. The distinct feature of the routing technique is that it does not use a rating curve because it requires as much time to develop the curve as to solve the problem. Instead it uses an iterative technique to size individual hydraulic control elements. The design method was validated by Brulo, Kibler and Miller in a series of computer modeled tests done separately on five hypothetical watersheds.

The Committee advocates implementation of detention ponds that utilize multiple recurrence interval events for a design basis. State-of-the-art practice currently favors control of two (rather than three or more) events.

5.4 Alternative On-site Detention Methods

Special attention must be given to the design of outlet structures for controlling runoff from rooftops, parking lots, and small on-site swales. Because runoff volumes from such areas are small, the required outlets are also small, which increases the potential for plugging by debris. In addition, because of the multi-purpose nature of these small on-site control facilities, the outlet must release temporarily stored water in a reasonable amount of time. As an example, parking lots must drain relatively fast in order not to be a nuisance.

Since the detained water may occasionally cause an inconvenience to those using the land for also intended other purposes, the temptation is great to modify or eliminate the on-site detention by changing the outlet. For example, perimeter curbing that serves as the outlet control for parking lot storage is frequently damaged by owners of the lot to release water at a faster rate, so that those using the parking lot immediately after a storm are not inconvenienced. This is an institutional problem that needs to be addressed locally to ensure continual and reliable functioning of such outlet facilities. Design of parking lot grading to minimize people/water conflicts and to recognize pedestrian movement needs and patterns can reduce unauthorized facility modifications.

Proper location and protection of such outlets can reduce these problems. In addition, public education programs may be helpful. Manufactured outlets that are more difficult to alter can be used to protect roof drains. Manufactured grates located on the surface discourage tampering of parking lot and swale outlets placed above ground. These grates also keep the outlets clear of debris and preserve their hydraulic capability even when the grates are partially plugged. Grated inlets are also notorious debris catchers and are often plugged when they are most needed; accordingly, the use of oversized grates should be considered where grate usage is unavoidable. The outlet facilities should be accessible and located where their maintenance needs are easily noticed, so that the public will be involved in the maintenance process. Regular maintenance of parking lots and other normally dry public detention areas should assure debris removal, especially removal of paper, plastics and gravel or rocks.

6.0 WATER QUALITY CONSIDERATIONS

Control measures used in stormwater management such as detention and retention basins (retention meaning ponds that maintain a constant, year round pool), infiltration techniques and vegetative swale systems have been demonstrated to provide water quality enhancement in addition to flood control.[19] Suspended solids are the primary pollutants found in urban storm runoff and are the pollutants most effectively trapped in detention facilities. Removal of the suspended solids also removes a range of pollutants that have a high affinity for particulates. Sorbed to the surfaces of these particulates are a variety of pollutants; of particular concern are petroleum-based organics, heavy metals, and un-dissolved phosphates.

Sedimentation analyses may not be applicable to trap efficiency of particulate pollutants in urban storm runoff, as the relationships between grain size and particle settling velocity do not account for the differing specific gravities of the various particles. In addition, the association of several pollutants with particulates, especially for varying pollutant concentrations, has not been determined. The trap efficiency of stormwater detention facilities will depend on the specific pollutant, its concentration, the storm event, detention times, flow velocities, and the type of management technique employed, as well as its specific design characteristics, among other factors.[20,21,22] In general, sedimentation designs can be utilized for the control of runoff pollution with the caveat that individual pollutant removals do not correlate well with total suspended solids (TSS) removal. In addition, sedimentation ponds usually will not adequately remove smaller particle sizes, which remain suspended and pass through the system. This can be of particular concern as the literature has associated these finer particles with many of the pollutants found in urban runoff. While significant removals of the smaller particle sizes have been reported due to agglomeration, coagulation may be necessary where the concentrations remaining after treatment threaten water quality goals, but such treatment is rarely justifiable or feasible.

Few studies of the settleability of urban runoff pollutants have been undertaken, although summaries of the water quality performance of detention facilities are available. Two of the more important studies are those performed at the New Jersey Water Resources Research Institute (NJWRRI) at Rutgers University[23] and the Occoquan Watershed Monitoring Laboratory (OWML) in Manassas, Virginia. In both cases, samples of urban runoff were placed in laboratory settling tubes and the quantity of each pollutant settling in a given time period determined. The results of

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[19] Randall, C. 1982. "Strom Water Detention Ponds for Water Quality Control." *Proceedings of the Conference on Storm Water Detention Facilities*. ASCE.

[20] Ferrara, R.A., and P. Witkowski. April 1983. "Storm Water Quality Characteristics in Detention Basins." *ASCE Journal of Environmental Engineering*, Volume 109, No. 2, pp. 428-447.

[21] Randall, C., K. Ellis, T.J. Grizzard, and W.R. Knocke. 1982. "Urban Runoff Pollutant Removal by Sedimentation." *Proceedings of the Conference on Storm Water Detention Facilities*, pp. 205-219. ASCE.

[22] Whipple, Jr., W., and J.V. Hunter. December 1980. "Detention Basin Settleability of Urban Runoff Pollution." Water Resources Research Institute, Rutgers University.

[23] Whipple, Jr., W., and J.V. Hunter. April 1980. "Settleability of Urban Runoff Pollution." Water Resources Institute, Rutgers University.

these studies demonstrate that substantial removals of pollutants in urban stormwater can be achieved by gravity sedimentation if turbulent mixing can be avoided and the detention time is long. The pollutants most susceptible to removal are TSS, hydrocarbons, lead, Biochemical Oxygen Demand (BOD), and total phosphates. Trap efficiencies of 70% for the TSS, hydrocarbons and lead, 50% of the BOD and phosphates and at least 33% of Chemical Oxygen Demand (COD), Total Organic Carbon (TOC), and Total Nitrogen (TN) were reported.

The NJWRRI study was followed by an analysis of pollutant settleability in detention basins fitted with restrictive outlet controls. The settleability of various runoff pollutants in large settling tubes over a 32-hour period was found to provide a reasonable approximation of the trap efficiency of a particular detention basin for the same pollutant. A similar study of detention basin trap efficiency by the Pollutant Control Division of Ottawa-Carleton[24] reported that treatment by sedimentation provided removal of over 95% of the pollutant load for TSS, total phosphorus, fecal coliform and fecal streptococcus and up to 50% removal of BOD. The results obtained in each of these studies showed considerable variation from site to site and indicate that similar variability should be anticipated in other regions. Therefore, these findings should be transposed only to locations assuredly having conditions similar to those at the research sites.

As a result of the studies by the OWML and NJWRRI, stormwater management (SWM) programs in Fairfax County, Virginia[25] and the State of New Jersey have been requiring various water quality controls in addition to flood and erosion controls. The water quality controls are designed to address a small design storm that can be treated by prolonged retention in detention basins or through infiltration. The two SWM programs have developed a series of design recommendations as reported by Kropp,[26] that directly deal with the quality control aspects of stormwater control facilities. The majority of these recommendations deal with detention basins, which are the most popular stormwater management technique in both the New Jersey and Fairfax programs. Design recommendations include:

1) Use long, narrow basin configurations with length to width ratios of 2:1 to 3:1. The distance between outlet structures and inflow points should be maximized to lengthen flow paths and detention times. The ratio of total inflow volume to detained volume obviously is also a significant factor and is frequency-related.

2) Riprap or other energy dissipating devices should be used at all inflow points to reduce flow velocities in order to promote settling of particulates and minimize resuspension of settled pollutants.

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[24] Gietz, R.J. March 1983. "Rideau River Storm Water Management Study—Urban Runoff Treatment in the Kennedy-Burnette Settling Pond." Pollution Control Division Works Department, Regional Municipality of Ottawa-Carleton.

[25] Regulations for the Review Zone of the Delaware and Raritan Canal State Park, Delaware and Raritan Canal Commission, New Jersey Department of Environmental Protection. November 1979.

[26] Kropp, R.H. 1982. "Water Quality Enhancement Design Techniques." *Proceedings of the Conference on Storm Water Detention Facilities*, ASCE.

3) Water tolerant vegetative cover should be used on the basin floor and sides to maintain high infiltration rates, promote filtration of runoff and aid decomposition of settled particles during dry cycles.

4) It is possible to incorporate pervious low flow channels (rock or grass-lined) under certain conditions. Pervious low flow channels increase travel time through the basin and promote sorbtion of pollutants to particulates. Rock-lined channels are suggested, with underdrains installed to provide soil filtration of all runoff. A disadvantage of pervious low flow channels is that they are more difficult to maintain than impervious channels, composed of concrete, asphalt, etc.

5) Construct basins in series with two or more stages or terraces. A sedimentation basin should receive the initial flows with an overflow to a larger flood control basin.

6) Outlet structures should be designed for release rates that produce approximately a 40-hour drawdown time for the runoff based on a water quality design storm or a specified depth of runoff. The New Jersey program uses the 1-year, 24-hour storm for its water quality design storm. The Fairfax County program uses a depth of runoff equal to 0.86 inches for impervious surfaces and lesser depths for pervious surfaces. The Fairfax County program suggests the use of a perforated riser with ½ to 1-inch diameter holes for orifice sizes. The New Jersey program does not allow orifices less than 3 inches in diameter to minimize the potential for clogging. Larger diameter pipes (12- to 15-inch) with cover plates containing the proper size orifice are recommended in place of small diameter conduits to facilitate maintenance. The orifice is required to be located at least 3-inches above the invert of the larger pipe to reduce clogging of this restricted outlet due to sedimentation.

7) Special care should be exercised when designing trash racks to protect these restricted outlets. Racks with large surface areas and properly sized openings are recommended. A riser pipe outlet should be encircled with a protective screen placed at a radius several diameters greater than the riser diameter. Orifice plates should be protected with trash racks with at least 10- to 15-square-foot surface areas. The openings in the protective devices should be small enough to restrict debris from reaching the outlet, yet large enough to prevent clogging of the screen or rack. Two-stage racks, or screens in series with the outer stage designed to restrict large debris and an inner rack or screen to restrict smaller debris, are suggested. In all cases the protective device must be hinged or in some way easily removable to facilitate clearing of accumulated debris and sediment.

8) Runoff entering the basin should be diverted from impervious areas to pervious areas, to promote infiltration, filtration, and sedimentation. This provides for maximum interaction between the runoff and groundcover, and promotes sorbtion of pollutants to particulates. Sedimentation and filtration will be optimized as long as the depth of flow does not exceed the height of

the vegetation, which may best be accomplished through the promotion of sheet flow by such techniques as level spreader berms.

The literature suggests that control measures can be tailored to specific pollution problems. Nutrient loadings are subject to higher removals in wet ponds due to the biological activity found in permanent pools of water. Dry ponds, on the other hand, have been found to have negative removals for nitrogen forms and are significantly poorer in the removal of phosphorus forms. This finding has resulted in the State of Maryland requiring that all water quality ponds be wet ponds.

For detention basins, the design can be based on the settleability of a target pollutant with the design settling velocity correlated to the desired percent removal of the target pollutant. It is suggested that column-settling analyses be performed on actual runoff samples from the planning areas to establish settleability data that will be site specific before deciding on local design requirements for ponds and outlets.

If the settleability data indicate that the trap efficiencies will not be sufficient to meet water quality goals, infiltration of the first flush may offer a solution. The use and design of infiltration techniques have been discussed in detail elsewhere.[27] These techniques have the potential for significant removals of suspended and dissolved loadings. However, care must be exercised when dealing with soils with rapid percolation rates, areas of high water tables, and pollutants that have a migration potential might threaten ground water quality.

7.0 SAFETY

A stormwater detention facility that encompasses an embankment and outlet works is, in effect, a dam and reservoir. Rapid failure of such dams may cause significant damage in urban areas. Foundation seepage, dam overtopping due to large floods, erosion, deficient spillway design, outlet works malfunction or failure, and inadequate freeboard are often causes of dam failures.[28] Litigation following dam failures often involves a rule of common law that is referred to as "doctrine of strict liability." The leading case on strict liability is Fletcher v. Rylands, L.R. 1, Ex. 265 (1866) from England. It has been applied throughout the United States under English Common Law. Here, the plaintiff was damaged by his property being flooded by water, which, without any fault on his part, broke out of a reservoir constructed on the defendant's land by the defendant's orders and that was maintained by the defendant.

The House of Lords, L.R. 3 H.L. 330 (1868) found that the dam owner was responsible for all consequences of the accidental release of the water. In the opinion of the House of Lords it was stated:

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[27] *U.S. Environmental Protection Agency Process Design Manual for Land Treatment of Municipal Wastewater.* USEPA Office of Technology Transfer. 1976.

[28] Wright, K.R. February 1983. "Safety of Small Dams and Owner Liability, Hydrologic and Hydraulic Considerations." Seminar sponsored by Professional Engineers of Colorado and Federal Emergency Management Agency, Denver, Colorado.

> *If a person brings, or accumulates, on his land anything that, if it should escape, may cause damage to his neighbor, he does so at his peril. If it does escape, and cause damage, he is responsible, however careful he may have been, and whatever precautions he may have taken to prevent the damage.*

As to large floods that are in excess of the spillway and outlet works capacity, and where embankment overtopping occurs with resulting dam failure, the "Act of God" defense is seldom available. The courts have held that a probable maximum flood can be computed and used for design purposes by prudent engineers. In effect, the design engineer who designs an outlet and spillway for a detention storage facility to pass the 100-year flood, without providing for larger floods, may be negligent in the event of dam failure caused by the larger flood, and the dam owner and/or operator may similarly be held liable.

Local governments cannot, however, escape liability by requiring private ownership and operation of detention facilities, because:

1) The local government has a fundamental responsibility for protection of the public safety.
2) The local government has review, approval, and permitting authority for construction.
3) Private runoff control facilities inescapably become links in the public drainage system chain and, therefore, a public responsibility.

7.1 Embankment Protection

The outlet works and emergency spillway should be sized to cause detention of and passage of the design inflow without causing the water level to rise above a pre-selected elevation. The storage level rise normally allows for additional vertical freeboard (typically about 3-feet) between the detained water level and the top of the embankment during peak design conditions.

The designer and owner are responsible for deciding whether or not the emergency spillway and outlet works for a detention pond will be sized to pass the probable maximum flood. If the spillway is not so sized, the reasons should be well documented. Whenever an emergency spillway is sized for less than a probable maximum flood, the embankment of the pond, if any, will be subjected to potential overtopping. Consequences of unplanned embankment overtopping range from simple excessive downstream face erosion to total failure of the embankment with release of the water in storage. If the failure is rapid, it may cause downstream flow greater than would result if the embankment had not been built. The designer must be aware that his actions can potentially increase the flood hazard rather than mitigate it. Owners may elect to take a "calculated risk," but the designing engineer may aggravate his liability by designing to a lesser standard.

In many instances, the size and economics of a detention pond virtually preclude a probable maximum flood spillway. In those cases, it is possible to mitigate the adverse effects of overtopping of the embankment by a number of means, such as, but not limited to:

1) Flattening of the downstream embankment face. SCS provides discussion regarding slope stabilities associated with different slopes.
2) Armoring the dam crest and downstream embankment face.
3) Using regulated floodplain delineation and occupancy restrictions downstream representative of conditions without the detention storage.
4) Providing extra channel capacity downstream.
5) Use of a wide embankment crest, such as is common with urban roads and streets where rapid failure seldom occurs due to modest overtopping depths.
6) Use of non-eroding dam material such as rolled soil cement or concrete.
7) Design and construction of off-stream detention facilities.
8) Use of a totally excavated pond.

Other factors that may mitigate overtopping effects include:

1) Small tributary basins where the rate and volume of discharge involved are limited, resulting in short duration overtopping flows of modest and non-hazardous proportions.
2) Floodplain uses downstream of the detention facility that are not susceptible to flooding damage.

It is appropriate to perform an incremental damage analysis for a detention facility where it is considered uneconomical or imprudent to construct an emergency spillway sized to pass the maximum probable flood. An incremental damage analysis needs to examine the extent of increased damages should the embankment fail from overtopping. If the additional flood flows would cause only a modest amount (or none) of additional (incremental) damages due to embankment failure, then it may be reasonable to allow the embankment to overtop and fail during large floods. In such a case, the cost/benefit analysis would usually show that a larger spillway was uneconomical. Nevertheless, if an embankment fails, regardless of cause or the fact that no incremental increase in damages has occurred, the owner or designer may be subject to claims for damage.

Outlet safety considerations include two aspects: safety of the structure and safety to the public at the facility. The outlet works create a potential hazard when in operation due to the possibility of a person being carried into the opening. Grating or trash racks are often used; however, a person can be forced against the grate or trash rack with substantial pressure which, in some cases, can be worse than being carried through the conduit. To mitigate this, low entrance velocities at the trash rack are recommended to reduce the potential pressure. Fencing or other effective measures should be provided to exclude people from potentially hazardous areas. Alternative measures include site grading, planting of thorny shrubs, or grading to assure "safety ledges" along the pond perimeter.[29]

Outlet works can be designed to reduce the hazard to the public where heavy recreational use is anticipated. For instance, a vertical riser of concrete, timber, or steel can have a series of openings of 12 inches or less from top to bottom with sufficient total area to cause low velocity at the entrances, if compatible with hydraulic requirements. The top of such risers can be grated, or even closed. In some instances the outlet works can be fenced. Fences are not universally recommended because of maintenance and operational needs, and because most fences do not fully prevent access. Appropriate signing is sometimes used to warn the public of the safety hazards involved at the outlet works.

During periods of no operation, there is little hazard at most outlet works, although outlet works can be attractive nuisances during operation. The designers need to be aware that an owner can be held liable, if an accident occurs, for having created an attractive nuisance. Design of outlets for which the orifices or weirs are not accessible from the embankment or shore when functioning is one method of reducing the hazard to playing children or curious adults, but this may complicate maintenance. Pipe openings on the upstream face of the embankment, where the pipe is only partially submerged, might be fenced on three sides to inhibit access from the embankment. Where the public (particularly children) has access to a potentially hazardous outlet works area, grating or trash racks having low entrance velocity should be used.

Finally, although not related to failure from overtopping, designers must recognize that a substantial percentage of embankment failures are due to inadequate outlet works design and construction, classified as "structural failures." Consequently a designer is reminded to direct special attention to the following:

1) Avoid potential piping of water along the outside of the outlet conduits by using cutoff collars, careful material selection and good compaction around the conduit.
2) Minimize the number of conduits through the embankment.
3) Ensure against leaky joints within the embankment.
4) Do not use thin walled conduit through the embankment without a protective exterior encasement.
5) Where reasonable, design the pipe to operate under little or no internal water pressure.
6) Provide a safety factor in outlet works openings to account for debris collection and design the pond to minimize debris migration to the outlet.

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[29] Jones, J.E., and D.E. Jones, Jr. October 21, 1983. "Floodway Delineation and Management." A presentation at the *ASCE Symposium on Floodplain Delineation and Management*, ASCE Annual Convention, Houston, Texas. Pp. 7 & 8.

7) Do not depend upon human intervention to operate gates or other controls during a storm runoff event. Gates can jam or become inoperable when needed during emergencies. People can be unavailable or diverted by other activities.
8) A spillway entrance is a natural location for debris buildup. Design the detention pond to minimize debris migration to the spillway. Provide emergency spillways with vehicle and crane access so that debris can be removed from a spillway when in operation.

8.0 MAINTENANCE

Maintenance of the outlet structure should be an integral part of a common sense, periodic program of maintenance for the entire facility. The responsibility for maintenance rests with the owner of the facility, whether it is a rooftop, parking lot, underground basin, or the usual surface storage facility. The local government should have the authority to inspect or review any private maintenance actions that would help to ensure that private maintenance is being provided.

Funding for maintenance, whether public or private, should be assured and may come from a long-term tax or service charge that is collected on a monthly or annual basis. In the case of private maintenance, two legal provisions should be made: 1) an allowance for public maintenance if the private organization fails in this responsibility, possibly with the cost added to property tax bills; and 2) recorded easements to allow public access for maintenance.

To reduce maintenance, outlet structures should be designed with no moving parts (i.e., pipes, box culverts, orifices, and weirs). Manually and/or electrically operated gates should be avoided. The only exceptions are excavated storage or surface storage affected by tides or other high water that must be pumped dry, and structures with water-actuated flap gates. To reduce maintenance, outlets should be designed with openings as large as possible, compatible with the depth-outflow curve desired and with water quality, safety, and aesthetic objectives. One way of doing this is to use a larger outlet pipe and construct an orifice or a V-notch weir in the headwall to reduce outflow rates.

Outlets should be designed somewhat massively to lessen the chances of damage from debris or vandalism. The use of thin steel plates as sharp crested weirs or orifices is best avoided because of potential accidents involving humans, especially children.

Outlets that are protected by a trash rack will accumulate trash during and between storm events. Such trash may consist of brush, tree limbs, leaves, grass clippings, paper, tumbleweeds, plastic, shopping carts, cans, and other urban debris. To facilitate outlet operation and maintenance, trash racks should be curved or inclined so that debris tends to ride up as the water level rises. Such a design leaves the rack clear and allows for easier cleaning during a storm event.

Outlet structures may be partially or completely plugged by a build-up of deposited sediment, by floating plant growths such as water hyacinths, and by vegetation growing in the sediment. Sediment deposition is a natural occurrence in basins and periodic removal of vegetation and sediment is necessary to ensure that the intended hydraulic function of the outlet is not impeded. Such periodic activities should be anticipated during design so that both maintenance access and a nearby waste disposal site assuredly will be provided.

The discharge end of the outlet structure also needs maintenance and good design can help reduce its needs. High velocity outflow can erode the downstream outlet, foreslope, and channel. A well-designed and constructed energy dissipater, surrounded by large, well-graded riprap on the foreslope and downstream channel can do much to reduce the maintenance needed at this location. Deep toe walls to resist scour (undercutting) should also be provided.

All portions of the outlet structure must be accessible to vehicles, equipment and personnel, both between and during storm events. This includes the floor of the basin as well as ramps to points above the upstream and downstream sides of the outlet structure.

Where pumping facilities are required, the pump house should be designed to provide security and resistance to vandalism, which could include fencing and vandal resistant doors and locks. Between storm events, maintenance should include lubrication and exercise of the pumps on a regular schedule to ensure they will operate when needed.

Basins designed to trap sediment and debris for water quality enhancement can be dry or can have a permanent pool. A pool tends to hide the deposited sediment and can be more aesthetically pleasing. To maintain aesthetic appeal, floating debris must be removed from the pool surface, and pool depths should be adequate to prevent weed growth. Access must be provided for vehicles and boats to remove floating debris. Accumulated sediment can be removed by dredging or by excavating equipment after dewatering. The pond can be emptied by pumping, but it is better to include a drainpipe to empty the pond by gravity in case of emergency or for bottom maintenance. The frequency of sediment removal will vary among facilities depending on the original volume set aside for sediment, the rate of accumulation, drainage area erosion control measures and the aesthetic appearance of the pond.

To encourage sedimentation, some normally dry basins use subsurface outlets of perforated pipe for the smaller runoff events. To reduce clogging, geofabrics have been employed around pipes that are placed beneath a layer of gravel and/or riprap. However, very little experience record exists to state if these fabric filters do indeed reduce outlet clogging and maintain the desired infiltration characteristics. Removal of deposited sediment from normally dry basins requires mechanical equipment to operate on the basin floor; appropriate practical measures are then needed to facilitate equipment mobility and prevent basin floor damage.

Larger diameter perforated pipes, placed vertically, wrapped with geofabric, and surrounded with a cone of gravel and/or riprap have also been employed to reduce clogging. The theory is that as the bottom holes become inoperative, holes higher up allow the water to drain from the basin. After a period of time, this cone will need to be removed, washed free of accumulated sediment, and replaced around the pipe.

Maintainability of outlet works of detention ponds should receive particular attention during design. Decision makers should recognize the life cycle costs of these facilities. Long-term maintenance costs are inevitable and can be minimized only by sensitive consideration and treatment during the design of a detention facility. The difference between a maintainable design and a design that is difficult and expensive to maintain will often also be the difference between an attractive operating facility and a neglected, degrading eyesore generating frequent public complaints.

9.0 AESTHETICS

As indicated by the Committee survey, designing an outlet structure that is visually appealing has been relegated to the bottom of the list of factors required to formulate a "successful" detention pond. The Committee believes, however, that two major factors argue for implementing attractive structures:

1) Public attention in an urban setting will naturally focus on the pond's outlet structure, both where the water enters the structure and where it is discharged downstream of the embankment. Because the outlet structure attracts attention, it should be pleasant to look at, or at least certainly not an eyesore.
2) Designing an outlet to minimize hazards is normally "ensured" through construction of trash racks or fences, which often become eyesores, trap debris, impede flows, hinder maintenance, and, ironically, fail to prevent access to the outlet. On the other hand, desirable conditions can be achieved through careful design and positioning of the structure, as well as through landscaping that will discourage access by even the most avid curiosity seeker (i.e., steep slopes, thorny bushes, positioning the outlet away from the embankment, etc.). Creative designs, integrated with innovative landscaping, are not only practical safety approaches, they also enhance the appearance of the outlet and pond and often are less expensive initially.

The Committee believes that the single greatest impediment to development of interesting, appealing outlet structures is lack of interest, sensitivity, and creativity on the part of the designer. This is in part attributed to reluctance on the part of designers to seek advice from planners or landscape architects on ways to transform uninspiring pipes and channels into interesting, attractive structures. Examples of attractive detention ponds are found in references.[30,31,32]

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[30] American Public Works Association. 1981. *Urban Stormwater Management.* Special Report No. 49.

[31] Melbourne and Metropolitan Board of Works. 1981. *Interim Drainage Basin Management Criteria Manual,* MMBW-D-0016.

9.1 Techniques for Enhancing the Appearance of Outlet Structures.

One of the most straightforward and effective mechanisms for enhancing interest in a structure is to utilize concrete that is "architecturally treated." Such concrete has its appearance altered through:

1) Color and surface texture variation.
2) Utilizing different shapes and forms of aggregate, and exposing the aggregate to varying extents.
3) Molding the concrete into patterns as it is poured (nearly 50 different surface patterns are commercially available).
4) Finally, as an alternative to incorporating a limited number of orifices of a large cross sectional area into a riser, numerous smaller holes can be fabricated into the structure, either randomly or in patterns. This approach, in addition to being potentially attractive, would improve the safety of the riser by limiting the cross sectional area of individual openings and would assure suitable hydraulic control if designed properly. (Although, as discussed previously, smaller openings may tend to clog more readily.)

Railroad ties or other treated wood can be tied together into "cribs" that surround inlets. Varied spacing between the ties serves to ensure appropriate hydraulic control. At low water levels, slot spacing can be small to limit discharges, while as the water levels rise, slot spacing can widen as needed. Such structures have been implemented successfully, and in addition to being attractive, they enhance safety by limiting access to open pipes.

Where risers are implemented, soil can be backfilled around a portion of the entire circumference of the riser, and either riprap or plants, or both, can provide protective cover for the backfill. Thus, in a wet pond an "island" may be created, while in a dry pond, an attractive landscaped mound may emerge from the pond bottom. In either case, an exposed corrugated metal or concrete pipe, or a yawning dark hole in a headwall, would represent less interesting alternatives. This concept can also enhance safety because firm material is provided around the pipe and thorny shrubs or riprap can be provided to surround the pipe to impede unauthorized access.

That portion of a riser visible to the public need not assume the form of simply a circular pipe. Instead, the portion of the riser normally visible can be shaped geometrically (into a pyramid, cone, sphere, etc.) or into some other interesting form. Through either prescribed fabrication or selective cutting of such objects, desired orifice control can be obtained. Although the concept of "sculpting" an outlet pipe is unusual, the idea is feasible if it does not significantly add to project costs, the public enjoys the approach, and the structure fulfills its function.

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[32] *Preliminary Design Manual for BMP Facilities*. Department of Environmental Management, Fairfax County, Virginia.

In dry ponds where a riser pipe will be visible, in addition to altering the conventional cylindrical shape of the structure, alternatives such as painting the structure or surrounding the base of it with low shrubs merit consideration. Furthermore, a riser pipe could be incorporated into the midst of a children's play area consisting of log, metal and concrete structures, provided that: 1) there is no possibility of a child entering the riser; 2) locating the riser amidst various other structures will not impede its hydraulic function; and 3) such a playground will not be inundated with such regularity that it will normally be muddy or covered with silt, debris, etc.

Potential for highlighting the aesthetic appeal of an outlet structure occurs when there is an opportunity to "cascade" water from one pond via an overflow channel into either another pond, stilling basin, or open channel. If such facilities require the use of rock-lined channels, plants frequently may thrive amidst or along the edge of the rocks, creating a pleasant appearance. Small, planted "islands" may be built in the cascade. Stilling basins can be made attractive through creative use of structural concrete or heavy riprap for energy dissipation. Spillway teeth need not necessarily assume a conventional rectangular block shape; instead, they can be fabricated into a variety of shapes and sizes and still function as energy dissipaters, often using large natural boulders and saving forming costs. There is an opportunity to utilize a spillway channel or stilling basin for multiple uses when such structures carry water only occasionally.

A design objective should be to provide outlet pipes that will not be noticed where they emerge from embankments. For example, a pipe that overshoots the toe of an embankment by 10-feet and terminates in a marshy area is not a good design. In one observed case, an outlet pipe from a dry pond terminates immediately adjacent to the edge of a residential street. Discharges flow across the street into a storm sewer inlet. A rusted grate that has trapped a variety of debris covers the end of the pipe. During even the very small runoff events, discharges from this pipe interfere with traffic. In the aftermath of every discharge, a wide swath of the street is littered with sediment and debris. For relatively little extra cost, this outlet pipe could have connected directly into the storm sewer across the street. Then it would not have been visible, and would not have demanded frequent maintenance expenditures. If the outlet cannot be hidden, the discharge area (stilling basin, ball field, or whatever) should appear attractive, thereby diverting attention from the pipe itself.

If an emergency spillway will carry water, on the average, less than once every 100-years, the potential for enhancing the appearance of the spillway by landscaping and/or recreational facilities should be assessed. As long as its flow capacity will not be impeded, aesthetic improvements to the emergency spillway will be appreciated by the public.

9.2 Interfacing Aesthetics with Economic, Safety, and Functional Considerations

Although most of the foregoing concepts for enhancing the physical attractiveness of outlet structures can be surprisingly inexpensive to implement, financial limitations—

especially in the form of long-term maintenance costs—may negate the viability of certain concepts. Consequently, cost analyses over perhaps a 20-year project period will be informative. Traditional cost analyses must, however, be tempered with recognition of the fact that many of the benefits of an attractive and recreationally functional pond and outlet structure cannot be quantified but have significant value.

In deciding on outlet features, safety should not be sacrificed for attractiveness. For example, placing an inlet in a dry detention pond in the midst of a children's playground can be risky and can increase potential liability. Generally, however, safety can be improved in conjunction with enhancement of physical appearance, as was suggested earlier.

A design technique that will contribute to the visual appeal of an outlet structure should not interfere with the hydraulics of that structure. For example, flowers or shrubs in the vicinity of the inlet could become dislodged and plug the structure. No hard and fast guidelines exist for integrating aesthetic approaches with hydraulics; common sense and imagination on the part of the designer is a necessity.

Surface ponds are by far the most common type of detention storage. Their outlet structures must:

1) Resist blockage both between and during runoff events,
2) Be accessible for maintenance both during and between events,
3) Perform their intended hydraulic function,
4) Discourage vandalism,
5) Be safe, and
6) Be aesthetically pleasing.

This is a tall order, but with good initial design and regular maintenance after construction, outlet structures can accomplish most, if not all, of these objectives.

10.0 RESEARCH NEEDS

Many stormwater management regulations were developed because of water quality considerations, such as those pertaining to streambank erosion and other sources of non-point source pollution. However, these same regulations used peak discharge control as the performance criterion because very little was known about the relationship between stormwater detention design and water quality control. Regulations often incorrectly assume that there is a direct correlation between quantity control and quality control. While this may be true, the need still exists to identify the relationship between detention designs and water quality. There is reason to believe that the use of an elongated detention basin or the presence of baffles may improve the water quality state of a basin. In addition, detention time is believed to be a primary determinant of water quality, and the outlet facility is certainly the primary factor controlling this. There is a need for research on the effect of detention basin design, including the design characteristics of the outlet, on water quality characteristics.

While in many cases stormwater management regulations were proposed because of detrimental changes to the quantity and quality of runoff from areas undergoing land use changes, all costs and benefits of stormwater control have not been fully evaluated. This issue becomes increasingly important as more localities adopt regulations requiring greater control. While new approaches to stormwater management, such as infiltration-based control, are being written into regulations, the detention basin remains the primary control method. Currently, the greatest change in detention basin design is with the outlet facility, with the trend towards multi-stage outlets. Given the wide array of outlet configurations, the need for research on the costs and benefits of the outlet design has never been greater.

For many in public works agencies, assuring proper long-term maintenance is the primary problem. After all, an improperly maintained facility will not function as the designer intended it to. In addition, there are many who believe that the cost of maintenance is a major public commitment. In addition to the maintenance that is associated with the volume of storage, the work necessary to keep both the outlet facility free of debris and the emergency spillway capacity adequate is a major effort. In fact, the frequency of maintenance is almost entirely due to the demands for maintenance associated with the outlet facility. There is a need for research on the effect of a lack of maintenance on the hydraulic efficiency of both the primary outlet facility and the emergency spillway. There is also a need for research on the most effective way to incorporate the effect of "failure to maintain" into design requirements. The economic significance of planned "failure to maintain," can materially benefit local maintenance budgeting.

Detention basin outlet design methods were developed by transferring technology from other areas of design, such as the design of small dams and culvert design. Thus, there is some concern whether or not the hydraulic characteristics of single and multi-stage outlet facilities presently being constructed duplicate the characteristics that are predicted with the currently used design methods. Of special importance is the accuracy of the depth-discharge rating curve. The current practice utilizes standard weir and orifice equations and applies them to very complex and composite control sections. Other factors that may lead to design inaccuracies include the applicability of the weir and roughness coefficients that are applied to the outlets. Weir coefficients typically were developed using carefully controlled approach conditions, and the coefficients reflect flow behavior through a significant flow distance upstream from and immediately downstream from the weir itself. There have been no reported studies of "stacked" weirs whose net hydraulic behavior may differ appreciably because of interference from performance anticipated using conventional entrance loss and discharge computations. All of these hydraulic considerations deserve further research.

Cavitation, piping, and vortexing are important to the hydraulic efficiency and the integrity of the detention system. Hydraulic model studies are needed to assess whether or not cavitation problems normally associated with major structures are a potential problem with typically smaller on-site detention basin. The degree to which vortex control is needed should also be determined from hydraulic model studies. While piping along conduits has been studied, the development of uniform design guidelines still needs to be addressed.

The integrity of a detention structure is important for reasons of both safety and costs. There is a need for research on the design and construction practices that affect the integrity of the structure, especially practical economical practices that can prevent the rapid failure of an embankment. In addition, there is a need to develop methods to guide engineers in assessing the effects on downstream properties of failure of either the outlet facility or the embankment.

The Committee endorses the study of these and other research needs for detention ponds.

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Index

Page numbers represent first page of paper