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Problem Oriented Evaluation Of Institutional Decision Making And Improvement Of Models Used In Regional Urban Runoff Management: Application To Indiana

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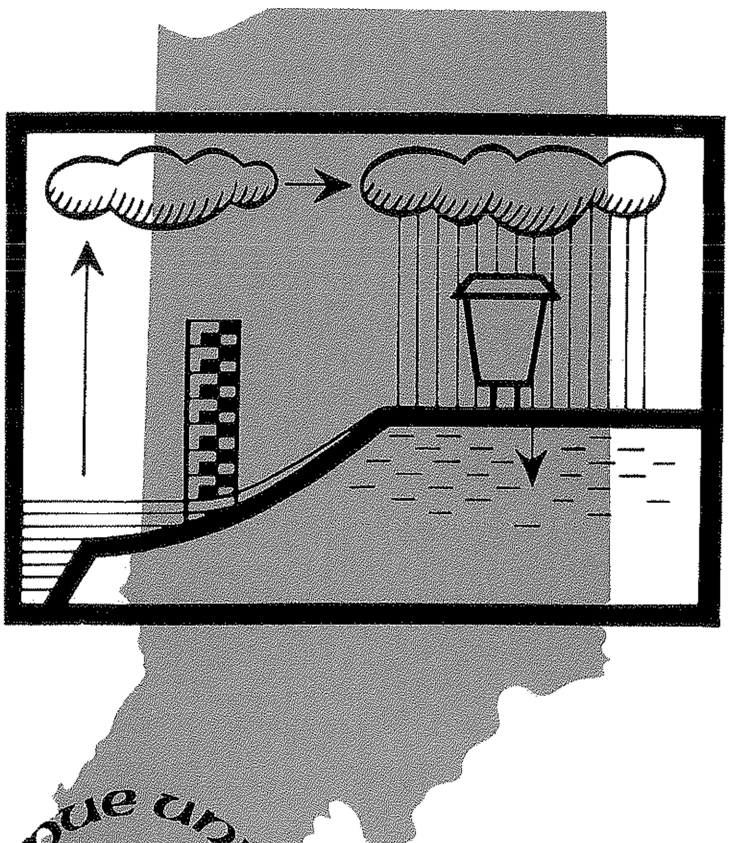

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"Problem Oriented Evaluation of Institutional Decision
Making and Improvement of Models Used in Regional
Urban Runoff Management: Application to Indiana"

DISCLAIMER

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The work upon which this report is based was supported in whole by funds provided by the United States Department of the Interior, Office of Water Research and Technology, as authorized under the Water Research and Development Act of 1978. The Office of Water Research and Technology has since been terminated effective August 25, 1982, by Secretarial Order 3084 and its programs transferred to other bureaus and offices in the Department of the Interior. The Urbanizing Areas Research and Development Program was among those transferred to the Bureau of Reclamation.

ABSTRACT

The principal objective of the research is to evaluate, in a multi-disciplinary framework, problems of regional runoff management, with special attention to Indiana. One focus of the research is on institutional issues affecting runoff management; specifically, interorganization relations and decision making among Section 208 water quality management planning agencies are investigated. It was found that although 208 planning was never a smooth process in the two study regions, and many participants were frustrated by their experiences and the lack of clear results from the planning process, it may in fact have laid the foundation for future water quality management programs within the regions. It was clearly a learning process for all participants, and as such, responsibility for any shortcomings in 208 planning cannot be laid solely on any one organization.

The other focus of the research is on models that may be used to improve runoff management. To support this work, a small watershed in West Lafayette, Indiana, was monitored with an automated data collection, data processing, and data storage system. An investigation of the effects of hydrologic conditions on the quality of urban runoff from this watershed concluded that: the watershed generally exhibits a first flush in concentration for all pollutants monitored; and there is no apparent relationship between the quantity of previous rainfall or the length of the antecedent dry period or the time elapsed since street sweeping and the quality of stormwater runoff.

An investigation of how to select the best critical duration for a design storm for hydrologic studies concluded with a procedure to make the selection using ILLUDAS. The first step is to determine the Huff quartile storm that yields the maximum value for the runoff from the watershed. The next step is to use this quartile storm to calculate the critical value of the paved entry area of the basin. The best estimate of the paved area entry time should be obtained by simulation because this parameter has the most significant influence on the value of the critical duration. Finally, by using the pipe distribution network which has been suggested for the basin, the effect of the percentage imperviousness on the critical duration should be investigated. The results from this analysis suggest the critical duration which must be used for that watershed.

The selection of parameter estimates for rainfull-runoff models is quite important. Two techniques to obtain "optimal" estimates are compared. The results from this analysis confirm the statement by Sorooshian (1981) that the objective function that makes use of the maximum likelihood estimator gives improved parameter estimates. This procedure is therefore recommended for obtaining parameter estimates for the urban rainfall-runoff model ILLUDAS. It should also be tried with other deterministic models of the rainfall-runoff process.

Another investigation has demonstrated the need for a general planning methodology for use in the design of dual purpose detention basins. Such a methodology is developed, tested, and appears to work very well. The general interaction of both storage and pipe costs and flowrate and pollutant constraints in relation to the overall system design is illustrated. The effect of various design frequency constraints on the overall system design is

investigated. Although some initial results are obtained in relation to the above considerations, any general conclusions should be delayed until more case studies have been investigated. It is quite possible, given the complexity of the system, that any conclusions may be site specific.

The new detention basin planning methodology should prove to be a valuable tool in the analysis and design of dual purpose detention systems. The new methodology can be used to obtain an individual system design or used in a sensitivity analysis of a given system. Such an analysis can be used to construct cost graphs as a function of different flowrate reduction and pollutant removal levels. By deriving such graphs, information can be obtained concerning the region of control of each constraint. This information could then be used in the selection of a design that provides the best trade-off between pollutant and flowrate objectives for a selected level of flowrate reduction or pollutant removal.

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CHAPTER 1
INTRODUCTION

1.1 Research Team and Publications

The focused research project titled "Problem Oriented Evaluation of Institutional Decision Making and Improvement of Models Used in Regional Urban Runoff Management" (Grant Number 14-34-0001-0498) began in September, 1980. The research team comprised the Project Director, Dr. J. W. Delleur, and four other Principal Investigators: Drs. J. M. Bell, Mark H. Houck, H. R. Potter, and A. R. Rao. In addition, M. Blumberg, H. R. Lemmer, L. E. Ormsbee, and H. Schweer were Research Assistants for the project.

Numerous technical reports, theses and papers have resulted from this work. Some of these are still in the prepublication stage and will not appear in the scientific literature for another year. Many, however, have been published or accepted for publication. A listing of these follows:

1. Bell, J.M., and Blumberg, M.S., "The Effect of Various Hydrologic Parameters on the Quality of Stormwater Runoff from a West Lafayette, Indiana, Urban Watershed", Technical Report No. 162, Purdue University Water Resources Research Center, West Lafayette, Indiana, January, 1984.

2. Blumberg, M.S., "The Effect of Various Hydrologic Parameters on the Quality of Storm Water Runoff From a West Lafayette, Indiana, Urban Watershed", M.S. Thesis, Purdue University, West Lafayette, IN, December, 1983.

3. Lemmer, H.R., "Critical Duration Analysis and Parameter Estimation in ILLUDAS", M.S. Thesis, Purdue University, West Lafayette, IN, 1981.

4. Lemmer, H.R., and Rao, A.R., "Comparison of Two Methods of Estimating the Parameters of a Deterministic Urban Rainfall-Runoff Model", Proceedings, Indiana Water Resources Association Symposium on Water -- Indiana's Abundant Resource, South Bend, IN, pp. 146-157, June, 1982.

5. Lemmer, H.R., and Rao, A.R., "Critical Duration Analysis of Design Storms", Proceedings, 9th International Symposium on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, Lexington, KY, pp. 11-17, July, 1982.

6. Lemmer, H.R., and Rao, A.R., "Critical Duration Analysis and Parameter Estimation in Illudas", Technical Report 153, Water Resources Research Center, Purdue University, West Lafayette, IN, June, 1983.

7. Ormsbee, L.E., "Systematic Planning of Dual Purpose Detention Basins in Urban Watersheds", Ph.D. Dissertation, Purdue University, West Lafayette, Indiana, December, 1983.

8. Ormsbee, L.E., Delleur, J.W., and Houck, M.H., "Development of a General Planning Methodology for Storm Water Management in Urban Watersheds", Technical Report No. 163, Purdue University, Water Resources Research Center, West Lafayette, Indiana, March, 1984.

9. Ormsbee, L.E., and Delleur, J.W., "Data Acquisition and Management Techniques in Urban Hydrology", Proceedings, Indiana Water Resources Association Symposium on Water--Indiana's Abundant Resource, South Bend, IN, pp. 14-23, June, 1982.

1.2 Summary of the Report

The remaining chapters of this report are summaries of the five major areas of investigation undertaken in this project. Chapters 2 through 5 are condensed versions of other technical reports for the project. Chapter 6 contains a description of the data collection system -- field hydrology station, telemetry, data processing, analysis and storage -- maintained throughout the project.

Chapter 2. Institutional processes are an important factor in the implementation of many natural resources policies which involve both federal and state governments and many units of local governments. This is particularly true for programs like 208 water quality planning. This study examines data from 38 respondents in 32 state, regional and local organizations in Indiana. They include the responsible state agency, state offices of federal and voluntary organizations, and numerous organizations in a designated and in an undesignated area.

Within the designated area there was much more emphasis on public participation and support for regional planning. The 208 agency reported adequate budget and staffing. The undesignated area had little public participation; their planning was done by the State 208 agency. The State 208 agency was not particularly supportive of either public participation or 208 regional planning; it also viewed its staffing and budget as less than adequate. Overall, there were few interorganizational problems reported; however, there were some goal/priority incompatibilities reported that affected relations with the State 208 agency. There was almost no local government involvement, which would likely be detrimental to any implementation.

Three inter-related factors appear important for future federally mandated policies. Adequate time must be provided. It should not be assumed that agencies have staff expertise in place for new or rapidly expanding programs. Sources of funds need not only to be adequate in the short run, but provide for organizational stability over time. There must be a deliberate effort to build local support, for example through public participation. Local support is needed to get local or state funding for staffing and program implementation.

Chapter 3. This study investigated the quality of stormwater runoff from a 29-acre, fully developed, residential watershed. The parameters monitored were BOD, suspended solids, total coliforms, and fecal coliforms. In most of the 36 discrete runoff events monitored, a first flush in concentration and mass (or number) was evident for each of the pollutants.

The importance of various hydrologic activities (such as average intensity of rainfall, peak intensity of rainfall, total rainfall, peak rate of runoff, total runoff, and duration of precipitation event) on stormwater pollutant concentrations, total mass (or number), and peak rates was evaluated by multiple regression analyses. The hydrologic activities accounted for 33-87% of the variance in the pollutant data.

No relationship was apparent between the quantity of previous rainfall or the length of the antecedent dry period and the quality of stormwater runoff. Neither was an overall trend apparent between the time elapsed since street cleaning and the quality of the runoff.

Seasonal variations in pollutant levels appear to exist in the data collected during this study. In general, concentration, mass (or number), and peak rates for all pollutants appear to be the greatest in the Fall and least in the Winter.

Chapter 4. Urban stormwater drainage networks are usually designed by assuming a design storm. The durations for these storms are arbitrarily selected or they are specified by drainage design codes. The first objective of the research reported in the hydrology section of this report is to develop guidelines which can be used by designers to select optimal storm durations which may be used in urban drainage design.

The urban rainfall-runoff model ILLUDAS is used to obtain estimates of the maximum runoff rate for a given set of parameters. The maximum runoff rates are studied to estimate the duration that yields the largest runoff rate, and this duration is called the critical duration. Each of the parameters in the rainfall-runoff model is varied systematically and its effect on the critical duration is studied. The rainfall-runoff variables that are studied are the total basin area, basin imperviousness, grassed and paved entry times, pipe slopes and pipe roughness, rainfall frequency and storm distribution. Data from First Street Watershed in Louisville, Kentucky, Mount Washington Watershed in Cincinnati, Ohio and Bar Barry Heights in West Lafayette, Indiana, are used in the analysis. The storm distribution, paved entry time and the ratio of paved to grassed area in the subbasins are the parameters which have the greatest influence on the critical duration. The procedure that is recommended to determine the critical duration is first to study the temporal storm distribution to obtain the Huff quartile storm giving the maximum runoff rate. Secondly, the most accurate value of the paved entry time which can be used for that particular watershed should be estimated. This will be the critical duration of the design storm. Finally, the pipe drainage network should be determined for the final selection of the critical duration, associated with the percentage imperviousness.

The two stage parameter estimation procedure proposed by Sorooshian (1981) to estimate the parameters of deterministic rainfall-runoff models has been tested by using ILLUDAS. The parameters estimated by Sorooshian's method are compared with those obta ned by using the least squares criterion. The method is applied to observed rainfall and runoff data from the Mount Washington watershed in Cincinnati, Ohio. The parameters from ILLUDAS that are estimated in the study are the grassed and paved abstraction values.

The parameter estimates obtained from Sorooshian's procedure yield better estimates of the maximum runoff rate and of the runoff volume than the parameter estimates obtained from the simple least squares parameter estimation procedure. On the basis of results obtained from this study, it is recommended that Sorooshian's method be used to estimate the parameters in urban rainfall-runoff models.

Chapter 5. As urbanization in a watershed increases, there is a corresponding increase in both runoff volume and rate. As a result, most rapidly developing urban areas are now finding themselves faced with the almost inevitable problem of storm sewer overloads. In addition to this problem, many municipalities are now facing problems related to the quality of stormwater runoff. In response to these and other problems, many municipalities are employing detention basins as the primary stormwater management control. While various policies have been proposed for improving the quality of urban runoff, probably the most effective stormwater management technique is the use of the detention basin. As a result, stormwater management basins are now being used to control water quality in addition to the quantity of runoff.

A general methodology has been developed for use in the planning of dual purpose detention basins. The methodology employs continuous simulation, statistical analysis, and a general design heuristic to obtain an integrated system of detention basins. Both water quantity and water quality constraints may be considered. The developed methodology can be used for the analysis of a particular detention system or in deriving general design guidelines. A description of the methodology along with two sample applications is provided in Chapter 5.

Chapter 6. An important portion of the study was the collection of hydrologic data for the Ross-Ade Watershed in West Lafayette, Indiana. The watershed comprises approximately 29 acres -- mostly single family residences with about 11 acres of impervious area.

The field station is currently set up to monitor rainfall, runoff, air temperature, and the water temperature of the runoff. The station is also equipped with an ISCO sampler that can draw water samples from the monitored runoff. Data collected from the various sensors are processed by a small microcomputer and then transmitted via a telephone line to a central station in the Hydromechanics Laboratory of the Purdue University Civil Engineering Building. Data sent from the field station are processed at the central station by a Heathkit H-8 microcomputer which receives the data and stores them on floppy discs.

As of April, 1984, 305 individual events have been analyzed. Associated with each event are minute data for rainfall, flowrate, air temperature, and water temperature. In addition, quality data have been recorded for 36 of these events. These data represent a very extensive data base for use in the analysis of the physical mechanisms involved in urban hydrology. Summary tables and graphs have been developed for the entire data base for use in screening and identifying critical events for subsequent analysis.

CHAPTER 2

INTERORGANIZATIONAL RELATIONS AND DECISION MAKING AMONG SECTION 208 WATER QUALITY MANAGEMENT PLANNING AGENCIES

2.1 Introduction

It is almost a truism that natural resources boundaries, such as watershed boundaries, do not correspond with political boundaries. Consequently efforts to control pollution often involve several units of government as well as various industries and voluntary organizations. Institutional processes, or relations among organizations, therefore often become an important factor along with technological and economic factors in solutions to environmental problems.

The purpose of this study is to examine how institutional processes may operate as constraints or restrictions on alternative solutions in water resources decision making. Institutional processes in water resources decision making arise on the one hand out of specialized bureaucratic structures designed to bring technical information and criteria to bear on issues, and on the other hand out of such basic governing principles as fairness and due process. While these two aspects of institutional processes may not always inherently conflict at a more abstract level, at the level of policy implementation often they at least place restrictions or constraints on what are acceptable solutions to problems. This results in part out of the need for interaction among the specialized agencies and the units of general purpose government. This is particularly likely in a program like 208 areawide water quality management (named after section 208 of the P.L. 92-500) where a federally mandated policy is implemented through state and local governments.

In this study institutional processes are examined through the interorganizational relations of 208 agencies with other local organizations and with state and federal agencies. These various organizations each have goals, domains, budgets and constituencies which are a result of their institutional structure, and which affect the extent and nature of their interaction with the 208 agency. The objective here is to examine how institutional processes may place constraints on decision making by looking at organization resources, with an emphasis on regional planning and public participation in relation to the extent and importance of interorganizational relations.

While 208 plans and planning have been in limbo since 1980, the relevance of this study lies in the fact that the problems of non-point pollution continue to exist, and represent a significant part of water pollution. Further legislation is being considered to address the problem. It seems clear that many environmental issues, from acid rain as an international problem to pollution of a modest size watershed covering a few towns and counties will continue to involve interorganizational arrangements as a means to working toward a solution.

2.2 Background

In response to increasing environmental concern over the previous decade, Congress passed the "Clean Water Act," Public Law 92-500, in 1972. This Act established that among other goals:

> "(1) it is the national goal that the discharge of pollutants into the navigable waters be eliminated by 1985;...
> (5) it is the national policy that areawide waste treatment management planning processes be developed and implemented to assure adequate control of sources of pollutants in each State..." (U.S. Congress, 1972).

It is Section 208 of this law that provided for areawide waste treatment management. Its scope was intended to be broad enough to cover both point and non-point pollution. Point pollution can generally be dealt with by a single governmental agency since it has a specific source. Non-point pollution however, is likely to span several governmental jurisdictions, thus no one agency typically could effectively deal with it. The broad scope and mechanism of 208 planning is described concisely by Barton as:

> "...a rational, comprehensive, integrated planning process enabling local areas to develop methods to control water pollution. Under the 208 process, areas were supposed to estimate growth and identify needs for municipal sewage treatment for a 20 year period; to inventory point pollution; to identify nonpoint pollution sources and develop regulations and land use measures (BMPs) to control them; to estimate the economic, social, and environmental impact of the plan; and to designate appropriate agencies to implement it. The plan was then to be submitted for approval to EPA, with implementation following plan approval" (1978: 17).

Several studies have pointed to various aspects of the problem, of fragmentation of local authority as a source of difficulty in controlling pollution. See for example, Page and Weinstein (1982), Centaur Management Consultants, Inc. (1978) Dersch and Hood, (1975) and Kaynor and Howards, (1973).

However, there was optimism that 208 planning efforts might help cope with the existing fragmentation:

> "...the structure of American Federalism simply does not permit such (large regional) arrangements to successfully cope with the problems of water quality...as a result, we feel the areawide arrangements such as those emerging under Section 208 should be the focus of research efforts. They not only have the potential to coordinate municipal, county and state initiatives, but are also more likely to allow in their structure for representation on the part of the variety of public and private interests which are concerned in this policy sphere. This is extremely important, since institutional arrangements must be such as to facilitate achieving supply/quality/land use interfaces at the local level." (Whipple, 1975).

There are several features of the law that are important for relations among organizations. (1) In administration of the law, a distinction is made between designated and undesignated planning areas. Designated planning areas are usually urban-industrial areas with severe water pollution problems, often from both point and non-point sources. Since many metropolitan areas already had councils-of-governments (COGs) or metropolitan planning agencies in existence, much of the administrative machinery that could be used for this new program was already in place. Responsibility for coordination and planning for the remaining undesignated areas, primarily rural areas with less severe or at least less identifiable pollution problems, fell to the state governments. (2) The Federal government was to supply 100 percent of the funding for initial areawide water quality planning though local or state agencies were expected to supply the funding for continued planning. (3) In addition, the EPA had large amounts of money available for sewage treatment construction grants and other water pollution control measures. (4) Another feature with implications for Indiana was that if the states would not accept the responsibility for undesignated area planning, the EPA would assume that responsibilty for the state and in effect bypass state government in the planning process. (5) Finally, the planning process was to take approximately two years--what was to prove to be an overly optimistic time frame for a complex process involving social, political, economic, and technical factors.

2.3 Theoretical Framework

A recent theory of interorganizational relations (IOR), the political economy model (Benson, 1975), forms the general conceptual framework used in this study. The central premise of this theory is that the context of IOR can be conceptualized as a political economy with the central problem being resource acquisition. The relevant organizations must compete for various resources, which can generally be categorized as money and authority. The emphasis is on the network of organizations and the resource flows that occur into and within the network. Thus, it is a genuinely IOR perspective rather than a modified micro-theory. While the individual organizations are still the actors, they are not the focus of the analysis. In addition, the model explicitly deals with the issue of power--both within the network and between the network and its socio-political environment. This model may be said to largely incorporate key elements of resource dependency theory in a macro-social context.

The approach to the analysis of interorganizational decision-making taken in this study draws on the works of Benson (1975), Van de Ven, et al. (1979), Esman and Blaise (1966), Raelin (1980), and Warren, et al. (1974). It focuses on what Warren, et l. (1974) in their study of community decision organizations termed "critical events" which were decision events that mobilized the decision-making network. In the present study, the formation of the water quality management plan for a planning area will be the "critical event" or issue that is the focal point of the interorganizational relations and this analysis.

The conceptual approach examines selected attributes of the single organization, including its age, size and resources. These organization attributes are important in influencing actions an organization takes. They describe past experience of the organization which involves both developing internal competence and external relations with other organizations, the resources the organization has to develop and operate programs, and the extent to which they are either able to make independent decisions based on technical and/or economic criteria or are vulnerable to outside influence. These external or institutional relations are also important to the decision-making process because, to the extent that functional linkages are not uniformly distributed throughout the set of water quality management agencies (and it is unlikely that they are uniformly distributed), these linkages form the basis for clusters of interconnected organizations or coalitions which can more effectively work to achieve similar goals in the decision-making process.

2.4 Implementing 208, Areawide Water Quality Management Planning

As one might expect, implementation of the water quality planning stage was not without its problems (c.f., Barton, 1978; Centaur Management Consultants, 1978; McPherson, 1979). An initial problem was that EPA deadlines required states to begin the planning process even before the EPA had formulated the guidelines and requirements for the plans. States (perhaps understandably) delayed taking action, and EPA had to extend the deadline for the filing of water quality management (WQM) plans until 1978.

EPA apparently made the implicit assumption that the planning agencies were mature organizations with appropriate technical staff, or if new organizations, that they would obtain highly qualified staff who were familiar with techniques required for the plan formation. In fact, existing organizations rarely had the technical staff necessary for data gathering, and new organizations required a good deal of startup time.

There were several problems involved with the local focus of the planning process (Centaur Management Consultants, 1978). The procedures for local approval were unclear. It was not unusual for the local planning process to be dominated by one or two powerful governmental units or agencies. This was especially problematic when, for example, planning was dominated by a county unit, but primary responsibility for implementation fell on a municipal unit. Second, although planning was to take place at the local level, the state was ultimately responsible for approving and submitting the final plan to EPA. In some cases, the state took the initiative for planning, but failed to seek local input into the plan. Finally, although the initial planning process was funded through EPA by the Federal government, EPA was not to fund the continuing planning and monitoring process that was supposed to occur during the implementation of the plan. This would have placed a heavy financial burden on local and state governments.

A problem that would have occurred at the implementation stage is that neither EPA nor the local WQM agencies had the authority to implement and enforce the plan. Rather, this was to be the responsibility of the state government. Given that some states (notably, Indiana) were less than enthusiastic about the entire water quality management planning process, the aggressive implementation and enforcement of the plans were less than certain. However, EPA plans called for full implementation of the plans within 5 years to meet the goal of "clean water by 1983".

One feature that seemed to be characteristic of the planning process (especially at the local level) was the desire to avoid controversy. The solution of choice was usually to plan to build improved sewage treatment facilities rather than grappling with the larger pollution problem which might require a more complicated solution (land use plans, etc.). In addition, although public involvement was mandated, environmental and consumer interest groups were usually under-represented. Public involvement often took the form of public education rather than public input.

Many problems in the planning stage can be traced to how the policy was implemented rather than to the policy itself. EPA apparently gave little consideration to the political and domain issues relating to state and local decision making though they did encourage broad local participation in planning. In fact, throughout the planning process, most of the emphasis was placed on technical solutions to pollution problems. In much the same way, local WQM agencies often sought state and local support for a plan as it neared completion rather than attempting to involve important individuals and organizations in the process at its early stages. There was little recognition of interorganizational dynamics and issues of domain, power, and autonomy that were important for organizations whether included in or excluded from the planning process.

A key feature of P. L. 92-500 for this research is areawide or regional planning and control of water pollution. This areawide focus includes mandated interorganizational relations among the various agencies and governmental units that, in some way, have some level of responsibility for water quality monitoring and management. Though perhaps to the architects of the law, mandated interorganizational relations (IOR) was only a means to achieve a comprehensive approach to water quality management, it occupies a very central role in the planning and implementation of the Section 208 process. While the primary focus of the legislation was creation and implementation of a water quality management plan, for this research, IOR or institutional relations is seen as the central mechanism for both the creation and implementation of the plan. Without effective IOR, it is unlikely that implementation of the plan could even have approached a level required for effectiveness.

2.5 Methods

The program objectives of Section 208 planning of P. L. 92-500 were to occur in both large urban areas and rural areas. Therefore, the criteria used to select study locations reflected this diversity. Two regions were selected. One was an 8-county area with a total population of 1,500,000 in 1980, a large, central urban population which extended out into the surrounding counties. This was a designated area for 208, meaning it had been classified as having relatively serious water quality problems. Indiana Heartland Coordinating Commission (IHCC) was the 208 agency. The second region also had 8 counties, but was considerably more rural, with extensive agricultural production. The total population was 250,000 in 1980 with twin cities of about 66,000 population but no other cities larger than 25,000.

This was an undesignated area for which the Indiana State Board of Health had planning responsibility, although through various subcontracting arrangements the Region 4 Planning and Development Commission (PDR4) had responsibility for land use surveys, inventories, and projections.

Not only did the areas differ socially and economically, but the 208 agencies were also different. IHCC was an active regional planning agency which acquired 208, and had a direct funding link with the U.S. Environmental Protection Agency, an important difference in the terms of institutional linkages (or factors). PDR4 was a relatively benign regional planning agency with economic development as a primary goal. When ISBH acquired overall responsibility for 208 planning for all undesignated areas in the state, it contracted with the State Planning Services Agency which in turn subcontracted some responsibilities to the Planning and Development Regions including PDR4.

The focus of the study is on the role that organizations played in 208 planning and they are the units of analysis. Our objective was to identify organizations that were involved in the 208 process and to obtain data on that involvement. Data were obtained from written records and from appropriate persons within those organizations. Interviews averaging 1.75 hours were conducted, usually with the head of such organizations, or head of a unit within that organization. A snowball sampling technique with multiple entry points was used for this (Babbie, 1975; Coleman, 1959). This resulted in 39 interviews with people in 32 organizations. Twenty-two people were in single-level organizations, such as a local governmental unit, and 17 were in multi-level organizations such as a local or state unit of a national organization. Four were with the Indiana State Board of Health with responsibilities over different units and levels. There were three organizations within which more than one person was interviewed, including both of the regional 208 agencies.

A problem with snowball sampling is determining when you have exhausted meaningful interviews. The criterion used for this decision is known as saturation sampling (Glaser and Strauss, 1967). Saturation is said to be achieved when additional interviews do not add new data. We also probed the boundaries of the set of organizations from which data were collected to determine if additional organizations should be included. This typically occurred when a respondent suggested we make a new contact. There were, then, additional preliminary contacts with potential respondents where it became apparent that the organization had no appreciable involvement, and so, were not interviewed. Another problem was that several respondents were involved with more than one organization, such as a voluntary organization or local governmental unit and a citizen's advisory board. Respondents were instructed to respond primarily in terms of one organization, with one exception, although information they had from their multiple roles was not ignored.

2.6 Findings

Organizational Characteristics and Resources. There are a number of organizational attributes that are important to describing and understanding the interactions in the 208 planning process. These attributes affect the resources available to the organization, and its relationships with other organizations.

There was great diversity in the types of organizations we found involved in the 208 process. The thirty-three organizational units represent a wide variety of organizations and groups including local, regional, state, and federal agencies or governmental units, voluntary associations, and business organizations. Slightly over half (21) of the interviews were conducted with officials or employees of governmental units or agencies while the remainder (18) were with members and employees of voluntary associations and business organizations. A total of sixteen interviews were obtained in Planning and Development Region 4 (PDR4), the non-designated area, twelve from Planning and Development Region 8 (or Heartland area), the designated area, and eleven from state level organizations, including state offices of federal organizations.

Multi-level organizations have in place a means of communicating information from national to state to local units. This has the potential to facilitate building grass-roots support or opposition for a program. This may give such organizations an advantage over single level organizations when dealing with federally mandated programs. About half (17) of the organizations had more than one administrative level. Typically, these were governmental agencies and voluntary associations that had a local unit (e.g., county office) that was administratively responsible to a state office. The majority of the state organizations in this study were multi-level organizations, while the majority of the organizations from the two regions (PDR4 and Heartland) were single-level.

Although the age of the organization varied from over 150 years to 8 years, nearly one-third of them had been formed since 1968. Thus, many of the organizations were relatively young--less than ten years old at the time of 208 planning. A slightly larger proportion of the youngest organizations were located in PDR4.

The size of the organization in terms of number of paid employees varied considerably, ranging from zero (voluntary organizations) to several organizations with over 200 employees (governmental agencies). Again, these were fairly evenly distributed, though smaller organizations (zero to seven employees) were predominant in PDR4.

As was expected, age of the organization was a moderately good predictor variable for a number of resource variables. Age was highly correlated with an organization's number of paid positions across the entire set of organizations ($r=.43$, $p<.01$).[1] However, it varied considerably by organizational locus. It w s strongest in PDR4 ($r=.66$, $p<.01$) but the

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[1]Although the sample of organizations in this study is not a probability sample, significance levels are reported (where appropriate) to provide an indication of the statistical strength of the relationship if it were a probability sample. This may help emphasize the need for caution in interpreting the data since they represent only a modest number of organizations.

correlation largely diminished in Heartland ($r=.03$, n.s.) and at the state level ($r=.04$, n.s.).

The size of organizations' total budgets during 208 also had a great deal of variation with a range of $250 to $8,500,000, although the median was about $70,000. Voluntary organizations tended to have small budgets while larger budgets tended to belong to government agencies. Organization age was highly correlated with budget also ($r=.40$, $p<.01$). As with organization size, this correlation was strongest in PDR4 ($r=.65$, $p<.01$), while dropping considerably in Heartland and the state level ($r=.11$, n.s. and $r=.01$, n.s. respectively).

However, there was little correlation ($r=.07$, n.s.) between age of organization and the amount of additional money received specifically for 208 planning activities. However, it should be noted that only eleven of the organizations received additional 208 money. A t-test on the mean age of the organizations receiving additional money versus those that did not showed no significant difference between the two groups.

One of the major problems encountered during 208 planning was that of having sufficient money to carry out planning activities. The respondents were asked how adequate their overall budget was during the 208 period. Although the responses ranged from "inadequate" (scored 1) to "very adequate" (scored 5) the mean response was between "fairly adequate" (3) and "adequate" (4) ($\bar{X}=3.3$ on a five point scale). Respondents in the Heartland region reported the greatest budget adequacy ($\bar{X}=3.7$), followed by PDR4 respondents ($\bar{X}=3.3$) and state level respondents ($\bar{X}=2.7$). The two agencies with responsibility for 208 planning, the Indiana State Board of Health (ISBH) and Indiana Heartland Coordinating Commission (IHCC) reported mean budget adequacy of 3.6 and 4.0 respectively.

There are however, some important differences in the adequacy of money specifically allocated to 208 planning. In this area, PDR4 respondents and Heartland respondents reported the highest budget adequacy ($\bar{X}=3.7$ on the same scale as above), followed by state level respondents ($\bar{X}=2.8$). The most interesting comparison is between the two organizations with coordination responsibilities for 208 planning. The ISBH respondents reported a much lower 208 budget adequacy ($\bar{X}=2.0$) than did the IHCC respondents ($\bar{X}=4.0$). This, according to the ISBH respondents, is one of the major reasons why they did not carry out a more vigorous planning program. In fact, both agencies received approximately the same amount of money ($1,300,000)--one to implement planning for one region, the other to implement planning in the 70 percent of the state that was classified as undesignated.

Closely related to these economic factors are technical factors such as the adequacy of staff and their expertise. The mean responses to the question on the adequacy of staffing were virtually identical to those for adequacy of overall budget. However, as with the adequacy of 208 budget item, the ISBH respondents reported much lower adequacy ($\bar{X}=2.0$) than did the IHCC respondents ($\bar{X}=4.0$). This was one of the major problems for the ISBH and led to the subcontracting of soil erosion assessment studies to the State Soil and Water Conservation Committee, coordination of public participation to Regional Planning and Development Agencies, etc. According to one of the respondents, 208 planning was such a large program nationally, that there simply weren't enough qualified water planners to meet the demand.

Staff expertise was rated as better than adequate by the entire set of organizations, but only adequate by IHCC and ISBH. This is perhaps to be expected since these two organizations, because of their centrality to the planning process, had the greatest demands made on their staff.

Finally, there was not a significant correlation overall ($r=.10$, n.s.) between organization age and the effect that participation in the 208 process had on perceived public support for the organization. The correlations varied considerably among the organizations in the different loci. While Heartland organizations had a similar but negative correlation ($r=-.11$, n.s.), PDR4 organizations had a strong positive correlation ($r=.48$, $p<.04$), and state level organizations had strong negative correlation ($r=-.51$, $p<.12$). Statistical control for various indicators of contact with the public did little to alter the reported coefficients. There is no convincing evidence for a consistent relationship based on these data, which may be in part a methodological artifact of the data.

One of the key questions asked of all respondents was intended to find out what kind of goals the organizations involved in 208 planning sought to achieve. The result was that the respondents for many organizations reported that their organization had no goals for 208 planning (N=4), their goals were whatever EPA said they were (3), were seeking information or were only monitoring the process (7) or were there simply to discuss water quality management (3). This is particularly interesting in light of the fact that when asked how important water quality management was to their organization, 75 percent of the respondents reported that it was either important or very important to their organization. Thus, nearly half of the organizations that participated in 208 planning took a reactive rather than a proactive approach to the 208 planning process. This can, of course, be taken as an indication that these organizations were satisfied with existing water quality and planning for water quality in their areas. If this is in fact the case, it is very unlikely that, without the Federal government's insistence, anything like the 208 process would have occurred in Indiana, given the State's reluctance to become involved in 208 planning.

Although the previous explanation was no doubt true for some organizations, an alternative explanation is that few organizations were aware of the potential of 208 planning or even what it involved. In PDR4, the public participation phase of the planning process didn't formally start until late in 1977, and the plan was completed in 1978. This allowed little time for citizen education and informed involvement. In PDR4, public participation primarily involved public reaction to a plan created at the state level by the Indiana State Board of Health's Division of Water Pollution Control and its subcontractors. This clearly inhibits the development of local support for long term water quality management programs funded even in part by local dollars.

However, not all organizations played a passive role in the process. A notable exception is the League of Women Voters. They were heavily involved in 208 planning at both regional and state levels. They applied for and received a small grant from EPA (approximately $5,000) to present public information and education workshops on 208 planning. It is important to note that their goal was not to influence the content of the 208 plans in these workshops. Rather, their goal was to increase the level and the quality of public input into the planning process.

To a lesser extent, groups like the Izaak Walton League, Soil and Water Conservation Districts, and other agricultural groups sought in various ways to provide information to the public, but more particularly, to their constituents.

Institutional Relations. Interorganizational relations are the central feature of institutional relations studied here. They are frequently a problem in multi-organizational efforts, and 208 planning was no exception. These problems were quite often reported as being related to the ISBH. ISBH's structural position is important, since it was the state agency responsible for 208 planning, and for water quality generally. Thus it lies between local organizations and federal organizations on many environmental matters. Heartland area organizations in particular reported problems in their relations with ISBH. Although IHCC staff and other participants in the process in that region were not directly responsible to ISBH, the Heartland 208 plan still had to be approved by the state personnel. While the overall goal/priority incompatibility with other organizations was generally low with little effect on relations, respondents reported that incompatibilities had a moderate effect on relations with the ISBH. This is consistent with anecdotal data in which the ISBH was described as inflexible and apparently unwilling to seriously consider ideas from outside of the agency. This is most likely a result of a combination of factors including their history as a regulatory agency, their staff shortages, and that as a whole, the agency had little enthusiasm for 208 planning.

Institutional relations constitute an important part of both 208 planning and of this study. Respondents were asked to name those organizations that they had contacts with concerning 208 planning. The number of other organizations named ranged from one to fourteen with a mean of seven. The frequency of contact was in most cases monthly or slightly less. Many 208 committees met monthly, so it is probable that most interorganizational contacts took place at scheduled meetings. Typically, these contacts involved information sharing, with resource transfer (money, personnel, etc.) occurring much less frequently. In nearly 85 percent of the contacts, there was no resource sharing and in slightly more than 10 percent of the contacts, it occurred less than monthly. Thus, although institutional relations were relatively frequent, they were not particularly intense, as few "hard" resources were exchanged.

However, most respondents felt that their contacts with other organizations were important for their own organization with nearly 60 percent rating them as important or very important. IHCC respondents said that their contacts were very important, while ISBH and PR04, the other two organizations with 208 responsibilities, reported them to be of lesser importance. There was little difference though in how other organizations rated their contacts with the three 208 planning agencies--all rated as fairly important.

One of the variables of particular interest is the influence that the participant organizations had on the planning process. The organizations were divided into two groups--those named eight or more times, and those named less than eight times. Frequently named organizations were rated as much more influential (self-ratings excluded) than those named less frequently. About 57 percent of the respondents rated the frequently named organizations as highly influential or very highly influential, while over 75 percent rated the less frequently named organizations as moderately influential or lower. The perceived influence of the three 208 planning agencies reflects the scope of the role they each played in the process. IHCC and ISBH were both seen by others as having quite high influence, with PDR4 having moderate influence on 208 planning. Agricultural organizations such as the Soil Conservation Service, Farm Bureau, etc., were generally rated as both important contacts, and also as at least moderately influential. The Soil Conservation Service and State Soil and Water Conservation Districts in particular were rated as quite important and were also highly influential.

Overall, there does not appear to have been a great deal of conflict among the organizations involved in 208 planning. For the entire set of named organizations, over 75 percent of the respondents reported either none or little effect from incompatibilities on their relationship with the organizations they named. However, of those naming ISBH as an organizational contact, nearly 60 percent said that incompatibilities had either moderate or great effects on their relationship. Clearly, there was not always agreement between ISBH and participants as to procedures and outcomes for 208 planning.

Public Participation. As was mentioned earlier, Congress mandated regional planning and public participation in 208 planning. Congress appeared to have two reasons for this. The first was that these two elements were necessary for the success of water quality management planning. Water quality management and especially non-point pollution are problems that tend to involve more than one governmental jurisdiction. Thus, any serious attempt to assess and reduce non-point pollution could not be bound by traditional city or county boundaries. Since the water quality plans could conceivably have had a great impact on the public (e.g., if the plan included land use regulations), it was important that those affected by the plan have some input into it. The second reason was that Congress apparently expected that local agencies would fail to carry out 208 planning unless it was mandated.

Thus, 208 planning featured an interesting combination of required participation and volunteerism. On the one hand, areawide or regional planning and public participation were required. On the other, no particular groups or organizations were required to be involved and it was left to the states (and regional planning bodies) to implement these specified features.

This led to some differences in implementation of 208 planning, depending on who the responsible agency was. In general, the respondents stated that their organizations were quite committed to public participation with over 75 percent of them responding that their organizations were either moderately or very committed to public participation, and only 8 percent reported opposition to public participation. However, respondents for the agency in charge of planning for undesignated areas reported that their agency was neither committed nor opposed (aggregated responses) to public participation in 208 planning. This may explain to some extent the relatively low level of public participation in PDR4. Although there were public meetings and a Citizens Advisory Committee, these were primarily vehicles for State Board of Health personnel to inform people about the status of 208 planning in their region rather than as channels for public input. In fact, several of the voluntary organization respondents reported that at least part of their efforts were directed toward getting the State Board of Health to implement as much public participation as they did. Even so, three respondents from the PDR4 area spontaneously said that they felt the plan for their region was "handed down" from the State Board of Health, and felt that there should have been more public input.

In the Heartland area, participation was much greater. Although the bulk of the planning work was done by IHCC staff and a citizen steering committee, they also had five large committees that dealt with problem areas such as agricultural non-point pollution, contamination from septic tanks, etc. Respondents for the Heartland agency reported very high committment to public participation (aggregated responses).

Although there was public participation in both of the regions studied, it is clear that it was much lower in the undesignated area, where planning occurred within the state agency, and higher in the designated area where there were greater opportunities for public participation.

Regional Planning. The second aspect of the mandate--regional planning--was quite controversial in some areas of the state where it was seen as an attempt to create another level of government between the county and state levels. Although not a major issue in either of the two areas studied, it was less popular than the public participation element. Slightly over 65 percent of the respondents reported moderate to very high committment to regional planning, while 15 percent reported some level of opposition to regional planning.

As with public participation, the State Board of Health respondents reported less committment to regional planning than most other respondents, and indicated that there was some opposition to it within the agency. The Heartland respondents, however, indicated strong committment to regional planning. This could be expected since the IHCC was a regional planning agency. This is highly consistent with the evidence that planning for the undesignated area (PDR4) occurred primarily at the state level, while planning for the Heartland area occurred at the regional level with associated public involvement.

There were, of course, a number of benefits from 208 planning in the two regions studied. In response to an open-ended question concerning benefits of 208 planning, close to half of the respondents (17) felt that it had increased their organization's and other organizations' awareness of water quality management problems. Additionally, twelve felt that 208 planning had heightened the general public's awareness of water quality issues. Sixteen respondents felt that 208 planning had led not only to increased inter-organizational contacts, but had improved the quality of interorganizational relations as well. Seven respondents noted an increase in information sharing among organizations involved in 208 planning.

2.7 Summary and Conclusions

To summarize the key points of this analysis, a comparison of the 208 planning process in the two study regions reveals a number of differences. The Heartland group had full responsibility for 208 planning and engaged in extensive data collection and documentation for their plan. They received additional money for a Model Implementation Project and for a study of on-lot disposal problems in one of their member counties. They also sought (unsuccessfully) to have the ISBH Stream Pollution Control Board approve a "limited-use stream designation" regulation. While it is difficult to determine with any certainty the level of their involvement, several hundred people representing all eight counties had committee appointments, and at least had formal opportunity for input into the plan. This is in addition to the public meetings sponsored by IHCC.

In PDR4, on the other hand, the regional agency had only partial responsibility for 208 planning, including land-use mapping, inventories, and projections. Soil erosion assessment, for example, was subcontracted to the State Soil and Water Conservation Committee, and overall public participation responsibility was retained by the ISBH. Thus, the advantages of using an existing regional agency were largely lost, and 208 planning and public participation became a "top-down" process in PDR4. Although the PDR4 Policy Advisory Committee did meet monthly for a six-month period in late 1978 and early 1979, the minutes of the last regular meeting (May 9, 1979) show that "It was felt by many committee members that the material presented was too technical and did not contain very much policy material." This, in large part, reflects the difference between data and information generated regionally as in the Heartland region versus data and information generated by the state agency and then presented to the region. Clearly, the ISBH took an "educational" approach in PDR4, while the IHCC implemented a more participatory approach.

The second major point of this analysis concerns the ISBH. The ISBH's Stream Pollution Control Board had overall responsibility for approving and submitting 208 plans for all designated and undesignated areas in Indiana. It did not necessarily wish to become involved in 208 planning, but had little choice in the matter for two reasons. First, the Governor assigned 208 planning to the ISBH (though presumably the Board had some input into this decision). Second, and more important, was that this agency had responsibility for water quality in Indiana, and the Indiana Department of Natural Resources, in general, had responsibility for water quantity. The ISBH either had to accept responsibility for 208 water quality management planning or else allow another organization to encroach on its domain. This lack of enthusiasm for 208 planning in conjunction with a shortage of funds for additional personnel and perhaps a lack of management expertise resulted in the ISBH's eventual compliance with EPA regulations and directives, but at the most minimal level possible. This is not to say there were not dedicated personnel in the Division of Water Pollution Control. Rather, there were not enough of them, and evidence suggests that those staff were probably not utilized as effectively as they might have been. This is perhaps not too surprising if, as one of the respondents suggested, promotion policies tended to advance personnel with engineering and scientific training rather than filling management positions with trained administrators.

The final point to be made concerns the interorganizational relations among the organizations participating in 208 planning. In general, agricultural organizations such as the Farm Bureau, Soil and Water Conservation District Boards, etc. were well represented and involved in 208 planning both at the regional and state levels. The remaining organizations primarily included citizen/environmental groups, city officials and employees, with very few business organizations involved. The organizations with a surprising lack of involvement were county-level governments. None of the elected county commissioners from the sixteen counties in the study regions played a central role in the 208 process, and only two were reported to have played even a peripheral role. There are several explanations for this. One, of course, is that 208 planning was not perceived as relevant to county government. An alternative is that it was unclear what the outcome was to be and therefore what the political consequences of 208 would be. Thus, the politically expedient course of action was to ignore the process entirely. This is a significant institutional factor, because the regional agencies had no real enforcement powers, and any implementation of a water quality management plan would almost certainly involve county government as well as the state government.

The nature of the contacts between organizations was generally perceived as being cordial with a relatively low level of conflict. Disagreements that did occur were most likely to occur between the ISBH and other organizations or between organizations from different sectors of the economy such as environmental and business organizations. But also, there was not a particularly high level of intense contact between the organizations. As mentioned before, many of the organizations did not have strong goal positions for the 208 process. Second, much of the process involved technical rather than policy decisions which no doubt averted some potential conflict.

Staffing was somewhat of a problem for 208 planning agencies. Organizations had to hire staff, which meant acquiring funds and positions to do so. When the impetus for the program has not come at the local level that may be difficult to do, for institutional factors become relevant here. Even though funding may come from the federal level at the present, the duration of federal money is uncertain. Local government may be reluctant to expand, fearing it will be faced with a choice of picking up program costs in the future, or cutting the program back and firing people, neither of which are seen as desirable. Also there may be local opposition to the program.

It needs to be recognized that the expansion of environmental protection in the early 1970's was enormous. For example when the level of detailed information needed for 208 planning alone was extended to non-designated areas as well as designated areas a great amount of work was entailed beyond what planning agencies were previously doing.

What conclusions can we draw for future programs? Three factors stand out as important to modify in future water policies involving federal-state-local relations: length of time required for planning and implementation, sources of funds, and local support.

The length of time provided for 208 planning posed problems both for the content of plans and personnel for doing the plans. Largely it was assumed the data and personnel were available. A longer period of time would have alleviated some problems by allowing agencies to carry the work out over a longer period which could be done with fewer staff, and also provided time for training more staff. While federal funding seemingly solves the direct costs problem for an organization doing 208 planning, its limited duration with no certainty for future funds doesn't do much for longer term stability in an organization that needs to hire personnel to carry out such planning. In addition, uncertainty about funds for implementation may make the activity seem trivial or futile.

208 planning contained a substantial citizen participation component, at least potentially. We found this component implemented much more in the designated than non-designated area. A third change to be considered in future programs is to place greater emphasis on the public participation component with two objectives in mind. It would develop the plan out of local citizens' concerns, not simply as a technical activity, and in the process build the basis for stronger local support for moving toward funding and implementing the plan. For example, the planning process might be structured as a large quasi-nominal group process, at the regional level for example, in which the stakes are potentially real. Presumably the objective of 208 was not to develop a set of plans, but to develop a means to deal with water quality problems. While there is a need to develop technical knowledge of water quality, there is also a need to develop a constituency to support such monitoring activity and to support action for alleviating problems.

In conclusion, although 208 planning was never a smooth process in the two study regions, and many participants were frustrated by their experiences and the lack of clear results from the planning process, it may in fact have laid the foundation for future water quality management programs within the regions. It was clearly a learning process for all participants, and as such, responsibility for any shortcomings in 208 planning cannot be laid solely on any one organization.

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CHAPTER 3
STORMWATER RUNOFF QUALITY

3.1 Introduction

Stormwater runoff, particularly from urban areas, can be a substantial source of pollutants entering streams and lakes. This research was undertaken to evaluate the characteristics of stormwater runoff from an urban watershed and to determine the relationship, if any, between such quality and various hydrologic activities such as rainfall intensity, total rainfall, storm duration, antecedent dry period, street sweeping, and season of the year. Water quality data included suspended solids, biochemical oxygen demand (B.O.D.), total coliforms, and fecal coliforms.

3.2 Literature Review

Studies of urban stormwater runoff have allowed certain generalizations to be made about the quality of these waters (see references 1-30 in Bell and Blumberg, 1984). The B.O.D. of urban runoff is approximately the same as the effluent from secondary wastewater treatment plant. The suspended solids are higher than those found in raw sewage. The bacterial concentrations are typically 2 to 4 orders of magnitude greater than those considered safe for water-contact recreational activities.

The effect of antecedent conditions, primarily street sweeping, on the quality of stormwater runoff has been studied by numerous investigators. Other investigators have studied the effect of such hydrologic conditions as rainfall intensity, total runoff, rainfall depth, rainfall duration, and season of the year (see references 7-10, 20-27, 30-40 in Bell and Blumberg, 1984).

3.3 Sample Analysis

As described in Chapter 6, a field station for the collection of water quantity and quality data was established for the Ross-Ade watershed in West Lafayette, Indiana. The station included an automatic sampling device to collect and store water samples during various rainfall events. Samples were transported to the laboratory as soon as possible after a storm. They were placed in a refrigerator at $4^{\circ}C$ and analyses were typically completed within two days.

All 28 samples collected from each storm were not analyzed. Typically 10-12 samples were analyzed per storm. Usually every second sample was analyzed from the first half hour, and every third sample thereafter. This produced a maximum sampling interval of thirty minutes. This scheme was altered on the basis of a visual check of the turbidity of the samples; i.e., samples with especially high or low turbidity were always analyzed.

The analyses performed in this study were chosen on the basis of their importance in characterizing the quality of the runoff, the relative ease in which they could be performed, and the frequency of their appearance in the literature dealing with stormwater quality. The 5-day biochemical oxygen demand (B.O.D.), suspended solids (SS), total coliform (TC), the fecal coliform (FC) tests were performed in this study . The analyses were performed in accordance with the procedures given in Standard Methods for the Examination of Water and Wastewater, 14th Edition (1975).

3.4 Runoff Characteristics

A total of 36 discrete runoff events were sampled. An event was considered discrete when the time between precipitations falling was equal to greater than the critical lag. The critical lag is that time period which separates rainfall sequences into non-related events. The critical lag for this watershed has been defined as 100 minutes (Rao and Chenchayya, 1974).

A summary of hydrologic data is presented in Table 3.1. Peak rainfall intensities and peak runoff were determined from the most intense ten minutes of each event.

A summary of runoff quality data is presented in Table 3.2. Average flows were used to determine total mass (or number) and peak rates. Average flows were calculated using the flow at the time of the sample plus those flows for a half-time interval both before and after such flow.

Data from all samples were entered on plots of cumulative load versus cumulative volume (Figure 3.1). Most of the points lie above the dashed 45-degree line indicating a greater portion of the total pollutant was contained in the earlier portion of the runoff; which is indicative of a "first flush".

Three generalized pollutograph patterns emerged in this study. The first was a classic first flush with a rise in concentration followed by a decrease and subsequent leveling off (Figure 3.2). The second was the same as the first, except the peak concentration occurs initially (Figure 3.3). The third pattern exhibits more than one peak (Figure 3.4).

Suspended solids exhibited 11 pattern-one storms, 15 pattern-two storms, and 8 pattern-three storms. BOD exhibited 9 pattern-one storms, 21 pattern-two storms, and 4 pattern-three storms.

Both total coliforms and fecal coliforms displayed a first flush. Cumulative loads are plotted against cumulative volumes in Figure 3.5. The same three generalized pollutograph patterns emerge; however, curves were more irregular than those for BOD and SS. Total coliforms exhibited 7 pattern-one, 4 pattern-two, and 13 pattern-three storms. Fecal coliforms exhibited 6 pattern-one, 7 pattern-two, and 10 pattern-three storms.

Table 3.3 compares runoff quality to treated and untreated domestic wastewater. Stormwater runoff contains significantly less BOD and SS than untreated sewage. More importantly, runoff contains higher concentrations of BOD and SS than treated effluent, and concentrations of fecal coliforms in runoff are over 100,000,000 times greater than those in chlorinated effluent.

Table 3.1

Summary of Hydrologic Data

Hydrologic Activity	Unit	Range
Peak Rainfall Intensity	in/hr	0.12 - 2.04
Average Rainfall Intensity	in/hr	0.03 - 1.60
Total Rainfall	in	0.03 - 1.83
Peak Rate of Runoff	MGD	0.28 - 19.04
Total Runoff	MG	0.004 - 0.634
Duration of Precipitation	min	5 - 299

Table 3.2

Runoff Quality Summary

		Total lb/storm	Peak lb/day	Avg. mg/l	Peak mg/l
BOD	range	0.3 - 110	21 - 3,220	2 - 61	6 - 136
	mean	16	554	19	40
	std. dev.	23	765	15	32
	# of storms	34	34	34	34
SS	range	1 - 706	59 - 27,800	12 - 199	35 - 1,380
	mean	78	4,690	78	335
	std. dev.	142	8,260	50	336
	# of storms	34	34	34	34

		Total #/storm	Peak #/day	Avg. #/100 ml	Peak #/100 ml
TC	range	5.6×10^{9} - 1.0×10^{13}	5.0×10^{11} - 6.3×10^{14}	8.8×10^{3} - 1.5×10^{6}	2.0×10^{4} - 5.1×10^{6}
	mean	1.2×10^{12}	7.7×10^{13}	3.2×10^{5}	1.0×10^{6}
	std. dev.	2.6×10^{12}	1.6×10^{14}	4.9×10^{5}	1.4×10^{6}
	# of storms	24	24	24	24
FC	range	4.8×10^{8} - 2.3×10^{12}	1.6×10^{10} - 1.1×10^{14}	6.7×10^{2} - 9.7×10^{5}	5.0×10^{3} - 1.3×10^{6}
	mean	1.9×10^{11}	1.0×10^{13}	2.0×10^{5}	7.3×10^{4}
	std. dev.	4.9×10^{11}	2.4×10^{13}	3.5×10^{5}	2.0×10^{5}
	# of storms	23	23	23	23

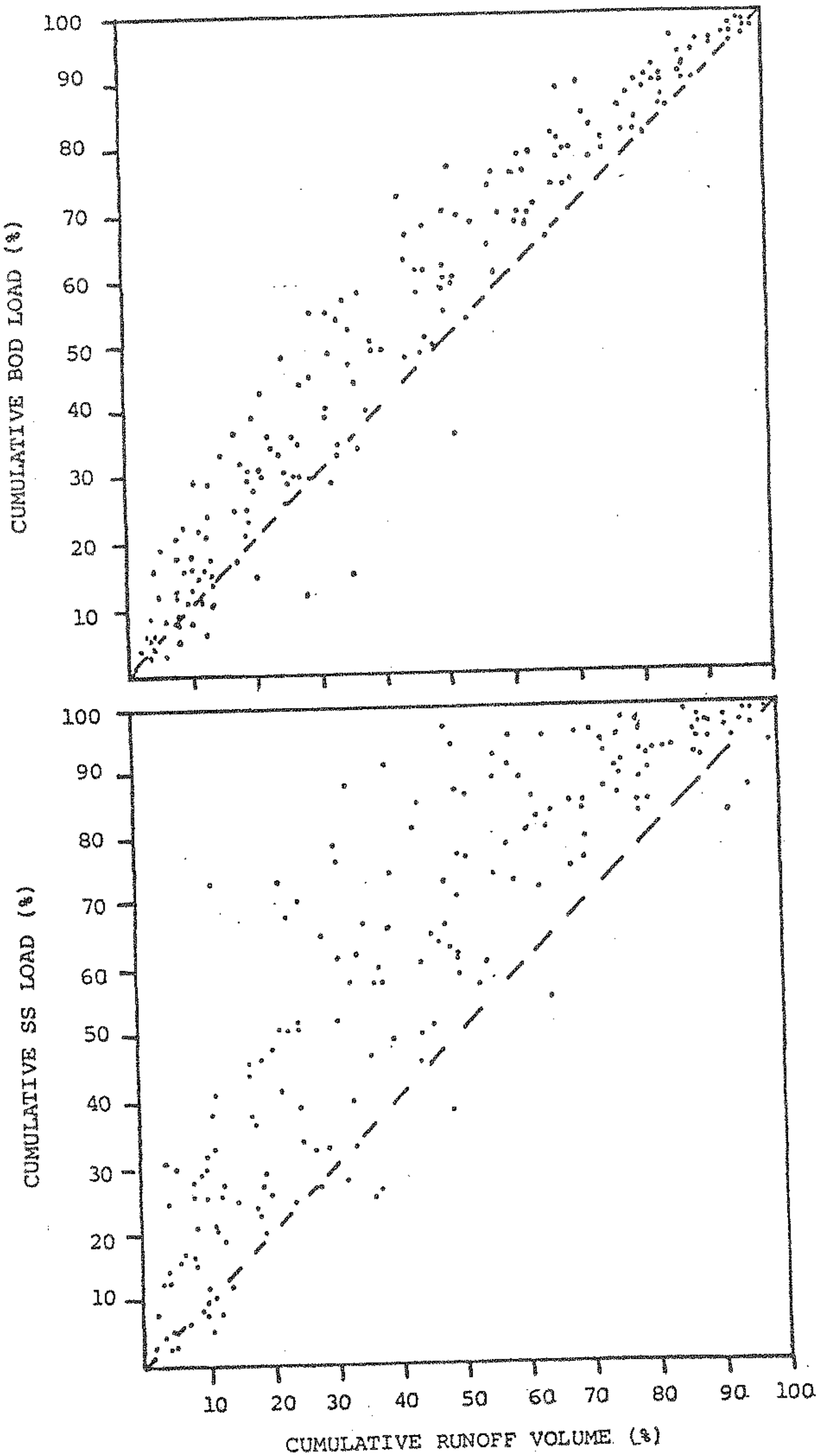

Figure 3.1. Cumulative BOD and SS Loads(lb) vs. Cumulative Runoff Volume(MG)

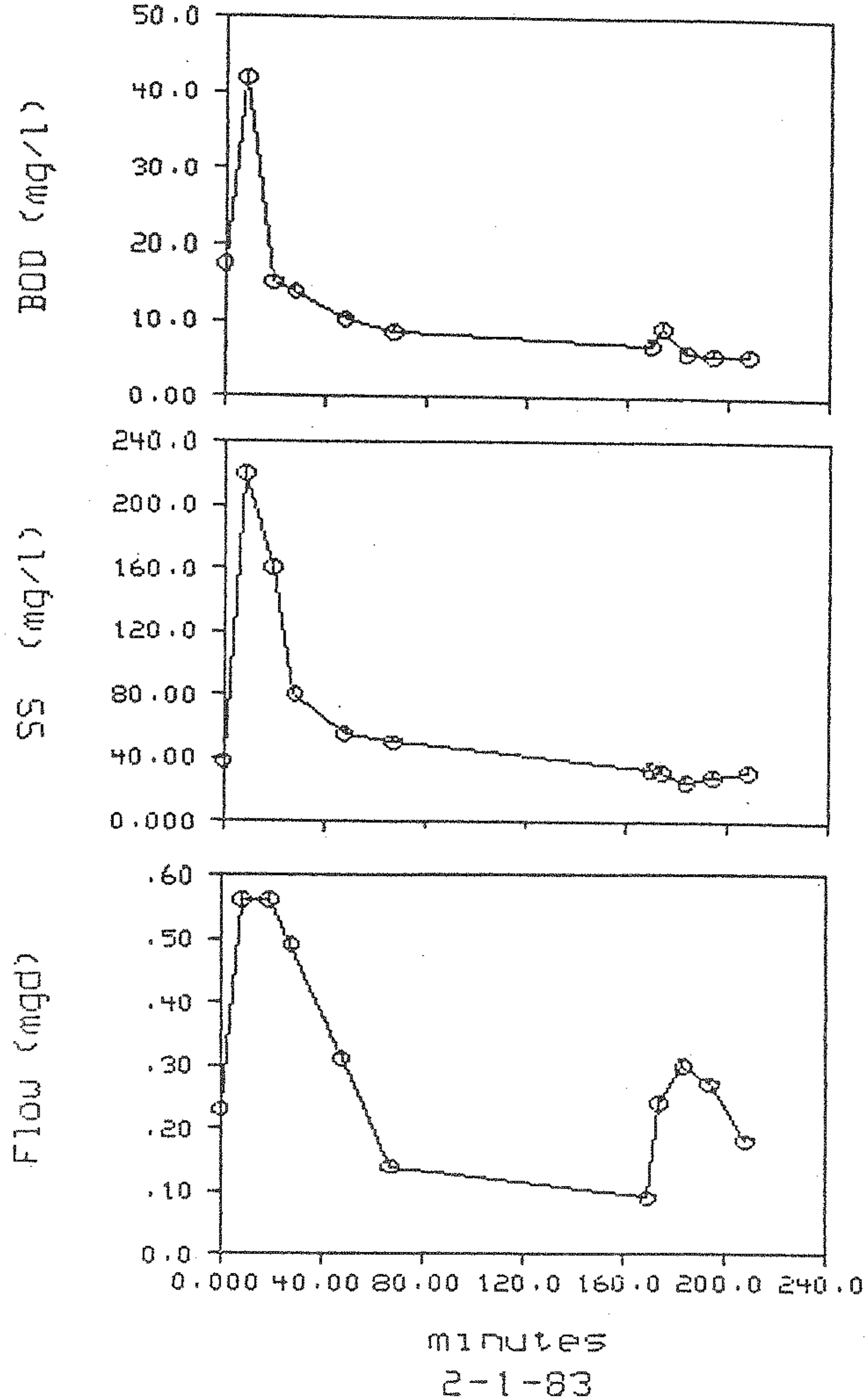

Figure 3.2. Plots of BOD, SS and Flow Exhibiting Pattern-One Pollutographs

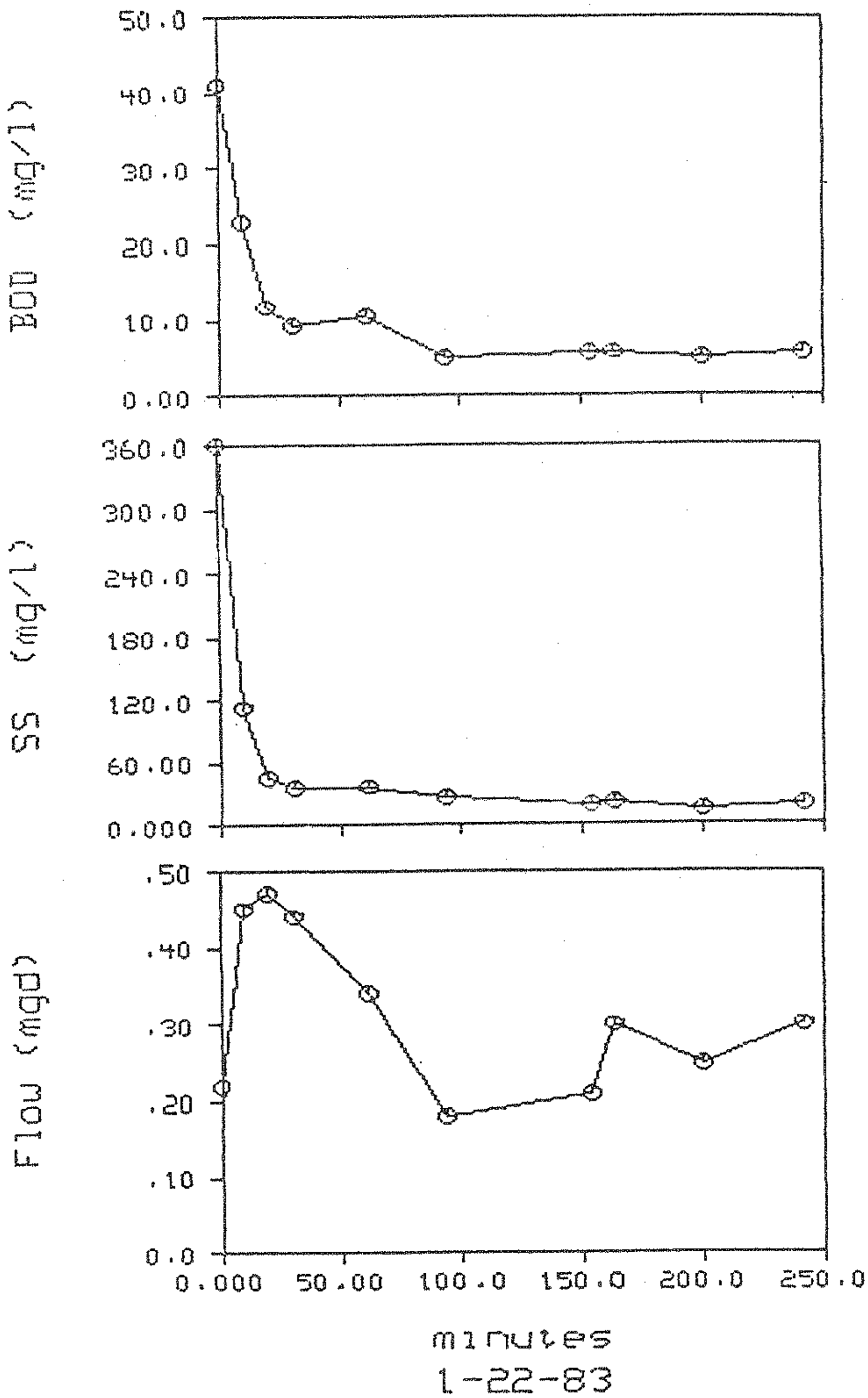

Figure 3.3. Plots of BOD, SS and Flow Exhibiting Pattern-Two Pollutographs

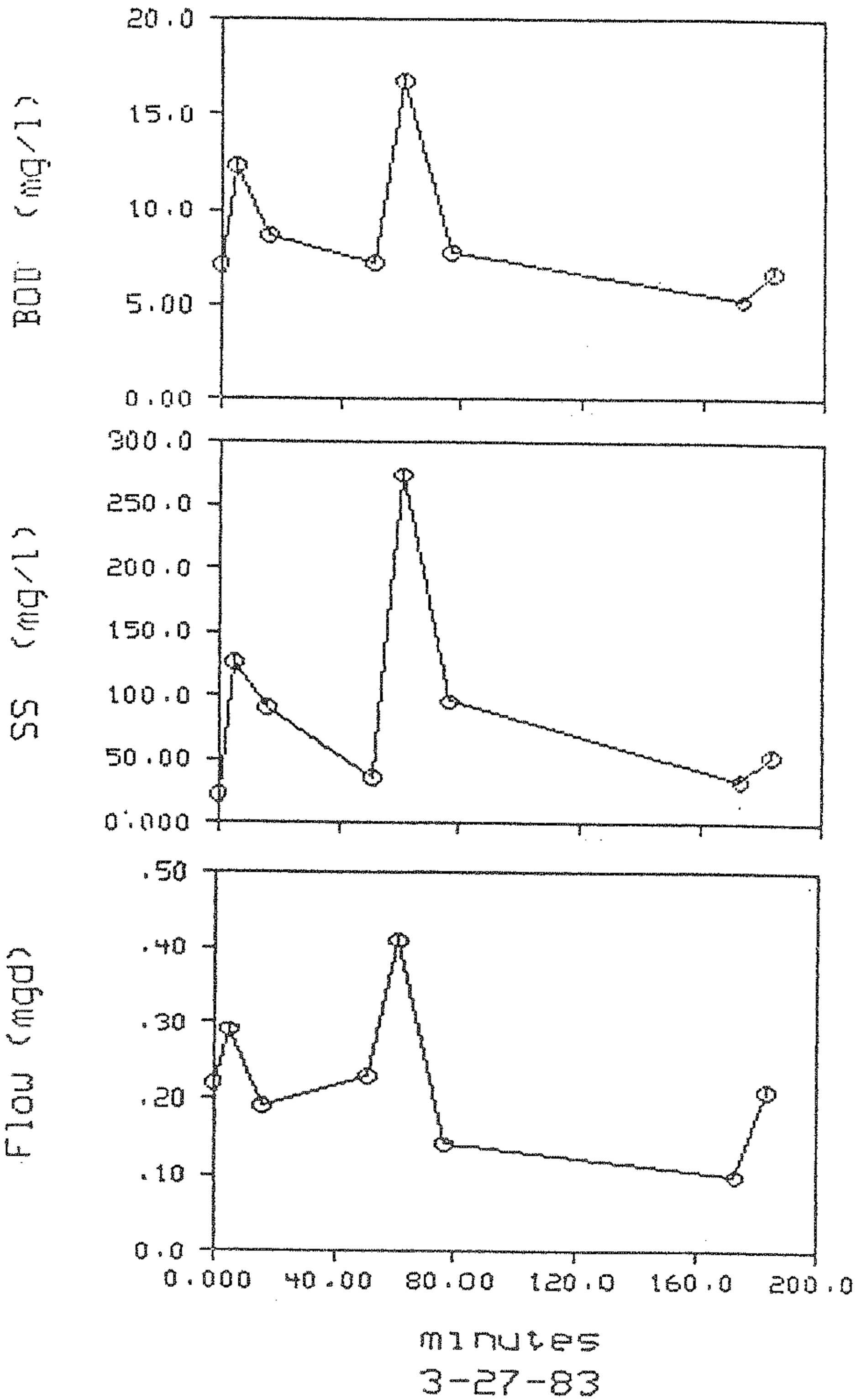

Figure 3.4. Plots of BOD, SS and Flow Exhibiting Pattern-Three Pollutographs

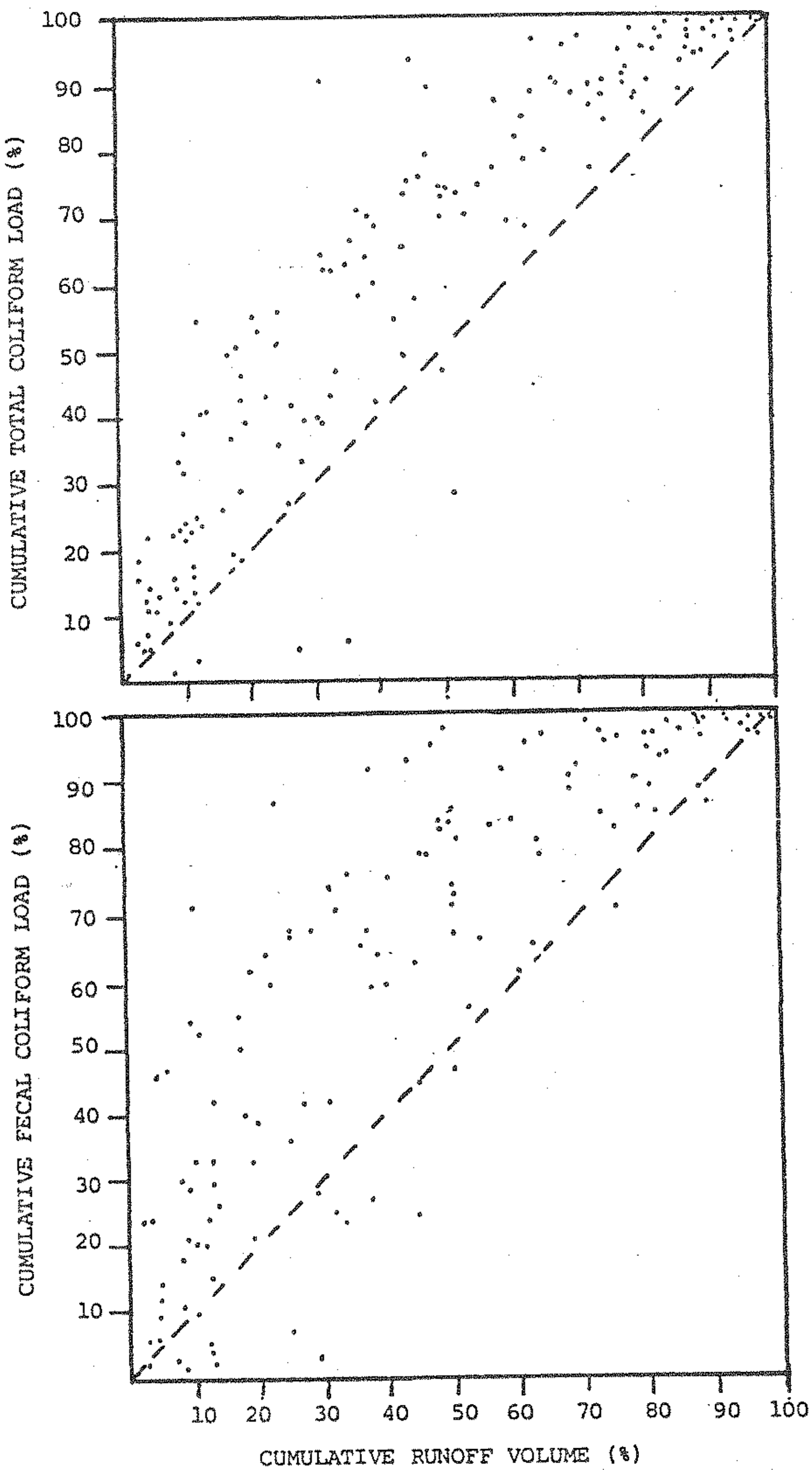

Figure 3.5. Cumulative Total and Fecal Coliform Loads(#) vs. Cumulative Runoff Volumes (MG)

Table 3.3
Characteristics of Stormwater Runoff and Domestic Wastewater

	BOD (mg/l)	SS (mg/l)	Fecal Coliforms (#/100 ml)
Stormwater Runoff[a]	16	78	1.9×10^{11}
Sewage[b]			
Untreated	166	229	
Secondary Treat.	6	7	1700[c]

[a]Average Concentrations

[b]Values from West Lafayette, Indiana treatment plant (1981 data for BOD and SS; 1981 & 1982 data for fecal coliforms).

[c]Chlorinated secondary wastewater

3.5 Regression Analysis

Introduction. In an effort to determine which hydrologic parameters have the most effect on water quality, multiple regression analyses were performed. The dependent water quality variables were biochemical oxygen demand, suspended solids, total coliforms, and fecal coliforms--in the forms of total mass (or number), peak rate, average concentration, and peak concentration.

The independent variables are those hydrologic activities involved in the collection and transport of pollutants. Pollutants are made available by the flushing action of rainfall. To account for that action, peak intensity, average intensity, and total rainfall were considered. Transport of pollutants is regulated by the carrying capacity of the flow. Those parameters involved in that action are peak runoff and total runoff. As total runoff and total mass (or number) are not independent, total runoff was omitted from total mass (or number) regressions. An additional parameter, storm duration, was included in all regressions. Independent and dependent variables and their units are listed in Table 3.4.

A correlation matrix (r values) of independent variables, inclusive of all 36 storms, is presented in Table 3.5. Peak intensity and peak runoff are highly related ($r^2 = 0.95$). The same is true of total rainfall and total runoff ($r^2 = 0.95$). A high correlation also exists between total rainfall and peak intensity, total rainfall and peak runoff, and total rainfall and total runoff.

Formulation of Models. The first step in the regression analysis was an attempt to determine which hydrologic parameters were most important. The BMD program P9R was used to estimate coefficients of determination (r^2) for the best subsets of predictor variables (Dixons and Brow, 1979). Best subsets were chosen by the adjusted r^2. Adjusted r^2 is $r^2 - p(1-r^2)/(N - p')$, where:

p = number of independent variables
$p' = p + 1$
N = number of storms

The inclusion of more independent variables will always increase r^2. The adjusted r^2 is used to account for that inflation.

The best possible adjusted r^2 for each dependent variable is listed in Table 3.6. Average and peak concentrations of BOD, SS, and fecal coliforms show no relation to hydraulic parameters ($r^2 \leq 0.25$). This is not unexpected because high levels of flushing (total rainfall, average intensity, or peak intensity) would not only wash off more pollutants, but also increase carrying capacity (total runoff or peak runoff). This increased flow would cause dilution of high pollutant concentrations, thus complicating relationships between those concentrations and hydraulic parameters. No reason, however, is apparent for the better relationships found between average and peak concentration of total coliforms and hydraulic parameters.

It was considered appropriate to generate regression relationships for the ten cases where hydraulic parameters accounted for more than 25 percent ($r^2 > 0.25$) of the variance in the data. The SPSS REGRESSION subroutine was

Table 3.4

Regression Variables

Dependent Variable	Abbreviation	Unit
Total mass BOD	tbod	lb/storm
Total mass SS	tss	lb/storm
Total mass Total Coliforms	ttc	#/storm
Total mass Fecal Coliforms	tfc	#/storm
Peak rate BOD	rbod	lb/day
Peak rate SS	rss	lb/day
Peak rate Total Coliforms	rtc	#/day
Peak rate Fecal Coliforms	rfc	#/day
Average concentration BOD	avbod	mg/l
Average concentration SS	avss	mg/l
Average concentration Total Coliforms	avtc	#/100 ml
Average concentration Fecal Coliforms	avfc	#/100 ml
Peak concentration BOD	pbod	mg/l
Peak concentration SS	pss	mg/l
Peak concentration Total Coliforms	ptc	#/100 ml
Peak concentration Fecal Coliforms	pfc	#/100 ml

Independent Variable	Abbreviation	Unit
Peak Rainfall Intensity	pint	in/hr
Average Rainfall Intensity	avint	in/hr
Total Rainfall	tot	in
Peak Rate of Runoff	pkrun	MGD
Total Runoff	run	MG
Duration of precipitation event	dur	minutes

Table 3.5

Correlation Matrix of Independent Variables (r values)

	pint	avint	tot	dur	run
avint	.43				
tot	.85	.40			
dur	.17	-.34	.39		
run	.81	.26	.95	.40	
pkrun	.95	.31	.90	.27	.91

Table 3.6

Best Adjusted r^2

Dependent Variable	Adjusted r^2	Independent Variables Included
Total mass BOD	.36	pkrun, dur
Total mass SS	.87	tot, pint
Total number Total Coliforms	.59	pkrun, pint, avint
Total number Fecal Coliforms	.51	pkrun, pint, avint
Peak rate BOD	.57	pint, tot, dur, pkrun
Peak rate SS	.79	pkrun
Peak rate Total Coliforms	.60	pkrun, pint, avint, tot, dur
Peak rate Fecal Coliforms	.43	pkrun, pint, avint
Average concentration BOD	.12	avint, tot, dur
Average concentration SS	.21	tot, run, pkrun
Average concentration Total Coliforms	.55	avint, tot, run, dur
Average concentration Fecal Coliforms	-.01	dur
Peak concentration BOD	.02	avint, tot, dur
Peak concentration SS	.05	dur
Peak concentration Total Coliforms	.35	avint, pkrun, pint
Peak concentration Fecal Coliforms	.04	pint, pkrun

used to generate these models (Nie et al., 1975). The independent variables were introduced into the regression one at a time. The variable entered in each step is the one which explains the greatest amount of variance unexplained by those variables already in the equation.

The best models are presented in Table 3.7. Table 3.7 lists the adjusted r^2 value and its significance. Significance indicates how likely it is that the r^2 for the model occurred by chance alone. The significance is obtained from the F-statistic.

Suspended Solids. The independent variable with the greatest amount of variance accounted for by the hydrologic parameters is Total Mass of Suspended Solids (TSS). Peak intensity, total rainfall, and peak runoff were the three independent variables selected by the regression analysis for the best (highest adjusted r^2) TSS models.

The model with the largest adjusted r^2 contains only peak intensity and peak runoff (adjusted $r^2 = 0.87$). A second model containing total rainfall and peak runoff, has an adjusted r^2 (0.86) which is only slightly lower than the first. The inclusion of more than two variables into either of these two models results in no improvement in adjusted r^2 suggesting that peak intensity, peak runoff, and total rainfall contain all the information of the other hydrologic parameters.

As with Total Mass of SS (TSS), the Peak Rate of SS (RSS) also had a high correlation with peak intensity, total rainfall, and peak runoff. The best resulting regression relationship for this dependent variable is shown in Table 3.7, but contains only peak runoff.

Biochemical Oxygen Demand. Total Mass of BOD (TBOD) has a high correlation with peak intensity, total rainfall, and peak runoff. The best regression model contains peak runoff and duration. Inclusion of other hydrologic parameters lowers the adjusted r^2, indicating they contain no additional information. The best model for Total Mass of BOD, although highly significant ($\alpha = 0.0001$), explains only 35 percent of the scatter in the data.

The Peak Rate of BOD (RBOD) had a high correlation with peak intensity, total rainfall, and peak runoff. The best regression relationship contains peak intensity, total rainfall, peak runoff, and duration.

Total and Fecal Coliforms. Although Total Number of Total Coliforms (TTC) has the highest correlation with peak intensity, total rainfall, and peak runoff, the best regression relationship was found with the use of the independent variables of peak intensity, average intensity, and peak runoff. The addition of duration to the model only increased the adjusted r^2 by 0.01, and was deemed, therefore, unnecessary.

The best regression relationship (Table 3.7) for the Total Number of Fecal Coliforms (TFC) contained the same hydrologic parameters as Total Number of Total Coliforms (TTC): namely, peak intensity, total rainfall, and peak runoff. The best regression model for the Peak Rate of Fecal Coliforms (RFC) incorporated peak intensity, average intensity, and peak runoff.

Table 3.7

Best Regression Models

Model	Adjusted r^2	Significance
tss = 183.98 tot + 70.29 pint - 32.03	.87	.0001
tss = 214.53 tot + 9.56 pkrun - 28.48	.86	.0001
tbod = 2.30 pkrun + .78 dur - 1.94	.36	.0001
ttc = 1.69×10^{12} pkrun - 7.42×10^{12} pint + 2.06×10^{12} avint + 4.07×10^{11}	.59	.0001
tfc = 5.90×10^{11} pkrun - 3.09×10^{12} pint + 7.64×10^{11} avint + 1.30×10^{11}	.51	.001
rss = 1466.61 pkrun - 291.03	.79	.0001
rbod = 580.96 pint - 1502.67 tot + 2.05 dur + 107.96 pkrun + 49.72	.57	.0001
rtc = 1.01×10^{14} pkrun - 3.28×10^{14} pint + 2.45×10^{14} avint - 3.63×10^{14} tot + 8.31×10^{11} dur - 5.17×10^{13}	.60	.001
rfc = 2.62×10^{13} pkrun - 1.37×10^{14} pint + 3.06×10^{13} avint + 7.80×10^{12}	.43	.003
avtc = 1.55×10^{6} avint - 1.78×10^{6} tot + 2.81×10^{6} run + 2.07×10^{3} dur + 4861.94	.55	.001
ptc = 3.94×10^{6} avint + 5.54×10^{3} dur - 4.84×10^{6} tot + 1.78×10^{7} run - 1.51×10^{6} pint + 6.29×10^{4}	.35	.022
ptc = 2.87×10^{6} avint + 7.18×10^{5} pkrun - 3.89×10^{6} pint + 6.13×10^{5}	.33	.012

Table 3.7 continued

tbod = Total mass BOD (lb/storm)
tss = Total mass SS (lb/storm)
ttc = Total number Total Coliforms (#/storm)
tfc = Total number Fecal Coliforms (#/storm)

rbod = Peak rate BOD (lb/day)
rss = Peak rate SS (lb/day)
rtc = Peak rate Total Coliforms (#/day)
rfc = Peak rate Fecal Coliforms (#/day)

pint = peak intensity of rainfall (in/hr)
avint = average intensity of rainfall (in/hr)
tot = total rainfall (in)
pkrun = peak rate of runoff (MGD)
run = total runoff (MG)
dur = duration of precipitation event (hr)
avint = average intensity of rainfall
tot = total rainfall

avtc = Average concentration of Total Coliforms (#/100 ml)
ptc = Peak concentration of Total Coliforms (#/100 ml)

A reasonable regression model (adjusted r^2 = 0.55) was found for the Average Concentration of Total Coliforms (AVTC). A relatively good correlation (r = 0.65) was found between that parameter and average rainfall intensity. The best model for AVTC incorporates average intensity, total rainfall, duration, and total runoff.

Peak Concentration of Total Coliforms (PTC) has the highest correlation with average intensity (r = 0.65). For this pollutant, the best model includes average intensity, duration, total runoff, and peak intensity. A second model for PTC, with a slightly lower adjusted r^2, includes only peak intensity, average intensity, and total runoff.

3.6 Previous Rainfall and Antecedent Dry Period Analysis

To evaluate properly the effects of previous storms and antecedent dry period on stormwater runoff quality, information on street cleaning activities was necessary. Street cleaning records were available from September, 1982, through May, 1983. This allowed information on antecedent dry period to be obtained for 20 storms. Suspended solids data were available for all 20 storms, BOD data for 19 storms, and total and fecal coliforms for 17 storms.

Previous rainfall and antecedent dry period were measured in several different ways. The different methods of measurement were: 1) the total amount of rainfall occurring during the preceding five days; 2) the five-day antecedent moisture categories (see Table 3.8) of the ILLUDAS Model (Terstriep and Stall, 1974); 3) the total amount of rainfall occurring during the previous storm activity; and 4) the number of days since the prior rainstorm.

The information generated by each method of measurement was used as the independent variable in linear regression analyses with water quality pollutants. The correlation coefficients (r values) and significance values (α) which resulted from these analyses are presented in Table 3.9. An inspection of Table 3.9 reveals no significant relationships between the water quality pollutants and either total rainfall occurring during the preceding five days, antecedent moisture categories, or number of days since prior storm activity.

Only a few significant relationships ($\alpha \leq 0.10$) were found between the total rainfall occurring during the previous storm and water quality parameters (such as, average concentration of SS, total number of fecal coliforms, and peak rate of total and fecal coliforms). Therefore, it was concluded that there is no relationship between that antecedent parameter and the water quality pollutants.

3.7 Street Cleaning Analysis

Street cleaning records from September, 1982, through May, 1983, indicated streets were cleaned six times in September, three times in October, and once in November, February, and May. Only five monitored storms had street cleaning, not prior rainfall, as the antecedent activity.

Samples from the five storms were analyzed for BOD and SS concentrations. Only four of the five storms were analyzed for coliform content. The correlations between days since street cleaning and water quality, and level of

Table 3.8

ILLUDAS 5 Day Antecedent Moisture Categories

Category	Rainfall Inches During Prior 5 Days
1 Bone Dry	0
2 Rather Dry	0 - 0.5
3 Rather Wet	0.5 - 1.0
4 Saturated	1.0 or greater

Table 3.9

Correlation Matrix of Antecedent Conditions and Water Quality (r values)

	Five	α	cat	α	one	α	last	α
Total mass BOD	.01	.970	.18	.458	.18	.448	-.15	.538
Total mass SS	.11	.654	.20	.388	.34	.138	-.10	.708
Total number Total Coliforms	.14	.586	.25	.324	.59	.012	-.11	.662
Total number Fecal Coliforms	.05	.850	.16	.540	.32	.204	-.10	.612
Peak rate BOD	.16	.516	.27	.270	.35	.142	-.16	.506
Peak rate SS	.08	.750	.17	.476	.09	.712	-.09	.714
Peak rate Total Coliforms	.17	.514	.27	.292	.63	.006	-.10	.714
Peak rate Fecal Coliforms	.08	.762	.18	.498	.45	.072	-.04	.886
Average concentration BOD	-.13	.590	-.10	.692	-.31	.196	-.11	.660
Average concentration SS	.14	.556	.06	.804	-.52	.018	-.01	.952
Average concentration Total Coliforms	-.13	.608	-.04	.864	.03	.908	-.09	.772
Average concentration Fecal Coliforms	-.13	.608	-.09	.728	-.18	.498	.02	.940
Peak concentration BOD	-.15	.546	-.13	.586	-.07	.762	.10	.690
Peak concentration SS	.03	.884	-.23	.322	-.19	.420	.39	.120
Peak concentration Total Coliforms	-.08	.760	.02	.930	.29	.378	-.11	.662
Peak concentration Fecal Coliforms	-.13	.610	-.12	.652	.06	.824	-.03	.916

Five = total rainfall occurring during preceding five days
cat = five day antecedent moisture category used by ILLUDAS
one = total rainfall during the last storm activity
last = number of days since last rainfall
α = significance level.

significance of those relationships, are presented in Table 3.10. The high significance values ($\alpha > 0.30$) suggest that chance is as likely as natural phenomena to have produced the degree of correlation indicated.

The storm of October 31, 1982, was much higher in peak and total runoff than the other four storms. As it is possible that these high hydrologic values might influence water quality, and mask the effect of days since street cleaning, regression relationships were re-evaluated without this storm (Table 3.11). This second set of regression relationships is specific to storms with total runoff less than 0.055 MGD, and peak runoff less than 1.50 MG, and was formulated with only four storms with BOD and SS data, and three storms with total and fecal coliform data.

The second evaluation found significant relationships ($\alpha \leq 0.10$) between days since street cleaning and only a few water quality parameters (total number, peak rate, average concentration, and peak concentration of total coliforms, and peak concentration of fecal coliforms, and peak concentration of BOD). Therefore, it was concluded that no overall trend exists between street cleaning and water quality pollutants.

3.8 Seasonal Analysis

To compare seasonal effects properly, either of two situations would be acceptable. The ideal situation would be to have available water quality data from many storms in each of the four seasons. Unfortunately, the number of summer and winter storms analyzed in this investigation was meager.

The second acceptable situation would be to eliminate hydrologic bias artificially. This would require finding high, medium, and low categories for the various hydrologic activities, separating storms into these categories, and testing within each category for seasonal differences. Total rainfall was the only hydrologic activity which had been categorized for this watershed. When the storms were divided into the peak, moderate, and trace total rainfall groups, it became apparent that no category had runoff quality data for all four seasons.

The only available method to test the water quality data for seasonal differences required using all the data broken down into the four seasonal categories. A test of significance was used to check for seasonal differences. Where only one seasonal value was available, the standard deviation was estimated from an average coefficient of variation determined for that pollutant measurement group. The results of seasonal testing indicated that, in general, concentrations, total mass (number), and peak rates of all pollutants appear to be the greatest in the Fall and the least in the Winter.

3.9 Conclusions

1. The total mass of BOD, in the stormwater runoff from the West Lafayette, Indiana, watershed investigated in this study, ranged from 0.3 to 110 lb/storm; and the peak rate ranged from 21 to 3220 lb/day. The average concentration of BOD ranged from 2 to 61 mg/l; and the peak concentration ranged from 6 to 136 mg/l.

Table 3.10

Correlation Between Days Since Street Sweeping and Water Quality
(Five Storms)

Water Quality Parameter	Correlation Coefficient (r)	Significance (α)
Total mass BOD	-.16	.796
Total mass SS	-.05	.938
Total number Total Coliforms	-.03	.972
Total number Fecal Coliforms	-.45	.552
Peak rate BOD	-.11	.860
Peak rate SS	-.12	.846
Peak rate Total Coliforms	-.07	.934
Peak rate Fecal Coliforms	-.44	.562
Average concentration BOD	.51	.382
Average concentration SS	.11	.852
Average concentration Total Coliforms	.58	.424
Average concentration Fecal Coliforms	-.08	.918
Peak concentration BOD	.39	.512
Peak concentration SS	-.09	.890
Peak concentration Total Coliforms	.38	.618
Peak concentration Fecal Coliforms	-.32	.684

Table 3.11

Correlation Between Days Since Street Sweeping and Water Quality (Four Storms)

Water Quality Parameter	Correlation Coefficient (r)	Significance (α)
Total mass BOD	.84	.160
Total mass SS	.73	.266
Total number Total Coliforms	.99	.080
Total number Fecal Coliforms	.27	.826
Peak rate BOD	.85	.152
Peak rate SS	.48	.552
Peak rate Total Coliforms	.99	.090
Peak rate Fecal Coliforms	.19	.880
Average concentration BOD	.42	.576
Average concentration SS	.50	.504
Average concentration Total Coliforms	.99	.040
Average concentration Fecal Coliforms	-.15	.902
Peak concentration BOD	.93	.074
Peak concentration SS	-.16	.836
Peak concentration Total Coliforms	.99	.070
Peak concentration Fecal Coliforms	.04	.978

2. The total mass of SS in the stormwater runoff from the watershed ranged from 1 to 706 lb/storm; and the peak rate ranged from 59 to 27,800 lb/day. The average concentration of SS in the runoff ranged from 12 to 199 mg/l; and the peak concentration ranged from 35 to 1,380 mg/l.

3. The total number of total coliforms in the stormwater runoff ranged from 5.6×10^{9} to 1.0×10^{13}/storm, and the peak rate ranged from 5.0×10^{11} to 6.3×10^{14}/day. The average concentration of total coliforms ranged from 8.8×10^{3} to 1.5×10^{6}/100 ml; and the peak concentration of total coliforms ranged from 2.0×10^{4} to 5.1×10^{6}/100 ml.

4. The total number of fecal coliforms in the stormwater runoff ranged from 4.8×10^{8} to 2.3×10^{12}/storm; and the peak rate ranged from 1.6×10^{10} to 2.3×10^{12}/day. The average concentration of fecal coliforms ranged from 6.7×10^{2} to 9.7×10^{5}/100 ml; and the peak concentration ranged from 5.0×10^{3} to 1.3×10^{6}/100 ml.

5. This watershed generally exhibits a first flush in concentration for all pollutants investigated. Three types of pollutographs are evident: 1) a rise in concentration followed by a decrease, 2) an initial peak concentration followed by a decrease, and 3) more than one peak in concentration. Suspended solids and BOD most often exhibited the second pollutograph pattern, next the first, and last the third. Total and Fecal Coliform most often exhibited the last pollutograph pattern.

6. The stormwater runoff contained, on the average, higher concentrations of BOD, SS, and fecal coliforms than the effluent from a secondary municipal wastewater treatment plant.

7. With 99% confidence, the multiple regression models developed for total mass (or number) and peak rates of all pollutants studied explain at least 30% of the variance in the data. The models, which contain combinations of various hydrologic activities (peak intensity of rainfall, average intensity of rainfall, total rainfall, peak rate or runoff, total runoff, and duration of precipitation event), are listed in Table 3.7.

8. With 97% confidence, the multiple regression relationship developed for average concentration of total coliforms, and with 99% confidence, the regression relationship developed for peak concentration of total coliforms explain at least 30% of the variance in the data. These models use different combinations of the same hydrologic variables used in the first group of regression equations; they are listed in Table 3.7.

9. The hydraulic parameters accounted for less than 25% (adjusted $r^2 < 0.25$) of the variance in the average and peak concentrations of BOD, SS, and fecal coliforms data. Therefore, regression relationships were not generated for these pollutants.

10. There is no apparent relationship between the quantity of previous rainfall or the length of the antecedent dry period and the quality of stormwater runoff.

11. There is no apparent relationship between the time elapsed since street cleaning and the quality of stormwater runoff.

12. In general, concentrations, total mass (or number), and peak rates for all pollutants appear to be greatest in the Fall and least in the Winter.

3.10 References

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CHAPTER 4
HYDROLOGIC STUDIES

The hydrologic investigation deals with two problems. The first of these is the selection of durations of design storms and the second one is the parameter estimation in rainfall-runoff models. These two investigations are separately discussed below.

4.1 Selection of Design Storm Durations

Introduction. The tendency towards the use of more sophisticated rainfall-runoff models in urban storm sewer design requires a detailed study of the parameters used in these models. The variables associated with design rainfall are the parameters that are used for the calculation of runoff from a basin of specific physiographic characteristics. The calculated runoff values are in turn used in sizing the pipes of the storm sewer network of a subdivision or other areas.

The design storm that is normally used for the design of storm drainage networks is defined by the storm duration, the storm frequency and the rainfall intensity. Curves relating storm duration, frequency and rainfall depth are available in the literature. The best known of these curves is the set in the Rainfall Frequency Atlas by Hershfield et al. (1961). In order to define a design storm uniquely, two of the three parameters, duration, frequency and depth must be specified. Due to variations in the rainfall depth at different locations, the storm frequency and duration are normally assumed in specifying the design storm.

As there is little work in the literature about the variation of runoff with storm durations, the storm duration was selected for further study. The storm duration that produces the maximum runoff from a watershed for a given set of watershed conditions is herein called the critical duration. It has been shown by Burke et al. (1981) that the critical duration is affected by the antecedent moisture condition, basin imperviousness and travel times. Wenzel and Voorhees (1978) have remarked that the critical duration for the design storm should be selected on the basis of model results and not arbitrarily.

Although the importance of the design storm duration on the peak and volume runoff from a watershed is very well known, very little is known about the effects of basin parameters on the critical duration. The first objective of the research reported in this paper is to investigate the effect of various parameters on the critical duration. In order to generalize the results, data from three urban watersheds have been analyzed and the results are compared.

The method used in the investigation of critical storm duration is as follows. The Illinois urban drainage area simulator, ILLUDAS (Terstriep and Stall, 1976), is the rainfall-runoff model used to study the effect of various physiographic variables on the critical duration. Each of the variables in ILLUDAS is systematically varied and the resulting changes in the critical duration are studied to isolate significant variables and trends. Various changes had to be made to ILLUDAS to obtain the critical duration for each set of watershed parameters efficiently. None of these changes affected the basic model but only changed the way in which ILLUDAS is used. These changes are discussed in detail by Lemmer and Rao (1983).

Rainfall-Runoff Variables. The Rainfall and Runoff variables that are analyzed in this study are defined with particular reference to ILLUDAS and are discussed in that context. Also, the results obtained from this study are based only on the analyses carried out with ILLUDAS. Similar studies must be conducted by using other urban rainfall-runoff models to confirm the trends observed in the present study. A summary of the Rainfall and Runoff variables that are investigated as well as the results obtained from the study are given in Table 4.1.

The percentage imperviousness, paved entry time and storm distribution have the most significant influence on the critical duration. The variation of critical duration with these parameters is discussed in greater detail in a later section. The other variables that are studied but yielded no significant influence on the critical duration are the total basin area, the pipe slope, length and roughness, grassed area entry time and the rainfall frequency. The rainfall depth or intensity is related to the storm duration. Consequently they are fixed by the critical duration and return period of the storm and can therefore not be varied independently. The soil type and antecedent moisture condition, which are the other physiographic variables that influence the runoff from a watershed were not studied because of the limited variation built into ILLUDAS for these variables.

Before using them in the analysis, the rainfall depth values given by TP 40, TP 25 and the mathematical relationship proposed by Fair et al. (1966) were compared. The result of this comparison is given in Figure 4.1 for the Mount Washington watershed in Cincinnati, Ohio. The rainfall depths for the same duration and frequency varies considerably depending on the source of data. It can be seen that the relationship between rainfall depth, duration and frequency is best represented for the data used in this study by the mathematical relationship proposed by Fair et al. (1966) and is given by equation 4.1. The input structure of ILLUDAS was changed to accommodate the relationship given in equation 4.1. A mathematical relationship among rainfall depth, durations and frequencies was also preferred since it can easily be evaluated at intermediate points and lends itself to inclusion in a computer program.

$$D = \frac{ctT^m}{60(t + d)^n} \tag{4.1}$$

In equation 4.1, D is the rainfall depth in inches, t is the storm duration in minutes, T is the recurrence interval of the storm in years and c, d, m, and n are coefficients that must be determined at each basin location. The coefficients for each of the watersheds used in the analysis are given in Table 4.2. The procedure that was used to develop these regional coefficients is given in Lemmer and Rao (1983). The storm duration was varied between 5 and 360 minutes and the recurrence interval of the storm between 1 and 25 years in the determination of the coefficients given in Table 4.2.

TABLE 4.1. Summary of Results of the Critical Duration Analysis

Parameter	Effect on Critical Duration	Comments
Total basin area	No significant effect	Effect better represented by parameters containing an element of time.
Percentage Imperviousness	A significant effect	The source of this effect is the layout of the storm sewer system.
Travel Times		
Pipe slope and length	Small effect	This effect can be contained within a band of 5 minutes.
Pipe roughness	Small effect	
Paved area	Very significant	Effect is also dependent on the percentage imperviousness of the basin.
Grassed area entry time	No effect	
Rainfall frequency	Small effect	Critical duration values lies within a 5-minute band.
Storm distribution	Significant effect	A particular set of Huff curves give a maximum runoff rate for one of the quartiles.

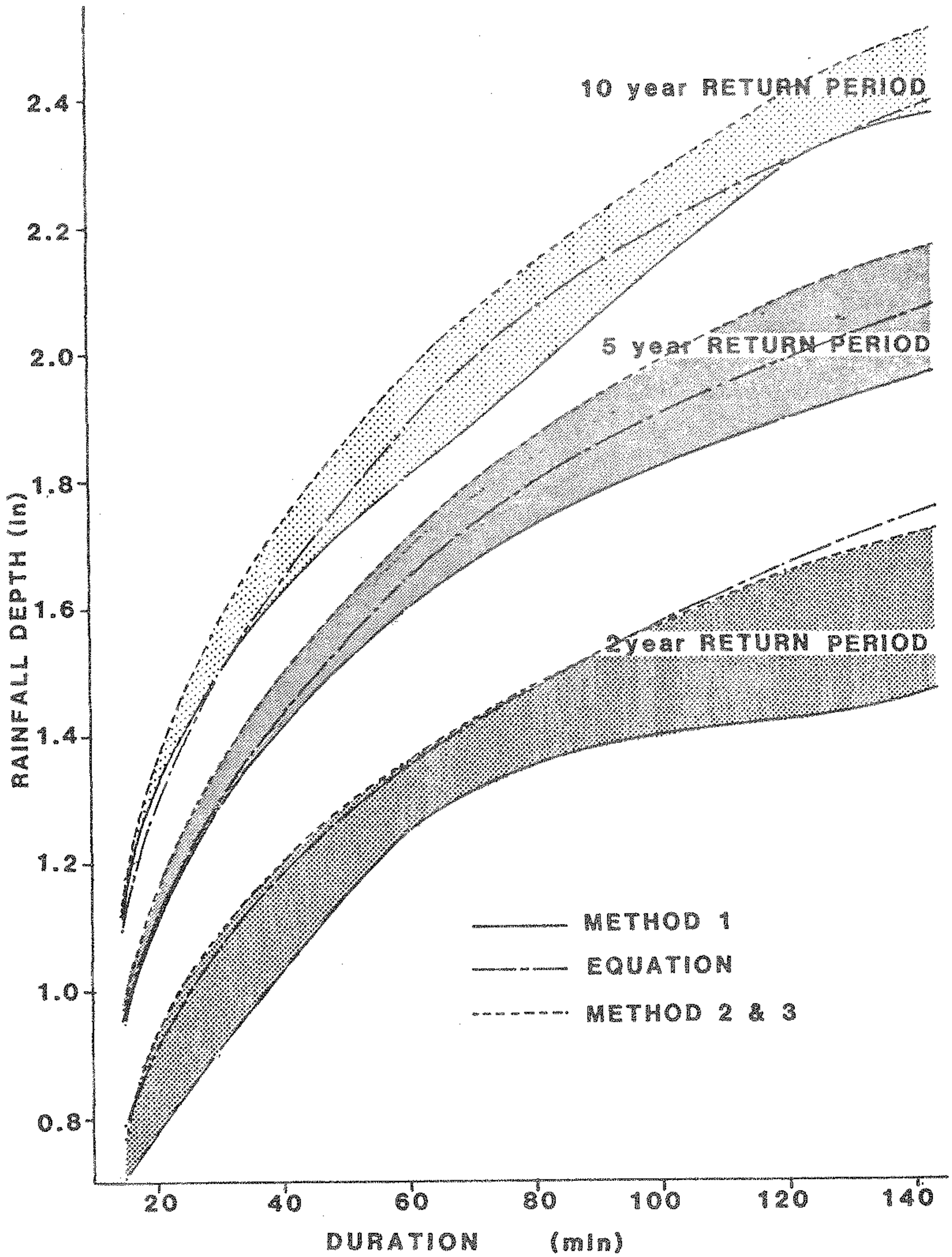

FIGURE 4.1. Rainfall Depths for Mount Washington Watershed Obtained by Various Methods.

TABLE 4.2. Summary of Regional Coefficients

Basin	c	m	d	n
Louisville	35.66	0.202	8.6	0.791
Cincinnati	32.11	0.204	8.1	0.783
West Lafayette	32.30	0.189	7.7	0.775

Some definition of the variables which have a significant influence on the critical duration is appropriate at this point. The percentage imperviousness of a basin, which is a variable significantly affecting the critical duration is defined in equation 4.2.

$$\text{Percentage Imperviousness} = (1-\text{Total unpaved area}/\text{Total basin area})\times 100 \tag{4.2}$$

As the total paved area of the basin increases, the percentage imperviousness will also increase. The second watershed parameter that shows a significant influence on the critical duration is the paved entry time. The paved entry time is defined in ILLUDAS as the time taken by the runoff to reach the outlet point from the most hydraulically distant point in the watershed. A uniform basin area is assumed in the calculation of the time-area curve in ILLUDAS. The validity of this assumption has been discussed by Burke et al. (1981).

Data Collection and Investigation Procedure. The following watersheds are selected for analysis in this study: i) First Street Watershed in Louisville, Kentucky, ii) Mount Washington Watershed in Cincinnati, Ohio, and iii) Bar Berry Heights Watershed in West Lafayette, Indiana. The data for the first two watersheds were obtained from Mr. M.L. Terstriep of the Illinois State Water Survey. These data were also used in the verification of ILLUDAS (Terstriep and Stall, 1974). The data for the third watershed was obtained from Mr. C.B. Burke of Purdue University and was used in a previous study (Burke et al. 1981). The relevant basin data are summarized in Table 4.3.

The temporal storm distribution at a given location is also needed in the analysis of the effect of storm distribution on the critical duration. The temporal storm distribution at a particular location is estimated by using a statistical procedure proposed by Huff (1967) if the necessary rainfall data are available.

The Chicago method proposed by Keifer and Chu (1957) is a good alternative to generate hyetographs at locations where rainfall data at a suitable time step or in sufficient quantity are not recorded. The values for the cumulative percent of precipitation and the cumulative percent of storm time of each quartile as defined by Huff have been obtained for central Illinois

TABLE 4.3. Summary of Data for Basins Used in the Present Study

Input	Basin: Louisville	Cincinnati	West Lafayette
Total Area (acres)	70.6	30.7	122.0
Paved Abstraction (in)	0.1	0.1	0.1
Grassed Abstraction (in)	0.2	0.2	0.2
Minimum Pipe diam. (in)	6.0	8.0	12.0
Mannings 'n'	0.014	0.014	0.013
Paved Area (acres)	58.74	16.24	36.6
Grassed Area (acres)	11.94	13.89	85.28
Percentage Imperviousness	83.09	54.8	30.0
Average Paved Time (min)	9.78	6.86	5.27
Average Pipe Length (ft)	329.0	151.36	296.23
Average Pipe Slope	0.48	1.72	0.59
Average Grass Flowpath Length (ft)	30.0	92.19	156.32
Average Grass Slope	2.0	6.5	2.0
Soil Group (ILLUDAS classification)	2.0	4.0	2.0
Time Increment (min)	1.0	1.0	1.0
Routing Option	Time Lag	Time Lag	Time Lag

from Huff (1967) and for West Lafayette, IN, from Rao and Chenchyya (1974). The default storm distribution that is built into ILLUDAS is the first quartile storm for central Illinois. This storm distribution is used throughout the study except in the case where the effect of storm distribution on critical duration is analyzed.

The approach adopted in studying the effect of the variables listed in Table 4.1 on critical duration is the usual method used in parametric sensitivity analysis. All the variables, other than the one presently being studied, are held at reasonable constant values while a single variable is systematically varied. The values of the variable under study are multiplied by a scale factor to keep the proportions between the values of the variable constant. Although this may sometimes lead to physically contradictory situations, the effect of a single variable on critical duration can be best isolated by this method than by following any other approach. During the present study ILLUDAS was used only in the design mode and no storage or flow restrictions were included in the analysis.

Variation of Critical Duration with Paved Area Entry Time, Percentage Imperviousness and Temporal Storm Distribution. The three physiographic variables which have the greatest influence on critical duration are the paved area entry time, the percentage imperviousness of the basin and the temporal storm distribution. The plots showing these relationships are shown in Figs. 4.2, 4.3, 4.4, and 4.5.

Paved Area Entry Time. The paved area entry time has the most significant influence on the critical duration as seen from Figure 4.2. As the percentage imperviousness of the basin increases, there is a resultant increase in the influence of the paved area entry time on the critical duration.

The maximum and minimum critical durations for each of the basins and the percentage imperviousness associated with that basin are given in Table 4.4. For the same basin, the critical duration value may be as low as about 20 mins. or as high as 1 hr. depending on the paved entry time values used in the study.

TABLE 4.4. Comparison of Critical Duration and Percentage Imperviousness for Changes in the Paved Entry Time

Basin Name	Imperviousness (%)	Critical Duration Maximum (min)	Critical Duration Minimum (min)
First Street	83.11	57	19
Mount Washington	52.90	36	16
Bar Barry Heights	30.20	31	17

The large influence of the paved entry time on critical duration can be attributed to the large runoff volume that originates from the paved areas. The way in which the hydrograph peaks from different subareas arrive at the outlet determines the runoff rate at the outlet for a given set of conditions. The paved area entry time also determines the time at which the hydrograph peak occurs. Changing the paved entry time by a constant factor therefore moves all the paved area runoff hydrograph peaks by the same amount. Only the grassed area runoff hydrographs reduce this effect.

As the percentage imperviousness of the basin increases so does the difference in the amount of runoff between the paved and grassed areas. This in turn increases the influence of the paved entry time on the critical duration. The paved entry time should therefore be studied very carefully in

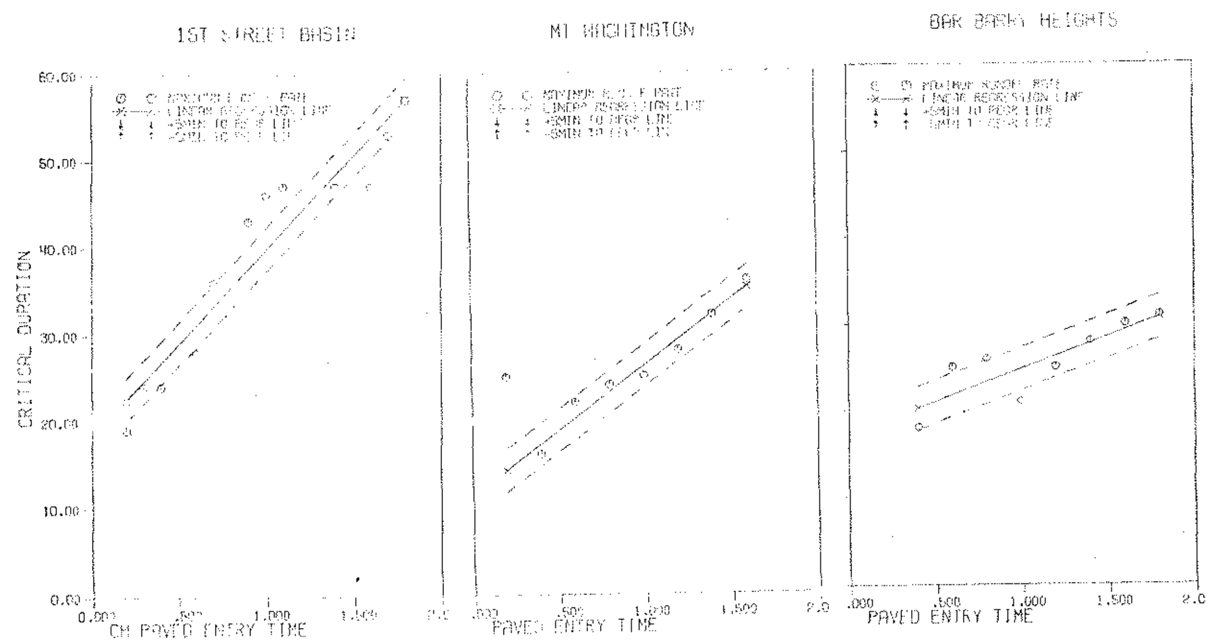

FIGURE 4.2. **Changing the Paved Area Entry Time.**

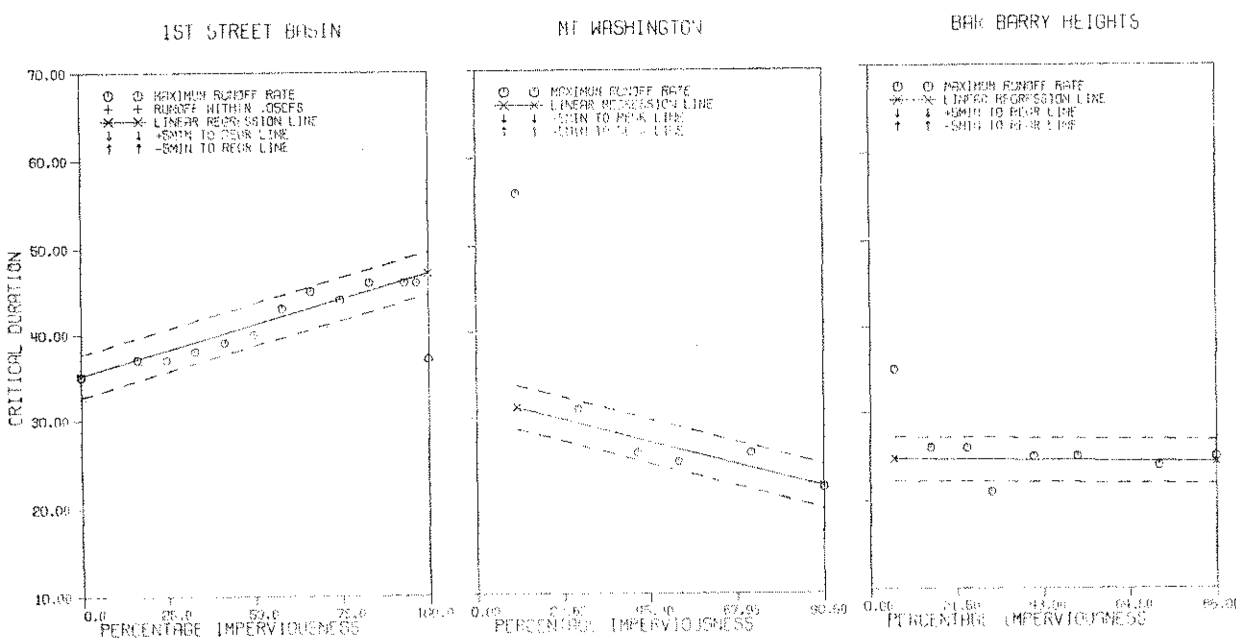

FIGURE 4.3. Changing the Percentage Imperviousness.

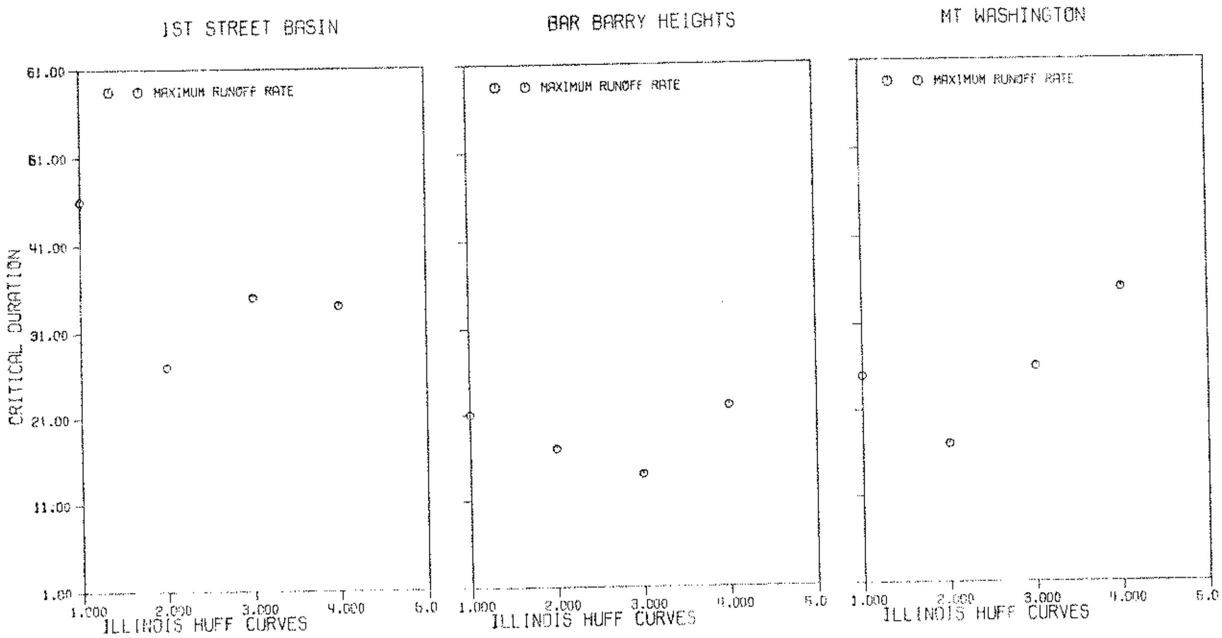

FIGURE 4.4. Changing Storm Distribution According to Illinois Huff Curves.

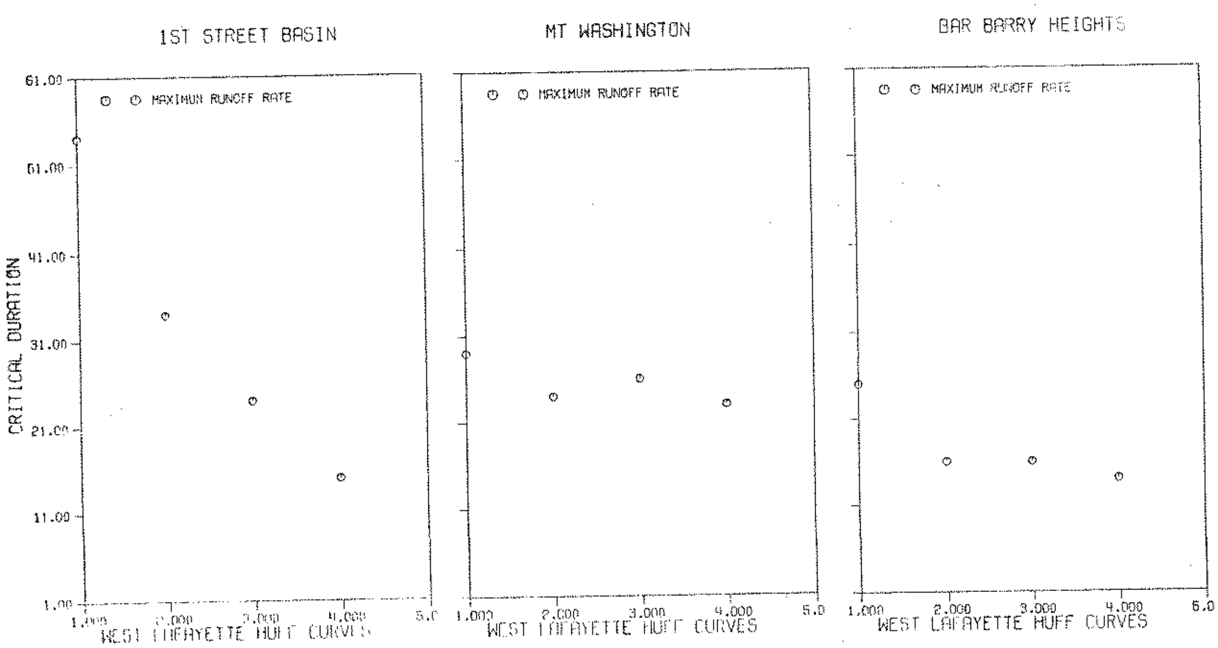

FIGURE 4.5. Changing Storm Distribution According to West Lafayette Huff Curves.

order to obtain a value for the critical duration especially for highly impervious basins.

Percentage Imperviousness. The total basin area was held constant during the analysis of the effect of percentage imperviousness on critical duration by multiplying the contributing paved area and the supplementary paved areas by a selected paved area factor. The contributing grassed area was then multiplied by the grassed area factor calculated from equation 4.3. These changed areas were then used to get a new value for the critical duration of the basin.

$$\text{Grass area factor} = 1 \times 2 \text{ paved area factor} \qquad (4.3)$$

The change in the slopes of the regression lines fitted through the points for each of the watersheds in Figure 4.3 were analyzed. It was hypothesized that the changes in these slopes could be accounted for by the layout of the sewer network of that watershed. To test this hypothesis a pipe network similar to that of First Street basin was assumed for the Bar Barry Heights and the analysis was repeated. The result is that the slope of the regression line for the Bar Barry Heights Watershed changed from zero to a positive slope as in the case for First Street basin. This change is shown in Figure 4.6. The result in Figure 4.6 should be compared to that in Figure 4.3. However, the interaction between the storm sewer network and critical duration is very complex and changes with changes in pipe layout. It was not possible to derive general relationships about this aspect.

Temporal Storm Distribution. All three basins used in this analysis are tested with the four quartile Huff curves from both central Illinois and West Lafayette, Indiana. Two approximations made in this analysis that could introduce some errors in the results are that the numerical values of the Huff curves had to be read from the published curves and the procedure in ILLUDAS that calculates the storm distribution is set up to place the emphasis on the first quartile rainfall values. This procedure could not be changed easily and hence was retained.

The results did not give a clear indication of the quartile which should be selected for determining the critical duration and also the maximum runoff. Results given in Table 4.5 clearly show that for each of the basins the same quartile of a set of Huff curves produces the maximum runoff.

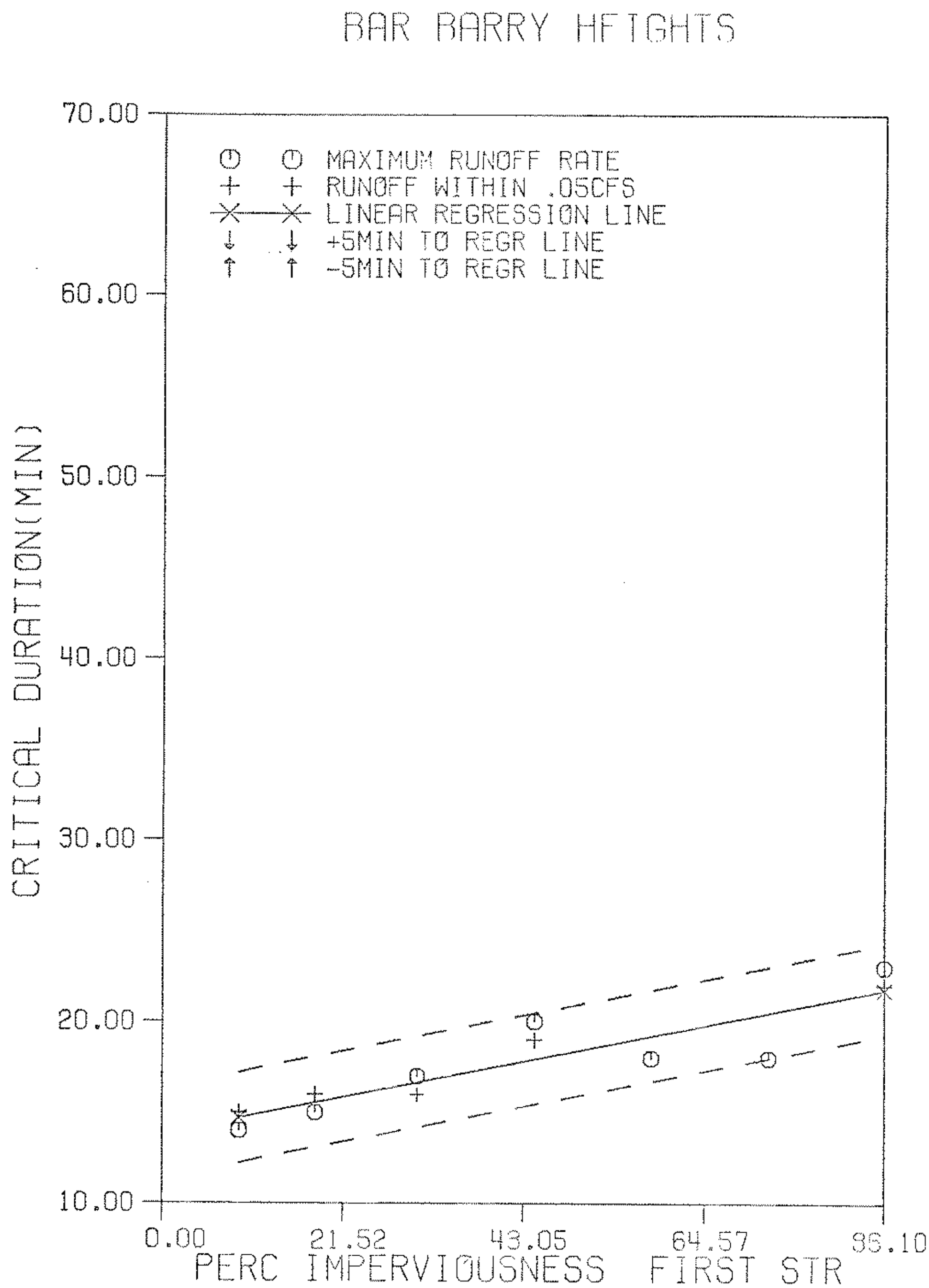

FIGURE 4.6. Changing the Percentage Basin Imperviousness for Bar Barry Heights Basin with the Pipe Network Equivalent to that of First Street Basin.

TABLE 4.5. Maximum Runoff Rate at Critical Duration for Different Huff Storm Distributions.

Basin Name	Maximum Runoff Rate [cfs] Illinois Huff Distribution				West Lafayette Huff			
	1	2	3	4	1	2	3	4
First Street	247	286	295	316	305	297	252	236
Mount Washington	54	61	66	71	67	62	56	50
Bar Barry Heights	163	180	193	209	231	183	149	121

Discussion and Conclusions. The above results suggest the following procedure for obtaining the best value of the critical duration of a watershed. The first step will be to determine the Huff quartile storm that yields the maximum value for the runoff from the watershed. The next step will be to use this quartile storm to calculate the critical value of the paved entry area of the basin. The best estimate of the paved area entry time should be obtained by simulation since this parameter has the most significant influence on the value of the critical duration. Finally by using the pipe distribution network which has been suggested for the basin the effect of the percentage imperviousness on the critical duration should be investigated. The results from this analysis suggest the critical duration which must be used for that watershed.

Strategies for alleviating flooding problems in urban areas can also be developed by using the results presented in this study. The sensitivity of the basin to the percentage impervious and paved entry times can be studied to see if increasing the pervious area or decreasing slopes or increasing flow path lengths can change the critical duration. Using the correct duration for the design storm will also produce increased runoff rates from the rainfall-runoff models. This increase in the runoff rate will lead to estimation of better pipe sizes. Since the increased pipe sizes are based on better estimates of runoff the design will be optimal instead of just being conservative.

4.2 Parameter Estimation in Urban Rainfall-Runoff Models

Introduction. The procedure that is used to obtain the optimal parameter estimates in a rainfall- runoff model is quite important. However, this aspect has not been given sufficient attention in deterministic rainfall-runoff models. Appropriate objective functions and estimation procedures will enable accurate parameter estimation and estimation of runoff. Consequently, objective function and optimization methods are being tested for use with deterministic rainfall-runoff models. For example, in a study of the objective functions of urban rainfall-runoff models Han and Rao (1980) concluded that the sum of the squared differences between the observed and the calculated runoff hydrographs gives the best estimate of the basin parameters that are being optimized.

The procedure adopted by Han and Rao (1980) to estimate the parameters optimally is used in this study also. This procedure, called OPTIL, is compared with another parameter estimation method developed by Sorooshian (1981). OPTIL can briefly be summarized as follows. The measured rainfall is read into the rainfall- runoff model together with parameter values which are initially guessed. The runoff values computed by the model are compared to measured runoff values. These parameter values are updated by using an optimization procedure and least squares objective function. The optimization procedure is designed to select new values for the parameters and the process is continued until the objective function values do not change more than a selected tolerance value..

Sorooshian (1981) has developed a two stage optimization method to estimate the parameters of deterministic rainfall-runoff models. In this method the parameters are optimized by using a maximum likelihood estimator which contains a weighting factor. This weighting factor is automatically estimated by means of a Bayesian formulation for the non-informative data case. Sorooshian tested this procedure on a four parameter model and compared the results to those obtained by using a simple lease squares criterion and the weighted objective function of HEC 1 of the U.S. Army Corps of Engineers (USACE, 1973). The models were evaluated by using synthetically generated data.

In the present study, the two stage procedure proposed by Sorooshian is applied to the urban rainfall-runoff model ILLUDAS (Terstriep et al., 1974). The parameter estimates obtained from this procedure are compared to the parameters estimated by using simple least squares criterion function as implemented by Han and Rao (1980) in the program OPTIL which is discussed later.

The objective of the research discussed in this section is to verify the claim by Sorooshian (1981) that the procedure developed by him gives better parameter estimates than methods based on the simple least squares criterion. In the following discussion, Sorooshian's method is called the MLE. The method based on OPTIL is called the SLS procedure. The procedure that is used, its implementation and the results from the study are presented below.

Data Used in the Study and the Parameters Estimated - Introduction. The procedure that is used in the analysis of the parameter estimation methods is to compute the best parameter estimates obtained from both the MLE and SLS procedures. These parameter estimates are first examined for consistency. The parameters are used to estimate the runoff. The correspondence between the computed and observed runoff hydrographs is also used to arrive at conclusions about the parameter estimates.

The data, parameters selected for optimization and the routines that are used in the analysis are discussed here. The implementation of the procedure is discussed in a subsequent section.

Parameters Which Are Optimized. Although four parameters are optimized in the model tested by Sorooshian (1981), only two parameters are optimized in the present study due to cost and computational time considerations. The two parameters in ILLUDAS that are optimized, the bounds within which they are allowed to vary and their initial values are given in Table 4.6.

TABLE 4.6. ILLUDAS Parameters that are Optimized and Their Values

Parameter	Variable Name	Parameter Values maximum	initial	minimum
Paved Abstraction	ABSTRT	0.2	0.1	0.05
Grassed Abstraction	DEPG	0.5	0.2	0.1

The grassed (DEPG) and paved (ABSTRT) area abstractions are selected for estimation because it is difficult to obtain reasonable values for them by measurement. It is also easier to visualize the storage effect implied in ABSTRT and DEPG than to visualize the effect of the infiltration coefficients which are the other parameters optimized by OPTIL (Han and Rao, 1980).

The maximum and minimum values for the parameters given in Table 4.6 for the abstractions are rough estimates taken from published values. The initial values used in the optimization of the parameters are recommended by Terstriep et al. (1974) for use with ILLUDAS.

Drainage Basin and Rainfall-Runoff Data Selected. Data from the Mount Washington watershed in Cincinnati, Ohio, are used in the analysis of the parameter estimation methods. This basin is selected from several which were available because it has a fairly complex storm sewer network. It has also been verified previously by Terstriep et al. (1974) and a good set of rainfall-runoff data is available for the basin. Several storms on Mount Washington watershed for which the runoff data are also available are used in the study.

Methods Used in the Study. The objective functions and parameter estimation procedures that are used to obtain optimal parameter estimates are now discussed. The simple least squares objective function is discussed first since it has already been implemented in OPTIL. The two step optimization procedure by Sorooshian (1981) is discussed next. The Box Complex (Box, 1965) optimization procedure is discussed last.

OPTIL with Least Squares Criterion. A modified version of the program OPTIL developed by Han and Rao (1980) is used in this analysis. The major change that is made to this program is to replace Rosenbrock's optimization routine (Rosenbrock, 1960; Kite, 1965) with the Box Complex optimization routine (Box, 1965) as developed by Gabriele et al. (1979). This change was necessary because it was found that the Box Complex method converges more rapidly and with fewer evaluations of ILLUDAS than Rosenbrock's method.

The alternations made to ILLUDAS by Han and Rao (1980) to evaluate the simple least squares criterion were retained. A listing of the final program with the changes made to implement the MLE of Sorooshian is found in Lemmer and Rao (1983).

<u>Two Stage Optimization Using a Maximum Likelihood Estimator.</u> The two stage procedure developed by Sorooshian (1981) to estimate the parameters in rainfall-runoff model was adapted for use with ILLUDAS. A flowchart of the procedure is given in Figure 4.7. The procedure is described in more detail below and the expressions to evaluate the objective function and for obtaining new values of the parameters $\hat{\sigma}$ and λ are also given.

The version of ILLUDAS used in OPTIL with the least squares criterion is used to calculate the runoff which is required in the maximum likelihood function. The maximum likelihood function is the objective function included in ILLUDAS for this two stage optimization procedure. Sorooshian's (1981) procedure can be summarized as follows:

a. The parameters σ and λ required in equation 4.4 given below are set equal to unity for the first evaluation of the maximum likelihood estimator.

b. The Box Complex optimization procedure is used to maximize the maximum likelihood function $L(\underline{\Theta}, \sigma, \lambda)$

$$L(\Theta,\sigma,\lambda) = \frac{\left(\sum_{t=1}^{n} w_t\right)^{1/2}}{(2\pi\sigma^2)^{n/2}\, e^{\left(-1/2\sigma^{-2} \sum_{n=1}^{n} w_t (q_{t.mes} - q_{t.com})^2\right)}} \tag{4.4}$$

$$\text{where, } w_t = q_{t.com}^{2(\lambda-1)}, \tag{4.5}$$

Θ : the vector of parameters to be estimated, σ : the standard deviation of the errors, λ : a weighting factor, $q_{t.mes}$: the measured runoff at time t, $q_{t.com}$: the computed runoff at time t.

The value for w_t was obtained from a Box and Cox (1964) transformation to obtain a set of flow values, $q_t(\lambda)$, with a constant variance σ^2 where λ is the unknown transformation parameter.

Sorooshian later recommended that better results may be obtained by maximizing the natural logarithm of equation 4.4. The logarithm of $L(\Theta, \sigma, \lambda)$ is given in equation 4.6

$$L(\Theta,\sigma,\lambda) = 0.5 \ln \left(\sum_{t=1}^{n} w_t\right) - (n/2) \ln (2\pi\sigma^2) - (0.5/\sigma^2)$$

$$\sum_{t=1}^{n} w_t (q_{t.mes} - q_{t.com})^2 \tag{4.6}$$

c. After obtaining the optimal parameters estimated by using equation 4.6 a new value of λ is selected. A simple peak seeking routine is used to

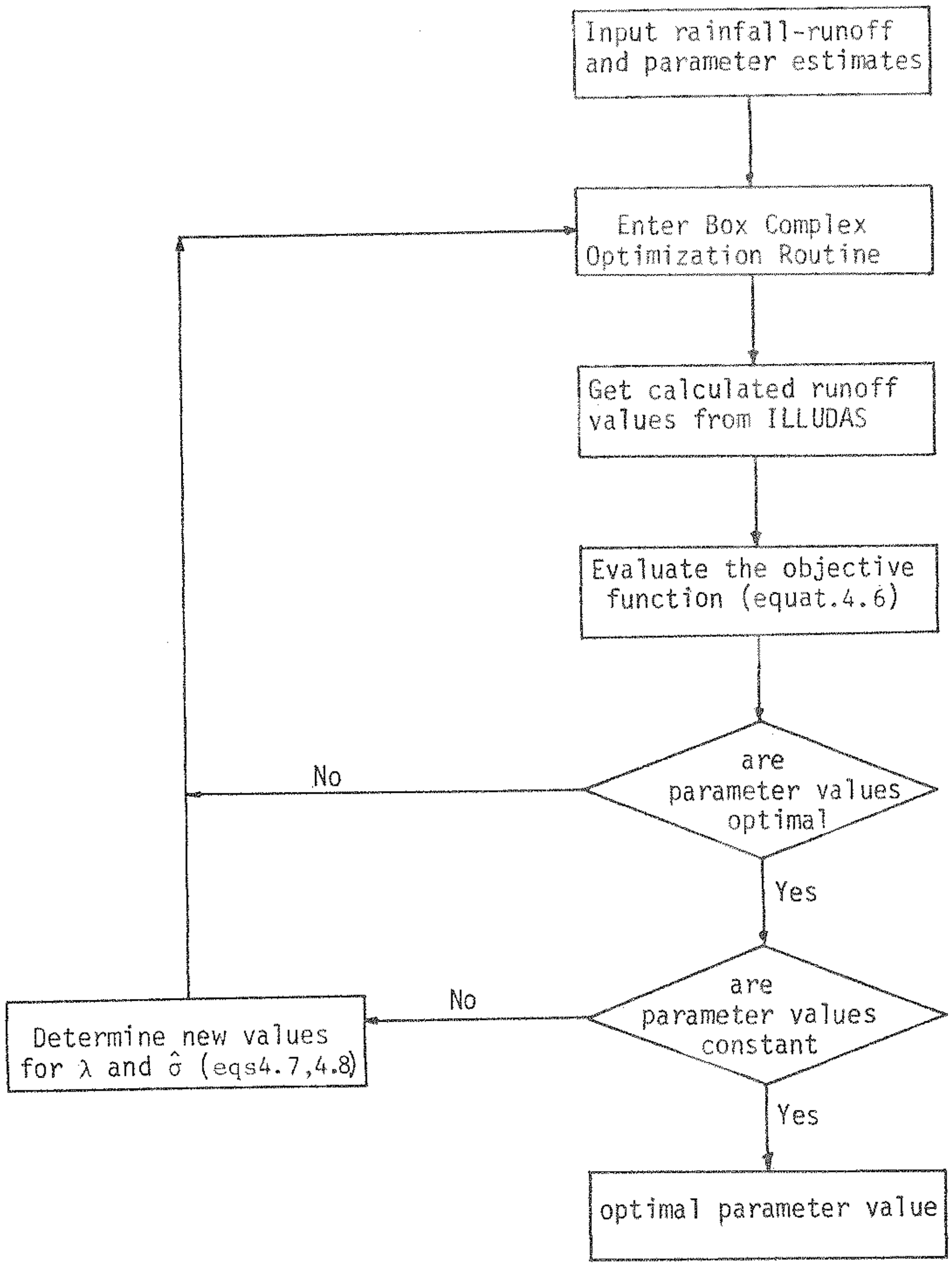

FIGURE 4.7. Flowchart of Sorooshian's Two Step Optimization Procedure.

solve equation 4.7 by trial and error.

$$\sum_{t=1}^{n} \ln(q_{t.com}) - \frac{n \sum_{t=1}^{n} w_t \ln(q_{t.com})(q_{t.mes} - q_{t.com})^2}{\sum_{t=1}^{n} w_t (q_{t.mes} - q_{t.com})^2} = 0 \tag{4.7}$$

After obtaining optimal estimates for λ the value of σ is obtained from equation 4.8,

$$\sigma^2 = \frac{1}{n} \sum_{t=1}^{n} (e_t - \frac{1}{n} \sum_{t=1}^{n} e_t)^2, \tag{4.8}$$

$$\text{where } e_t = q_{t.mes}^{2(\lambda-1)} - q_{t.com}^{2(\lambda-1)} \tag{4.9}$$

d. Steps (b) and (c) are repeated until the difference between two successive values of λ are within a prescribed tolerance or the changes in the parameter values are within preselected limits.

Box-Complex Method of Constrained Minimization. The decision to use the Box-Complex method in the present study rather than Rosenbrock's method is based on the conclusion arrived at by Box (1965) that it is a better procedure for solving constrained problems than Rosenbrock's procedure. The simplex method of Nelder and Mead (1965) is suggested by Sorooshian et al. (1981) for estimating the parameters as an alternative to the pattern search technique which was originally used by them.

The Box Complex optimization method as implemented by Gabriele et al. (1979) is used to find the optimal estimates of grasses and paved area abstractions in ILLUDAS. The SLS and MLE procedures are both used to determine the optimal parameter estimates.

Implementation. All the procedures discussed above are brought together in a program called 'ILLBOX' that is used to obtain optimal parameter estimates. The procedure that is adopted to compare the results from the two different parameter estimation procedures by means of the program ILLBOX is first to estimate the parameters by using the simple least squares objective function. The objective function is then changed to the MLE function and the parameters are refined in a stepwise fashion. λ was determined in order to get a better understanding of the effect that the weighting factor has on the values of the parameters. The results obtained from this analysis are presented and discussed next.

Results and Discussion - Introduction. During the development of the program 'ILLBOX', the parameters which were estimated by using the SLS objective function and the MLE objective function with a weighting factor of 1 were compared. The parameter values obtained from these two objective functions must be the same since maximization of equation 4.6 with $\lambda = 1$ and $\sigma = 1$ leads to the maximization of the function given below:

$$- \sum_{t=1}^{n} w_t(q_{t.mes} - q_{t.com})^2 \tag{4.10}$$

and from equation 4.5 $w_t = 1$ for all flows. This assumption of $\lambda = 1$ and $\sigma = 1$ therefore reduces the likelihood function to a simple least squares objective function. The parameters estimated by these two methods are found the same as those estimated by using the SLS objective function. The SLS objective function was not used again and only the MLE objective function was evaluated.

Results. The values for the paved and grassed abstractions for the three different storms analyzed, together with the value of the weighting constant are given in Table 4.7. The values of the objective functions are also given but they cannot be compared due to the change of the values of w_t and σ in equation 4.6.

Examples of the outflow hydrographs computed by using these methods are given in Figures 4.8 through 4.13. In Figures 4.10 and 4.12 the uncalibrated hydrographs, which are hydrographs computed by using parameter estimates which are guessed, are compared to hydrographs obtained by using parameters obtained by using the least squares objective function and to the measured runoff hydrographs. The results in Figures 4.9, 4.11, and 4.13 are the final calculated outflow hydrographs obtained by using the parameter estimates from Sorooshian's method as well as measured hydrographs.

The observed peak flows, time to peak flows, and the runoff volumes are compared with those of calculated hydrographs. The calculated outflows produced by the optimal parameters from the SLS and the MLE procedures are given in Table 4.8.

Discussion of Results. The optimized values for the paved and grassed abstractions for the first storm are closer to the initial values and the outflow hydrograph computed by using the optimal estimates also resembles that which is computed by using the initial parameter values. In the case of the remaining two storms the optimized values for the paved and grassed abstractions yield outflow hydrographs that are better representations of measured hydrographs. The magnitude of the paved and grassed abstractions for both these storms are of the same order of magnitude and markedly different from the paved and grassed abstractions obtained by using the first storm.

The difference between the parameter estimates for the first and for the second and third storms can be attributed to the differences in the storms themselves. The first storm has a multiple peak runoff hydrograph while the second and third storms produce single peaked hydrographs.

It should also be noted that the SLS objective function leads to the unlikely situation that the paved abstractions are greater than the grassed abstractions for the third storm. In contrast all the results obtained from the MLE analysis seem reasonable representations of the actual situation.

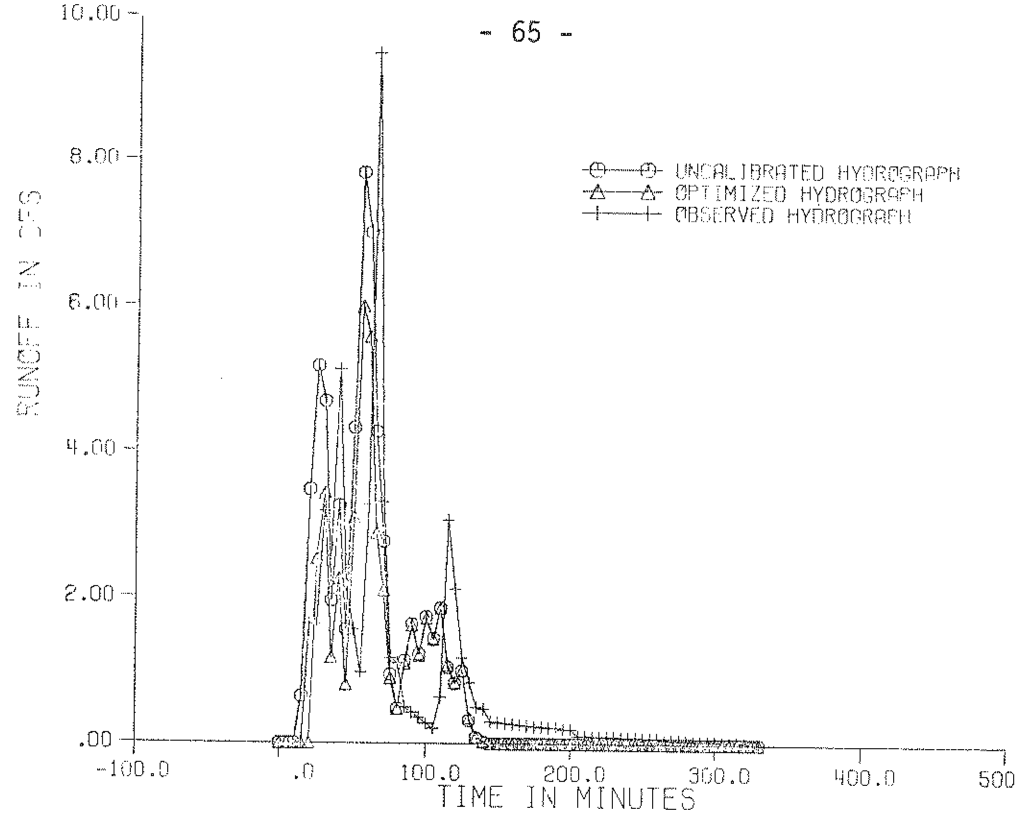

FIGURE 4.8. Uncalibrated and SLS Optimized Runoff Hydrograph for Storm 1.

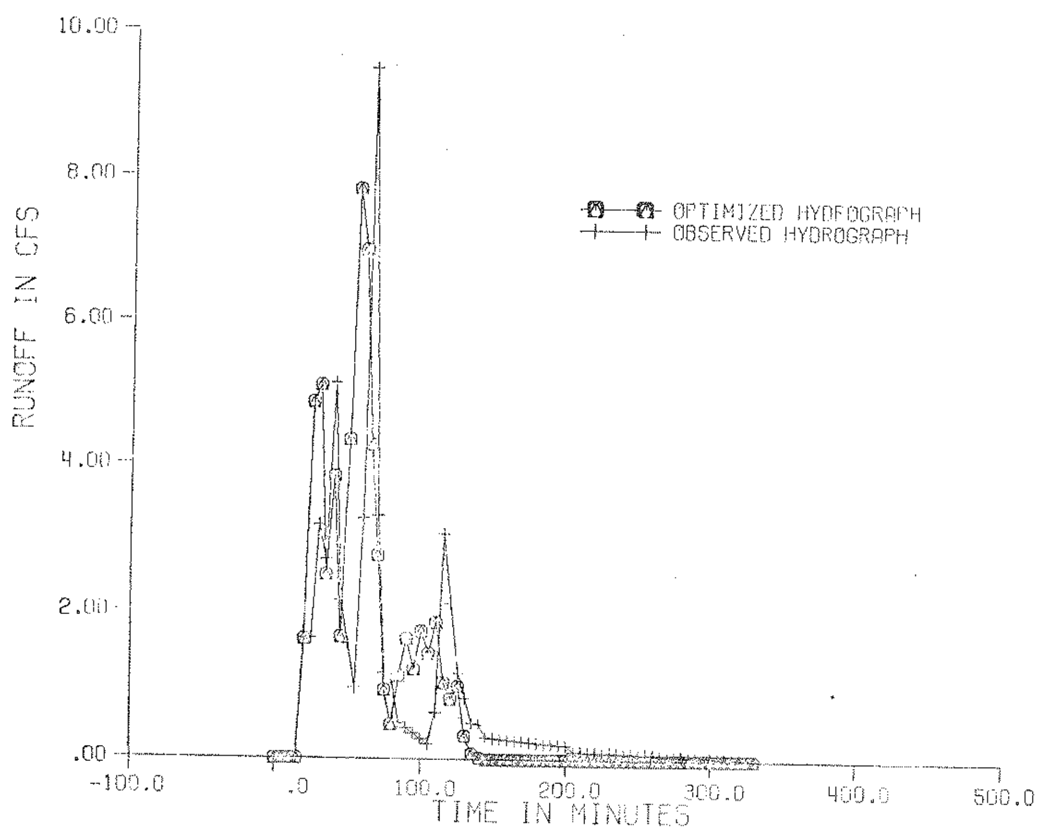

FIGURE 4.9. MLE Optimized Runoff Hydrograph for Storm 1.

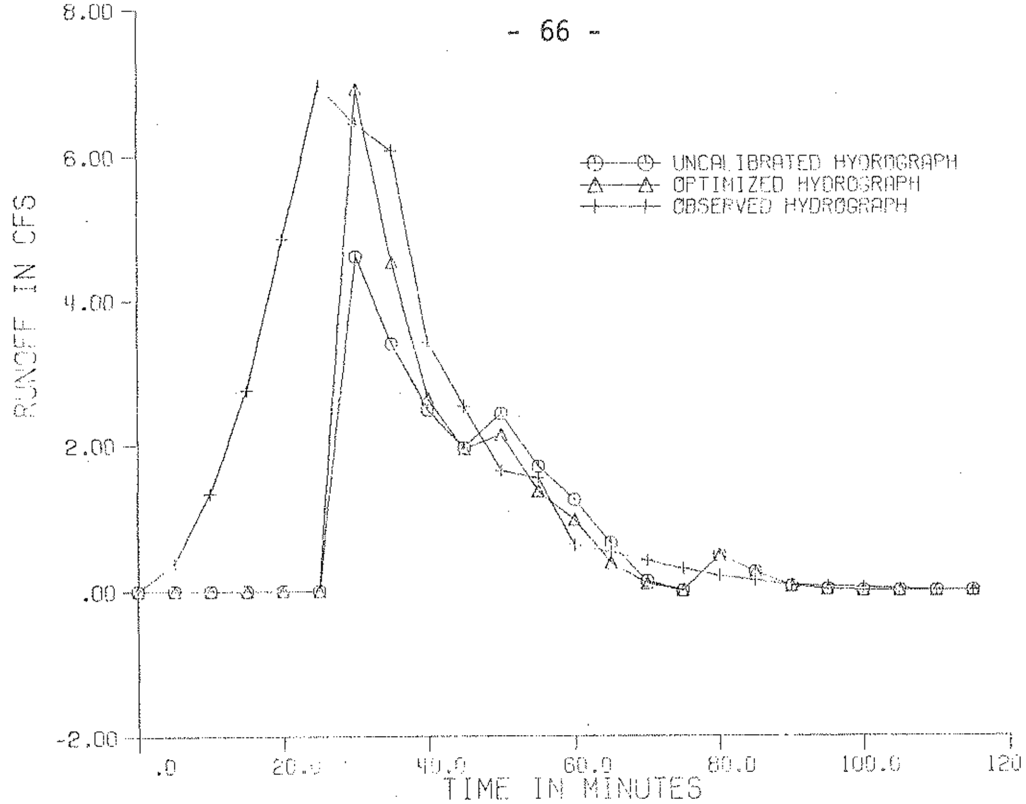

FIGURE 4.10. Uncalibrated and SLS Optimized Runoff Hydrograph for Storm 2.

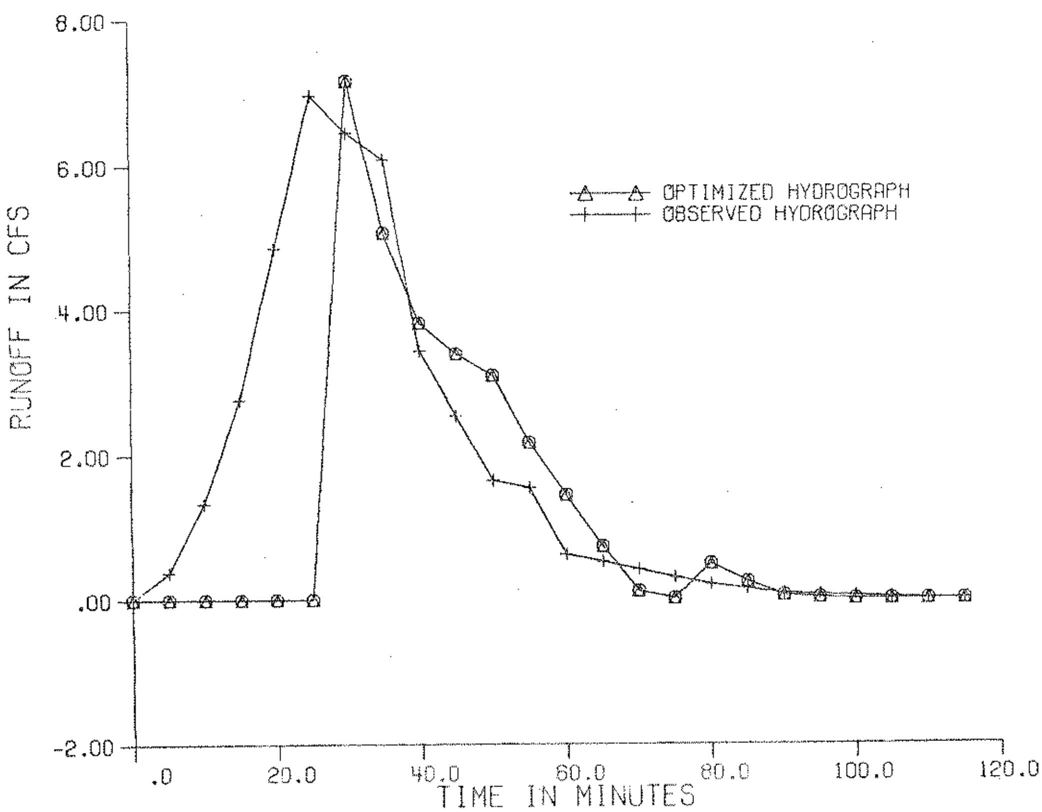

FIGURE 4.11. MLE Optimized Runoff Hydrograph for Storm 2.

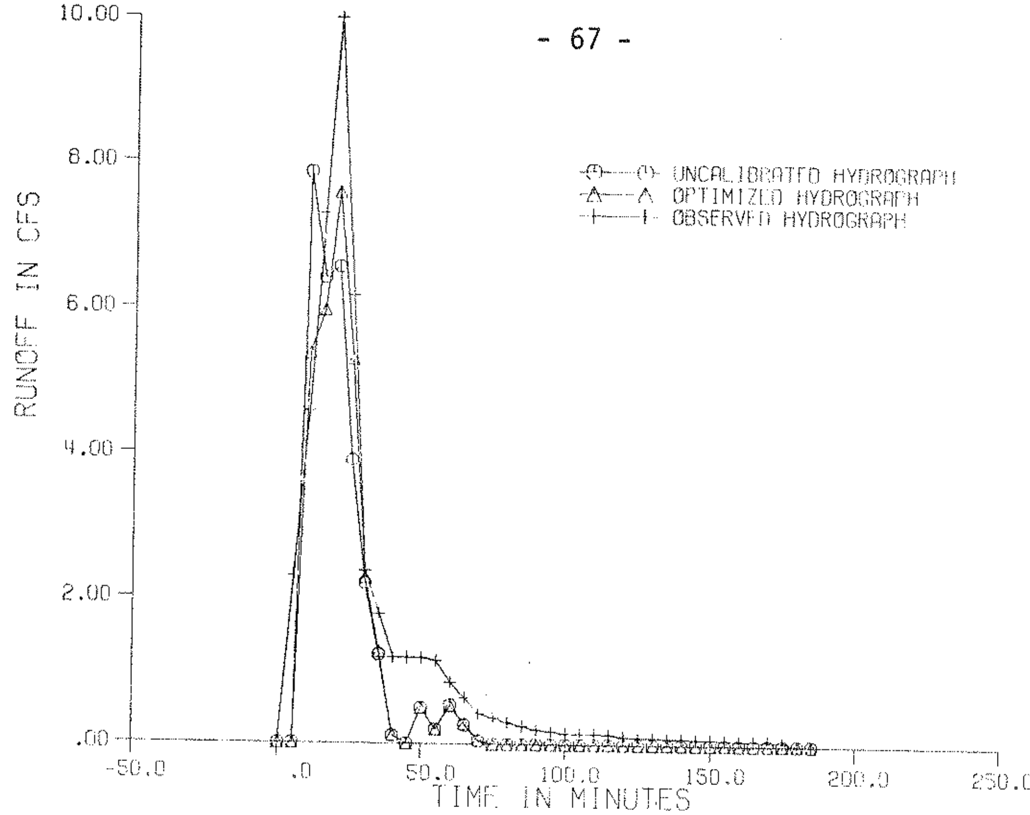

FIGURE 4.12. Uncalibrated and SLS Optimized Runoff Hydrograph for Storm 3.

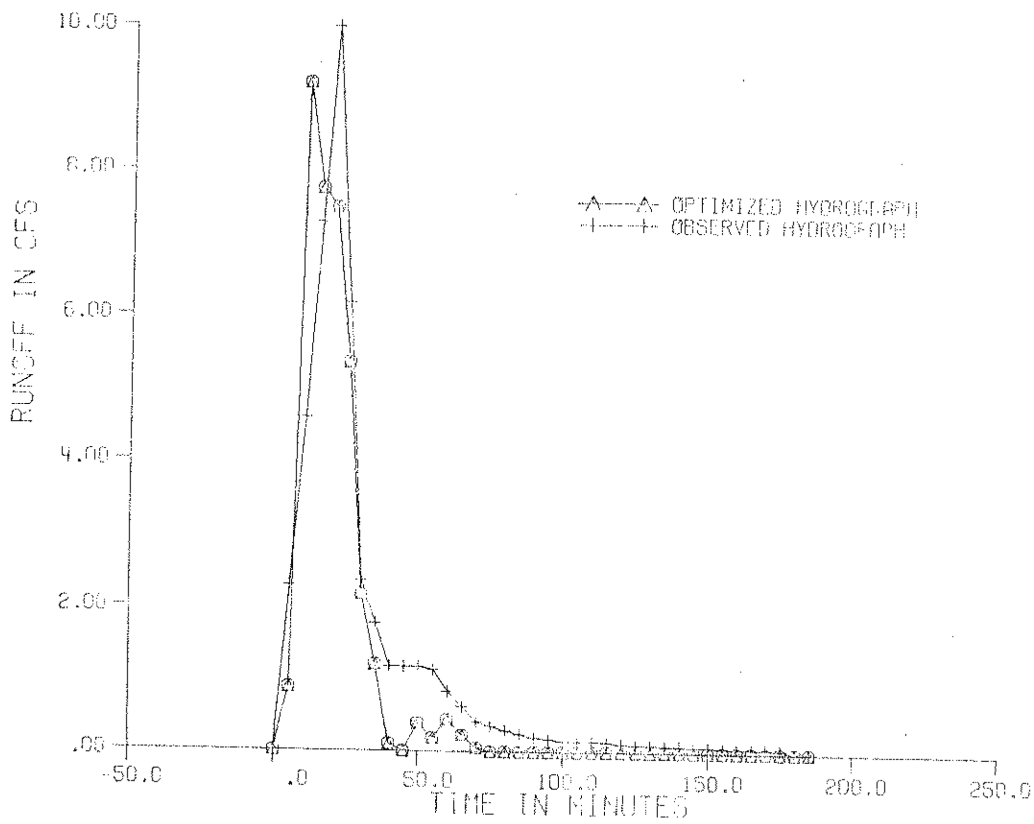

FIGURE 4.13. MLE Optimized Runoff Hydrograph for Storm 3.

TABLE 4.7 Results of Analysis of Parameter Estimation Procedures

Storm of	Run No.	Simple Least Squares			Maximum Likelihood Estimates			
		Paved Abstraction Value	Grassed Abstraction Value	Objective Function Value	Paved Abstraction Value	Grassed Abstraction Value	Objective Function Value	λ
April 19, 1963	1	0.200	0.359	8.2	0.200	0.359	8.2	1.0
	2					0.156	0.227	23998.0
	3				0.165	0.200	14327.3	0.521
	4				0.150	0.161	13957.6	0.523
July 22, 1963	1	0.052	0.278	19.2	0.052	0.278	19.2	1.0
	2				0.056	0.100	34036.6	0.553
	3				0.67	0.100	227565.1	0.600
April 20, 1964	1	0.176	0.112	27.0	0.176	0.112	27.0	1.0
	2				0.073	0.100	509.5	0.828
	3				0.058	0.100	96.7	0.907

TABLE 4.8 Comparisonof Results Obtained With Optimal Parameter Values From the SLS and MLE Procedures

Storm	Method	Peak Runoff Rate (cfs)		Time to Peak (min)		Runoff Volume (cfs)	
		Observ.	Calcul.	Observ.	Calcul.	Observ.	Calcul.
April 19, 1963	SLS	9.5	6.0	70	60	15765	13077
	MLE		7.8		60		17892
July 22, 1963	SLS	7.0	6.9	30	35	12088	6523
	MLE		7.2		35		8823
April 20, 1964	SLS	10	7.6	25	25	13177	8823
	MLE		9.2		15		10660

The MLE procedure leads to closer approximations of the peak runoff rate. The time to peak produced from both optimization procedures are the same except in one instance where the MLE procedure leads to a worse estimate of the time to peak. In the case of two of the storms the total runoff volume estimated by using the MLE procedure is closer to observed values. For the third case, the volume of runoff is overestimated with the MLE procedure and underestimated by the SLS procedure.

4.3 CONCLUSION

The results from this analysis confirm the statement by Sorooshian (1981) that the objective function that makes use of the maximum likelihood estimator gives improved parameter estimates. This procedure is therefore recommended for obtaining parameter estimates for the urban rainfall-runoff model ILLUDAS. It should also be tried with other deterministic models of the rainfall-runoff process.

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CHAPTER 5
URBAN DRAINAGE SYSTEM DESIGN

5.1 Introduction

As urbanization in a watershed increases, there is a corresponding increase in both runoff volume and rate. As a result, most rapidly developing urban areas are now finding themselves faced with the almost inevitable problem of storm sewer overloads. In response to these and other problems, many municipalities are employing detention basins as the primary stormwater management control. Although detention storage has been shown to be an effective stormwater control, random or unplanned placement can significantly reduce its effectiveness, and in some cases, can actually aggravate potential flood hazards. In addition, designs which fail to consider the long term performance of a basin can result in ineffective management for a wide range of runoff events.

In addition to problems of storm sewer overloads, many municipalities are now facing problems related to the quality of stormwater runoff. Recent studies such as the National Urban Runoff Program have shown that dual purpose detention basins can be very effective in the removal of various pollutants. While various policies have been proposed for improving the quality of urban runoff, probably the most effective stormwater management technique is the use of the detention basin. As a result, stormwater management basins are now being used to control water quality in addition to the quantity of runoff.

The widespread use of detention basins is reflected in the results of a recent AWPA survey of 325 public agencies. The results of this survey indicated that over 50 percent of the drainage master plans of the surveyed municipalities included detention basins. Nearly 40 percent of those communities without detention facilities said that facilities are being built, are in the planning stage, or have been considered and are a priority item for the near future (Poertner, 1981). Four of the top eight design objectives reported by the public agencies responsible for establishing detention facilities fall in the category of water quality enhancement (Smith, 1982).

The relevancy of the general problem of detention basin design and utilization was recently highlighted at the 1982 ASCE specialty conference on detention basins, which was held at Hennicker, New Hampshire. The conference addressed both institutional and design issues. Among the recommendations of the conference was a need to integrate both quality and quantity objectives into the design of individual basins. In addition, the need for a better understanding of the interaction of various detention basins within a watershed was identified. The need for general planning methodologies was also addressed.

5.2 Development of a General Planning Methodology

In response to the general recommendations of the Henniker Conference, a general planning methodology has been developed for use in the planning of dual purpose detention basins. The methodology employs continuous simulation, statistical analysis, and a general design heuristic to obtain an integrated system of detention basins. Both water quantity and water quality constraints may be considered. The developed methodology can be used for the analysis of a particular detention system or in deriving general design guidelines. A flowchart of the general planning methodology is provided in Figure 5.1. A description of the methodology along with two sample applications is provided in the following sections. (See Ormsbee (1983) for a more detailed description and a listing of all computer programs used.)

SIMULATION OF THE SELECTED WATERSHED

↓

SELECTION OF A DESIGN EVENT

↓

SELECTION OF A FINAL DESIGN

Figure 5.1 General Planning Methodology

Simulation of the Selected Watershed. In order to optimize the design and placement of detention basins in an urban watershed, it is necessary to model the hydrologic response of the watershed. The last decade has seen the development of several urban watershed simulation models. In order to consider the interaction of both water quantity and water quality objectives effectively, it is necessary to use a model that simulates both processes. Four models that do consider both processes are STORM (U.S. Army, 1976), DR3M-QUAL (Smith and Alley, 1982), HSPF (Hydrologic Simulation Program, 1979), and SWMM III (Huber et al., 1981).

Although all four models can simulate both quantity and quality processes, STORM cannot simulate the storm sewer network of a watershed. DR3M-QUAL is relatively new and is not as comprehensive as SWMM III. SWMM III is generally easier to use than HSPF and has been tested and updated over the last ten years. In addition, SWMM III can use National Weather Service rainfall tapes directly for use in continuous simulation. Because of this fact, and because of the availability of the model, SWMM III was selected as the watershed simulation model to be used in the overall planning methodology.

Selection of a Design Event. The proposed planning methodology uses a continuous simulation model (SWMM III) to obtain a time series of runoff events. The various events are then ranked on the basis of peak, average, and total statistics for both flowrate and pollutant loadings. As a result of these rankings, a set of critical events is obtained for each desired return frequency. In general, the various discrete events will not have the same ranking for the different selected statistics (i.e., peak, average, total). Thus, for a given design variable such as flowrate, one must decide whether to select the design event based on a peak ranking, average ranking, or a total ranking. When pollution parameters are considered in addition to flowrate, the selection process can become very difficult.

In general, for a detention basin design, peak statistics are more important in the selection of design constraints while total statistics are more important in the selection of design events. However, the selection of a final design event will still require an examination of both statistics. For example, an event with a medium volume and a very high peak will probably be more severe than an event with a very large volume and much lower peak. In general, the final selection will tend to involve a certain degree of engineering judgment.

The selection of the most appropriate pollutant statistic will tend to depend on the specific pollutant objectives of the overall design. In this study, the total load statistic was considered to be most important because the design heuristic determines removal efficiencies based on total load.

Although a single design event for a given frequency may be obtained based on an examination of simulation results, a more appropriate approach would be to select a set of design events. By using a set of events, the design corresponding to a particular event can be tested by applying the remaining events. Ideally, one of the individual designs will be satisfactory for all of the events. If no single satisfactory design can be found, then some manual adjustment must be made. This could be accomplished by generating a composite event from the selected set of design events. Such an event could have the hydrograph corresponding to the event with the peak discharge or maximum runoff volume and the pollutograph corresponding to the event with the maximum load. Although the use of a composite design event could result in designs that correspond to larger return frequencies for a particular hydrologic statistic, such an approach will result in designs that satisfy the design frequencies of all of the hydrologic parameters and not just one or two.

Selection of a Final Design. The basic objective of the general planning methodology is to determine the locations and sizes of selected detention basins so as to minimize the overall design cost of the system while satisfying both water quantity (flowrate) and water quality (pollutant load) constraints at specified control points in the watershed. In order to accomplish this objective, some type of design algorithm is needed.

The design algorithm employed in this study uses dynamic programming to obtain an initial feasible solution. Once an initial solution is obtained, the algorithm continues using the Complex Method of Box. Four different state variables may be considered at each detention site: basin length, basin width, basin side slope, and the orifice outlet dimension. For the purpose of this study, square orifices were assumed. In addition, three different costs may be considered: storage cost, area cost, and orifice cost. The cost of the required downstream pipe or channel may also be included in the problem if desired. The design heuristic is very flexible and may include the storm sewer network in the overall design problem. For the purpose of this study, the slope of any designed pipe is assumed to equal a specified ground slope.

Embedded in the design heuristic is a simulation program that routes the inlet hydrographs and pollutant loads through the watershed. Hydrographs are routed through the channels using a simple time lag approach. Hydrographs are routed through the detention basins using Newton's iteration technique.

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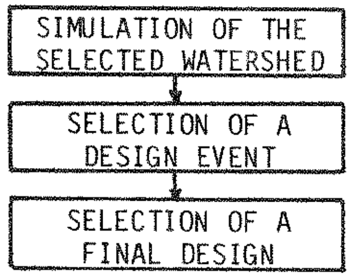

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5.3 First Application of the Planning Methodology - Glen Ellyn Watershed

The general planning methodology was applied to two case studies. The first involved a watershed in Glen Ellyn, Illinois and the second case study involved a synthetic watershed constructed from average geomorphic data for the state of Indiana. In this section, the application of the planning methodology to the Glen Ellyn watershed is described.

In order to test the general planning methodology, it was applied to a watershed in Glen Ellyn, Illinois. Glen Ellyn is located in Dupage County, Illinois, just west of Chicago. The Glen Ellyn watershed encompasses 534 acres of moderately sloped land. The watershed is composed of two major subsheds that drain into a small lake at the outlet of the watershed. Lake Ellyn is one of nine detention facilities currently being investigated as part of the National Urban Runoff Program. Approximately 18 months of data have been collected with regard to the Lake Ellyn study. These data include five minute rainfall and flow data as well as data for 47 different quality constituents. A map of the Glen Ellyn watershed is provided in Figure 5.2. A summary of the physiographic, lane use and hydrologic characteristics of the Glen Ellyn watershed is provided in Table 5.1.

Before applying the general planning methodology to the Glen Ellyn watershed, SWMM was first calibrated using three different storms selected from the 18 months of record. Final selected parameter values were obtained from an average of the various calibration parameters. Both flowrate and water quality parameters were adjusted in calibrating the model. Once SWMM was calibrated, it was used to generate 18 months of simulated flowrate and pollutant data. These results were then analyzed statistically and a ranking of the top five events for nine different hydrologic statistics was obtained (see Table 5.2). Based on these results, a composite design event was derived for a design frequency of 18 months. The resulting composite event was made up of the hydrograph corresponding to the event with the peak flowrate and the pollutograph corresponding to the event with the maximum pollutant load.

The watershed detention system may be designed using interior flow constraints along with constraints at the watershed outlet, or using constraints at the outlet only. In addition, the system may be designed considering the connecting pipe or channel network, or designed assuming that an existing network is already in place. In applying the general design heuristic to the Glen Ellyn watershed, all four of the above possible design considerations were examined. This resulted in four different case studies. A description of each case study is provided in Table 5.3.

Associated with each case study were 12 different designs. Each design was obtained by considering a different combination of flowrate and pollutant constraints. The flowrate constraints were based on percentages of the flowrate corresponding to the design event which had the largest peak flowrate. For the purpose of this study, percentages of 25, 50, and 75 were used. Total load constraints were selected based on percentages of the total load corresponding to the design event which had the largest load. For this study, percentages of 0, 25, 50, and 75 were used. The results of the application of the general design heuristic are presented in Figures 5.3 to 5.6.

Table 5.1 Physiographic and Hydrologic Characteristics of Glen Ellyn Watershed

Total Drainage Area	534 acres
Impervious Area	182 acres
Effective Impervious Area	91 acres
Land Use	
Low Density Residential	427 acres
High Density Residential	16 acres
Institutional	53 acres
Commercial	27 acres
Wetland	11 acres
Pollutant Loading (TSS - lbs/curb mi)	
Low Density Residential	711 lbs
High Density Residential	267 lbs
Institutional	460 lbs
Commercial	611 lbs
Average Hydrologic Soil Group	C
Main conveyance slope	49 ft/mi
Average basin slope	220 ft/mi
Population Density	5000 pn/mi
Street Density	21.6 mi/sm

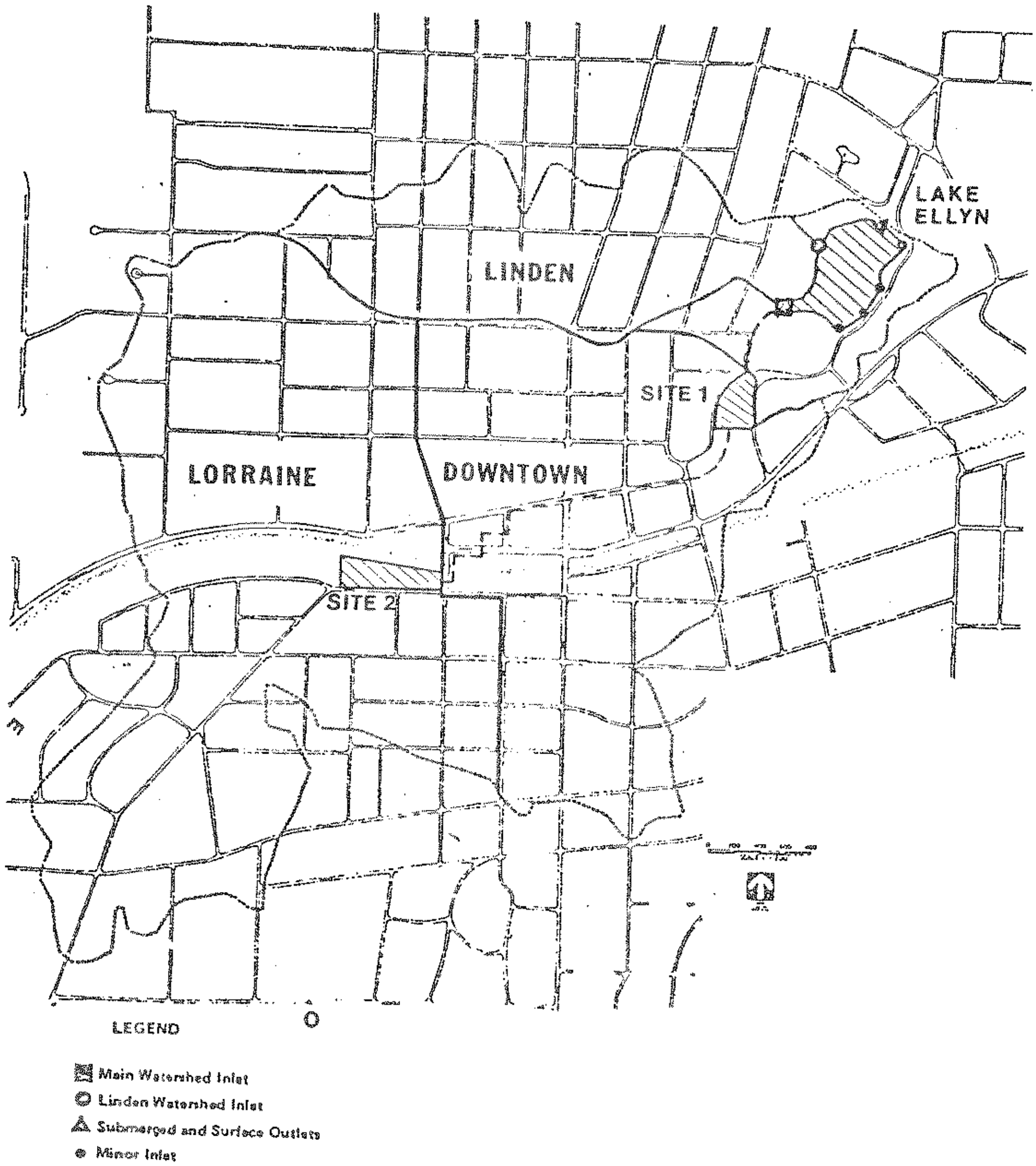

Figure 5.2. Watershed Discretization

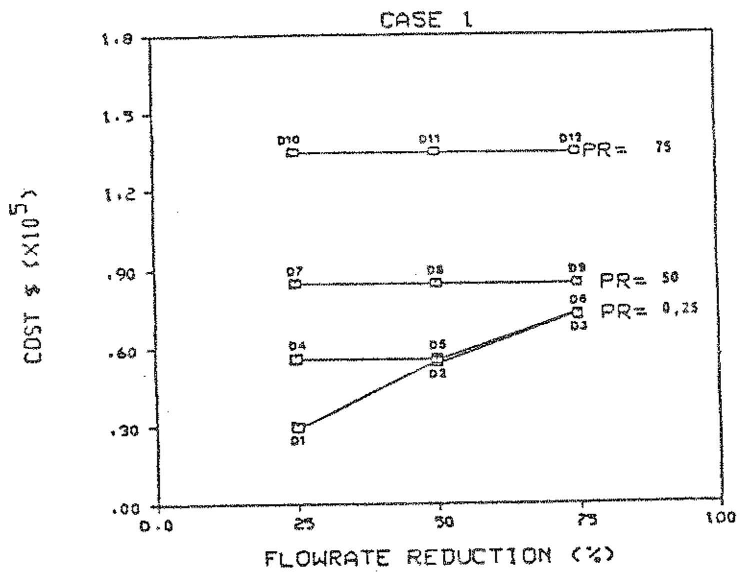

Figure 5.3. Summary of Results for Case 1
(PR = Pollutant Removal %)

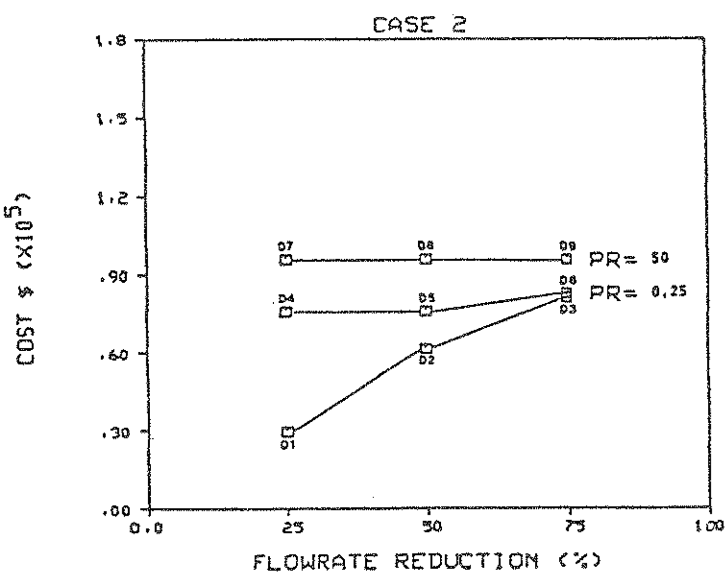

Figure 5.4. Summary of Results for Case 2
(PR = Pollutant Removal %)

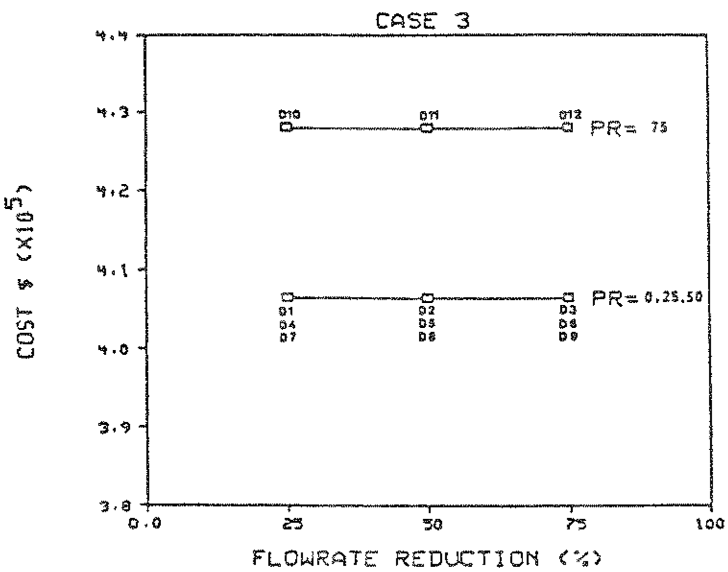

Figure 5.5. Summary of Results for Case 3
(PR = Pollutant Removal %)

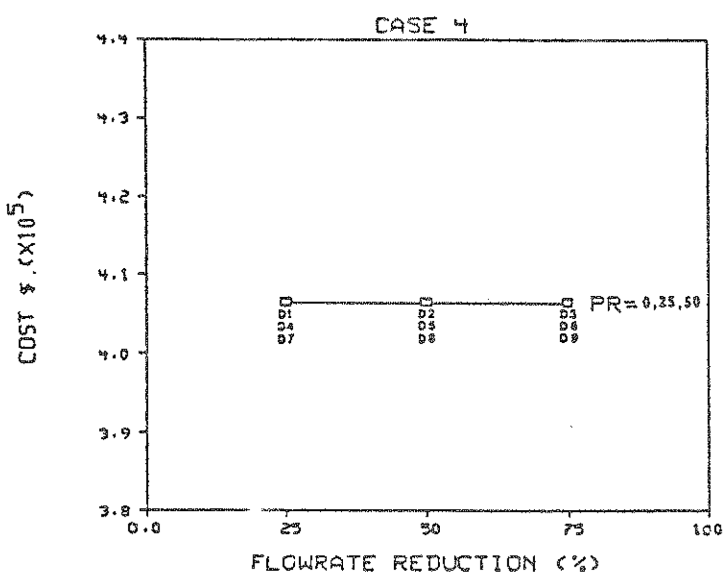

Figure 5.6. Summary of Results for Case 4
(PR = Pollutant Removal %)

Table 5.2 Discrete Simulation Results

RAINFALL DATA

Rank	Peak (in/hr)	Avg (in/hr)	Vol (in)
1	5/29/80 (5.16)	7/13/81 (.710)	9/16/80 (2.05)
2	6/07/80 (5.16)	5/29/80 (.413)	6/07/80 (1.77)
3	4/08/81 (3.72)	7/25/81 (.340)	5/28/80 (1.69)
4	4/28/80 (3.24)	7/20/80 (.298)	7/20/80 (1.49)
5	7/20/80 (3.12)	6/07/80 (.295)	5/29/80 (1.24)

FLOWRATE DATA

Rank	Peak (in/hr)	Avg (in/hr)	Vol (in)
1	5/29/80 (.605)	5/29/80 (.125)	6/07/80 (.380)
2	6/07/80 (.566)	7/13/81 (.096)	9/16/80 (.378)
3	4/08/80 (.406)	6/07/80 (.065)	5/28/80 (.323)
4	4/28/81 (.384)	7/25/81 (.054)	5/29/80 (.312)
5	6/28/80 (.263)	6/28/80 (.048)	5/29/81 (.232)

POLLUTANT DATA

Rank	Peak (mg/l)	Avg (mg/l)	Total (lbs*E3)
1	4/08/80 (2432)	5/24/81 (1243)	5/28/80 (26.2)
2	4/28/81 (2231)	7/20/80 (1062)	9/16/80 (24.2)
3	5/24/81 (2225)	6/28/80 (948)	4/28/81 (23.4)
4	6/28/80 (2150)	4/08/80 (926)	4/08/80 (20.3)
5	7/20/80 (2061)	4/28/81 (916)	7/25/81 (19.7)

Table 5.3 Description of Case Studies

Case Study	External Constraints	Internal Constraints	Storage Costs	Pipe Costs
1	X		X	
2	X	X	X	
3	X		X	X
4	X	X	X	X

The design of a detention basin system has been shown to involve a trade-off between storage and pipe costs. In this particular study, the pipe costs were the controlling factor in the overall design. In the absence of pipe costs, the overall design may be dominated by either the flowrate constraint or the pollutant constraint. In general, designs corresponding to high pollutant removal levels and low flowrate levels tend to be dominated by the pollutant constraint. Likewise, designs corresponding to high flowrate reduction levels and low pollutant levels tend to be dominated by the flowrate constraint. In addition to these two regions, there tends to be a middle region where neither constraint is dominant. The degree of control of the solution space by either constraint will depend on the specific case study being investigated. For case studies 1 and 2, the pollutant constraint tended to be more dominant than the flowrate constraint. In addition, the incremental cost of the pollution removal tended to be higher than the incremental cost of flowrate reduction.

5.4 Second Application of the Planning Methodology - A Synthetic Watershed

Using average stream length and slope data for streams in Indiana, a typical watershed representative of watersheds in Indiana was constructed for use in the application of the planning methodology. For the purpose of this study, average parameter values for the entire state were used. Using mean link lengths and slopes, a simple network configuration was constructed. The subshed areas associated with the various channel lengths were then obtained by application of a mainstream length to area relationship for the state of Indiana. The constructed watershed is shown in Figure 5.7. A conceptualization of the watershed is shown in Figure 5.8.

The synthetic watershed was simulated in both a developed and undeveloped condition. For the natural state, infiltration parameters were selected assuming a hydrologic soil group of C. A listing of the assumed geomorphic and hydrologic parameters for the natural watershed is provided in Table 5.4.

For the developed condition, the original watershed was modified using the regression relationships developed by Gray (1977). For the purpose of this study, average land use values for the entire state were used. Only one pollutant, total suspended solids (TSS), was modeled in this study. Pollutant buildup was assumed to be linear. The initial loadings for TSS were obtained using average state land use values. An exponential washoff equation was used to generate the pollutant loadings during each storm. For this study, an exponential decay coefficient of 1.5 was assumed. For pervious areas,

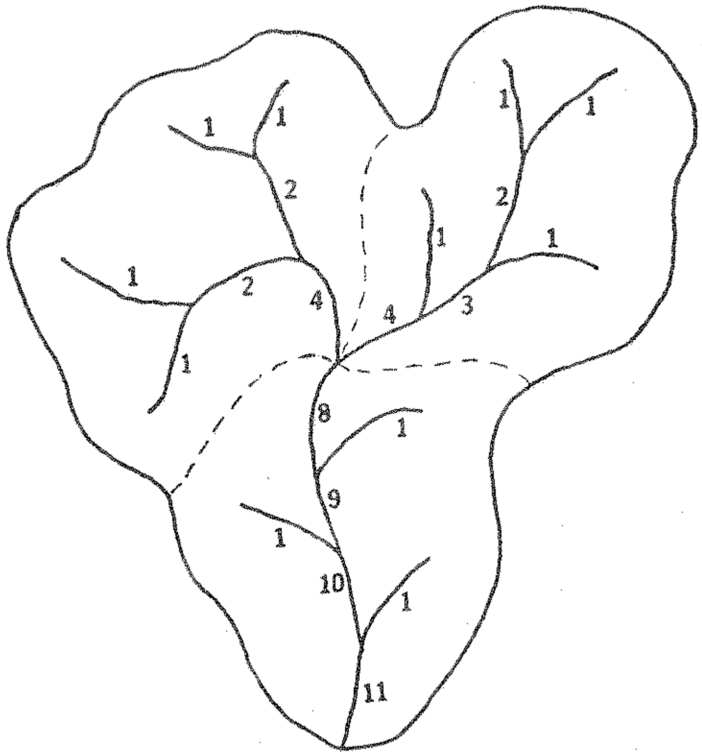

Figure 5.7. Map of Synthetic Waterhsed

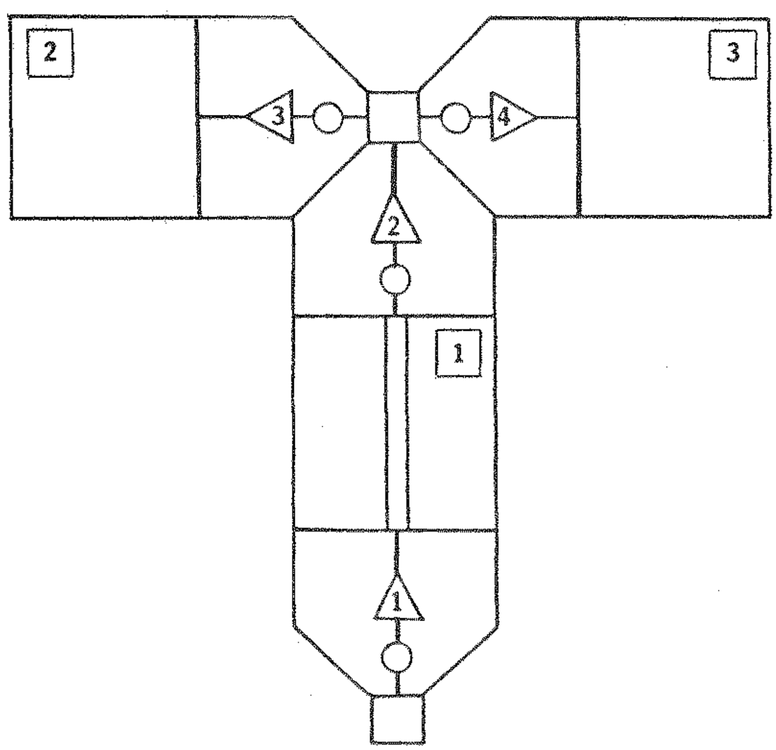

Figure 5.8. Watershed Conceptualization

Table 5.4 Assumed Parameters for Synthetic Undeveloped Watershed

Subshed 1

Subshed Area	95 acres
Eff Imp Area	5 acres
Subshed Slope	97 ft/mi
Subshed Length	2634 feet
Effective Width	9451 feet

Subshed 2

Subshed Area	71 acres
Eff Imp Area	4 acres
Subshed Slope	73 ft/mi
Subshed Length	1985 feet
Effective Width	9400 feet

Subshed 3

Subshed Area	95 acres
Eff Imp Area	5 acres
Subshed Slope	73 ft/mi
Subshed Length	2653 feet
Effective Width	9488 feet

infiltration parameters were selected assuming a hydrologic soil group of C. A summary of the assumed parameter values for the developed watershed are listed in Table 5.5.

Using the constructed synthetic watershed, the detention basin planning methodology was applied for a design period of 20 years. A 20 year rainfall record for West Lafayette, Indiana, was obtained from NOAA for use in the analysis. Design frequencies of 5, 10 and 20 years were selected. For this particular application, the channel network was assumed to be able to carry 20 year predevelopment flow and was not considered in the overall design.

The synthetic watershed was simulated for both developed and undeveloped conditions. In addition to flowrate, suspended solids loading and washoff were also simulated. A 20 year continuous simulation was conducted for both conditions using a one hour time step. After the continuous simulations were completed, a statistical analysis was performed for both simulation runs. A listing of events based on their frequency of occurrence for both the developed and undeveloped conditions is provided in Table 5.6.

For the purpose of this study, a composite design event was derived for each selected design frequency. The composite event for a given frequency was constructed using the hydrograph associated with the peak flowrate and the pollutant load associated with the peak pollutant load.

For this particular application, flowrate constraints and pollutant load constraints were set only at the watershed outlet. Flowrate constraints were based on the undeveloped simulation results for the associated return period. Pollutant load constraints were based on 50 percent of the maximum total load associated with the selected design frequency.

In applying the design heuristic to the synthetic watershed, an attempt was made to derive a single overall design which would meet the pollutant and flowrate constraints of all three design frequencies. In deriving the final design, two different design strategies were investigated.

The first strategy (Case 1) involves a sequential design process. In this case a design is first obtained for the lowest design frequency (e.g. 5 years). Once a design has been derived for the initial frequency, that design is then fixed and the design event corresponding to the next higher frequency is applied (e.g. 10 years). This event is then used to obtain a design that satisfies the constraints of that particular frequency, and also the constraints of all the lower frequencies. Once this design has been obtained, the process is then repeated for the next design frequency until a final design is obtained. Such a strategy will thus insure that the final design will satisfy the constraints of all the selected design frequencies.

The second strategy (Case 2) involves a single design approach. In this case the largest design frequency (e.g. 20 years) was used to obtain a single design. The performance of this design is then tested via simulation for all of the lower design frequencies (e.g. 5 and 10 years). If the derived design satisfies all of the lower frequency design constraints, then a final design is obtained. If the derived design violates a lower frequency design constraint, then the design must be modified in some way until an acceptable

Table 5.5 Assumed Parameters for Synthetic Developed Watershed

Subshed 1

Subshed Area	95	acres
Eff Imp Area	28	acres
Subshed Slope	97	ft/mi
Subshed Length	2634	feet
Effective Width	22724	feet
Solids Loading	366	lb/dy

Subshed 2

Subshed Area	71	acres
Eff Imp Area	21	acres
Subshed Slope	73	ft/mi
Subshed Length	1985	feet
Effective Width	23336	feet
Solids Loading	278	lb/dy

Subshed 3

Subshed Area	95	acres
Eff Imp Area	28	acres
Subshed Slope	73	ft/mi
Subshed Length	2653	feet
Effective Width	23510	feet
Solids Loading	360	lb/dy

Table 5.6 Event Statistics for Continuous Simulation of the Synthetic Watershed

RAINFALL DATA

Return Period	Peak (in/hr)	Avg (in/hr)	Vol (in)
20.0	12/31/65 (2.80)	12/31/65 (2.80)	5/15/68 (4.00)
10.0	8/01/61 (2.64)	7/14/58 (2.56)	9/14/56 (3.90)
6.7	7/11/58 (2.56)	6/13/58 (2.01)	7/02/62 (3.49)
5.0	8/12/56 (2.04)	1/29/59 (1.70)	8/01/61 (3.14)
4.0	6/10/58 (2.01)	8/02/67 (1.40)	1/26/66 (3.10)

FLOWRATE DATA
(Undeveloped Condition)

Return Period	Peak (in/hr)	Avg (in/hr)	Vol (in)
20.0	8/01/61 (1.27)	12/31/65 (.726)	9/14/65 (3.10)
10.0	7/11/58 (1.26)	8/01/61 (.598)	5/15/68 (2.75)
6.7	12/31/65 (1.16)	9/14/56 (.517)	7/11/58 (2.43)
5.0	9/14/65 (1.07)	6/10/58 (.504)	8/01/61 (2.39)
4.0	12/31/65 (.902)	7/08/71 (.443)	12/31/65 (2.18)

FLOWRATE DATA
(Developed Condition)

Return Period	Peak (in/hr)	Avg (in/hr)	Vol (in)
20.0	12/31/65 (1.75)	12/31/65 (1.25)	9/14/56 (3.43)
10.0	7/11/58 (1.65)	6/10/58 (.900)	5/15/68 (3.10)
6.7	8/01/61 (1.62)	7/08/71 (.839)	8/01/68 (2.72)
5.0	8/12/56 (1.27)	1/20/59 (.701)	7/11/58 (2.70)
4.0	6/09/58 (1.20)	9/14/65 (.686)	12/31/65 (2.51)

POLLUTANT DATA

Return Period	Peak (mg/l) (+E03)	Avg (mg/l) (+E03)	Total (lbs) (+E6)
20.0	9/15/60 (11.93)	9/19/60 (10.6)	5/10/57 (.241)
10.0	8/10/61 (11.92)	5/10/67 (10.3)	4/18/70 (.238)
6.7	5/15/58 (11.88)	6/15/57 (10.2)	7/29/70 (.236)
5.0	6/12/73 (11.86)	12/12/65 (10.1)	1/26/54 (.230)
4.0	5/26/65 (11/86)	9/29/70 (10.1)	5/09/55 (.228)

design is obtained.

In applying the general design heuristic to the synthetic watershed, both design strategies were employed. In addition, two different constraint conditions were examined for each strategy. This resulted in a total of four different case studies. A description of each case study is provided in Table 5.7. The results of the application of the design heuristic are presented in Figure 5.9.

Table 5.7 Description of Case Studies

Case Study	Single Strategy	Sequential Strategy	Flowrate Constraint	Pollutant Constraint
1A		X	X	
1B		X	X	X
2A	X		X	
2B	X		X	X

The results of this study indicate that neither strategy can be guaranteed always to be better. While the single design strategy should always produce the least cost design, the sequential strategy will generally always produce a feasible design. Instead of considering the single and sequential design strategies separately, a more appropriate approach would be to combine both strategies into a single design methodology. In using such a methodology, the single design strategy would be employed first. If this strategy yields an acceptable design for all other design frequencies then this design should be selected. If the single design violates constraints of the lower frequency designs than the sequential design strategy should be used.

5.5 General Conclusions

The current study has demonstrated the need for a general planning methodology for use in the design of dual purpose detention basins. Such a methodology has been developed and tested. The general interaction of both storage and pipe costs and flowrate and pollutant constraints in relation to the overall system design has been illustrated. The effect of various design frequency constraints on the overall system design has also been investigated. Although some initial results have been obtained in relation to the above considerations, any general conclusions should be delayed until more case studies have been investigated. It is quite possible, given the complexity of the system, that any general conclu-ion obtainable might be site specific.

The general planning methodology has been applied using several design constraints to yield a wide range of designs. Although there can be no guarantee that the resultant designs are globally optimal, the designs do tend to follow a consistent pattern. Thus, although no formal proof has been presented to guarantee the optimality of the algorithm (if such a proof is even possible), the heuristic does yield improved designs which do correspond to the expected results for given constraint sets.

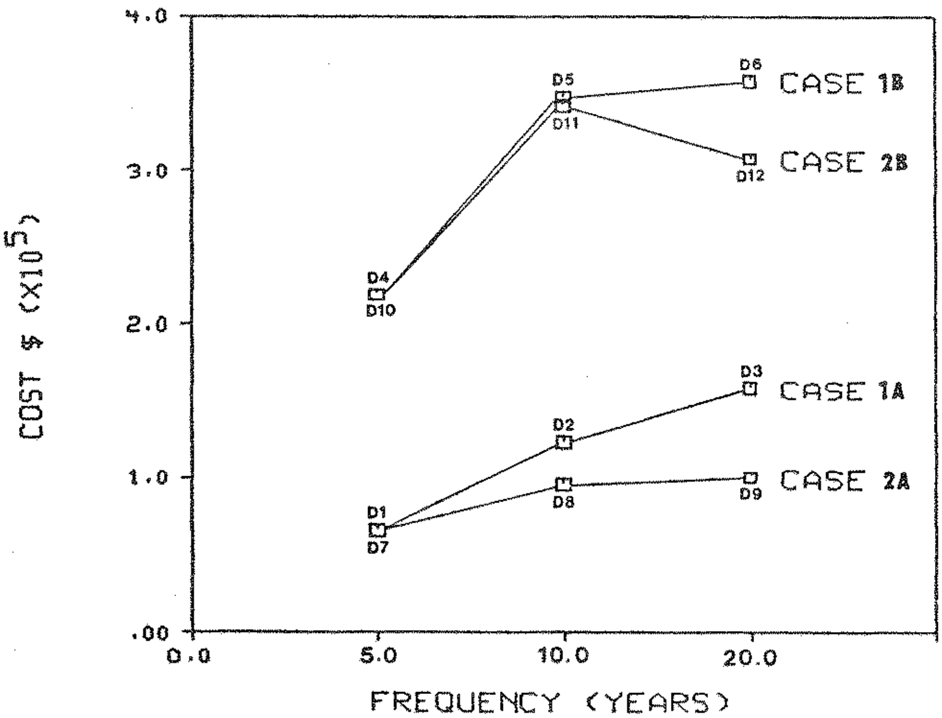

Figure 5.9. Summary of Results

The new detention basin planning methodology should prove to be a valuable tool in the analysis and design of dual purpose detention systems. The new methodology can be used to obtain an individual system design or used in a sensitivity analysis of a given system. Such an analysis can be used to construct cost graphs as a function of different flowrate reduction and pollutant removal levels. By deriving such graphs, information can be obtained concerning the region of control of each constraint. This information could then be used in the selection of a design that provides the best trade-off between pollutant and flowrate objectives for a selected level of flowrate reduction or pollutant removal.

5.6 References

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CHAPTER 6

DATA COLLECTION

The School of Civil Engineering of Purdue University has, for many years, operated field stations for the collection of hydrological data from basins on or near the West Lafayette Campus. As a part of this research project, one of these stations was significantly upgraded (Ormsbee and Delleur, 1982). The selected station is located at the outlet of the Ross-Ade Watershed, just west of Northwestern Avenue at the north edge of the Purdue University campus. The Ross-Ade Watershed has a drainage area of approximately 29 acres. The basin is composed mainly of single family residential houses, with about 11 acres or 38 percent of the watershed being composed of impervious surfaces.

6.1 Description of the Field Station

The station is currently set up to monitor precipitation, runoff, air temperature, and the water temperature of the runoff. The station was also equipped with an ISCO sampler that can draw water samples from the monitored runoff. At the present time, with the exception of the sampler, the station is operating on a year round basis. Data collected from the various sensors is processed by a small microcomputer and then transmitted via a telephone line to a central station in the hydromechanics laboratory of the Purdue University Civil Engineering Building. Data sent from the field station are processed at the central station by a Heathkit H-8 microcomputer which receives the data and stores them on floppy disks. A hard copy of the data is also printed on a DECwriter in the lab. In addition, a backup copy of all transmitted data is also printed on a small thermal printer that is located in the field station.

The monitoring instruments of the field station are housed below ground level in a concrete pit. A trap door and ladder provide access to the instruments. A padlock on the trap door provides the primary security for the station. The concrete pit is located above a concrete storm sewer that drains the Ross-Ade watershed. The gauging control structure is a Columbus type weir and is located adjacent to a concrete stilling basin. During the colder months of the year, a heat lamp is placed in the station to help maintain a milder environment and help control moisture.

The field station power supply consists of two, 12 volt, deep cycle storage batteries and the associated wiring, fusing, and circuitry to allow for independent or parallel operation of the batteries. Also, 115 VAC is available for recharging the batteries, and for providing power to the catchment heater for melting snowfall during winter operation. A switchboard with auctioneering circuitry allows the batteries to be paralleled without regard to the relative states of charge of the two batteries. This feature allows the batteries to be switched without losing power to the micrologger system. The switchboard also allows the standby battery to be isolated from the system during charging, thus minimizing the chance of a line transient damaging the microcomputer and its associated circuitry.

Data Processing. The heart of the data acquisition system is a CR21 micrologger which is produced by Campbell Scientific, Inc. The CR21 micrologger is a battery-powered microcomputer with a real-time clock, a serial data interface, and a programmable analog-to-digital converter. The micrologger can handle up to seven analog inputs and two pulse counting inputs. User programmable signal conditioning in the CR21 can measure volts, millivolts, AC and DC resistance, and pulse counts. The CR21 micrologger is well-suited to monitor signals from a wide variety of transducers recording such parameters as temperature, humidity, solar radiation, wind, pressure, precipitation, event occurrences and many others.

Once each ten seconds, the micrologger samples the input signals according to input programs specified in a user-entered input table, processes that data, and stores them according to output programs specified in a user-entered output table (see Table 6.0). The input and output processing capabilities of the micrologger are determined by programs contained in an applications program module. The input processing programs allow the user to define how each input channel is read and converted to engineering units. The user can also enter a multiplier and an offset with each input for sensor scaling. The output processing programs allow the user to operate on the sensor data and generate time-related data summaries at the output interval specified for that table. Sample, average, maximum, minimum, and histogram are examples of output processing programs.

Data Measurement Equipment. Precipitation at the station is monitored by a standard type tipping bucket rain gauge with a sensitivity of 0.01 inch. The rain gauge is mounted on a 10 foot metal pole which is bolted to a concrete slab that serves as the roof of the concrete pit. Each tip of the tipping bucket generates an electrical pulse that is sent as an input to the micrologger. The rain gauge is equipped with heaters to keep the tipping bucket operating during cold weather and to melt snow for precipitation measurement.

The stage water level signal is generated by a 12 inch diameter float which is positioned in a stilling well connected to the flume channel upstream of the flow measuring weir. The float and its counter weight turn a pulley with a circumference of exactly one foot. Thus an increase of one foot in the stage level will result in one revolution of the pulley. The pulley in turn, operates a set of gears (from a Leupold and Stevens recorder) which have a ratio of 1:3. The output of the gears drives a 10 turn, 2K ohm potentiometer which is connected to the micrologger. With this arrangement, it is possible to measure stage levels over a range of 3 feet.

The air temperature and water temperature are both monitored with Model 101 temperature probes manufactured by Campbell Scientific, Inc. The Model 101 incorporates the Fenwal Ele tronics UUT51J1 thermistor in a waterproof probe with 10 foot cables. A 249K ohm pickoff resistor is molded into the termination end of the cables. Each temperature probe is connected to the micrologger.

Water samples from the storm sewer draining the Ross-Ade watershed are obtained using an ISCO Model 1680OR/1730 refrigerated sampler. The Model 1680R is a refrigerated wastewater sampler which preserves biological

Table 6.0

OUTPUT KEY FOR RAW DATA FORMAT (SEE TABLE 6.1)

TABLE 0001 (ONE MINUTE DATA OUTPUT TRANSMISSION)

01; TABLE INDENTIFIER
02; JULIAN DATE
03; TIME IN MINUTES (2400 HOUR)
04; PRECIPITATION (#-1)/10 = PRECIPITATION IN 1/100'S INCH
05; STAGE LEVEL IN 1/100'S FOOT
06; AIR TEMPERATURE IN DEGREES FAHRENHEIT
07; WATER TEMPERATURE IN DEGREES FAHRENHEIT
08; SAMPLE DRAWN INDICATOR (1 = SAMPLE DRAWN)

TABLE 0002 (HOURLY SUMMARY DATA OUTPUT TRANSMISSION)

01; TABLE INDENTIFIER
02; 03; SPACERS
04; TOTAL PRECIPITATION IN PAST HOUR (#-60)/1000 = PRECIPITATION (INCHES)
05; AVERAGE STAGE LEVEL DURING PAST HOUR
06; AVERAGE AIR TEMPERATURE DURING PAST HOUR
07; AVERAGE WATER TEMPERATURE DURING PAST HOUR
08; NUMBER OF SAMPLES DRAWN DURING PAST HOUR

TABLE 0003 (DAILY SUMMARY DATA OUTPUT TRANSMISSION)

01; TABLE IDENTIFIER
02; MAIN BATTERY VOLTAGE
03; PRINTER BATTERY VOLTAGE
04; TOTAL PRECIPITATION IN PAST 24 HOURS (#-1440)/1000 = PRECIPITATION (INCHES)
05; MAXIMUM AIR TEMPERATURE IN PAST 24 HOURS
06; TIME OF MAXIMUM AIR TEMPERATURE
07; MAXIMUM WATER TEMPERATURE IN PAST 24 HOURS
08; TIME OF MAXIMUM WATER TEMPERATURE

specimens during and after collection. It is designed to collect up to 28 separate sequential samples. The sampler pumps, meters, and by means of a rotating distributor funnel, distributes the desired samples into one of 28 high density polyethylene bottles of 490 ml each. The samples can be collected on a time proportional basis using an internal sampler timing circuitry or on a flow proportional basis using flow inputs from an external flow meter. The sampler is presently set on a time proportional basis and is activated by a signal from the micrologger when a rainfall event begins and when the stage in the storm sewer exceeds 6 inches.

Data Transmission. Output from the micrologger is controlled by a small circuit board designed specifically for that purpose. The circuit board controls the transmission of data to the central station as well as the activation of various other components in the gauging station, such as the ISCO sampler. Data are currently being collected in a discrete event recording mode. Although the micrologger samples each input signal once every 10 seconds, data are output to the central station at predetermined discrete time intervals (see Table 6.0). Normally, during periods of no precipitation, the circuit board permits only transmission of hourly summary data. Once every 24 hours a daily summary table is transmitted that includes the status of the main battery voltage and the printer battery voltage so that the central station operator can monitor and switch the batteries as needed.

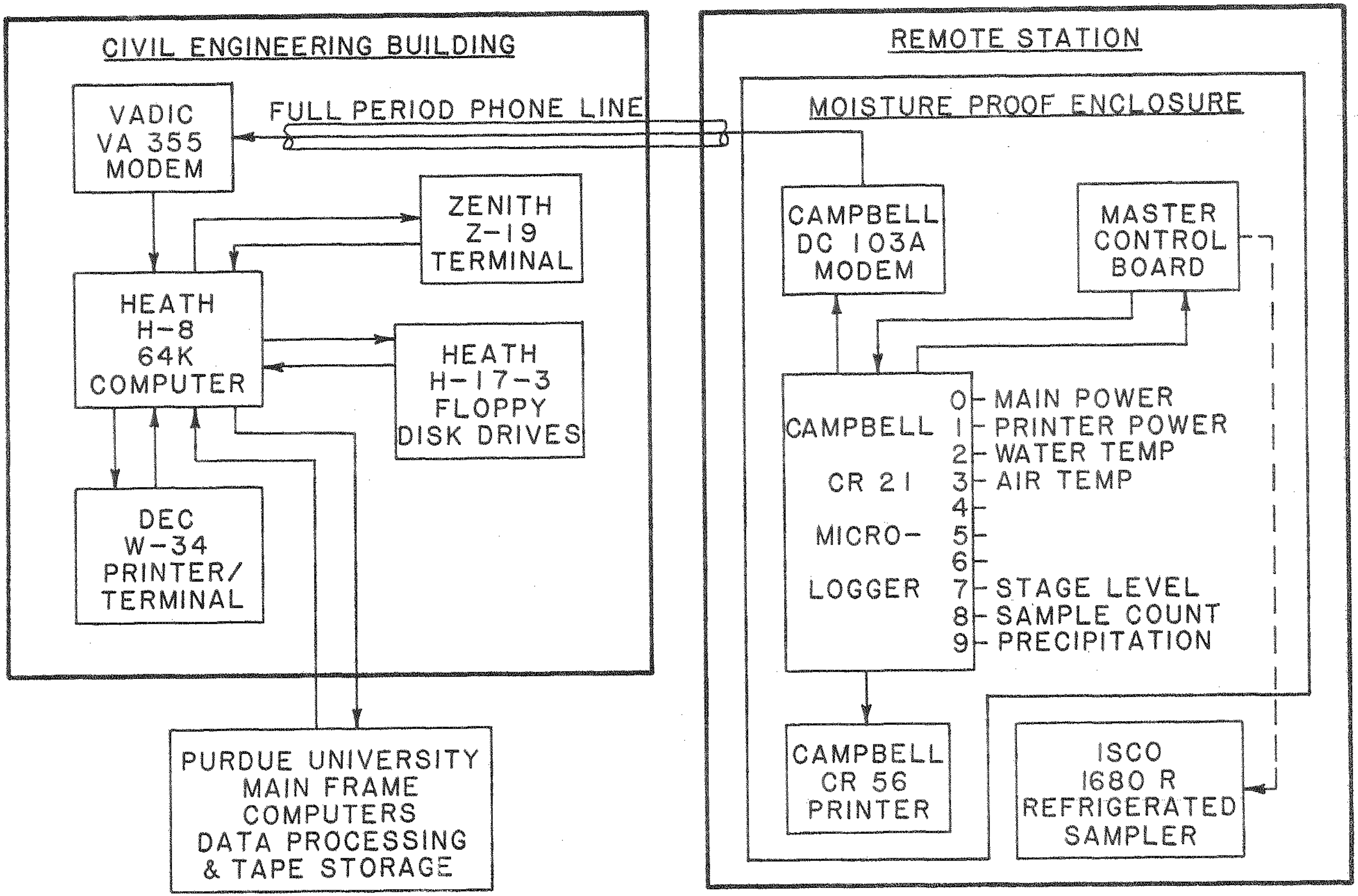
Whenever the tipping bucket is tripped, the circuit board permits the micrologger to send data once every minute. Data will continue to be sent once every minute until 4 hours after the last tip of the tipping bucket. This lag insures that the recession limb of the runoff hydrograph will be adequately recorded. After the 4 hour lag, the circuit board switches back to the hourly summary transmission mode. A schematic drawing of the data acquisition and data management system is presented in Figure 6.1.

6.2 Description of the Data Management System

The heart of the data management system is a Heathkit H-8 microcomputer. The microcomputer has a 64K workable memory and uses three 5 and 1/4 inch single sided, single density floppy disk drives. Each floppy disk has a storage capacity of 90K allowing up to 17 hours of continuous one minute data. The microcomputer and peripheral hardware are located in the Hydromechanics Laboratory of the Purdue University Civil Engineering Building. The microcomputer is connected to the field station micrologger via a restricted telephone line. Data processed by the micrologger are changed to an acoustic signal by a sending modem, sent to the central station over the telephone line, and then changed back into a digital signal by a receiving modem. Data received from the micrologger are first stored in the microcomputer storage buffer. Once the storage buffer fills up, the contents of the buffer are dumped onto a specified floppy disk. An individual disk may accept up to two complete buffer dumps. (See Figure 6.1.)

Data Reception. The data management system can basically be separated into three different steps. The first step involves reception and temporary storage of the incoming data. At least once a day, the central station operator checks the status of the system and the amount of storage space left on the currently accessed floppy disk. If the disk is almost full, the operator

Figure 6.1 System Schematic



will remove the existing data disk, record the contents of the disk, insert a new blank disk, and reinitialize the system. If at all possible, this process is performed between periods of precipitation so that a gap is not created in the record. If a gap does occur, the information may be recovered from the backup printer in use at the field station.

Data Verification and Editing. The second step of the data management system involves verification and editing of the raw data disk. This process consists of a combination of manual editing and use of a preliminary processing program. Manual editing of the data first involves a visual inspection of the central station printer output for an indication of any possible transmission errors. If an error exists, the raw data disk is edited and the error corrected. Transmission errors can usually be corrected from examination of the output from the DECwriter or from the output from the backup printer in the field station. An illustration of the raw data format is provided in Table 6.1.

After the raw data disk has been edited, the data are transferred from the microcomputer to one of the University's larger-scale computers. The transmission of the raw data file from a floppy disk to permanent file storage is accomplished using an assembly language program that links the H8 microcomputer to the larger computers. In transferring the raw data, a new raw data file is created, evaluated and processed using a small Fortran program. The Fortran program deletes unnecessary data, rescales several of the parameters, checks the ranges of the parameter sets, and checks for any previously unobserved data transmission errors. If any range or transmission error is present, the program will identify the location of the error for subsequent editing and re-processing.

The Fortran program, which processes the raw data file, creates two new output files. One file contains processed minute data while the remaining file contains only hourly summary data. An illustration of the processed minute data is provided in Table 6.2. An illustration of the processed hourly data is provided in Table 6.3.

After the raw data have been processed, any available quality data may be added to the processed file. This addition is accomplished through a manual editing of the processed minute file. The results of any quality analysis performed on the water samples are simply input in the appropriate column of the processed file.

In addition to the general processing program, a graphics program has also been developed. This program generates graphs of the four primary measured parameters: precipitation, flowrate, air temperature, and water temperature. The graphs depict 480 minute segments of each event. Therefore, events of 24 hours will have three graph sets. The four graphs from each event are saved in a binder for future reference in selecting storms of interest for subsequent analysis. An illustration of one such set of graphs is provided in Figure 6.2. In addition to providing a valuable catalog of all the various events, the graphs can also be used to identify any anomalies in the recorded data that may not have been identified by the processing program.

Table 6.1 Raw Data Format - Storm of May 1, 1983

01 0001.	02 0121.	03 1404.	04 11.00	05 26.70	06 59.48	07 53.66	08 0.000
01 0001.	02 0121.	03 1405.	04 1.000	05 26.85	06 59.25	07 53.58	08 0.000
01 0001.	02 0121.	03 1406.	04 1.000	05 26.85	06 59.17	07 53.58	08 0.000
01 0001.	02 0121.	03 1407.	04 1.000	05 26.85	06 59.33	07 53.58	08 0.000
01 0001.	02 0121.	03 1408.	04 1.000	05 26.85	06 59.17	07 53.58	08 0.000
01 0001.	02 0121.	03 1409.	04 1.000	05 26.85	06 59.17	07 53.51	08 0.000
01 0001.	02 0121.	03 1410.	04 1.000	05 26.85	06 59.10	07 53.51	08 0.000
01 0001.	02 0121.	03 1411.	04 11.00	05 26.85	06 59.10	07 53.51	08 0.000
01 0001.	02 0121.	03 1412.	04 1.000	05 26.85	06 59.10	07 53.51	08 0.000
01 0001.	02 0121.	03 1413.	04 1.000	05 26.85	06 59.33	07 53.51	08 0.000
01 0001.	02 0121.	03 1414.	04 1.000	05 26.85	06 59.17	07 53.44	08 0.000
01 0001.	02 0121.	03 1415.	04 1.000	05 26.85	06 59.10	07 53.44	08 0.000
01 0001.	02 0121.	03 1416.	04 1.000	05 27.15	06 59.17	07 53.44	08 0.000
01 0001.	02 0121.	03 1417.	04 1.000	05 27.30	06 59.33	07 53.44	08 0.000
01 0001.	02 0121.	03 1418.	04 1.000	05 27.75	06 59.33	07 53.37	08 0.000
01 0001.	02 0121.	03 1419.	04 1.000	05 28.65	06 59.33	07 53.37	08 0.000
01 0001.	02 0121.	03 1420.	04 11.00	05 29.70	06 59.25	07 53.37	08 0.000
01 0001.	02 0121.	03 1421.	04 1.000	05 31.20	06 59.17	07 53.37	08 0.000
01 0001.	02 0121.	03 1422.	04 1.000	05 32.85	06 59.25	07 53.37	08 0.000
01 0001.	02 0121.	03 1423.	04 1.000	05 34.95	06 59.25	07 53.37	08 0.000
01 0001.	02 0121.	03 1424.	04 11.00	05 36.90	06 59.17	07 53.29	08 0.000
01 0001.	02 0121.	03 1425.	04 1.000	05 39.00	06 59.17	07 53.29	08 0.000
01 0001.	02 0121.	03 1426.	04 1.000	05 41.25	06 59.17	07 53.29	08 0.000
01 0001.	02 0121.	03 1427.	04 11.00	05 44.25	06 59.17	07 53.29	08 0.000
01 0001.	02 0121.	03 1428.	04 1.000	05 48.15	06 59.10	07 53.37	08 0.000
01 0001.	02 0121.	03 1429.	04 11.00	05 52.80	06 59.10	07 53.44	08 0.000
01 0001.	02 0121.	03 1430.	04 1.000	05 58.95	06 59.10	07 53.51	08 1.000
01 0001.	02 0121.	03 1431.	04 11.00	05 65.10	06 59.17	07 53.95	08 0.000
01 0001.	02 0121.	03 1432.	04 11.00	05 70.95	06 59.17	07 54.47	08 0.000
01 0001.	02 0121.	03 1433.	04 11.00	05 77.40	06 59.10	07 55.13	08 0.000
01 0001.	02 0121.	03 1434.	04 1.000	05 081.0	06 59.10	07 56.02	08 0.000
01 0001.	02 0121.	03 1435.	04 11.00	05 085.2	06 59.25	07 56.84	08 1.000
01 0001.	02 0121.	03 1436.	04 11.00	05 088.6	06 59.10	07 57.29	08 0.000
01 0001.	02 0121.	03 1437.	04 1.000	05 091.8	06 59.17	07 57.74	08 0.000
01 0001.	02 0121.	03 1438.	04 11.00	05 093.6	06 59.10	07 58.11	08 0.000
01 0001.	02 0121.	03 1439.	04 1.000	05 094.9	06 59.10	07 58.34	08 0.000
01 0001.	02 0121.	03 1440.	04 11.00	05 096.0	06 59.17	07 58.64	08 1.000
01 0001.	02 0121.	03 1441.	04 1.000	05 096.3	06 59.10	07 58.72	08 0.000
01 0001.	02 0121.	03 1442.	04 11.00	05 097.2	06 59.10	07 58.87	08 0.000

01; TABLE IDENTIFIER
02; JULIAN DATE
03; TIME IN MINUTES (2400 HOUR)
04; PRECIPITATION (#-1)/10 = PRECIPITATION IN 1/100'S INCH
05; STAGE LEVEL IN 1/100'S FOOT
06; AIR TEMPERATURE IN DEGREES FAHRENHEIT
07; WATER TEMPERATURE IN DEGREES FAHRENHEIT
08; SAMPLE DRAWN INDICATOR (1 = SAMPLE DRAWN)

Table 6.2 Processed Data Format (Minute Data) - Storm of May 1, 1983

yrdaytime	rain (in)	stage (f-t)	flow (cfs)	atemp (f)	wtemp (f)	bod	fco	tco	ss	p
831211404.	.01	.27	.05	59.48	53.66	.00				
831211405.	.00	.27	.05	59.25	53.58	.00				
831211406.	.00	.27	.05	59.17	53.58	.00				
831211407.	.00	.27	.05	59.33	53.58	.00				
831211408.	.00	.27	.05	59.17	53.58	.00				
831211409.	.00	.27	.05	59.17	53.51	.00				
831211410.	.00	.27	.05	59.10	53.51	.00				
831211411.	.01	.27	.05	59.10	53.51	.00				
831211412.	.00	.27	.05	59.10	53.51	.00				
831211413.	.00	.27	.05	59.33	53.51	.00				
831211414.	.00	.27	.05	59.17	53.44	.00				
831211415.	.00	.27	.05	59.10	53.44	.00				
831211416.	.00	.27	.05	59.17	53.44	.00				
831211417.	.00	.27	.05	59.33	53.44	.00				
831211418.	.00	.28	.05	59.33	53.37	.00				
831211419.	.00	.29	.05	59.33	53.37	.00				
831211420.	.01	.30	.06	59.25	53.37	.00				
831211421.	.00	.31	.07	59.17	53.37	.00				
831211422.	.00	.33	.08	59.25	53.37	.00				
831211423.	.00	.35	.10	59.25	53.37	.00				
831211424.	.01	.37	.12	59.17	53.29	.00				
831211425.	.00	.39	.13	59.17	53.29	.00				
831211426.	.00	.41	.16	59.17	53.29	.00				
831211427.	.01	.44	.20	59.17	53.29	.00				
831211428.	.00	.48	.25	59.10	53.37	.00				
831211429.	.01	.53	.34	59.10	53.44	.00				
831211430.	.00	.59	.48	59.10	53.51	1.00				
831211431.	.01	.65	.68	59.17	53.95	.00				
831211432.	.01	.71	.90	59.17	54.47	.00				
831211433.	.01	.77	1.26	59.10	55.13	.00				
831211434.	.00	.81	1.48	59.10	56.02	.00				
831211435.	.01	.85	1.83	59.25	56.84	1.00				
831211436.	.01	.89	2.11	59.10	57.29	.00				
831211437.	.00	.92	2.44	59.17	57.74	.00				
831211438.	.01	.94	2.65	59.10	58.11	.00				
831211439.	.00	.95	2.81	59.10	58.34	.00				
831211440.	.01	.96	2.94	59.17	58.64	1.00				
831211441.	.00	.96	2.97	59.10	58.72	.00				
831211442.	.01	.97	3.08	59.10	58.87	.00				
831211443.	.01	.97	3.04	59.10	58.95	.00				
831211444.	.00	.96	2.95	59.17	58.95	.00				
831211445.	.01	.97	3.02	59.17	59.10	.00				
831211446.	.01	.97	3.09	59.02	58.95	1.00				
831211447.	.00	.97	3.08	59.02	58.95	.00				
831211448.	.01	.98	3.16	59.17	58.95	.00				
831211449.	.00	.99	3.27	59.17	58.95	.00				
831211450.	.01	1.00	3.41	59.10	59.02	.00				

Table 6.3 Processed Data Format (Hourly Data) - Storm of May 1, 1983

yrdaytime	rain (in)	pint (in)	timep (min)	avgflow (cfs)	maxfl (cfs)	timem (min)	minfl (cfs)	timem (min)	atemp (f)	wtemp (f)
831211600.	.38	.02	1553.00	3.68	7.96	1559.0	2.38	1509.0	59.13	58.72
831211700.	.86	.07	1632.00	13.03	25.31	1637.0	5.80	1624.0	58.42	58.39
831211800.	.73	.04	1750.00	16.43	24.79	1755.0	10.56	1711.0	58.37	57.66
831211900.	.55	.05	1809.00	19.57	25.59	1814.0	11.21	1900.0	57.66	57.43
831212000.	.01	.01	1943.00	5.59	10.97	1902.0	3.19	2000.0	57.37	56.43
831212100.	.00	.00	2100.00	2.31	3.19	2002.0	1.79	2059.0	64.17	56.02
831212200.	.00	.00	2200.00	1.49	1.84	2102.0	1.28	2157.0	68.39	56.39
831212300.	.42	.05	2230.00	4.95	15.80	2236.0	1.16	2223.0	62.42	57.41
831212400.	.03	.01	2339.00	4.92	9.02	2302.0	2.90	2400.0	57.87	56.95
831220100.	.00	.00	100.00	2.07	2.88	2.0	1.62	100.0	59.48	55.88
831220200.	.00	.00	200.00	1.35	1.66	102.0	1.18	158.0	60.23	55.06
831220300.	.00	.00	300.00	1.06	1.20	202.0	.94	300.0	60.49	54.44
831220400.	.00	.00	400.00	.85	.94	302.0	.79	400.0	59.76	54.06

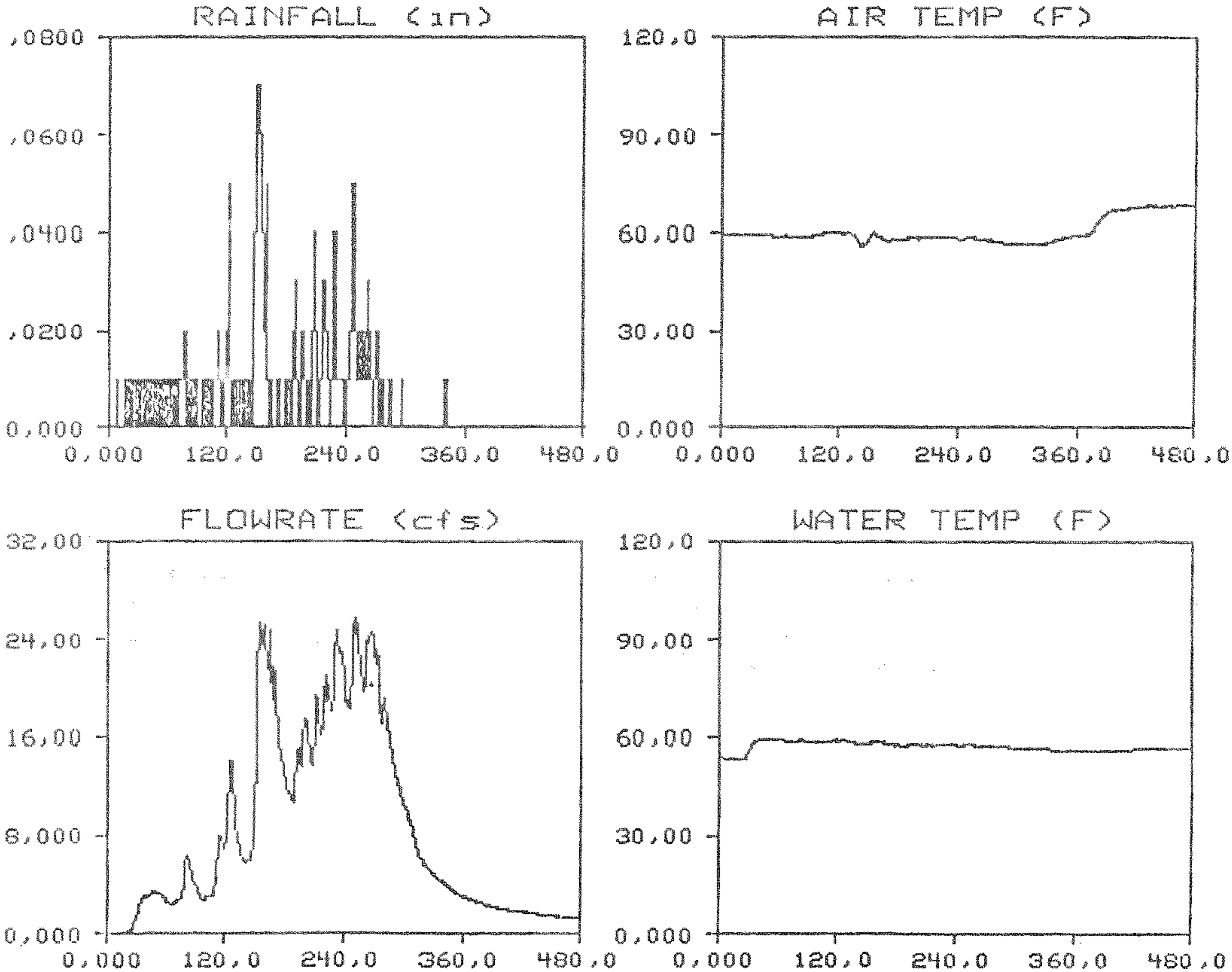


Figure 6.2 Graphs of Hydrometeorologic Data - Storm of May 1, 1983

Data Storage. The final step in the data management procedure involves the transferral of the file onto a magnetic tape for permanent storage (see Figure 6.1). The hydrometeorological data are stored on the magnetic tape in a random access mode. Any individual event or group of events may be pulled off the tape for subsequent analysis without reading the remaining files. Each event file has a name consisting of a letter (A for minute data, B for hourly data and R for raw data), and a 6 digit number corresponding to the year, the julian date, and the hour of the event. A binder containing a summary of all the different files has been created for use in identifying different selected events.

6.3 Implementation and Operational Considerations

Several problems were encountered in upgrading the Ross-Ade field station. The associated problems can basically be divided into two different groups: those problems associated with the data acquisition system, and those problems associated with the data management system.

Data Acquisition Considerations. The decision of what type of equipment to utilize in the improvement of the field station was based on several factors, such as remote acquisition capability, component precision, and system reliability. After considering several different possible systems, it was decided that a remote field station transmission system would be the most economical and efficient system. After consulting with personnel involved with similar projects, it was decided that the data acquisition equipment should be purchased from Campbell Scientific, Inc. This decision was based on the past performance of their products, system reliability, and cost. Although much of the necessary hardware was obtained from an external manufacturer, a few of the operational components, such as the control switchboard, were constructed by Purdue personnel. Although some initial problems were encountered with data transmission noise and a few circuit malfunctions, the problems have now been solved, and the system is working very well.

Data Management Considerations. One of the initial considerations involving the data management system concerned the type of data management hardware to use. Both cassette and floppy disk systems were investigated. Because of the versatility of the disk recording system and due to budgetary constraints, a microcomputer - floppy disk system was utilized. The microcomputer system chosen for use in the project was a Heathkit H8 microcomputer. It was selected due to budgetary constraints and because the Heathkit microcomputer was capable of using a CP/M operating system. After a couple months use, it was determined that the system operated most efficiently with three floppy disk drives. Although a few minor hardware problems were encountered, they were solved fairly easily with assistance from Purdue personnel and with assistance from Heathkit.

After the data management hardware was installed, several other factors remained to be considered. One of the primary problems was the development and/or the acquisition of necessary software to run the system. Several beneficial programs were obtained from persons with the University while the remainder of the software was developed specifically for the data management system. The verification and occasional modification of available software and the writing of the remaining software proved to be a tedious task.

However, after a couple months work, a general operational system was developed, documented, and implemented.

6.4 Summary

In order for hydrometeorologic processes to be adequately investigated and properly understood, it is imperative that a reliable hydrologic data base be established. In order to maximize the quality and utility of such a data base, it is necessary that the data acquisition and management system be designed so as to meet these objectives. In upgrading the Ross-Ade field station, an attempt was made to increase the operational capability of the system as well as the reliability and the utility of the collected data. As of April, 1984, 305 individual events have been analyzed. Associated with each event are minute data for rainfall, flowrate, air temperature, and water temperature. In addition, quality data have been recorded for 36 of these events. These data represent a very extensive data base for use in the analysis of the physical mechanisms involved in urban hydrology. The data have been recorded on magnetic tape in a random access format for easy access and usage. Summary tables and graphs have been developed for the entire data base for use in screening and indentifying critical events for subsequent analysis.

6.5 References

Ormsbee, L.E., and Delleur, J.W., "Data Acquisition and Management Techniques in Urban Hydrology", Proceedings Water-Indiana's Abundant Resource, J.W. Delleur, editor, Indiana Water Resource Association, pp. 14-23, 1982.

6.6 Disclaimer

Mention of trade names or commercial products does not constitute their endorsement or recommendation for use. They are given only to provide a complete description of the Ross Ade watershed installation.