METROPOLITAN WATER INTELLIGENCE SYSTEMS

COMPLETION REPORT - PHASE III

By

Neil S. Grigg, John W. Labadie and Harry G. Wenzel

Principal Investigators:

Maurice L. Albertson
Neil S. Grigg
George L. Smith

Submitted to:

U. S. Department of the Interior
Office of Water Resources Research
Grant No: 14-31-0001-9028

MWIS PROJECT

Water Resources Systems Program
Department of Civil Engineering
Colorado State University
Fort Collins, Colorado

June 1974

TABLE OF CONTENTS

Page

CONTRIBUTING AUTHORS

P. Warren Bell

Bruce H. Bradford

Marshall Flug

George L. Smith

Michael A. Stevens

Paul D. Trotta

L. Scott Tucker

FOREWORD

This is the final Completion Report prepared under a grant by the Office of Water Resources Research for a project at Colorado State University entitled, "Metropolitan Water Intelligence Systems." The basic objective of the project was to develop criteria and information for the development of metropolitan water intelligence systems (MWIS). The MWIS is a specialized urban water system form of the management information and control system concept which is emerging as a technological innovation in industrial applications.

The project consisted of three phases, each lasting about one year. This report was prepared during Phase III. Basic objectives for Phase I were to:

1. Investigate and describe modern automation and control systems for the operation of urban water facilities with emphasis on combined sewer systems.
2. Develop criteria for managers, planners, and designers to use in the consideration and development of centralized automation and control systems for the operation of combined sewer systems.
3. Study the feasibility, both technical and social, of automation and control systems for urban water facilities with emphasis on combined sewer systems.

Basic objectives for Phase II were to:

1. Formulate a design strategy for the automation and control of combined sewer systems.
2. Develop a model of a real-time automation and control system (RTACS model).
3. Describe the requirements for computer and control equipment for automation and control systems.
4. Describe nontechnical problems associated with the implementation of automatic and control systems.

In Phase III the project objectives were focused into three basic categories:

1. Development of control strategy for automated combined sewer systems.
2. To interrelate computer and control equipment system design with the control strategy adopted.
3. To identify and describe the socio-political and economic factors to be considered in implementation.

This report mainly describes factors associated with the three objectives above. It also attempts to interrelate results from Phase I and II as well.

* * * * *

This report was supported by OWRR grant number 14-31-0001-9028, Title II, from funds provided by the United States Department of Interior as authorized under the Water Resources Research Act of 1964, Public Law 88-379, as amended.

* * * * *

Maurice L. Albertson, Neil S. Grigg and George L. Smith are co-principal investigators.

* * * * *

The following reports have been prepared during the CSU-OWRR project, Metropolitan Water Intelligence Systems. Copies may be obtained from the National Technical Information Service, U. S. Department of Commerce, Springfield, VA 22151. (When ordering, use the report title and the identifying number noted for each report).

Technical Report No. 1 - "Existing Automation, Control and Intelligence Systems of Metropolitan Water Facilities," by H. G. Poertner. (PB 214266)

Technical Report No. 2 - "Computer and Control Equipment," by Ken Medearis. (PB 212569)

Technical Report No. 3 - "Control of Combined Sewer Overflows in Minneapolis-St. Paul," by L. S. Tucker. (PB 212903)

Technical Report No. 4 - "Task 3 - Investigation of the Evaluation of Automation and Control Schemes for Combined Sewer Systems," by J. J. Anderson, R. L. Callery, and D. J. Anderson. (PB 212573)

Technical Report No. 5 - "Social and Political Feasibility of Automated Urban Sewer Systems," by D. W. Hill and L. S. Tucker. (PB 212574)

Technical Report No. 6 - "Urban Size and Its Relation to Need for Automation and Control," by Bruce Bradford and D. C. Taylor. (PB 212523)

Technical Report No. 7 - "Model of Real-Time Automation and Control Systems for Combined Sewers," by Warren Bell, C. B. Winn and George L. Smith. (PB 212575)

Technical Report No. 8 - "Guidelines for the Consideration of Automation and Control Systems," by L. S. Tucker and D. W. Hill. (PB 212576)

Technical Report No. 9 - "Research and Development Needs in Automation and Control of Urban Water Systems," by H. G. Poertner. (PB 212577)

Technical Report No. 10 - "Planning and Wastewater Management of a Combined Sewer System in San Francisco," by Neil S. Grigg, William R. Giessner, Robert T. Cockburn, Harold C. Coffee, Jr., Frank H. Moss, Jr. and Mark E. Noonan. (PB # to be assigned)

Technical Report No. 11 - "Optimization Techniques for Minimization of Combined Sewer Overflow," by John W. Labadie. (PB # to be assigned)

Technical Report No. 12 - "Optimal Control of Flow in Combined Sewer Systems," by P. Warren Bell. (PB # to be assigned)

COMPLETION REPORTS

"Metropolitan Water Intelligence Systems Completion Report - Phase I," by George L. Smith, Neil S. Grigg, L. Scott Tucker and Duane W. Hill. (PB 212529)

"Metropolitan Water Intelligence Systems Completion Report - Phase II," by Neil S. Grigg, John W. Labadie, George L. Smith, Duane W. Hill and Bruce H. Bradford. (PB 221992/1)

"Metropolitan Water Intelligence Systems Completion Report - Phase III," by Neil S. Grigg, John W. Labadie and Harry G. Wenzel.

* * * * *

ACKNOWLEDGMENTS

Several persons deserve credit for their assistance in the conduct of this research. Appreciation is extended to Dr. George Mangan, OWRR Project Monitor for his helpful suggestions and advice. Mr. Murray B. McPherson, Director of the ASCE Urban Water Resources Research Program, deserves a great deal of credit for arranging liaison between the CSU project and the Department of Public Works of the City of San Francisco, and for his continued professional counsel. This arrangement has been extremely valuable to the investigators, both in their understanding of the real world problems involved and also because of the technical expertise made available by the City of San Francisco. San Francisco Public Works personnel have been extremely helpful in advising us on the project. In particular the help of Messrs. A. O. Friedland,

W. R. Giessner, Frank Moss and Harold Coffee is acknowledged. Some of the reviewers of this Phase III report have contributed substantially, notably Mr. Donald Hey of Hydrocomp, Inc. and Mr. Forrest C. Neil of the Chicago Metropolitan Sanitary District.

ABSTRACT

METROPOLITAN WATER INTELLIGENCE SYSTEMS

COMPLETION REPORT - PHASE III

Neil S. Grigg, John W. Labadie, Harry G. Wenzel

The results of the three Phases of the Colorado State University project "Metropolitan Water Intelligence Systems" (MWIS) are reported. The special type of MWIS considered is the fully automated control system for combined sewer systems. The report principally contains technical data on the solution of the control strategy problem and on optimization techniques for developing control logic. The socio-political problems associated with implementing a MWIS are discussed as well as the problems facing local decision makers who must comply with shifting standards under heavy time, technological, financial and political constraints.

LIST OF FIGURES AND TABLES

METROPOLITAN WATER INTELLIGENCE SYSTEMS

PHASE III

SUMMARY OF REPORT

National and local water quality problems are receiving prominent attention in the press. The 1972 Water Pollution Control Act Amendments have far reaching implications for urban water managers. The Act, and related federal policy, gives them a mandate to move to clean up wet and dry weather water pollution problems, a mandate which must be followed by sufficient funds to do the job.

Water quality management is only one of many high priority national problems, others being transportation, housing, education, social programs and other environmental problem areas. In all of these areas public works investments must achieve high levels of cost-effectiveness or they will simply fall short of public needs.

Controlled storage may be the most attractive alternative for the solution of wet weather pollution problems. It can be used to optimize the effectiveness of sewer transport-storage-treatment systems. A great deal of research has been conducted on the different aspects of this problem and engineers have many design and planning alternatives available to them, but the key to optimizing effectiveness may be the use of computer control utilizing field data collected in real-time. This was the basis for the term *Metropolitan Water Intelligence System* first coined by Murray McPherson, Director of the ASCE Urban Water Resources Research Program.

This report presents information on an innovation for wastewater management systems. (For a comprehensive discussion of innovation in the urban water field, see [2]). Automatic control is being increasingly adopted in industry and offers an opportunity--and a challenge--to urban water managers for finding cost effective solutions to wastewater management problems. Closing the automatic control loop on the first large wastewater system may be an event first seen in the 1980's rather than the 1970's, although individual components, such as treatment plants, may reach closed loop control earlier.

One of the principal technical results of this study is the demonstration that the use of a control strategy which makes optimum use of available system storage can result in significant improvement best described in statistical and probabilistic terms. The method used to demonstrate this is a combination of simulation and optimization techniques described in Chapter IV and V. Although a subbasin within the city is used as a vehicle for demonstration, it is clear that even greater improvement in total city system performance can be expected since advantage can be taken of spatial variations in the storm patterns.

Another important technical conclusion is a recognition of the importance of storm prediction in the achievement of optimum system performance. The approaches used in this study for generating control strategies all assume that the storm pattern is known in advance. This was a necessary and reasonable assumption at this stage of development. System performance under any control strategy will be subject to a certain level of uncertainty, however, and the major cause of uncertainty is inadequate storm prediction capability. This points out the need for eventual strategy evaluation in terms of this uncertainty and from this, specific research needs in the area will result.

The ASCE Urban Water Resources Research Program has documented in several recent reports the large-scale but decomposable nature of total city drainage problems. Typically, a city will have one to a hundred *catchments*, each tributary to an interceptor sewer or, in the case of storm sewers, a receiving water body. The concept of separable catchments suggests that computer control systems might be approached in a hierarchical fashion, with control strategy being implemented at the catchment level and that the drainage from catchments be scheduled into a treatment plant by a master computer. This concept fits the trend toward hierarchical control evident in industry. The control approach presented in this report is thus based on *hierarchical control*, meaning that a total city problem is viewed from the *large-scale* standpoint whereas the catchment or *subbasin* problem is viewed as a separate problem. The focus of this report is on optimal control of total city systems, to be achieved by linking together optimally controlled subbasins into a master problem. The *strategy* for developing these levels of control is described in detail. Chapter III describes the hierarchical nature of the problem. The advantages include:

1. More efficient optimization, since the various Subbasin Problems can be solved independently by use of techniques particularly suited to them.
2. Considerable lessening in required core computer storage, both on-line and off-line.
3. Flexibility in utilizing totally on-line optimal control, or a mixture of off-line and on-line optimization.
4. Less complexity in software design through decomposition into several simpler tasks.

5. Implicit storage of a large number of simulated storm events, through off-line solution for only a few representative events.
6. Compatibility with hierarchical computer control.

This large-scale approach provides a framework for dealing with the complex optimal control problem, but many obstacles remain before implementation becomes a reality. The most important include: development of rainfall-runoff and routing models that are adequate, and yet compatible with optimization (i.e., they don't render the optimization problems virtually unsolvable on-line); properly calibrated data, hardware configuration design, and storm prediction.

There are a number of additional problems to be overcome before a large-scale sewer system can be brought under computer control. Each city's system represents a unique design case, both from the standpoint of control *facilities* design and control *strategy* design. The latter is crucial to the selection of computer and control hardware and must be regarded as the most important technical problem in the implementation of a control system.

The concept of a control strategy for an urban sewer storage-treatment system is similar to that which would be used to operate a flood control reservoir, but differs in time scale and in complexity. In order to program a control strategy for a reservoir, the control objectives must be known and then, by an analysis of the reservoir inputs and responses, a control program can be developed. A reservoir would appear to be a rather simple problem compared to typical urban combined wastewater systems which are characterized by shorter response times, more political sensitivity, water quality regulations and other factors.

Because of the complexity of the problem, this report describes three approaches: (a) simulation alone, (b) optimization in conjunction with simulation, and (c) direct application of optimization techniques. The latter, however, is only described for a hypothetical case. Chapter IV describes the development of a simulation model for a particular subbasin in the San Francisco system (Vicente Subbasin). A subbasin was chosen because it is the basic unit of the system and its simulation can provide valuable insight into the effects of various designs and control strategies. This chapter shows how simulation can be used to develop a control strategy and how the results of the strategy can be displayed in terms of an upper-bound or zero overflow curve for storm depth and duration which will cause overflows. This curve is then used to develop probability relationships for performance variables which can be used for system evaluation. In order to compare this approach to that of continuous simulation a semi-continuous simulation program was developed which used the same historical data to generate probability relationships directly without using the overflow curves. These results serve to demonstrate the improvement which can be obtained using control at the subbasin level where spatially uniform rainfall is assumed.

The application of formal optimization techniques in conjunction with simulation is demonstrated in Chapter V using Vicente Subbasin. Results again indicate the significant improvement in system performance. That is possible through use of control strategies. Though many of the results were comparable, the use of optimization was found to have some advantages over the approach of Chapter IV. Chapter V concludes with discussion of a *flow projection* technique that may open the way for utilizing optimization in connection with complex, realistic models of subbasin behavior.

Chapter VI deals with use of optimization techniques for the design of a system of auxiliary storage reservoirs to control overflows. Vicente Subbasin of San Francisco is again used as a case study, where the design variables are numbers, locations, and sizing of the auxiliary reservoirs utilized for detaining flows and therefore minimizing overflows, based on a given 5-year design storm. To complete the study, many more representative storms must be used before the optimal design can be determined. However, a general strategy for solving the optimal design problem is developed which utilizes an efficient search algorithm over the design variables.

Chapter VII discusses the importance of storm prediction in real-time optimal control. It is pointed out that the actual level of performance achieved is directly related to the accuracy of storm prediction and that one of the steps necessary in developing optimal control strategy is a determination of the sensitivity of the system to uncertainty in storm prediction. A review of present analytical storm simulation models is presented as well as a discussion of the nature and potential value of weather radar as a predictive tool.

The approaches to design and control strategy discussed in Chapters IV and V have assumed prior knowledge of the depth and duration of any storm for which the system is to be designed or operated. Real-time operation requires this knowledge, or at least some estimate of it. This means that a control strategy developed without considering uncertainty in storm parameters cannot be truly *optimal*. Although knowledge of best possible system performance is useful, it is not self-sufficient for purposes of design and evaluation. At some point in the development of the system, a method of storm prediction must be incorporated in the control prediction model. Real-time optimal control requires that estimates of interior and overall storm parameters be made. These estimates may be updated as the storm progresses in time as actual data become available, so that the uncertainty of these estimates should decrease. The uncertainty, however, will never reach zero until the particular event is over. Thus, the best level of performance obtained is directly related to storm prediction capability.

Because of the difficulties in optimal control development presented by storm uncertainty, a logical approach is to assume zero uncertainty in the initial stages of the process. Once a model or procedure is developed to generate optimum strategy, a sensitivity analysis can be performed in which the effect of various degrees of storm uncertainty on system performance is evaluated. The specific manner in which the uncertainty is specified may depend on the nature of the strategy prediction model or method. Various depth-duration probability distributions or specific errors in depth and/or duration could be assumed and their effect on system performance observed. With this information, a quantitative judgment can be made as to the degree of importance and nature of the storm prediction model to be developed for inclusion in the final control software package. This model should be optimized in the sense that it should be designed for the specific purpose of providing necessary input to the control model rather than being the best general purpose storm prediction model.

It can be concluded that the entire question of storm prediction for application to real-time urban wastewater system control is a topic for much-needed research. As discussed in the previous section, it is first necessary to determine the detail and accuracy required. When this is accomplished, the prediction method can be developed with these requirements as objectives using perhaps the advantages of both the analytical and experimental approach. The use of long-range radar data as input to a mathematical prediction model with continuous update capability could result in predictions with much lower uncertainty than present methods. The use of weather radar as alternatives to urban raingage networks should be seriously considered. Raingage networks have a number of built-in disadvantages.

Although the report shows that an optimum control system will produce improved system performance, the question of cost-effectiveness remains. How can we move to clean up wet weather discharges in a cost-effective manner? The goals set by the 1972 Amendments to the Federal Water Pollution Control Act are broad: [4]

1. To achieve wherever possible by July 1, 1983, water that is clean enough for swimming and other recreational uses, and clean enough for the protection and propagation of fish, shellfish and wildlife,
2. and by 1985, to have no discharge of pollutants into the nation's waters.

These goals are too broad for operational use in planning, designing and operating pollution control facilities and, in fact, may require more national commitment than is currently in evidence [5]. These activities require more detailed specification of objectives. Since the objectives established will determine the ultimate cost of a system, this first step is the *single most important one* in the systems analysis process.

The attainment of the water quality goals in the 1972 Act will come only at great cost. The goals are, however, the *motivation* for halting combined sewer overflows. Congress has implicitly agreed that it is desirable to control wet weather discharges. The next question is, "What degree of control can we afford?" "What are the tradeoffs?" In the general area of water pollution control the costs skyrocket as the degree of control approaches 100%.

In the case of effluent from treatment plants, the degree of control is relatively well-understood and easy to measure compared to combined sewer overflows.

To attain 100% reduction in pollution may be prohibitively expensive. There may be some economic *optimum* point for pollution control investment, but the point we are currently at is described thus:

> "Since it is difficult to quantify and attach dollar price tags to benefits (from pollution control), it is usually more practical to establish environmental quality goals or *standards* by political means. Once such standards have been set, the economic problem becomes one of achieving these objectives at minimum cost." [1]

Since the design of pollution control facilities proceeds on the above basis, our measure of system *worth* is only present to the extent that an alternative system satisfies the political goal statements. Since the concept of system worth is basic to systems engineering and cost-effectiveness analysis, there is an apparent dilemma in planning for wastewater systems.

Cost-effectiveness analysis is still very much an art as applied to wastewater plans, not withstanding the fact that considerable quantitative analysis can be applied as part of a cost-effectiveness study.

A great deal of work needs to be done to relate storm and combined sewer regulations to water quality goals. The stage has been set for some careful demonstrations of cost-effective solutions to this problem. Control systems offer a promising alternative for consideration.

During the process of formulating alternative solutions to controlling combined sewer overflows, automation should be identified. Political and social feasibility of the various alternatives should also be considered because a project's worth is constrained by its political feasibility or its chances of being adopted. If the chances are slim for acceptance, then its *stock* with regard to other alternatives is certainly lowered. Once an alternative solution is selected, it then becomes a matter of successful implementation. Feasibility is an important consideration during the planning and implementation phases. Constraints are important for management to recognize during these phases because they can have a profound bearing on feasibility.

Five basic categories of constraints should be considered:

1. Technical
2. Political
3. Economic
4. Social
5. Environmental

Constraints are further categorized into 1) those that constrain adoption of control systems and 2) those that constrain or impede continued successful operations of such systems. A number of social constraints have been identified, some of which relate to the adoption phase, some to the implementation phase and some to both phases.

Automation is just one of the new technologies available to urban water managers. How does an agency evaluate the vast array of new technologies? If a promising new technology is identified, how is it assimilated into an organization? Why do some agencies tend to use new technology more rapidly than others? These and other similar questions should be answered to provide insight as to how urban water agencies can identify needs and put new technology to better use.

An urban water agency manager can obtain copies of the myriad of reports summarizing various research efforts that may pertain to his agency's operation. He in fact may never do this. It is very difficult to determine what research project may be useful, if any.

A manager is continuously weighing time and effort versus possible success. If it is difficult to detect a payoff, then the project is not given much consideration. To be most effective, research needs should be generated *from* the user (urban water agency) *to* the researcher. The user should take the lead in identifying the research need and should participate in the research effort. In this way, the research is pertinent and the user is involved, has a stake in the effort, and will be more inclined to attempt a transfer of the newly developed technology.

It is helpful to keep computerized solutions in perspective. A computer is incidental to operation of a system. A computer will not solve the mysteries of a combined sewer system although it can be a big help. A computer can receive, store, manipulate, calculate, and evaluate much more rapidly than a human, but it can only be brought to bear on an issue after the human has gained an understanding of the problem and system.

Because automatic computer control systems will be a new innovation for urban water systems, there are socio-political, economic, and technological problems to be solved for their implementation. Since automation has been a pervasive influence on American society during the last two decades, there exists a vast array of literature concerning its ramifications. Most of this literature is pertinent in some respects to Metropolitan Water Intelligence Systems. With the rapid pace of technological development in the computer industry there is a dramatic trend toward total automatic control of industrial processes. This trend is reflected in the interest in automation evident in the urban water agencies.

Just as in the Apollo Program, successful implementation of this technological venture requires careful advance planning, research, and policy implementation. The rewards may be clean water at lower cost and many spinoff contributions to the urban water field. The cost-effectiveness, and the acceptability of the inherent risks associated with automatic controlled wastewater systems, have not been extensively documented. In fact, the control strategy necessary for implementation of such systems has not been demonstrated yet. The contributions of this research report are in these areas, principally toward the development of appropriate control strategy. Unless it can be demonstrated that such automatic control systems (call them MWIS) are technically feasible, we might as well find other alternatives.

Although this report is mostly technical, presenting information on the development of combined sewer system control strategy at the macro and micro urban scales, it also contributes criteria for planning and designing the facilities of such systems. Much of this criteria was developed from interchanges with personnel from the San Francisco Department of Public Works who are in the midst of implementing an innovative new wastewater plan. Examples of planning and design information available in the report are: information on cost-effectiveness analysis of alternative systems; simulation and optimization procedures for sizing, locating, and controlling detention facilities; and information on planning for data collection and modeling programs.

The work reported on the development of control strategy may at first glance appear futuristic or irrevelent to practitioners (especially those beset by the day-to-day difficulties of meeting standards. Information on control strategy must, however, be considered even in the early stages of planning because it affects the options available to the manager. The eventual capability to reach automatic control will influence planning options and design criteria and must therefore be considered from the beginning of a project.

From a user's standpoint, the prospect of implementing MWIS systems is an intriguing one. In reviewing the draft of this report, a user made the following comments:

> "The prospect of computerized real time monitoring and control of water resources systems is an exciting one and promises to be a great tool for the use of agencies with responsibilities in this area. The development of such a tool is still in its infancy as is evidenced by the suggestions for further study made in your report. However, the first major step has been taken, that is, identification of the problem and outlining of the course to be taken in the solution of the problem.
>
> Recent developments in modeling can be incorporated into the system in order to refine the accuracy of the system's predictions. These refinements can be made concurrently with the installation of a small scale pilot system. Such a pilot system, once installed and operational, can be evaluated extensively in order to assess the worth and reliability of such a system.

Finally, the system is only as strong as its weakest link. In the case of real time monitoring and control of water resources systems in urban areas, this link is rainfall prediction. A real time control system's utility is highly sensitive to rainfall predictions. The use of probabilistic rainfall prediction methods will, perhaps, result in improved overall performance of the system, but such a system can never reach its full potential until it includes an accurate deterministic model of precipitation processes. The usefulness of such a model would extend beyond water resources management in many other areas. For these reasons, it is believed that far greater emphasis should be placed on the study of the physical processes which create precipitation and on the development of deterministic models to be used in precipitation prediction." [3]

The implementation of automated combined sewer systems represents a challenge and an opportunity to urban water managers. On the one hand, such systems promise to deliver high levels of performance for the funds invested; on the other hand, questions of reliability, political visibility, feasibility, technology, and social acceptance have not been answered. The contribution of this report should be measured in terms of its demonstration that mathematical modeling and programming can be fruitfully applied to the combined sewer control problem. If the concept of automated sewer control ultimately passes the above feasibility tests, the control techniques described here will be necessary for successful implementation.

SUMMARY

REFERENCES

1. Freeman, A. M. III, R. H. Haveman and A. V. Kneese, The Economics of Environmental Policy, John Wiley and Sons, New York, 1973.

2. McPherson, M. B., "Innovation: A Case Study," ASCE Urban Water Research Program, TM # 21, February, 1974.

3. Neil, F. C., Letter dated May 9, 1974 with review comments on draft MWIS report.

4. U. S. Congress, "Federal Water Pollution Control Act Amendments of 1972," Washington, D. C., 1972.

5. U. S. Comptroller General, "Research and Demonstration Programs to Achieve Water Quality Goals: What the Federal Government Needs To Do," Report to the Congress, January, 1974.

CHAPTER I

INTRODUCTION

The water pollution potential of combined sewer overflows and stormwater runoff are by now well documented [5]. Solution strategies for this problem are now converging to the point where water quality levels are fixed as goals and stormwater management systems are to be planned, designed and operated at minimum cost. In other words, systems are to be designed for *cost-effectiveness* in meeting water quality goals [13]. While the above is not the final word to be written on this subject, it does appear that one attractive solution to the problem may be the use of automatic control systems which in some cases may be the most cost-effective approach for adoption. In a recent comprehensive assessment report of the problem it was stated that

> "Storage is perhaps the most cost effective method available for reducing pollution resulting from overflows of combined sewage and to improve management of urban runoff."
>
> and
>
> "System controls using in-line storage represent promising alternatives in areas where conduits are large, deep, and flat (i.e., backwater impoundments become feasible), and interceptor capacity is high. Reported costs for storage capacity gained in this manner range from 10 to 50 percent of the cost of like off-line facilities. Because system controls are directed toward maximum utilization of existing facilities, they rank among the first of alternatives to be considered." [5]

As will be discussed later, in-line *and* auxiliary storage systems may represent cost effective solutions. The case for automation of storage control has been presented well by McPherson [9]. This report is concerned with the development of automatic control capability for combined sewer systems specifically. The needs for techniques to plan, design and control these systems are related; therefore the development of *control* capability has implications for the planning and design problems as well. It is expected that the technology presented in the report will be applicable to stormwater systems as well, with certain modifications.

For the purpose of this report, *automatic control capability* means the capability to place a system under total (closed loop) computer control. This is a level of automation beyond the level of operation in existing urban water systems.

The development of automatic control for combined sewer systems promises substantial increases in efficiency of operation for existing and proposed combined sewer systems. With required levels of water quality being established politically, the net result may be substantial economies to urban water agencies.

A. BACKGROUND

An urban water automatic control system may be defined for the purposes of this report as a real-time computer (control) system to operate an urban water system or subsystem automatically, in real-time. A real-time computer (control) system may be defined "...as one which controls an environment by receiving data, processing them, and taking action or returning results sufficiently quickly to affect the functioning of the environment at that time." [8]

The term, *Metropolitan Water Intelligence System* (MWIS) was coined by the Director of the ASCE Urban Water Resources Research Program to describe an urban water automatic control system. He described the MWIS concept as

> "... the hardening concept of multiservice automatic operational control, wherein field intelligence on all aspects of urban water might be acquired, including: precipitation; stream stages and flow rates; water and wastewater treatment facilities; water demands and distribution system rates and pressures; settings of regulating structures; quality parameters for watercourses and impoundments, and within conveyance systems; and the status of special facilities such as recreational ponds and lakes. Possibilities exist for incorporating non-water related service intelligence, including traffic and air pollution monitoring, because these are affected by precipitation and the trend for their control is towards a centralized operation. In its ultimate form, the intelligence system would be in the computer centered closed-loop mode. Using field intelligence as inputs, the computer decision program would resolve best service-least operating cost options, taking into account estimated reliability and risks, and would actuate field regulating and control facilities to approach elected option states. Feedback features would be such as to permit manual supervisory intervention at any time." [9]

McPherson described in detail the opportunities for implementing MWIS in different applications. Another recent report contains additional summaries of automation activities [11]. These reports, although already three years old, constitute valuable source material for the study of urban water applications. It should also be mentioned that the AWWA has a committee on automation of water distribution systems and the ASCE Urban Water Resources Research Council has several members active in this field.

The urban water system may be grouped into subsystems as follows:

URBAN WATER SUBSYSTEMS

Water Supply and Distribution

Sanitary Sewer and Wastewater Management

Urban Drainage and Flood Control

In order to focus on the opportunities for implementation of MWIS technology in urban water subsystems, consider the operational objectives of selected subsystems and elements.

Considering *treatment plants*, four types are now being constructed, including conventional water treatment and sewage treatment plants, as well as advanced wastewater treatment (AWT) plants and stormwater treatment plants. The basic objective for the operation of a treatment plant is to use the capacity at maximum effectiveness. This implies producing drinking water or plant effluent with a fixed level of quality (measured by a number of parameters) in the quantities demanded, at minimum cost. Treatment plants can be viewed as production processes and, as such, all of the advances in industrial automation should be applicable to them. They are subject, of course, to varying environmental standards and to the difficulty that exact mathematical models do not exist for most parts of the production process. Nevertheless, the treatment plant represents a promising application for automation techniques. In addition to McPherson [9], other useful references for automation of treatment plants are APWA [2] and some of the literature of the American Water Works Association such as the August, 1971 issue of their Journal, which was devoted to automation of water utilities.

Considering *water distribution systems*, customer service demands must be met by providing the desired quantities of water to the points of demand at given levels of pressure. The water must meet specified quality standards and the time variation of demand must be met. This problem is analagous to that of transporting goods to distribution points in the quantities demanded, at least cost. There must be provision to meet certain emergency fire demands and it would be desirable if the intelligence system could detect problems such as system leaks. The mathematical model of a water distribution system is probably the best understood of all urban water subsystems and therefore may be the first to reach full automatic control.

Urban Drainage Systems have heretofore not enjoyed the priority afforded water supply and sewer systems. Even the hybrid *combined sewer system* has only recently become an issue of major concern. The present operation of combined sewer systems was not the result of careful planning, but evolved as a process of providing storm and wastewater management at least cost. Now, faced with urbanization and environmental pressures, the existing systems are not adequate to meet the demands placed upon them. Stated as objectives, the demands on the combined sewer system are:

1. To operate as a conventional sewerage system during dry and wet weather.
2. In addition, to operate as an urban drainage system during wet weather, providing a level of service specified by local decisionmakers.
3. To deliver combined wastewater during wet weather to treatment plant(s) at the schedule prescribed by operational capacities and *to meet the regulatory requirements concerning leaks and overflows from the system.*

The emphasis given objective no. 3 reflects a national goal that has emerged in about the past ten years and has become more urgent with the passage of the 1972 water quality legislation.

In their implementation of the 1972 Act, the U. S. Environmental Protection Agency (EPA) may well eventually allow no overflows from existing combined systems. To meet this challenge, cities will be forced to find least cost methods of design, of which automatic control may well prove to be the most cost effective. The economics of this question are expanded in a later section.

B. PLANNING, DESIGN AND CONTROL PROBLEMS IN PERSPECTIVE

The planning, design, construction and operational phases of projects need to be distinguished from each other for a number of reasons. In the technical development of a combined sewer automation project, these phases are especially important because of the *new technology* which has to be developed and because of the careful, step-by-step development process required to collect data necessary for system control.

In this report, research results are presented which are concerned with technical problems to be expected in planning, design or operation of a system. It is important to distinguish carefully between these phases when discussing these technical problems. An example of this may be seen in a discussion of stormwater prediction models. The *criteria* for these models depend on the application, and consequently, the phase in which a model is used. To illustrate the spectrum of technical activities required for implementation, Figure I-1 has been prepared.

Figure I-1 shows a coarse breakdown of technical activities which support implementation of a control system. The real heart of the tech-

Control Development Phase / Project Phase	Preliminary Planning	Planning	Design	Implementation	Operation
Planning for Control Development	Establishment of Data Needs and Control Criteria	Articulation of Control and Design Criteria			
Development of Control Strategy	Experimental Design	Development of Prediction and Optimization Models for Subbasins, Simulation Mode	Simulation--Optimization. Transition and Testing Period	Large Scale Optimal Control Development	
Programming and Testing Control Strategy			Design of Hardware and Software	Programming, Testing	Updating, Expanding Data Base
Implementation					On-Line Optimal Control

FIGURE I-1

TECHNICAL ACTIVITIES FOR IMPLEMENTATION OF COMBINED SEWER CONTROL SYSTEMS nical capability development lies in the computer models which must be developed and utilized at the different stages of implementation. Detailed discussion of some of these models follows in the report and a brief discussion to clarify some points follows here. For the purposes of the discussion, consider that an automated control system has been selected as the solution to the problem such that technical developments will lead up to implementation of the system.

The ultimate operational system will be as shown on Figure I-2. From this figure the different *functional elements* can be seen. They will vary from system to system, of course. The blocks shown in the computer control system are explained more fully in a later section. For the purposes of this discussion, Figure I-2 demonstrates the basic elements of concern in the *planning*, *design* and *operational* problems. The planning and design problems are basically concerned with the combined sewer system itself, with the selection, location, sizing and timing of the system elements. The operational problem is concerned with the determination of a best or *optimal* strategy for controlling the combined sewer system. The assumption is that optimal control will more than pay its way, as compared to no control or suboptimal control. This will be clearly demonstrated in a later section.

A number of predictions will be needed for the computer control system shown in Figure I-2. The requirements for the models will vary in the planning, design and operational phases, however. A list of the types of models that will be needed is shown as follows:

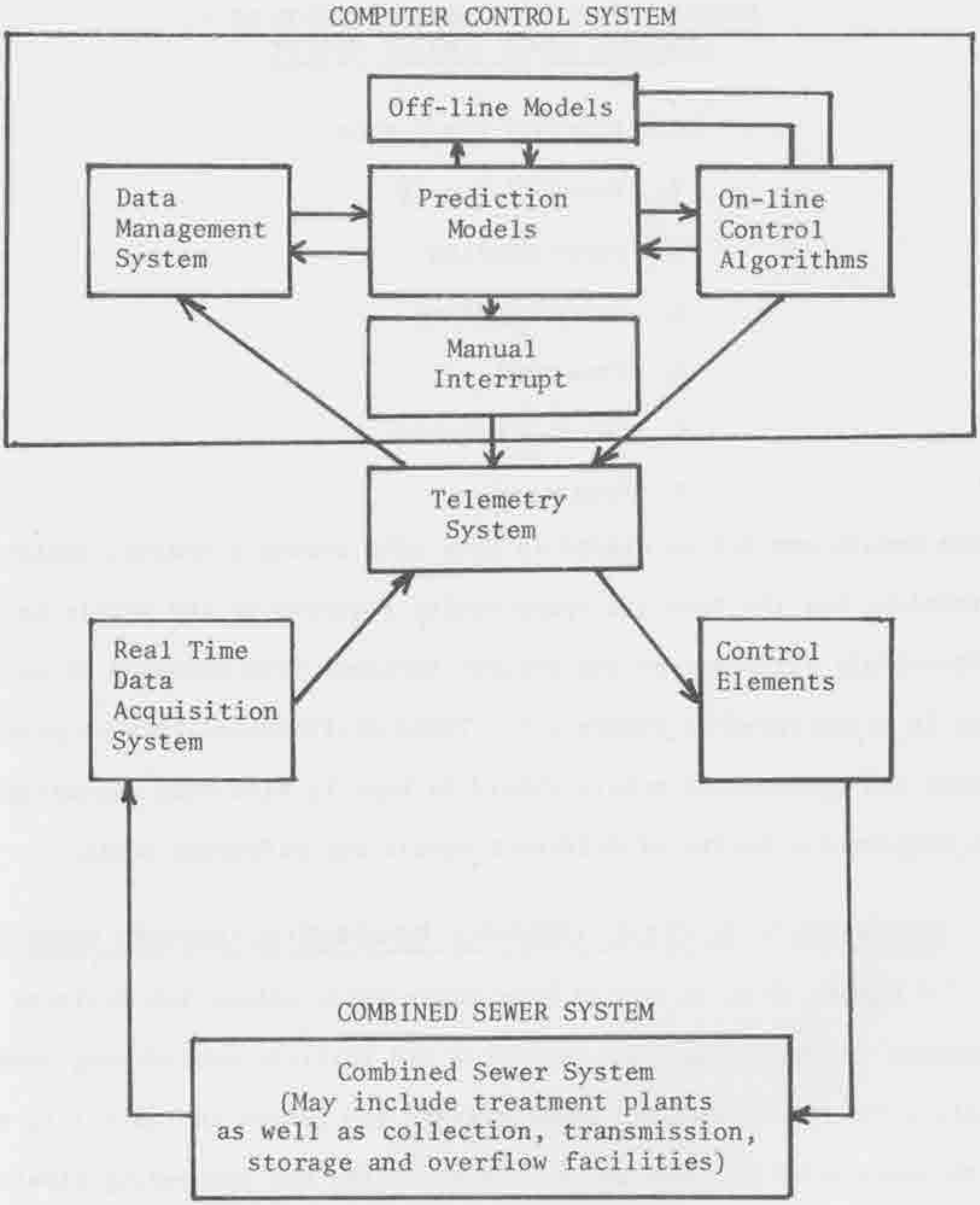


FIGURE I-2

FLOW DIAGRAM FOR COMBINED SEWER SYSTEM
IN AUTOMATIC CONTROL MODE

MODELS NEEDED FOR IMPLEMENTATION OF
COMBINED SEWER CONTROL SYSTEM

1. Rainfall Prediction
2. Rainfall-Runoff
3. Sewer Routing
4. Runoff Quality
5. Treatment
6. Control Response
7. Optimization

These models may all be needed in *some form* during planning, design and operation, but the time and space scales required of the models becomes increasingly stringent as the project proceeds from planning to operation. This is illustrated in Figure I-3. These distinctions between planning, design and operational models should be kept in mind when discussing the comparative merits of different models for different tasks.

C. SOME MAJOR U. S. CITIES CURRENTLY IMPLEMENTING COMBINED SEWER MWIS

A number of U. S. cities have urban water automation projects underway. Some cities that currently are actively considering computer control for their combined sewer systems are listed in Table I-1, along with representative references. These cities are proceeding slowly through the stages of automation and can offer substantial experience and guidance for urban water managers interested in implementing such systems. As a start into the literature, Table I-1 gives representative references.

D. APPLICATION OF RESEARCH TO THE SAN FRANCISCO COMBINED SEWER SYSTEM

The City and County of San Francisco, Department of Public Works, is in the midst of implementing an innovative Master Plan which ulti-

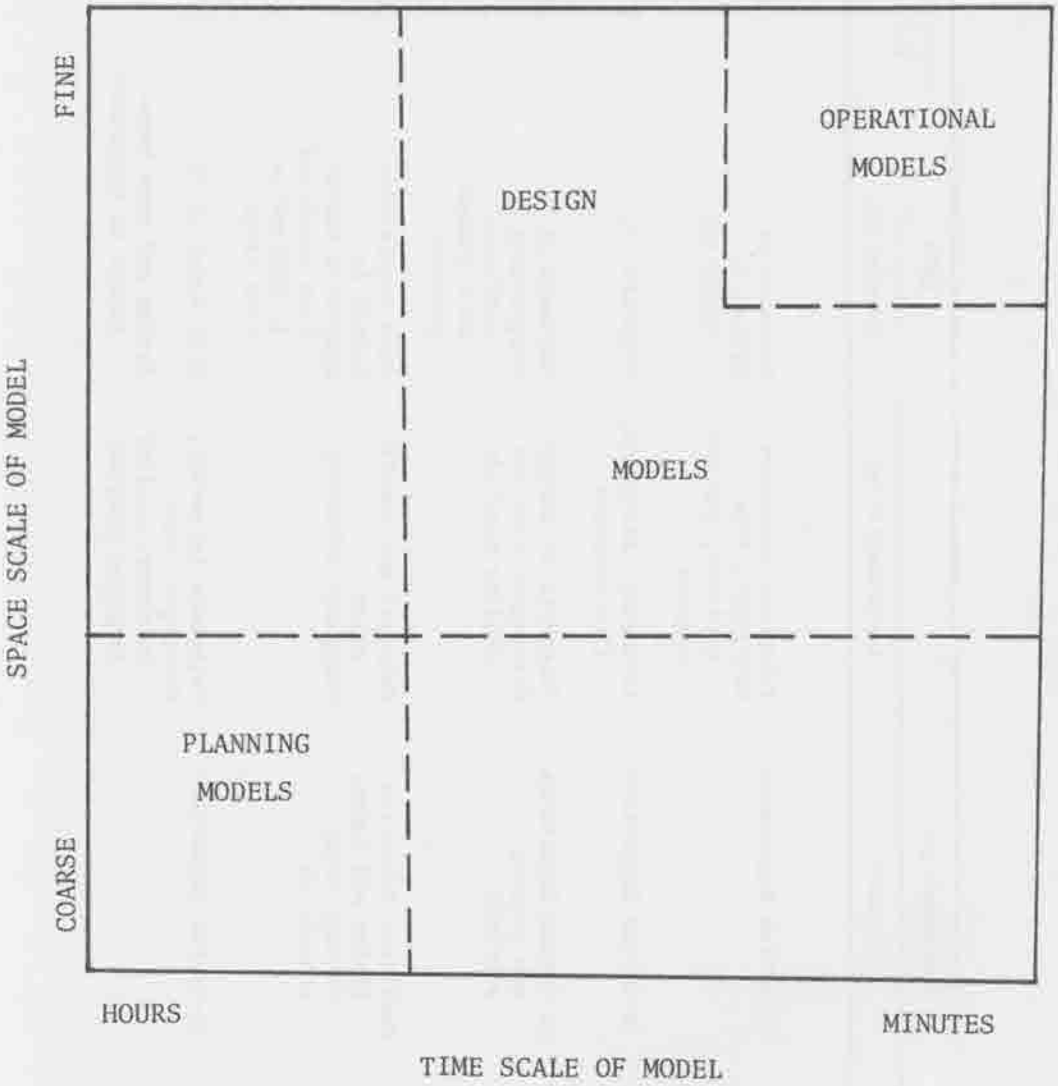


FIGURE I-3

TIME AND SPACE SCALES FOR MODELS REQUIRED

TABLE I-1

SUMMARY OF SELECTED ACTIVITIES IN COMBINED SEWER CONTROL SYSTEMS

	Equipment in			1974 Stage of Completion	Representative References
	Computer	Control	Instrumentation		
Minneapolis-St. Paul	DEC PDP/9	In-System Regulators.	Telephone telemetry. Raingages, flow monitors, quality sensors	Hardware in. System in operation.	[6]
Detroit	CDC SC-1700 (Replaced PDP-8)	In-System Regulators	Telemetry, raingages, flow monitors.	Hardware in.	[1,3]
Seattle	XDS Sigma 2	In-System Regulators and pumping stations.	Telephone telemetry. Raingages, flow and quality monitors.	Hardware in. Programming continues on a phased program.	[4,7]
San Francisco	Data Acquisition Honeywell 316	Auxiliary Detention Basins and Tunnel Storage being considered.	Rainfall and runoff gages. Telephone telemetry.	Data Acquisition System in. Extensive Master Plan completed in implementation stage.	[12]
Cleveland	Data Acquisition and regulator control: GE PAC 30	In-System Regulators	Telephone telemetry, Raingages, flow monitors, quality sampling program	Data Acquisition System and some regulators in operation.	[10]

mately will solve both dry and wet weather water quality problems. Because of the large scale ($500 - $1000 million) and complex nature of the undertaking, implementation is proceeding only after thorough study and testing of each concept has been completed. The basic reference for the plan is [12].

A number of the concepts presented in this MWIS report are based on needs perceived from this "San Francisco Master Plan for Wastewater Management" (SFMPWWM) and have been tested on actual San Francisco data. Personnel from San Francisco has contributed to the research reported here. The elements of the Plan have been summarized several times in other reports and will not be repeated here.

There are a number of reasons for basing the research reported here on an actual case study such as the SFMPWWM. The principal ones are as follows:

1. A wealth of real data and sophisticated analysis of the system is available.
2. The San Francisco DPW has an innovative automatic rainfall-runoff data collection facility in operation.
3. The San Francisco physical system breaks neatly into total city or subbasin packages.
4. Environmental constraints on the Plan are extremely stringent.
5. Public acceptance is an extremely critical factor in San Francisco.
6. The developers of the Master Plan have thoroughly thought out the details of planning, designing and operating their conceived system and are able to

react to suggestions and questions about control strategy.

Because of these and other reasons, the San Francisco system has been studied at two levels, the citywide system and the subbasin level, the latter having been studied in greater detail because of the need to master subbasin control as a prerequisite to citywide control.

E. OBJECTIVES AND SCOPE OF THE REPORT

This report presents technical and non-technical aspects of implementing combined sewer control systems. Much technical material has been omitted from the report, especially where it would duplicate material available elsewhere. Also, separate technical reports have been issued on a number of subjects at different phases of the project.

This report does not constitute the final word on the subject, rather it presents the current state-of-the-art of several facets of automation. It is an extension of some of the technical points presented three years ago by McPherson [9]. The report should provide engineers with technical depth for developing automatic control systems. It should focus for public works managers the choices they have for implementing control systems and illuminate some of the problems they will face. The objective of the report is therefore to present information of several aspects of this subject or in other words, to provide a state-of-the-art document of some important aspects of implementing combined sewer control systems.

The scope of the report includes coverage of technical, socio-political, economic and technological aspects of the problem. Technical aspects are stressed, in particular the difficult problem of developing optimal control strategies for large scale and subbasin level sewer systems. Each chapter is self-contained and represents in-depth coverage of a particular subject area. The chapters are interrelated to the extent that the different problem areas cannot be totally separated.

REFERENCES

CHAPTER I

1. American Public Works Association, "Combined Sewer Regulator Overflow Facilities Report," A report for the U. S. FWQA, July, 1970.

2. American Public Works Association, "Feasibility of Computer Control of Wastewater Treatment," A report for USEPA, December, 1970.

3. Detroit Metropolitan Water Department, A report for USEPA on Detroit Monitoring System to be issued in Spring, 1974.

4. Gibbs, C. V., S. M. Alexander, and C. P. Leiser, "System for Regulation of Combined Sewage Flows," Journal of the Sanitary Engineering Division, ASCE, December, 1972.

5. Lager, J. A. and W. G. Smith, "Urban Stormwater Management and Technology: An Assessment," A draft report for USEPA, December, 1973.

6. Minneapolis-St. Paul Metropolitan Sewer Board, "Dispatching System for Control of Combined Sewer Losses," A report to the USEPA, March, 1971 (11020 FAQ).

7. Municipality of Metropolitan Seattle, "Maximizing Storage in Combined Sewer Systems," A report for the USEPA, December, 1971.

8. Martin, J., Design of Real-Time Control Systems, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1967.

9. McPherson, M. B., "Feasibility of the Metropolitan Water Intelligence System Concept," Technical Memorandum #15, ASCE, Urban Water Resources Research Program, December, 1971.

10. Pew, K. A., R. L. Callery, A. Brandstetter and J. J. Anderson, "Data Acquisition and Combined Sewer Controls in Cleveland," Journal of the Water Pollution Control Federation, Vol. 45, No. 11, November, 1973.

11. Poertner, H. G., "Existing Automation, Control and Intelligence Systems of Metropolitan Water Facilities," Colorado State University, Technical Report #1, MWIS Project, 1972.

12. San Francisco Department of Public Works, "San Francisco Master Plan for Wastewater Management," 3 volumes, 1971 and 1973 Supplement.

13. U. S. Congress, "Federal Water Pollution Control Act Amendments of 1972," Washington, D. C., 1972.

CHAPTER II

ECONOMIC, SOCIO-POLITICAL AND TECHNOLOGICAL ISSUES

A. CLEAN WATER--A NATIONAL GOAL

The problem of combined sewer overflows and pollution from urban stormwater has been described many times. Recently the report of the National Water Commission presented it in this fashion,

> "Storm Water Runoff: A second source of water pollution attracting increasing attention is stormwater runoff from urban areas. Urban land runoff is commonly collected in strom sewers and discharged into waterways. Frequently, stormwater inlets connect directly with sanitary sewers. Where a combined storm and sanitary sewer system is used, heavy storm runoffs result in temporarily overloading or bypassing of local waste treatment plants so that raw or partially treated sewage is discharged into watercourses. Even where separate storm sewers are utilized, stormwater poses a pollution threat. Accidental interconnections with sanitary sewers are common, and recent studies have revealed that the first *flush* of stormwater often carries a pollution load of some constituents greater than that of raw sanitary sewage. It should be noted that the early runoff from heavy rainfall on rural agricultural land and even on wilderness areas also transfers a heavy pollution load to watercourses. [19]

There have been several other definitive reports describing this problem. Examples are the APWA Assessment Study of 1967 [1] and recent overview reports [5,13].

The question arises concerning how we can move to clean up these discharges in a cost-effective manner. What should we take as our goals and objectives? The goals set by the 1972 Amendments to the Federal Water Pollution Control Act are broad: [20]

1. To achieve wherever possibly by July 1, 1983, water that is clean enough for swimming and other recreational uses, and clean enough for the protection and propogation

of fish, shellfish and wildlife,

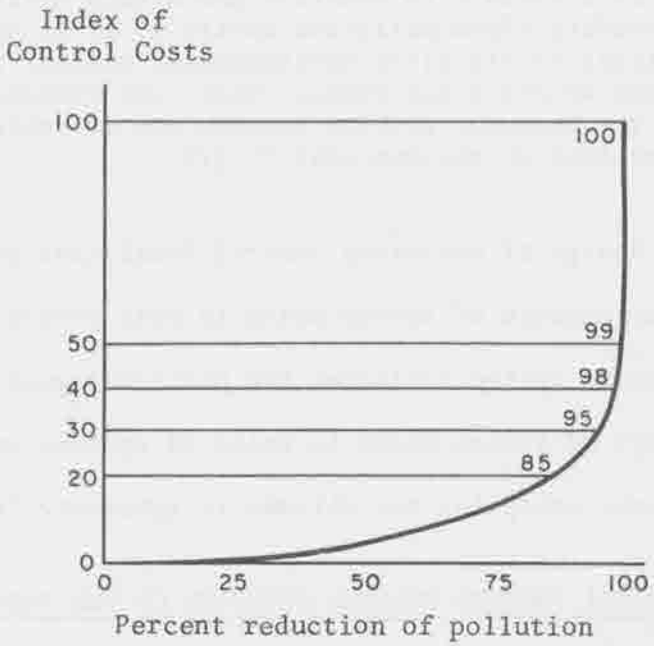
2. and by 1985, to have no discharge of pollutants into the nation's waters.

These goals are too broad for operational use in planning, designing and operating pollution control facilities. These activities require more detailed specification of objectives. Since the objectives established will determine the ultimate cost of a system, this first step is the *single most important one* in the systems analysis process.

The current national discussion about the economic feasibility of implementing the 1972 Amendments is well-founded. The projected national costs to meet the requirements of the Act are staggering. Just as an example, the National Water Commission Report cites costs of $120 billion to clean up *storm* flows by 1983 such that they meet 1965 water quality standards or $248 billion to meet the *best known technology* criterion by the same date. Obviously, expenditures of this magnitude simply cannot be committed for this important, but not highest priority problem (compared to national defense, human resources and other national problem areas).

The attainment of the water quality goals in the 1972 Act will, however, come only at great cost. This is a well-known fact, of course, and cannot easily be resolved. Two opposing questions are: "What quality water do we want, and how much can we afford to pay?" Ostensibly, an incremental increase in funds allocated to water pollution control takes away an increment of investment in another crucial social area.

The *motivation* for halting combined sewer overflows comes from the national water quality goals. We have implicitly agreed that it is desirable to control these overflows and the companion problem of pollution from stormwater discharges. The next question is, "What degree of control can we afford?" "What are the tradeoffs?" In the general area of water pollution control the costs skyrocket as the degree of control approaches 100%. From Figure II-1 this is evident. The same relationship holds for combined sewer overflow.



(Source: U. S. Environmental Protection Agency (1972). The Economics of Clean Water, Vol. I. U. S. Government Printing Office, Washington, D. C., p. 151.)

FIGURE II-1

TOTAL CONTROL COSTS AS A FUNCTION OF EFFLUENT CONTROL LEVELS

In the case of effluent from treatment plants, the parameter *percent reduction of pollution* is relatively well-understood compared to the case of combined sewer overflows.

We have found that to attain 100% reduction in pollution may be prohibitively expensive. This is evident from Figure II-2. There may be some economic *optimum* point for pollution control investment, but respected economists have argued that we can have "too much pollution control." [7]

Furthermore, it is stated that

> "Since it is difficult to quantify and attach dollar price tags to benefits (from pollution control), it is usually more practical to establish environmental quality goals or *standards* by political means. Once such standards have been set, the economic problem becomes one of achieving these objectives at minimum cost." [7]

Since the design of pollution control facilities proceeds on the above basis, our measure of system *worth* is only present to the extent that an alternative system satisfies the political goal statements. Since the concept of system *worth* is basic to systems engineering and cost-effectiveness analysis, our dilemma is apparent. [4]

B. SOCIO-POLITICAL FACTORS FOR THE SOLUTION OF THE PROBLEM

In a systematic approach to problem solving, the first step should be the identification of the problem. The problem to be solved is not always easy to identify because it depends on the goals and objectives of the local area or region as tempered by Federal and State regulations. Regional goals and objectives vary widely from individual to individual, group to group and local governmental entity to local governmental entity. Decisions at this level are largely nontechnical in nature although much of the input is technical. One such decision may be the level of control of combined sewer overflows; should all overflows be treated to the minimum level as prescribed by State and Federal regulations? Other decisions may be whether separated stormwater should be treated as well as combined sewer overflows and the degree or level of treatment.

Probably the most important questions are concerned with how action can be precipitated. What will cause decisionmakers to initiate and complete the combined sewer problem solving process? Will it be early action stimulated by individuals with foresight, or will the action be in response to some crisis, such as unacceptable pathological pollution?

Whatever it is that precipitates decisions to proceed is usually if not always a political decision. That part of the work that provides the basis for a decision is often referred to as the policy development. This is not to be confused with the initiation of the planning process which occurs after the political decision to proceed has been made.

The relationship between technical decisionmaking and political decisionmaking is not clearly understood. A great deal of research is needed in this area. For the purpose of this report it will be sufficient to make the reader aware of and be concerned that political decisions are not made by technical people and technical decisions dictated by the political system. The cases of failure of the program to be implemented or eventual failure of the system once implemented are too numerous to be ignored.

One the problem is identified and a commitment made to the solution, then various alternative solutions to solving the problem can be developed and evaluated. The motivation up to this point is to solve a problem; so far it is the *end* that is important and not the *means*. The selection of an alternative means to solving a problem in most instances will involve political decisions although not to the same extent as problem identification. The degree and level of political or nontechnical involvement in both problem identification and alternative selection will depend (or at least it should) on the extent of the problem. If it is a combined sewer overflow control problem involving or extending over several political jurisdictions, then the involvement will be significant. For example, in San Francisco the most important agencies are the San Francisco Department of Public Works (local), Environmental Protection Agency (Federal), California State Water Rsources Board (State), Regional Water Quality Control Board (State), Association of Bay Area Governments (regional planning), and Bay Area Sewerage Services Agnecy (regional).

During the process of formulating alternative solutions to controlling combined sewer overflows, automated systems should be identified. It is during this phase that cost and effectiveness of various alternatives are developed. Political and social feasibility of the various alternatives should also be considered because a project's worth is a direct function of its political feasibility or its chances of being adopted. If the chances are slim for acceptance, then its *stock* with regard to other alternatives is certainly lowered.

Once an alternative solution is selected, it then becomes a matter of successful implementation. It is in the area of implementation of automated control systems that the MWIS project has been principally directed although some attention was given to the feasibility question in Phase I. Feasibility is a consideration during the processes of formulation and evaluation of alternatives and the implementation phase. Efforts were primarily directed toward the implementation phase and particularly toward the identification of constraints. Constraints are important for management to recognize because they can be addressed and considered prior to their emergence or occurrence. Also, if a constraint is severe enough, it can have a profound bearing on feasibility.

B.1 Constraints to Consider

Five basic categories of constraints have been identified. They are:

1. Technical
2. Political
3. Economic
4. Social
5. Environmental

Constraints are further categorized into 1) those that constrain adoption of control systems and 2) those that constrain or impede continued successful operations of such systems. Several social constraints have been identified, some of which relate to the adoption phase, some to the implementation phase and some to both phases. These constraints are discussed below.

1. Where and how a manager can bring about desired affects. A manager has to be careful how, where and when he tries to affect the social system to achieve his goals. In some cases the common sense approach may jeopardize his case particularly since there is a tendency to treat symptoms rather than causes. A related constraint is the lack of points where a system can be effected with desired results. Both of these related phenomena are presented in "Counterintuitive Behavior of Social Systems," by J. W. Forrester presented in Technology Review, January 1971 [6].

2. Inability to measure and/or quantify benefits. Many benefits of a given proposal cannot be measured in dollar terms and as a result may not affect the decision. Also, it is sometimes difficult to include some costs. The inability to quantify benefits and costs can be countered by careful enumeration of such cost and benefits. The manager can *lay them before* the policy-makers who can then consider them to whatever extent they desire.
3. Lack of political savvy. In some cases a manager may lack the political and social skills required for implementation. This constraint is hard to cope with because the manager is reluctant to admit this failing or may not realize it until it is too late.
4. Organizational effects. Computers and automation have effects on an organization. The location of a computer control center, for example, can have an effect on personnel and should be carefully considered. Introduction of a computer system means the introduction of a new group of people. Managers must realize that introduction of a computerized system is the introduction of a new human process. Visibility within the organization is important. Effectiveness and productivity will be constrained if only one or a few people in high places have a comprehensive understanding of system needs and potentials.
5. Communication constraints. Adequate communications is an important element in any viable organization. Four factors were identified as contributors to communication failures.

The first involves the tendency that there is far more contact between persons of simular status than between vertical layers of an organization. Strong support should be developed in high places to maximize the tendency for high downward flow of information.

The second is related to the difference in skills which tends to reduce the flow of communications. In computer-based systems, there is always the *computer type* and the operation expert that might in some cases find it difficult to communicate.

The language barrier is the third communications constraints. Each profession has a *screen of terms* that are not always understood by persons outside the profession. A conscious effort must be made by the manager to foster communications between professions.

The fourth is attitudinal formations which are developed over a lifetime and shift slowly. Attitudes need to be identified and the potential effects defined.

6. Institutional constraints. If the problems are regional, then the solutions need to be regional also. In many cases a regional framework does not exist for coping with regional problems. This can be a *stopper* and is difficult for any one manager to handle. In any event, the manager must understand the institutional process if he is to succeed.
7. Timing. Usually there is pressure to achieve results quickly. This may sabotage ultimate productivity just as moving too slowly can be a problem. The process should

be such that there is an opportunity to test a control system's effect and then modify it. It could take years to fully understand a sewer system and then to modify a control system to react to it.

8. Skill level. Computer-based control systems require a high skill level. Management must insure that the skills necessary to a successful operation are available. Certain skills can often be combined in one or more persons, but such skills must be available at all times, must be of high quality, and must not be in short supply.

9. Financial commitment. Inherent in any successful operation is adequate financial support. Suffice it to say that without it, the manager must operate on a *shoestring* or in too short a time frame, both of which adversely effect successful implementation.

10. Personality Conflicts and Ego Factors. Organizational objectives are very often forgotten or deliberately set aside in situations of personality conflicts between principals involved in decisionmaking roles. This is understandable when it is recognized that it is strong drives and often high ego factors to seek, obtain, and retain management positions at all decisionmaking levels. To be effective, personal motives such as recognition, remuneration, adventure and personal security that differ from one leader to the next must be orchestrated toward successful program decisions. These considerations are too often left to chance and the logic of what is good for all is good for each part.

Because computer application is a relatively innovative alternative some constraints to its implementation may be unique or more profound than for more *traditional* solutions. Several constraints were enumerated and discussed in this section which could be useful to implementers in identifying potential problem areas.

The following by Murdick and Ross summarizes some of the major causes of difficulties with implementation. Most of these difficulties can be overcome by managerial involvement.

1. Computer rather than user orientation. This is reflected in the use of computers to assist in clerical activities rather than in managerial decision making. The orientation of computers should be toward operational improvement although the record-keeping function is not to be denied.
2. Improper definition of user requirements. System objectives and information requirements should be clearly defined. This is the step in which the manager-user must be actively involved.
3. Organization of the systems function. Traditionally computer operations are placed under the responsibility of the department or division using the computer. As a company expands its use of the computer the basic pattern is to place the overall responsibility for computer applications near the top and to involve operating management in increasing the effectiveness of computer operations. Computer appli-

cation to the combined sewer overflow problem have to date been placed under the user agency and they have not progressed or changed to more centralized control. The question is--should they?

4. Overlooking the human side of information systems. Something must be done to alleviate or overcome the human fears and natural resistance to change. This can be attempted with better communications regarding the nature, purpose and impact of the computer. Gaining acceptance is a function of top management.
5. Underestimating complexities and costs. The tendency is to underestimate the cost and complexity of systems design and implementation. [15]

It is helpful to keep the computerized solution in perspective. A computer is incidental to operation of a system. A computer will not solve the mysteries of a combined sewer system although it can be a big help. A computer can receive, store, manipulate, calculate and evaluate much more rapidly than a human, but it can only be brought to bear on an issue after the human has gained an understainding of the problem and system. A computer is simply no use unless management understands the problem and knows what it wants.

Emphasis in any computer application should be on how management can affect the computer and not on how the computer can affect management. Management should concentrate on how to solve problems and not on how to find uses for a computer.

B.2 Urban Water Agencies and Changing Technology

How does an agency evaluate the vast array of new technologies? If a promising new technology is identified, how is it assimilated into an organization? Why do some agencies tend to use new technology more rapidly than others? These and other similar questions should be answered to provide insight as to how urban water agencies can identify needs and put new technology to better use.

An urban water agency manager can obtain copies of the myriad of reports summarizing various research efforts that may pertain to his agency's operation. He in fact never will do this. It is very difficult to determine what research project may be useful, if any.

A manager is continuously weighing time and effort versus possible success. If it is difficult to detect a payoff then the project is not given much consideration. To be most effective, research needs should be generated *from* the user (urban water agency) *to* the researcher. The user should take the lead in identifying the research need and should participate in the research effort. In this way, the research is pertinent and the user is involved, has a stake in the effort, and will be more inclined to attempt a transfer of the newly developed technology.

It is recommended that research be initiated to gain an insight as to why some agencies are more apt than others to be interested in new technologies. This would permit the identification of progressive or potentially progressive agencies and promising new technologies could be directed toward them. More importantly, it would permit the identification of those agencies where research needs could be identified and possible user agency participation obtained. Once a technology is successfully applied, other agencies will follow. A method is needed to identify those parameters which can be used to identify urban water agencies most likely to utilize new technology.

B.3 Organizational and Behavioral Factors

The state-of-the-art study reported in reference [18] confirmed the need for continuing investigation of the organizational and behavioral factors related to the use of electronic data processing. The USAC report stated

> "...that a well-functioning EDP unit with small resources and dedication can achieve as much or more than a poorly functioning unit with large resources operating routinely. It is clear, therefore, that the organization and management of EDP operations in a municipality is critical to successful use of the technology by the cities." [18]

The report goes on to say that

> "...where the environment is satisfactory the presence of a computer clearly acts as a change agent. Equally important are the organizational and procedural arrangements under which an EDP resource base is operated. Some 20 organizational and behavioral factors potentially relating to successful EDP operations were identified in this study. But continued investigation will be required to learn more of the impact of each factor." [12]

The above stated need is loosely related to social modeling. The organizational and behavioral factors are important elements of a social model of any urban water agency. One can intuitively weigh the impacts, as has been done in this report, but firm, hard data is practically non-existent.

Research efforts to study the impacts of organizational and behavioral factors should be undertaken and should use the USAC work as a starting point. Such research could be related to a social model such as the one described in [9], eventually defining the critical elements of the model and their relationship to one another.

The USAC study identified the computer as an agent of organizational and behavioral change in a city administration. The study suggests that while promoting efficiency the computer at the same time would yield more subtle, less traceable but more important consequences in the balance of power, organizational restructuring, the delivery of services, employee roles and the relative excellence of the planning processes. In order that the range of benefits of computer applied technology can be more clearly defined the above impacts need to be better defined.

C. COST-EFFECTIVENESS ANALYSIS

In order to perform a cost-effectiveness analysis on alternative combined sewer systems, a standardized approach is needed. One example of such an approach is as follows: [11]

1. Define the desired goals, objectives, missions, or purposes that the systems are to meet or fulfill.
2. Identify the mission requirements essential for the attainment of the desired goals.
3. Develop alternative system concepts for accomplishing the missions.
4. Establish system evaluation criteria (measures) that relate system capabilities to the mission requirements.
5. Select fixed-cost or fixed-effectiveness approach.
6. Determine capabilities of the alternative systems in terms of evaluation criteria.
7. Generate systems-versus-criteria array.
8. Analyze merits of alternative systems.

9. Perform sensitivity analysis.

10. Document the rational, assumptions, and analyses underlying the previous nine steps.

It has been pointed out earlier that the goals and objectives of a combined sewer control system are derived politically. They turn out to be of the *fixed effectiveness*-type as described above, with cost-effectiveness being mostly an analysis leading to a minimum cost solution. The question of trade-offs with fines, penalties, incentives, etc. has not been yet extensively debated.

Not many actual broadly based examples of cost effectiveness studies on combined sewer systems exist in the literature. Many examples of localized engineering studies have used this approach, with varying degrees of sophistication.

C.1 <u>An Example: The San Francisco Master Plan</u>

A recent comprehensive example of a cost effectiveness analysis applied to a combined sewer plan was presented by J. B. Gilbert and Associates in their review of the San Francisco Master Plan [8]. The development of the Master Plan itself had been a good example of systems engineering applied to the problem (See reference I-12).

In reviewing the Master Plan, Gilbert shows that the process San Francisco went through was indeed based on politically-set standards followed by a search for the minimum cost to meet the prescribed effectiveness. To demonstrate the process of cost effectiveness analysis used in this case, consider the following extracts from the Gilbert report:

1. Demonstration of political and regulatory basis for establishment of control objective:

"In the case of San Francisco, the State Water Resources Control Board and the Environmental Protection Agency have placed the following condition on approval of a grant for construction of waste collection and treatment facilities:

> 'The Municipality shall by April 1, 1973, submit a staged wet weather program whose aim shall be to eliminate the discharge of untreated wastewater to the aquatic environment...consideration of Stage I improvements shall include a thorough cost-effective evaluation of various alternative plans taking into account pollutant removal accomplishments, costs, impact on beneficial uses of receiving waters, and environmental impacts. A sufficient number of alternative Stage I improvement plans shall be compared and presented to display a broad range of investments...'"

"In addition, the following provision of the 1972 Federal Water Pollution Control Act bases grant eligibility for wastes containing storm waters on cost-effectiveness analysis:

> '...Treatment works' means any other method or system for preventing, abating, reducing, storing, treating, separating, or disposing of municipal waste, including storm water runoff, or industrial waste, including waste in combined storm water and sanitary sewer systems. Any application for construction grants which includes wholly or in part such methods or systems shall...contain adequate data and analysis demonstrating such proposal to be, over the life of such works, the most cost efficient alternative...'

A formal cost-effectiveness analysis provides assurance to governmental agencies and the public that funds are being invested in projects that will provide the maximum benefit.

The San Francisco Master Plan was developed, in part, in response to a requirement of the California Regional Water Quality Control Board specifying that the City must submit a plan to eliminate the bypassing of untreated wastewater. This requirement raises numerous questions related to project cost-effectiveness."

2. Translation of goals into operational objectives

"Complete elimination of bypass from a system in which flows are dependent upon rainfall is an unrealistic goal. Hydrologic events occur randomly and when presented as a frequency distribution illustrate the declining necessity and increased cost per benefit for facilities designed to control rare events. In most cases, the cost-effect ratio for controlling such infrequent events is unfavorable. While elimination of bypass may be impractical, elimination of deleterious effects from bypasses is a goal which should be diligently pursued. Unfortunately, the level of control for degree of bypass reduction, at which deleterious effects cease or become insignificant is not easily defined. Definition of the desirable level of control thus becomes the aim of the cost-effectiveness analysis."

3. Formulation of alternatives

"Numerous alternate approaches could be used to control wastewaters from San Francisco. However, many can be eliminated in cursory comparisons because of inconsistencies with regulatory requirements or obviously inferior cost-effect relationships. The Master Plan Report eliminates several alternatives in this manner.

Factors or restrictions leading to early elimination of several alternative facility configurations include:

- Secondary treatment of dry weather flows required by the Federal Water Pollution Control Act of 1972.

- Extremely high project costs.

- Extreme disruption of normal urban activity.

- Economics of scale favoring construction of a small number of regional facilities over a large number of local facilities.

- Operation and maintenance problems associated with dispersed wet weather flow treatment facilities.

- Land use and value restrictions.

- Better dispersion of pollutants by ocean discharge as opposed to bay discharge.

- Spatial and temporal variation of rainfall on San Francisco.

- Pollution potential of urban stormwater runoff.

- Dense population of the San Francisco Peninsula.

The first alternative normally considered for all combined systems in recent years has been sewer separation. This can be quickly eliminated because of extremely high costs (in excess of $3 billion), excessive disruption of normal urban activity, and the water quality problems that may result from separate stormwater discharge in the highly urbanized San Francisco area. After elimination of this option, alternate means of controlling combined sewer flows can be considered. Although numerous facility combinations are possible, some basic approaches are definable. Since combined sewage contains large quantities of pollutants, some treatment process is a basic requirement of all options.

One approach is to install treatment devices at each of the 41 bypass locations or at a smaller number of consolidated bypasses. This alternative has been eliminated due to its high costs, severe operation and maintenance problems from the many treatment plants, and the fact that bay discharge would continue.

Expanding existing facilities to handle wet weather flows is unacceptable due to land use conflicts at the North Point and Richmond-Sunset treatment plants, inability to take full advantage of temporal and spatial rainfall variation, and continued bay discharge.

To eliminate bay discharge, some type of transport system must be constructed. The need for such a facility, the economy of scale, and spatial and temporal variation of rainfall favor a single large treatment facility for control of wet weather flows. However, the size and operational problems of a treatment facility required to handle any significant storm flow are prohibitive.

An alternate approach to treatment as a means of control is storage. Sufficient storage to allow utilization of the excess capacity of existing treatment facilities would require space not available in San Francisco without excessive disruption of normal urban activity.

The obvious solution is some combination of storage and treatment capacity. Central storage receiving flow from the entire area would take best advantage of rainfall variation. However, drainage in San Francisco is outward from the center, which reduces the feasibility of this approach. To take full advantage of capacity, storage should have the largest possible tributary area.

From this point, the Master Plan concept was developed:

- One wet weather flow treatment plant located at Lake Merced.
- Shoreline storage basins for the fifteen major drainage basins (upstream basins have been utilized to reduce the inadequacy of sewers and control flooding).
- A crosstown transport system.

Alternative combinations and staging programs for these basic facilities are the subject of the cost-effectiveness analysis."

4. Measurement of effectiveness

"The first step in analyzing the effectiveness of alternative combinations and staging programs is to define methods of measuring and presenting project effectiveness.

A definite relationship exists between water quality control equipment or facilities, the level of control attainable from operation of those facilities, and the benefits resulting from the available level of control. Facilities are precisely definable in all respects. The level of control attainable with the facilities generally can be determined in most respects. Benefits accrued from construction and operation of facilities or a given level of control are the most difficult to precisely define.

In some cases a level of control may be a direct benefit. Such is not normally true in water quality projects. Control parameters such as removal efficiency, effluent quality, and mass emission rates are not direct measures of benefit. Attempts to relate these parameters to benefits, that is, improvement or protection of the receiving waters, have been imprecise at best.

For combined sewage and stormwater overflows, control facilities perform the following functions:

- Reduce untreated bypass

- Remove pollutants from waste stream

- Divert flows to more desirable discharge locations

- Reduce street flooding

The degree of control is directly related to facility design.

Control related parameters commonly used as measures of project effectiveness for water quality control are:

- Effluent bacteriological quality

- Mass emission rates

- Bypass frequency and quantity

- Initial dilution of effluent with receiving water

- Location of discharge

- Frequency of street flooding

Benefits which can be accrued from control are:

- Reduction of health hazard to swimmers and recreationists

- Improved aesthetics of water bodies and shoreline areas

- Protection of the aquatic environment

- Reduction of the numerous problems associated with street flooding

The degree of benefit is related to level of control, receiving water condition, public exposure to wastes, public use of the waters, value of the aquatic environment, and other factors.

Some benefit-related parameters which can be used as a measure of project effectiveness for water quality control are:

- Days per year aesthetics are impaired

- Miles of contaminated or polluted beaches

- Days per year water is safe for swimming
- Number of recreationists adversely affected
- Receiving water quality for physical and chemical parameters
- Diversity, number, and health of aquatic species

The optimum cost-effectiveness comparison results when effectiveness can be quantified. To obtain sufficient data for comparison, control parameters are normally used. Benefits are difficult to quantify and frequently the data obtained are based on qualitative observations.

Substantial information is available concerning the degree of control from various hydraulic and wastewater treatment facilities. The major difficulty in applying benefit parameters as a measure of effectiveness results from the predictive nature of cost-effect analyses. Quantifying benefit parameters from measured or observed data is difficult but numerically defining the benefit resulting from various facility configurations is much more complex.

In view of the differences in availability of information, control parameters are used in this analysis to quantify effectiveness while benefit parameters will be used only as qualitative or general indicators."

C.2 Another Example: The Chicago Underflow Plan [3]

Another recent, large-scale plan for solving pollution and flooding problems in a large city with combined sewers is the Chicago *underflow plan*. This plan is also geared to meet politically set standards with the cost-effectiveness study being limited to a search for the least cost plan to meet the standards subject to a set of constraints.

The Underflow Plan is a composite of a number of alternatives developed through the years. It is designed to capture the runoff from all but the most severe storms as recorded during the rainfall periods of 1949-69 with ultimate land use assumed. By capturing the runoff as described the prescribed objectives of the study are to be met.

The cost for the plan is $1223 million (1972 costs) with other pollution control costs needed estimated at $1430 million for a total cost of $2653 million. If inflated at six percent over a ten-year construction period, the ultimate cost is $3301 million.

In establishing objectives for the plan the evaluation team used the following:

> "The criteria established for the comparison of system plans are as follows:
>
> 1. Prevention of backflow to Lake Michigan for all storms of record.
>
> 2. Meet the applicable waterway standards established by the State Pollution Control Board and the Metropolitan Sanitary District of Greater Chicago.
>
> The criterion of *no backflow to the Lake* derives from two considerations, as follows:
>
> a. It is the announced policy of the Metropolitan Sanitary District and the City of Chicago to strive for the elimination of all discharges of wastewater to the Lake, so as to protect the public water supply, maintain the Lake as a great natural resource and guarantee its continued use as an economic and recreational asset to the Metropolitan area and all other areas adjoining it.
>
> b. The water quality standards for the discharge of treated effluents into the Lake are more stringent than those for the discharge of treated effluents to restricted waters, therefore, backflow should be eliminated, so as not to violate these standards.
>
> The adoption of the *no backflow* criterion is consistent with water quality standards and national goals for the preservation of the water quality in Lake Michigan." [3]

The measures of effectiveness adopted were based on cost, subject to the constraint that the plan meet environmental acceptance standards.

The formulation of alternatives was rather complex because so many alternatives had been proposed and because they met differing levels of service standards. The evaluation team solved this problem by modifying the alternatives so that they all met substantially the same levels of service.

The alternative selected, and the reasons for selection, are described as follows:

> "An interim report entitled *Evaluation Report of Alternative Systems* was prepared by the Technical Advisory Committee covering much of the preceding sections. After extensive review of that report, the Flood Control Coordinating Committee unanimously agreed on the following course of action:
>
> > 'The final plan for Flood and Pollution Control in the study area should be in the form of the *Chicago Underflow Plan* (Alternatives *G*, *H*, *J* and *S*) with the Mod 3 level of storage. These Alternatives are less costly and would be more environmentally acceptable to the community than any of the other plans presented. Detail studies along the lines of these Alternatives should proceed to develop the final plan layout.'" [3]

It has been conclusively demonstrated in the interim report that the Mod 3 level of storage (54,000 acre-feet of reservoir storage) should be adopted for the study area. Some of the reasons for this conclusion are as follows:

"1. It will provide flood protection for the recurrence of the heaviest storms of record without the need of releasing flood waters to Lake Michigan.

2. It will capture the combined sewer overflows and provide subsequent treatment of this water before discharging to the waterways, for all but the largest storms of record. Exhibit 8, indicates that a reduction of 99.7 percent of the BOD, which currently is discharged through the existing combined sewer outlets, would be captured and treated.

3. The *Mod 3* system with ultimate land use would overflow a substantial quantity of water only during a recurrence

of the three storm periods, see bottom of Exhibit 8; these three storms, October 1954, July 1957 and September 1961, are the heaviest ever recorded for their respective durations. Because the first major portion of these storms would be captured by the 54,000 acre-feet of storage, overflows would be relatively clean and not objectionable.

4. The prevalent dry weather dissolved oxygen (DO), temperature during the summer, and ammonia-nitrogen levels would not be conducive to game fish life in the restricted watercourses. Warm water fish and associated biota, however, would not be greatly affected by the short dips in DO during the infrequent overflow events. Consequently, fish kill would not be a factor during the rare overflow events with the *Mod 3* level storage.

5. The *Mod 3* level of reservoir storage is considerably less costly than either the *Mod 2* (125,000 AF of reservoir storage) or the *Mod 4* (24,000 AF of reservoir storage with channel improvements). For example, as shown in Exhibit 12: Alternative H, Mod 3 is $645 million less than H, Mod 2; and $444 million less than H, Mod 4. Since all Alternatives having the *Mod 3* level of storage will provide about the same degree of environmental benefits with regard to improving waterway quality, it appears that those Alternatives which cause the least disruption to the urban community and provide for the minimum relocation of people, as well as having the least cost, should be adopted.

Exhibit 12, shows that the *Chicago Underflow Plan* either in the form of Alternatives *G*, *H*, *J* and *S* with the Mod 3 level of storage ranges in cost from $912 million to $1,002 million (Present Worth) and $85 million to $90 million equivalent annual cost. These Alternatives are less costly than the others and would result in little disruption to the urban community. Practically no relocation of people will be required." [3]

This demonstrates that cost effectiveness analysis is still very much an *art* as applied to plans of this nature, not withstanding the fact that considerable quantitative analysis can be applied as part of a cost-effectiveness study.

A great deal of work needs to be done to relate storm and combined sewer regulations to water quality goals. The stage has been set for some careful demonstrations of cost effective solutions to this problem. Control systems offer a promising alternative for consideration.

D. COMPUTER AND CONTROL EQUIPMENT

In the earlier phases of the MWIS study, attention was directed to the types of computer and control equipment which might be adapted to MWIS applications. References [9,16] reflect the results of these investigations. Hardware questions really must be solved on a case-by-case basis. There exists an abundant supply of *guideline* type books for implementing automation efforts and this type of information would not be appropriate here. The valuable *guidelines* are contained in the experience of process control engineers and a number of urban water managers who have grappled with hardware questions.

Technological change is a phenomena that cannot be ignored; it augers a trend toward increased automation of processes of all types.

including urban water systems. It also makes it difficult to specify and select equipment in this dynamic industry. The manager interested in implementing an MWIS must therefore exercise caution in selection of vendors, consultants and operating staff. He should expect the hardware/software implementation phase to be difficult, expensive and filled with unknowns and he should be prepared to ask difficult questions about verification, reliability, redundancy and back-up, service requirements and related items.

D.1 The Trend to Automation

Business and industry are exploding with examples of automatic control and automated information processing. The technology is available and new control applications are reported frequently. Some of the most advanced applications may be closely held by industry and not appear in the literature for years.

The literature abounds with descriptions of new computer applications. A recent roundup article appeared in Business Week magazine [14] which might be of interest to readers. Another description of trends in business which portends trends in urban government is given below in another quote from Business Week magazine [17]. This information summarizes many of the technological trends that will accelerate the pressure for implementation of MWIS.

> "A 20th Century revolution in business operations, teleprocessing could be as important to today's management as replacing the pony express with long distance telegraph. Today's revolution stems from the marriage of computers and communications.
>
> Teleprocessing provides the means for today's managers to keep split-second control over their increasingly complex national and international operations, indeed, the teleprocessing revolution represents a new way of doing business in which...

1. Computer power can be extended to the end of any communications line;

2. Data can be captured as it originates within an organization;

3. Widely dispersed information can be quickly accessed;

4. Production can be more easily geared to match inventory levels and incoming orders;

5. Customers get faster, more responsive service;

6. Management has better control over cash, resources and people." [17]

"Today's supercomputers are designed with data communications and the management of large data bases in mind. And at the other end of the EDP spectrum, the plunge in minicomputer prices is boosting performance per dollar for computer networks as a new breed of remote, minibased *intelligent* terminals moves computing power out to the user."...

"Top management--after years of encouragement and premature promises from suppliers--is beginning to realize and acutally accept the fact that a properly designed teleprocessing network gives a company tremendous competitive advantages in terms of better customer service and operating efficiency. To forward-looking management, the on-line computer network means:

--Better information flow...
--Cost savings...
--More scientific management"

There are trends in the *staging* of business information and control systems which appear the same as we observe in MWIS applications:

"First, the company or organization keeps its scattered computers, but reprograms existing batch applications for operation in a communications mode, preparing for later equipment concolidation...

Next, more computer power is moved into a central location and small computers or even intelligent terminals replace larger machines at other computer sites. More terminals are added, and cost-saving applications--such as inventory control--are put on the system. Concurrently, an in-house timesharing system may be introduced to cut spending on outside services.

In the third phase, the enormous data bases built up during earlier phases finally begin to provide a foundation for simulation and modeling techniques--tools for more knowledgeable decision making. In the network configuration, intelligent *front-ends* or controllers relieve the central computer of some *housekeeping* tasks, and gradually the entire business operation becomes dependent on the teleprocessing network.

This transition takes time, often five years or more, but is hard to stop once the commitment is made. Thus far, only the airline industry is moving solidly into *phase three* networks. However, many utility companies, several consumer goods suppliers such as Gillette and Coca-Cola, cumputer giant IBM, and a handful of consumer loan organizations are definitely entering this sophisticated level of development. In other industries..." [14]

In 1971, Poertner (I-4) surveyed a number of urban water utilities to determine the stage of automation they were at. His results were broken down into the following groups, by level of automation.

1. Data logging and analysis
2. Data processing and reduction
3. Conventional supervisory control
4. Automation of parts of systems
5. Computer control

Generally, his survey showed different units at different points of the automation spectrum. This survey has not been updated for 1974 but trends would suggest these developments as it were:

1. Much greater *interest* in automation at the user level.
2. Large *technological advances* in the automation equipment available from vendors.
3. Increased *technological expertise* in user organizations--in particular, engineers from aerospace firms may be finding their way into water agencies.
4. *Centers* of automation experts being formed in different water agencies around individuals who have taken automation projects *under their wings*.

5. Increased *consideration* of automation projects because of changing environmental standards.

6. A *maturing* of viewpoint among early advocates of automation in water agencies, reflecting their operating experience.

The levels and stages of automation projects were previously mentioned. Concerning this there appears to be a trend toward decentralized control with minicomputers as well as toward staged adoption of systems step-by-step. This latter viewpoint is expressed from the vendor's viewpoint by the Honeywell Corporation. [10]

> "It is far more beneficial to first identify and classify distinct levels of control within a plant, to study these levels in depth and then to develop cost-optimized standard application packages using minicomputers dedicated to unit and process operations.
>
> We first identify the four distinct levels of control within a plant. Dedicated packaged systems are then first applied to the lower levels.
>
> 1. Individual loop control (analog or digital) for control of operating VARIABLES such as temperature, pressure, flow, etc.
>
> 2. Multiloop systems for control of specific PROCESSING UNITS that involve several related variables, i.e., distillation columns, compressor systems, reaction systems, activated sludge process, electric arc furnaces.
>
> 3. Systems to control PROCESSING OPERATIONS where several related operations must be controlled to obtain desired performance, i.e., ethylene plants, crude units, catalytic crackers, sewage treatment plants, melt shops.
>
> 4. Plant-level systems which optimize over-all PLANT OPERATIONS, i.e., a refinery linear programming program that relates operation of the crude unit, the cat cracker and the reformer to over-all plant profitability." [10]

Development of Hierarchical Computer Control Systems

The trend toward development of hierarchical computer control systems answers the need for step-by-step rather than *one-shot* development of control systems. With the cost of minicomputers falling drastically, hardware is not a constraint on implementation of control systems.

Urban water systems, being public facilities which must operate at high levels of reliability, can afford to innovate, but not to the extent that private industry can. We should therefore look to industry for trends in computer control in order to anticipate techniques which can be adapted in utilities. Concerning the development in hierarchical computer systems, some industrial trends are discussed below [14].

> "Out of this has come a new concept of the automated factory called the *hierarchical* approach. This minis, programmed for simple, single tasks, direct the machines on the factory floor. And as they do, they feed information on what they are doing to successively higher levels of computers. These higher-level machines compile and analyze the data they get and provide factory managers with the information they need on output, costs, and so on. But if one of the bigger machines fails, the factory does not shut down. The minis go on about their work..."
>
> "As minicomputer prices decline--and they have been dropping about 20% a year since the 1970s began--the small machines can economically handle more and more of the tasks on the factory floor."...
>
> "As they work, they generate a lot of information: How many good or defective parts were made, how many parts were used to make them, how much raw material was used, how much inventory is stored in the warehouse. The information is the key to the hierarchical factory computing system, for the minicomputer can serve not just as a controller but as a communications channel to and from higher-level supervisory computers."
>
> "Such a hierarchical system would come close to being the long-discussed automated factory. Indeed, a growing number of plant designers believe that this evolutionary, building-block approach is the only feasible way to achieve total plant automation.

"As yet, no company has gone this far in linking together its computerized operations."...

"Most of those that are working on automation systems are moving deliberately and cautiously... GE's plan calls for a hierarchy starting at the top with the Appliance Group's central business computer and working down through plant-level computers to the minicomputers that run production, testing, and storage.

"But GE is at least three years away from linking its computer systems together."

D.2 Equipment Needs for MWIS

Hardware needs for combined sewer MWIS can be classified into generally two groups. The first group is associated with the physical system, the sensors, and the control elements required. The second group is associated with the computer center and the need to process and to utilize information in order to develop the proper control strategies. The latter group of hardware needs will be discussed in this section. Hardware which is electro-mechanical in nature is abundant in quantity and quality. Examples of these items are: pumps, weirs, valves, sensors, telemetry, gages and other control instrumentation. With some limitations, a device can be found to perform most data collection and control tasks needed by MWIS. Economic constraints are significant, however. An example where this constraint binds is the case of the sewer flowmeter which has been discussed elsewhere.

Computer hardware requirements are dependent on the control *strategy* adopted. The computer hardware limitations will strongly affect the sophistication and level of detail of these models.

The control model is the most significant factor determining what type of computer hardware will be required in the computer center. The control techniques available range from rather simple rule curves to more sophisticated on-line optimization techniques. Needless to say, the technique adopted will determine the computer storage and computation time requirements, these being the basic characteristics which will determine resulting computer costs. It appears at the present time that control can be done with computers as small as those in the minicomputer range, but with simple models, or control can be via the largest computers now available, obviously not a cost-effective arrangement. There must be some optimum point of computer size with a trade-off in model accuracy. This is a crucial problem in any type of process control.

Currently adopted computers are those falling into the category of *third generation* of process control computers. This includes machines similar to the GE 4020, the CDC 1700, the IBM 1800, and the SDS Sigma 2. [21] As examples, the Santa Clara Water and Flood Control District uses an IBM 1800, and the City of Seattle has an SDS Sigma 2.

The trend in industry to automation and the resultant need for direct physical control hardware will develop technology that can be transferred into the urban water industry. For example, transferable questions that have been asked in these applications address the following concerns:

1. Control valve actuation
2. Feedback control laws
3. Sampling
4. Input and output quantization
5. Type of digital computer
6. Interface equipment requirements
7. Operator communication

8. Computer utilization
9. Reliability
10. Economics

As another example, the requirements for computers used EDP applications in process control are related to the requirements for urban water applications. Note the following such requirements: [2]

1. The highest reliability possible with the existing state-of-the-art. Minimum requirements: an availability of 99.95 percent of possible plant operating time, or a maximum of 4 hr./yr. when the computer system is unavailable for normal palnt service.
2. An ability to convert input analog signals from the plant to an accuracy of at least 0.1 percent, i.e., to at least one part in 1,024 (ten binary bits).
3. A computational accuracy of at least 0.1 percent or one part in 1,024 (ten bits) throughout the computer control system.
4. Provision for direct data interchange with an optimizing or supervisory control computer to allow one to take advantage of a computer optimization of plant operations at minimum cost.
5. Provision for rapid and easy communication with plant operating personnel through an associated operator's console or panel.
6. Provision for a rapid, automatic switchover to some *locked* or manual standby method of plant control in the event of a computer operating failure.

7. A maximum accessibility of components and ease of trouble-shooting to minimize required maintenance time in the event of a computer failure.

Other questions which are pertinent to both process control and the MWIS are related to sampling frequencies and to stroking speeds for control valves.

Economics plays an important part in the implementation of automatic control and the process control industry has determined that the sophisticated and advanced control algorithms which are necessary to implement CDC, are the most expensive part of an automated system. This difficutly must especially be overcome in urban water application because the number of such applications is many-fold and the financial capabilities of water utilities do not rival their industrial partners to this date.

CHAPTER II

REFERENCES

1. American Public Works Association, "Problems of Combined Sewer Facilities and Overflows - 1967," Report for the USEPA, December, 1967.

2. Bernard and Cashin in "DDC in Process Control - A Review," Proceedings of the ISA, 1969.

3. Chicago Flood Control Coordinating Committee, "Summary of Technical Reports - Development of a Flood and Pollution Control Plan for the Chicagoland Area," August, 1972.

4. English, J. M., "Introduction," Cost Effectiveness, J. M. English, (ed.), John Wiley and Sons, New York, 1968.

5. Field, R., "Management and Control of Combined Sewer Overflows - Program Overview," paper presented at 44th Annual Meeting of New York WPCA, New York City, January, 1972.

6. Forrester, J. W., "Counterintuitive Behavior of Social Systems," Technology Review, January, 1971.

7. Freeman, A. M. III, R. H. Haveman, and A. V. Kneese, The Economics of Environmental Policy, John Wiley and Sons, New York, 1973.

8. Gilbert, J. B. and Associates, and Metcalf and Eddy, Inc., "Evaluation - San Francisco Wastewater Master Plan," March, 1973.

9. Grigg, N. S., et.al., "Metropolitan Water Intelligence Systems, Completion Report - Phase II," Colorado State University, Fort Collins, Colorado, June, 1973.

10. Honeywell, Inc., "Computer Control Systems Capabilities," Minneapolis, 1972.

11. Kazanowski, A. D., "A Standardized Approach to Cost-Effectiveness Evaluations," Cost Effectiveness, J. M. English (ed.), John Wiley and Sons, New York, 1968.

12. Kraemer, K. L., "Integrated Municipal Information Needs: The USAC Approach," Long Island University, Natick, Massachusetts, January, 1972.

13. Lager, J. A. and W. G. Smith, "Urban Stormwater Management and Technology: An Assessment," Report for the USEPA, December, 1973.

14. "Minicomputers that Run the Factory," Business Week, December 8, 1973.

15. Murdick, R. G. and J. E. Ross, Information Systems for Modern Management, Prentice-Hall, 1971.

16. Smith, G. L., et.al., Metropolitan Water Intelligence Systems - Completion Report, Phase I, Colorado State University, Fort Collins, Colorado, June, 1972.

17. "Teleprocessing," A special advertising section of Business Week, February 16, 1974.

18. University of Southern California, "Municipal Information Systems - The State of the Art in 1970.

19. U. S. National Water Commission, Water Policies for the Future, USGPO, Washington, D. C., 1973.

20. U. S. Congress, "Federal Water Pollution Control Act Amendments of 1972," Washington, D. C., 1972.

21. Williams, T. in "DDC in Process Control - A Review," Proceedings of the ISA, 1969.

CHAPTER III

HIERARCHICAL CONTROL

A. INTRODUCTION

A.1 Optimal Automated Control

The focus in this Chapter is on presenting an overall strategy for dealing with the automated control problem associated with the proposed San Francisco Master Plan for Wastewater Management. It should be emphasized, however, that the concepts developed here have general applicability to other cities interested in utilizing the automated control approach to wastewater management.

Development of automated control strategy for the proposed San Francisco Master Plan is a complex, large-scale problem. Figure III-1 shows the large number of detention reservoirs associated with the Master Plan, where the goal is to effectively utilize this storage capacity in order to control overflows. The complexity of the control problem is evident when considering that, potentially, each reservoir has remotely controllable valves and, in many cases, pumps that must be properly coordinated. This coordination is carried out in such a way that storm flows can be detained in the reservoirs so that pollution-causing overflows at the bypass points in Figure III-1 are minimized for any given storm event, while satisfying capacity constraints associated with the trunk sewers, interceptors, and the treatment plant. Optimization methods, such as linear programming, provide a systematic means of determining the *best* way to operate and coordinate all of the control elements.

In applying optimization techniques to the control problems, mathematical models are utilized to simulate the behavior of the system.

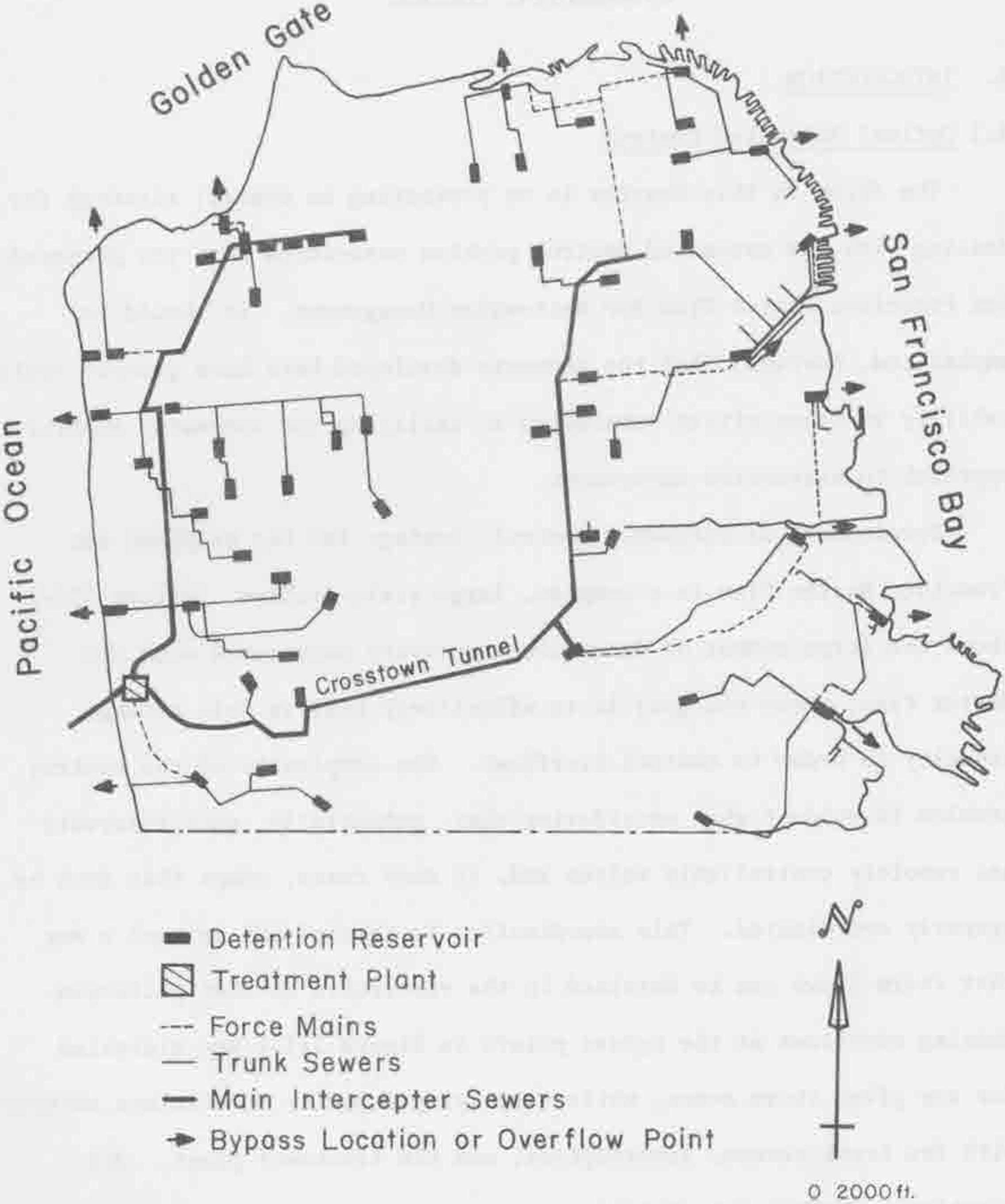


FIGURE III-1

PROPOSED SAN FRANCISCO MASTER PLAN [3]

The basic components are (i) a model transforming rainfall into runoff, (ii) models for routing flows through the sewer system and detention reservoirs, and (iii) models predicting temporal and spatial water quality variations throughout the combined sewer system. The emphasis in this study has been primarily on (i) and (ii), where the goal is to minimize total overflows without regard to their variability in polluting effects. A mechanism has been established, however, for incorporating (iii) as these models become available. This is based on the use of weighting factors associated with temporal and spatial quality variations in overflows and localized street flooding. For example, bypass points with a history of overflows with high concentrations of pollutants would be penalized more heavily than less polluting overflows at other bypass points. Another example would be quality variance as a function of time. Due to initial *flushing* effects, overflows occurring early in a storm would be given a higher weight. The objective then is to minimize total *weighted* overflows rather than just total overflows.

The control problem is further complicated by the fact that, for real-time control, important aspects of the storm in progress must be spatially and temporally predicted. What these aspects are will depend to a large extent on the control strategy used. In general, accuracy of control is only as good as the prediction accuracy. It should be emphasized that little has been done in this kind of short-term storm prediction. Chapter VII gives a comprehensive review of the current state-of-the-art of storm modeling, with some insight into the importance of accurate prediction.

A.2 Large-Scale Control Problem

Consider the following discrete-time formulation of the optimal control problem:

Given: A predicted, historical, or synthetically generated storm event, or series of events, defined spatially and temporally.

Minimize:
$$\sum_{i=1}^{N} \sum_{k=1}^{M} \omega^{i}(k) O^{i}(k) \tag{1}$$

Subject to:

1. Dynamic and mass balance equations describing the transport and storage of combined sewage throughout the sewer system.
2. Capacity constraints on detention storage, trunk sewer and interceptor flow, and treatment plant input.

By: Proper operation of valves and pumps in the sewer system at discrete-time intervals.

Where: $O^{i}(k)$ = the overflow occurring at bypass point i during some discrete-time period k.

$\omega^{i}(k)$ = weighting factors

N = the total number of bypass points

M = the total number of discrete-time periods.

The optimal control problem is illustrated in Figure III-2. The input to the system is some given storm event which may either be a predicted or tracked storm in real-time or a historical or synthetically

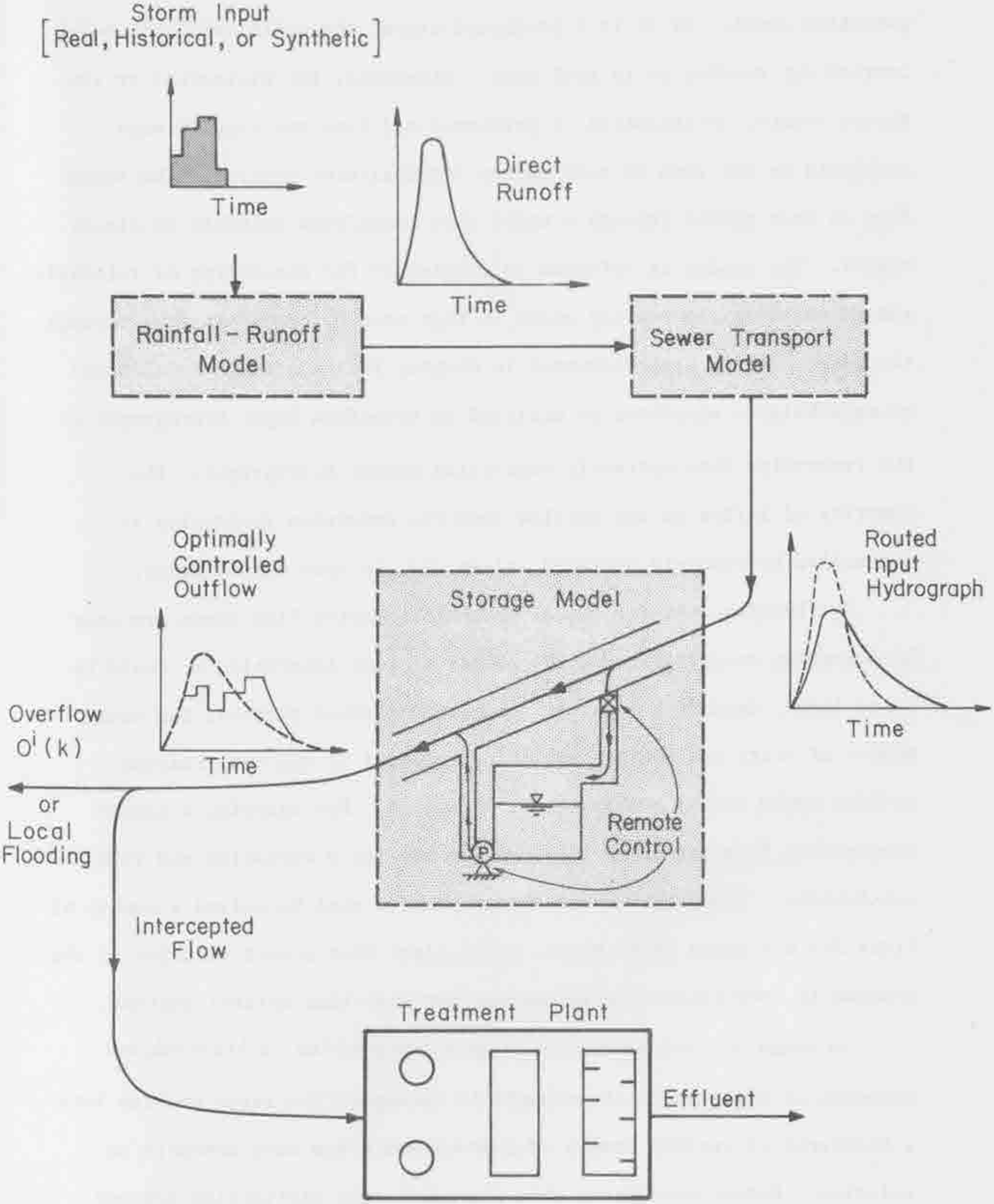


FIGURE III-2

COMPONENTS OF THE OPTIMAL CONTROL PROBLEM generated event. If it is a predicted storm, the optimization is being carried out *on-line* or in real-time. Otherwise, for historical or synthetic events, optimization if performed *off-line* and results made available in the form of rule curves for real-time control. The stormflow is then passed through a model that transforms rainfall to direct runoff. The reader is referred to Chapter IV for discussion of rainfall-runoff modeling. A routing model is then used to transport flow through the sewer system, also discussed in Chapter IV. A storage model based on mass-balance equations is utilized to transform input hydrographs to the reservoirs into optimally controlled output hydrographs. The quantity of inflow to and outflow from the detention reservoirs is controlled by remotely operated valves and, in some cases, pumps.

Considering that for the San Francisco Master Plan there are over 50 detention reservoirs, and the number of time intervals M could be quite large, depending on storm duration and other factors, the total number of state and control variables involved in the optimization problem could easily number in the thousands. For example, a linear programming formulation of this problem has 108 M variables and 78 M constraints. Since this optimization problem must be solved a number of times for any given storm event, it is clear that direct solution of the problem is computationally infeasible for real-time optimal control.

In order to deal with this large-scale problem, a hierarchical approach is utilized which attempts to decompose the large problem into a hierarchy of various levels of control which are more amenable to solution. Before discussing this framework, the distinction between off-line and on-line optimization must be clearly established.

B. OFF-LINE VS. ON-LINE OPTIMIZATION

Having formulated the large-scale optimal control problem in very general terms, the question now arises as to whether optimization should be off-line, on-line, or a combination of the two. As stated previously, off-line optimization refers to optimization carried out independently of the actual real-time control situation. Many optimizations are performed for an assumed range of most probable storm events that could occur, based on historical and synthetically generated events. The resulting optimal strategies are then stored in the on-line computer system for real-time control. As a current storm is sensed in the field, a prediction of the rest of the storm is carried out. That particular stored optimal control strategy, computed from an assumed storm closest to the prediction of the actual storm in progress, is then retrieved and applied to the field control elements. The process continues with predictions updated as more data become available. This off-line optimization format is illustrated in Figure III-3.

On-line optimization means that the optimization is carried out in real-time on the on-line control computer as a storm is progressing. This approach is also illustrated in Figure III-3. It is difficult to generalize as to which method is preferable. There are strengths and weaknesses associated with each. It can be said, however, that off-line optimization is more amenable to use of realistic models of the sewer system. The resulting complex optimization can be performed off-line. without the tight constraints of time associated with on-line optimization.

For on-line optimization, the computations must be completed within the specified control time interval. If the optimization problem is

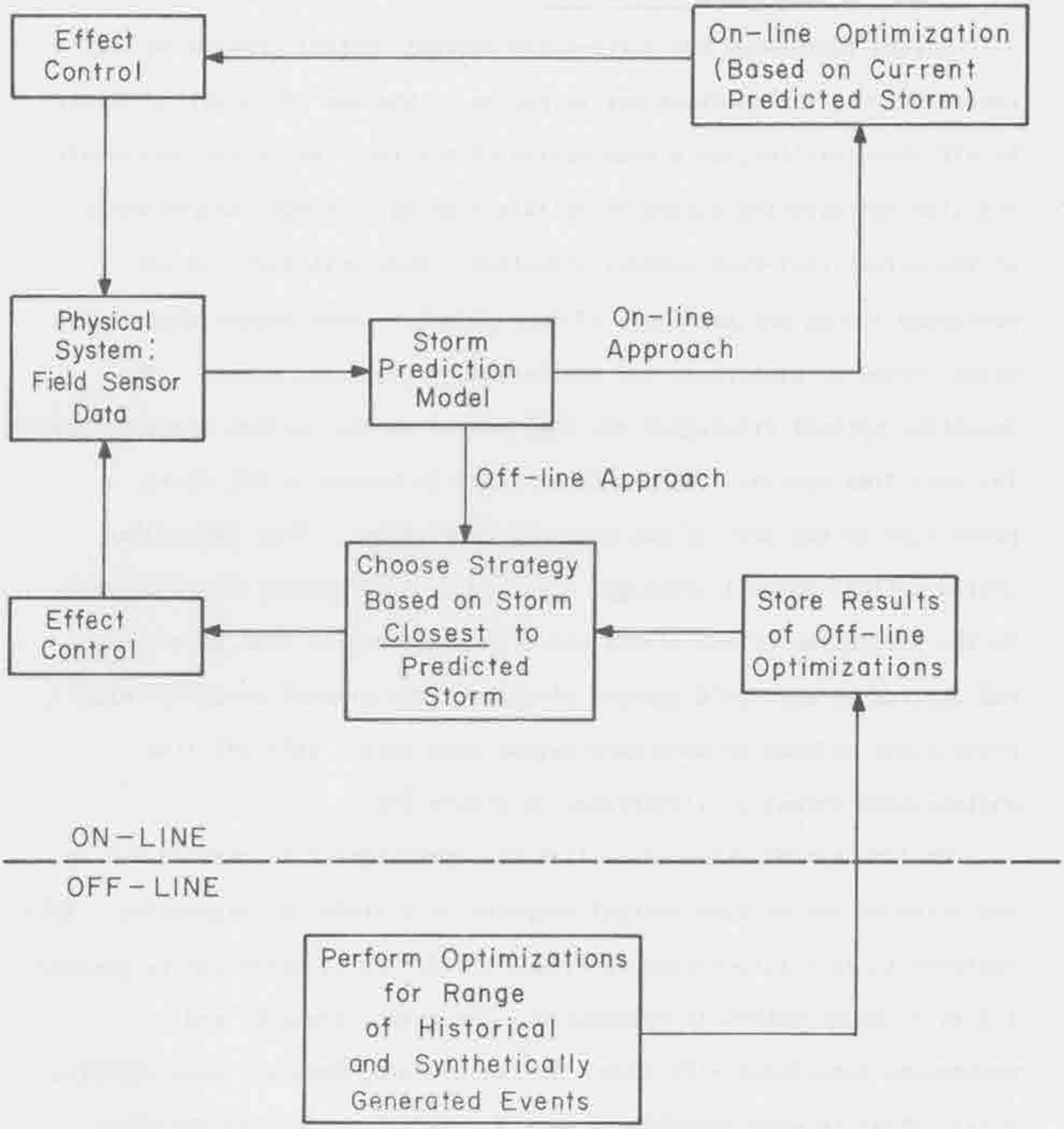


FIGURE III-3

OFF-LINE AND ON-LINE OPTIMIZATION

too complex, or the optimization algorithm excessively time consuming, or both, then solutions may not be reached within the time allotted. Hence, simpler optimization problems, and therefore simpler sewer models (e.g., linear transport models) as well as reliable and efficient optimization algorithms, are a must for effective on-line optimization. Since sewer model development is constrained in this way, there is question as to the accuracy of the on-line optimization, due to its dependence on the accuracy of the system models. Model simplicity is usually related to its degree of nonlinearity. If linear or quasi-linear models are adequate, totally on-line optimization is probably viable. Otherwise, at least some off-line work may be necessary.

An advantage of the on-line approach is that optimizations are performed on the basis of the current predicted storm event only. For off-line work, optimizations must be carried out for a wide range of possible events that could occur. The result may be a massive computational burden.

The best approach may be a combination of off-line and on-line optimization that takes advantage of the strengths of each method, while diminishing the weaknesses. Based on the above discussion, the following trade-offs can be listed:

1. Off-line vs. on-line optimization
2. Simplified vs. realistic mathematical system models.
3. Analysis of trade-offs in optimization codes according to their sophistication, reliability, applicability, and efficiency.
4. Simplified vs. complex storm prediction models.

C. HIERARCHICAL FRAMEWORK FOR CONTROL

C.1 Decomposition of the Large-Scale Problem

Though the large-scale optimal control problem previously discussed is impractical as it stands for use in real-time control, attempts can be made to decompose it into smaller control problems coordinated through a hierarchical control structure. For the San Francisco Master Plan, this can be accomplished by dividing the city sewer system into tributary areas referred to as *subbasins*. Figure III-4 depicts the boundaries of these subbasins for a portion of the Richmond-Sunset area on the west side of the city. This area is isolated in order to more effectively illustrate the use of the hierarchical approach. The boundaries are defined such that all rainfall falling within them is tributary to the trunk sewers contained in the subbasin.

The goal is to be able to develop optimal control strategies for each of the subbasins independently, and recombine them together in such a way as to achieve an overall optimum strategy. Thus, the computationally infeasible large-scale problem is replaced by several smaller problems which are more readily solvable.

The advantages of this kind of decomposition can be listed as follows:

1. Greater conceptual grasp of the complex problem is attainable through attempting to define its component parts and their interconnections.
2. Since the control strategies for each of the subbasins are developed independently, special structure of the subbasin problems can be exploited.

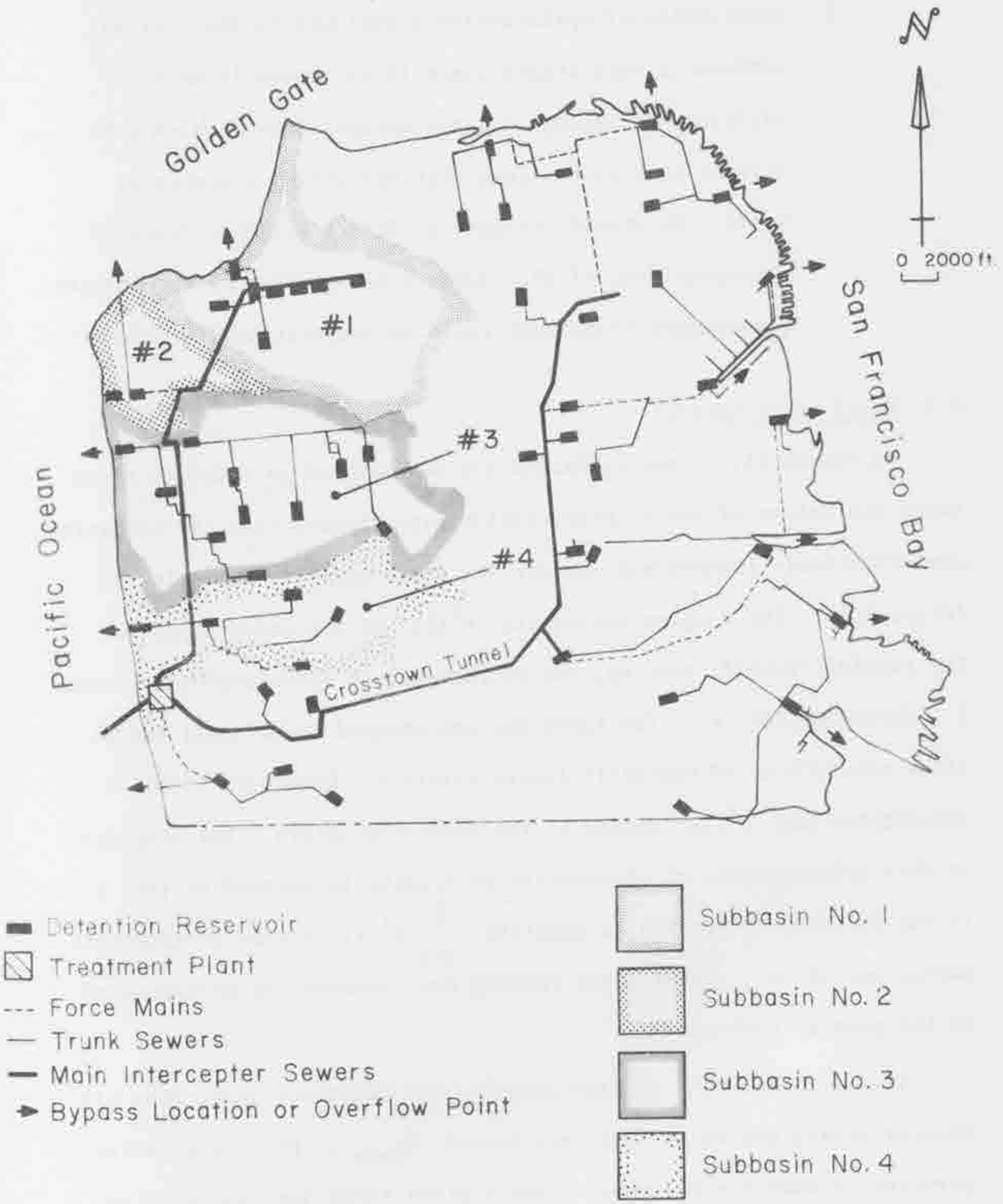


FIGURE III-4

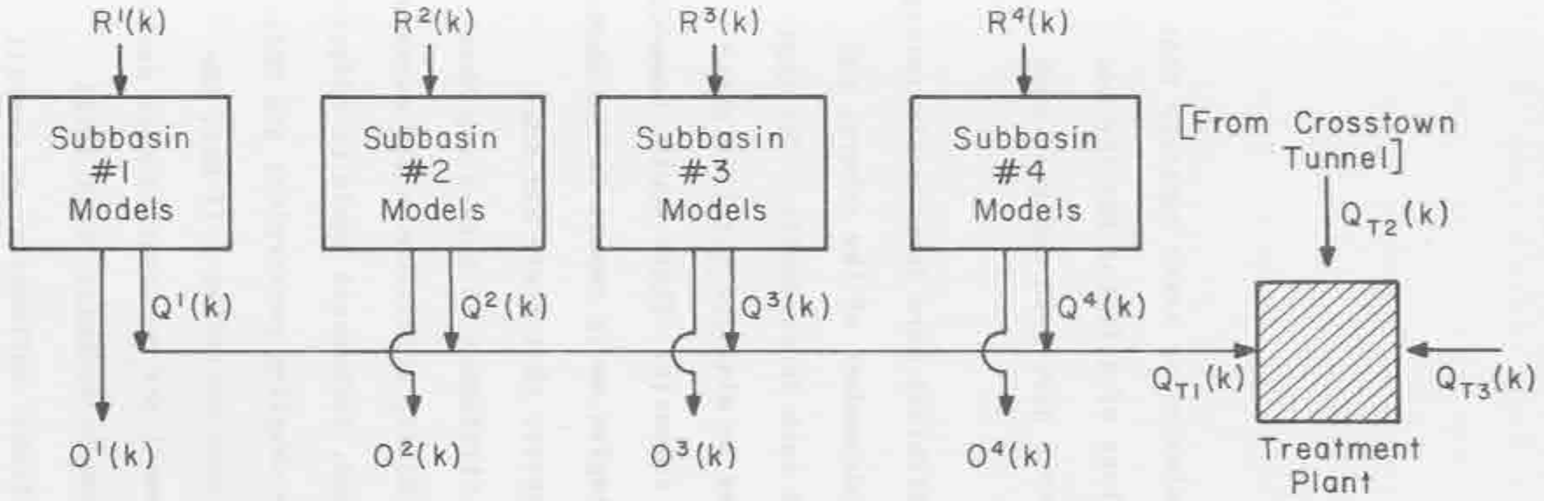
SUBBASINS IN THE RICHMOND-SUNSET AREA OF SAN FRANCISCO [3]

3. Application of optimization techniques to the control problem is facilitated since it is generally more efficient to replace a large optimization problem with several smaller problems that are solved a number of times. As stated previously, direct solution (without decomposition) of this large-scale problem by mathematical programming techniques would be computationally infeasible.

C.2 Multi-Level Control

In Figure III-5, the subbasins are schematized in order to accentuate the nature of their interrelationship. Notice that the subbasins are effectively independent, except for their parallel input to the interceptor. The diagram represents $R^i(k)$ as the storm input to the rainfall-runoff, routing, and storage models associated with subbasin i, during period k. The subbasins are assumed to be small enough to allow assumptions of spatially lumped rainfall. Quantity $Q^i(k)$ is intercepted and $O^i(k)$ passes to the receiving waters. For simplicity in this illustration, no attenuation or lagging is assumed to take place in the interceptor, so that a quantity $\sum_{i=1}^{N} Q^i(k)$ passes to treatment during period k. Interceptor routing can, however, be incorporated in the general problem.

The total quantity of flow entering the treatment plant from all sources during period k must not exceed Q_{Tmax}. For illustrative purposes, assume for the moment that a given storm has concentrated on the Richmond-Sunset area, with little or no rainfall on other portions



$$\sum_{i=1}^{4} Q^{i}(k) = Q_{T1}(k) \qquad \text{[Assuming no Routing in the Sewer]}$$

$$Q_{T1}(k) + Q_{T2}(k) + Q_{T3}(k) \leq Q_{TMAX}$$

FIGURE III-5

SCHEMATIC ILLUSTRATION OF SUBBASIN INTERACTION of the city. The constraints that bind the subbasins together are therefore

$$\sum_{i=1}^{4} Q^{i}(k) \leq Q_{Tmax} \qquad \text{for all } k=1,\ldots,M \tag{2}$$

There may be additional constraints on interceptor sewer capacity that also couple the subproblems. These are less of a factor for the San Francisco Master Plan since the interceptors have been designed such that (2) is the major constraint.

Suppose that the above coupling constraints were temporarily relaxed. Each subbasin would then be completely independent of the others, and control strategies could be developed for each individually. The individual subbasin objectives would simply be to minimize total weighted overflows contributed by that particular subbasin. There would, however, be no guarantee that the individual strategies would result in the above constraint being satisfied. Suppose, however, that there was some mechanism for influencing the individual strategies in such a way that the coupling constraints and overall optimality conditions would eventually be satisfied. In an iterative fashion then, influences would be effected on each subbasin control strategy and the coupling constraints and optimality conditions checked to see if they were satisfied. If not, the process would be repeated several times until the constraints were satisfied. Since each of the subbasins continue to minimize their total weighted overflows, subject to these additional influences, an overall optimum would be attained when the binding constraints and optimality conditions are eventually satisfied.

This kind of iterative strategy is illustrated in Figure III-6 as a hierarchical control structure. The first level of the hierarchy represents the individual subbasin optimizations, which are influenced by a second level Master Problem whose major purpose is to make sure that the coupling constraints and overall optimality conditions are eventually satisfied. In subsequent sections, this hierarchical structure will be developed in greater detail for application to totally on-line optimization and a combination off-line/on-line optimization. The reader is referred to Lasdon [8], Wismer [10], and Mesarovic, et.al. [9] for detailed discussion of the hierarchical approach. Applications of the hierarchical approach to water resource systems can be found in [4], as discussed by Haimes.

D. HIERARCHY FOR ON-LINE OPTIMIZATION

D.1 Development of Subbasin Problems

The following hierarchical methodology is best suited to a totally on-line optimization framework, where all optimization is carried out in real-time as a storm is actually progressing.

The large-scale optimization problem is repeated as follows:

$$\text{minimize} \quad \sum_{i=1}^{4} \sum_{k=1}^{M} \omega^i(k) O^i(k) \tag{3}$$

subject to:

$$\left.\begin{array}{l}\text{Constraints associated with subbasin } i \text{ only; based on} \\ \text{mass-balance equations associated with a given sewer} \\ \text{transport model and bounds on storage and trunk sewer} \\ \text{capacities.}\end{array}\right\} \tag{4}$$

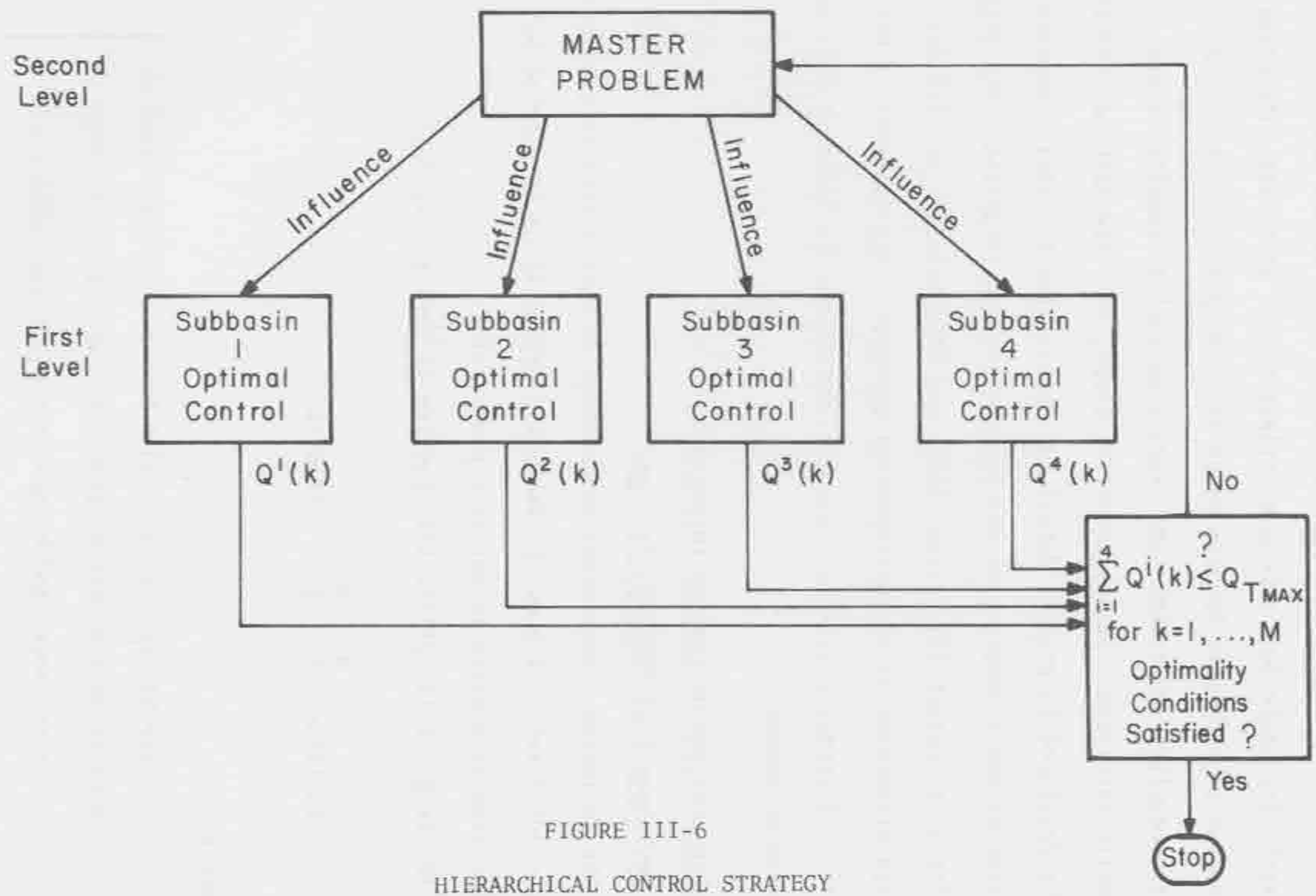


FIGURE III-6

HIERARCHICAL CONTROL STRATEGY

$$\sum_{i=1}^{4} Q^i(k) \leq Q_{Tmax}\,, \quad k=1,\ldots,M \tag{5}$$

$$0 \leq Q^i(k) \leq Q^i_{max}\,, \quad k=1,\ldots,M\ ;\ i=1,\ldots,4 \tag{6}$$

where Q^i_{max} is the fixed physical upper bound on the contribution from subbasin i to flow in the interceptor, where it is assumed

1. There is no lag or attenuation of flows in the interceptor sewer.
2. Constraints on interceptor capacity are not binding, as compared to treatment plant capacity Q_{Tmax}.
3. Inflows to the treatment plant from other sources (e.g., from the crosstown tunnel and the subbasin south of the treatment plant) are negligible. Again, this is not a necessary assumption, but is placed here to simplify illustration of the hierarchical approach.

Again, on-line optimization is probably feasible only if the constraints (4) associated with each subbasin are linear or quasi-linear (i.e., the sewer routing model is basically linear). If the constraints are linear, Dantzig-Wolfe decomposition is applicable to this problem. Application of this method would give a hierarchical structure slightly different than that discussed previously. In particular, the goal of the Master Problem would be to satisfy certain overall optimality conditions, while the coupling constraints would remain satisfied throughout the iterative process. The reader is referred to [8] and [1] for detailed treatment of Dantzig-Wolfe decomposition.

For the more general case, where there is some degree of nonlinearity in the flow routing model, the following approach can be utilized. Let

$$V^i(\underline{\alpha}^i) = \sum_{k=1}^{M} \omega^i(k) O^i(k) \tag{7}$$

represent the total weighted overflow from subbasin i, as a function of a decision vector $\underline{\alpha}^i$ representing the control policy used. The coupling constraints (5) can now be placed into the overall objective function (3) by use of Lagrange multipliers. The Lagrangian function is written as

$$\begin{aligned} L &= \sum_{i=1}^{4} V^i(\underline{\alpha}^i) + \sum_{k=1}^{M} \lambda(k) \left[\sum_{i=1}^{4} Q^i(k) - Q_{Tmax} \right] \\ &= \sum_{i=1}^{4} \left\{ V^i(\underline{\alpha}^i) + \sum_{k=1}^{M} \lambda(k) Q^i(k) \right\} - \sum_{k=1}^{M} \lambda(k) Q_{Tmax} \\ &= \sum_{i=1}^{4} L_i - \sum_{k=1}^{M} \lambda(k) Q_{Tmax} \end{aligned} \tag{8}$$

Suppose that for some given $\lambda(k)$, $k=1,\ldots,M$, the Lagrangian function is minimized. That is

$$\text{minimize } L \tag{9}$$

subject to: constraints (4) and (6),

for $i=1,\ldots,4$

Since L is separable into L_i, $i=1,\ldots,4$, then minimizing L is equivalent to minimizing each L_i, independently and adding the result together minus the term $\sum_{k=1}^{M} \lambda(k) Q_{Tmax}$. Suppose that optimal solutions $\underline{\alpha}^{*i}$, $i=1,\ldots,4$, result from these minimizations. Notice that constraints (5) may not be satisfied since they have been removed from the constraint set. Their being placed in the objective function with the nonnegative Lagrange multiplier is much like adding a penalty term to the original objective function. The penalty increases as the constraints are violated. If the correct values of $\lambda(k)$ are chosen, the original problem may be indirectly solved. Suppose that for some given $\lambda^*(k)$, it turns out that the following conditions are satisfied:

$$\text{Condition 1:} \quad \sum_{i=1}^{4} Q^i(k) \leq Q_{Tmax} \qquad \text{(feasibility)} \tag{10}$$

$$\text{Condition 2:} \quad \lambda^*(k)\left[\sum_{i=1}^{4} Q^i(k) - Q_{Tmax}\right] = 0 \qquad \text{(complementary slackness)} \tag{11}$$

$$\text{for } k=1,\ldots,M$$

then the original problem has been solved, as proved by Lasdon [8].

Therefore, if given $\lambda^*(k)$, the original large-scale problem can be replaced with four independent problems associated with each subbasin i:

On-Line Subbasin Problem

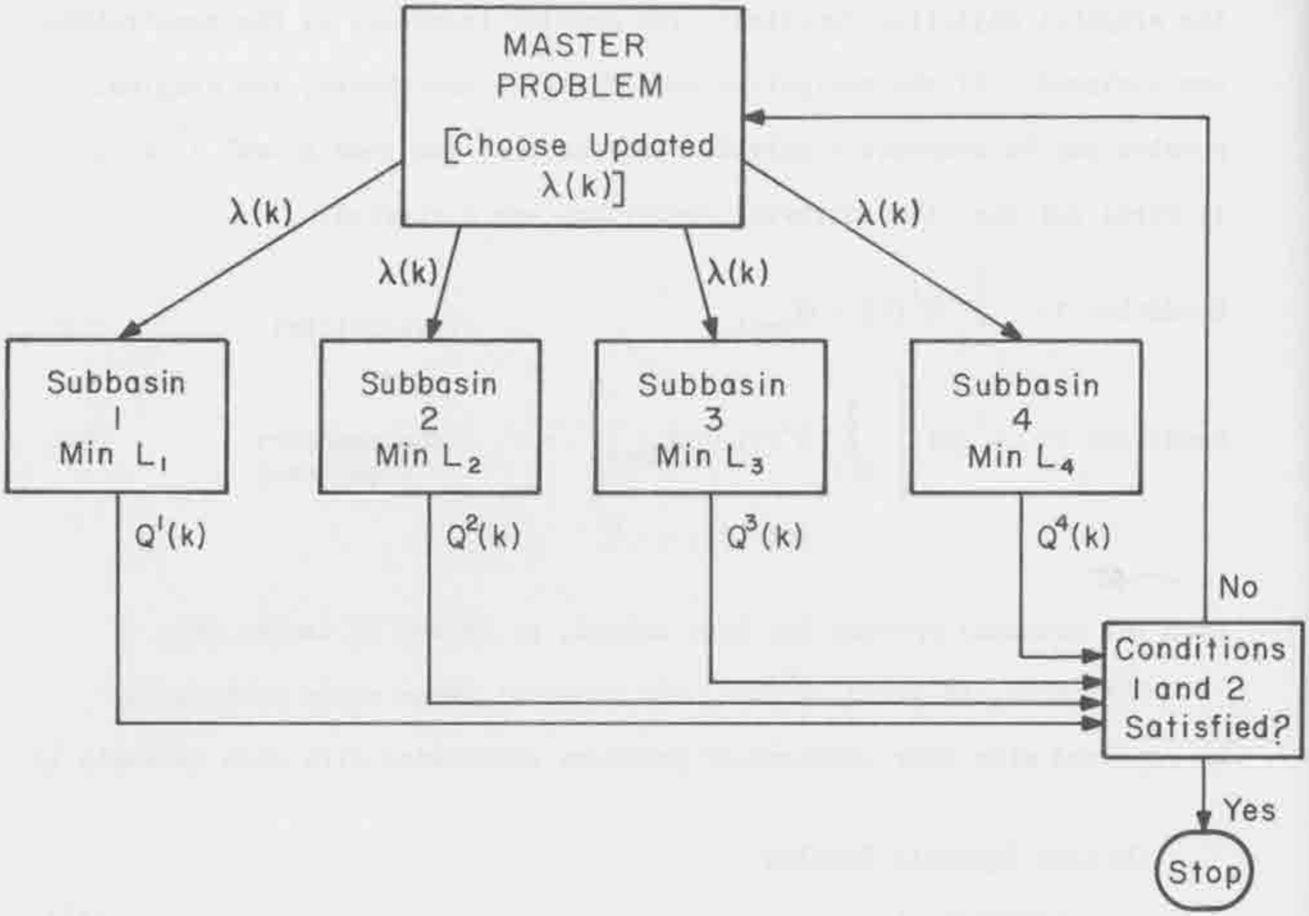
$$\text{minimize} \quad L_i \tag{12}$$

subject to: constraints (4) and (6), for subbasin i.

D.2 The Master Problem

The question is, do such $\lambda^*(k)$ always exist and, if so, how can they be found? Lasdon [8] has shown that such $\lambda^*(k)$ always exist if the objective function and constraints are convex. It may be difficult to assure convexity of the problem, in which case it would not be known a priori if such $\lambda^*(k)$ exist. Even if they do not exist, however, (10) and (11) may be satisfied within a tolerable error.

The goal of the Master Problem is to find $\lambda^*(k)$. Figure III-7 illustrates the iterative procedure, where initial guesses for $\lambda(k)$ are



Condition 1: $\sum_{i=1}^{4} Q^i(k) \leq Q_{TMAX}$, $k = 1, \ldots, M$

Condition 2: $\lambda(k)\left[\sum_{i=1}^{4} Q^i(k) - Q_{TMAX}\right] = 0$, $k = 1, \ldots, M$

FIGURE III-7

HIERARCHY FOR ON-LINE OPTIMIZATION

sent to the subproblems, resulting in throughflows $Q^i(k)$ based on the individual optimal control strategies for each subbasin. These values are then checked to see if Conditions 1 and 2 are satisfied. If they are, the overall optimum has been found. If not, the Master Problem chooses a new set of $\lambda(k)$ that will insure an improvement towards satisfying Conditions 1 and 2. Detailed discussion on how $\lambda(k)$ is updated can be found in Lasdon [8].

For the on-line framework, adjustments on the $\lambda(k)$ can be carried out in real-time. Given the $\lambda(k)$, each subbasin problem can perhaps be solved simultaneously on a minicomputer allocated to each subbasin. A central computer would process the results, and choose new values of $\lambda(k)$. Since the large problem has been replaced by several smaller problems with the potential of simultaneous solution, considerable savings in computation time is achieved.

This methodology can easily be extended to more realistic situations where constraints on interceptor capacity cannot be excluded, and routing occurs in the interceptor. For the former, these additional constraints are included in the Lagrangian function in the same way that the treatment plant capacity constraints were. The result is more Lagrange multipliers that must be adjusted by the Master Problem.

It should again be emphasized that through this hierarchical structure, the subbasin control problems can be treated individually and independently in real-time control, resulting in a significant computational advantage.

E. HIERARCHY FOR OFF-LINE OPTIMIZATION

E.1 Off-Line Subbasin Optimization

When simplified linear models are not adequate for describing the dynamics of sewer flow in the subbasins, some off-line optimization may be necessary. The results of these optimizations would then be placed in auxiliary storage of the on-line computer (e.g., on drums or disks) in the form of rule curves. The previous hierarchical approach would not be computationally feasible, in this case, due to the wide range of values that $\lambda(k)$ could take on, and the large number of $\lambda(k)$ that would exist when additional binding or coupling constraints due to interceptor capacity limitations were considered. The subproblems would have to be solved off-line for all combinations of a reasonable number of values of the $\lambda(k)$. An alternative approach is clearly needed. The following discussion is based on the assumption that all subbasin optimization is performed off-line. A more realistic approach is a blending of off-line and on-line subbasin optimization, but that analysis will be left for future reports.

Suppose that Q^i_{max} is now considered to be a control variable, and allowed to take on discrete values between zero and its actual physical value. It is now used to control throughflow from each subbasin. Suppose also that L historical or synthetically generated storm events have been identified as a reasonable range of events that could take place in the future. Then, objective function (3) can be minimized, subject to constraints (4) and (6) for various given levels of Q^i_{max} and the given L storm events, $\ell=1,\ldots,L$. The minimum total *weighted* overflow resulting from these off-line Subbasin Problems can then be designated as $O^{*i}(Q^i_{max}|\ell)$, since it is a function of the given level Q^i_{max} and given storm event ℓ . These optimizations are carried out for each subbasin $i, i=1,\ldots,4$, independently, and for all given levels of Q^i_{max} and storm events $\ell=1,\ldots,L$. The results of these optimizations must be stored in the auxiliary files of the on-line computer system for use in real-time control. Not only must $O^{*i}(Q^i_{max}|\ell)$ be stored, but also the optimal control strategy that gave $O^{*i}(Q^i_{max}|\ell)$.

E.2 The On-Line Master Problem

With the off-line subbasin optimization stored on-line, the Master Problem can be solved *on-line*. The Master Problem is basically an allocation problem, where the resource to be optimally allocated is treatment plant capacity Q_{Tmax} .

The use of a time-invariant Q^i_{max} is of course only an approximation. Allowing Q^i_{max} to be time-variant would require a myriad of subbasin optimizations for all possible Q^i_{max} levels over time. In illustrating the Master Problem, assume that time-lag and routing in the interceptor can be ignored, and that storm ℓ has been predicted as the storm in progress.

Master Problem:

$$\min_{\substack{Q^i_{max} \\ i=1,\ldots,4}} \sum_{i=1}^{4} O^{*i}(Q^i_{max}|\ell) \tag{13}$$

subject to:

$$\sum_{i=1}^{4} Q^i_{max} \leq Q_{Tmax} \tag{14}$$

$$0 \leq Q^i_{max} \leq \bar{Q}^i_{max} \ , \ i=1,\ldots,4 \tag{15}$$

where $\bar{Q}^i_{max}$ is the actual physical value of Q^i_{max} .

This problem is easily solvable on-line as a one-dimensional dynamic programming problem, requiring little core storage and computer time. Admittedly, these assumptions are not realistic for most situations, particularly the neglecting of time-lag in the sewer. In these cases, some kind of routing model must be utilized for the interceptor and incorporated into the optimization. Otherwise, treatment plant capacity will not be effectively utilized. The Master Problem should be solved on-line, however, so that complex routing may not be possible if it results in complex optimization for which finite, global convergence cannot be guaranteed for a limited quantity of computer time and storage.

Even simple routing in the interceptor (e.g., using time-lag only) can result in Master Problems which cannot be easily solved by dynamic programming. Initial experience with subbasin optimization indicates that $O^{*i}(Q^i_{max}|\ell)$ can be closely approximated by a convex, piecewise-linear curve. Since the constraints remain linear for simple routing, linear programming can be used to solve the Master Problem on-line.

E.3 Storm Prediction and Feedback Control

An important advantage of this decomposition approach for use in real-time control, if it is desired to mix off-line and on-line optimization, is the immense number of storm events which can be simulated from a few representative events. Suppose that the individual subbasin problems are each solved off-line for each of the L given storm events.

If the results of these $L \times N$ optimizations are stored in the on-line control computer, then a total of L^N events over the entire large basin have been implicitly stored through all the possible subbasin input combinations that could occur in the future.

The ultimate goal is to be able to successively update control policies as a storm progresses. These changes in control policy would primarily be due to modified storm predictions, based on accumulation of sensor data from the current storm which are periodically *fed back* into the model; hence, the term *feedback control*.

The approach taken here is deterministic, in that either a series of optimizations are carried out totally off-line for several representative storm events, or on-line optimization proceeds for a predicted storm, resulting in control policies based on the given simulated or predicted events. Another possibility, of course, would be a blending of these two approaches. At any point in real-time, the remainder of the storm in progress is successively predicted, and the control policy based on that storm *closest* to the storm predicted in real-time is utilized. A realistic approach would be to consider risk and uncertainty directly in the optimization problem. For example, in the early phases of a storm, predictions will tend to be poor, since information is sparse. Therefore, optimal control policies should properly reflect the degree of uncertainty associated with storm prediction. As the storm continues, and more information is collected, the degree of uncertainty should decrease. This *stochastic* approach will be given increasing attention in future research, but the deterministic approach will suffice here as a first step.

If the Subbasin Problems are solved on-line, optimizations can be carried out for whatever the current states of the subbasins (i.e., current levels in the detention reservoirs). Again, only simplified routing can be used in this case, in order to insure convergence within the limited amount of time between control opportunities. If it is desired to use more realistic routing, off-line optimization seems the only alternative. In this case, off-line optimizations would have to be carried out for all possible discrete initial storage conditions at any time k, for all $k=1,\ldots,M$. With this information stored in the real-time control computer, optimal policies based on a predicted storm event would be known for any state the system might be in at any time t_k.

The difficulty with this approach is the enormous number of off-line optimizations required. Suppose storage in the detention reservoirs of a particular subbasin are discretized into 10 levels, and there are 5 reservoirs. Then at least 10^5 optimizations must be carried out, corresponding to the possible combinations of reservoir levels that could occur in real-time at any time t_k.

Some kind of simplification is necessary if an off-line/on-line approach is to be used. One approach is to solve the Subbasin Problems off-line for zero storage initial conditions (i.e., all the detention reservoirs are assumed to be empty at the beginning of a storm) at initial time $k=1$ only, and store the optimal storage levels $\underline{S}^{*i}(k)$ in the detention reservoirs and minimal *weighted* overflows $O^{*i}(k)$ from subbasin i for $k=1,\ldots,M$, and for all discrete levels of Q^i_{max} and storms $\ell=1,\ldots,L$. To gain some insight into the amount of storage required, experience with the proposed San Francisco Master Plan for Wastewater Management has indicated that the following values may be reasonable, as an example: 10 subbasins, 100 representative storm events, 10 discrete time periods, 10 levels of Q^i_{max}, and an average of five detention reservoirs per subbasin. For this example, a total of about 6×10^5 words must be stored; well within current disk and drum capacities.

For real-time control using off-line optimization, information is retrieved from auxiliary memory to core memory in the on-line computer as storm predictions are successively updated. For the above example, only 6000 of the total 6×10^5 words would be needed in core at any time. This strongly suggests the possibility of using minicomputers. Many large industries are now considering the use of *hierarchies* of minicomputers instead of one large computer to run plant processes, control quality, and increase reliability [7]. If the large computer fails, production suffers greatly. If a minicomputer shuts down, the other *minis* can still carry on. In addition, software design is greatly simplified through use of several minicomputers doing simpler tasks, rather than one large computer trying to do it all.

Hierarchies of minicomputers and decomposition into subbasins go hand in hand. Perhaps one minicomputer can be allocated to each subbasin, with another minicomputer, or larger computer, tying them together through solution of the Master Problem. These are only suggestions, and the number of possible computer hardware configurations is great. As always, the basic trade-off is control effectiveness and reliability versus cost and complexity.

The authors suggest that it is perhaps best to start with something simple. Use simple linear routing methods, so that some attempt at on-line optimal control can be initiated as soon as possible. As experience is gained, more realistic modeling can be incorporated.

Subbasins that can tolerate simple routing models can continue to be optimized on-line, whereas off-line computational work can proceed for those subbasins requiring more complex models.

Now, suppose that at time k, storm ℓ is predicted as the storm in progress. The Master Problem is then solved using

$$O^{*i}(Q^i_{max}|\ell) = \sum_{k'=k+1}^{M} O^{*i}(k) \tag{16}$$

yielding optimal Q^{*i}_{max}, $i=1,\ldots,N$. This is only valid, however, if the actual states of the subbasins $\underline{S}^i(k)$ at time k can be brought to optimal states $\underline{S}^{*i}(k+1)$ at time $k+1$, for given Q^{*i}_{max}. This off-line/on-line framework is illustrated in Figure III-8.

This simplification results in the following: Given that current storm predictions have been carried out just prior to real-time t_k, $k < M$, on-line subbasin optimization (as discussed in the previous section) assures optimal policies from time t_k to t_M, assuming that the predictions are correct. Time period k is used to bring the subbasins to the optimal states. It may not be desirable, however, to bring the states of the subbasins to $\underline{S}^{*i}(k+1)$ exactly, if this is possible only at the expense of considerable overflow.

The critical factor in feedback control is the accuracy of the storm prediction. Considerable research must be carried out in this area, since *optimality* of control becomes virtually meaningless without accurate prediction. Again, prediction accuracy should increase as the storm progresses and more information is collected.

All of the discussion on the hierarchical approach has been related to the restricted subbasin configuration of Figure III-4. It

Subbasins that can tolerate simple routing models can continue to be optimized on-line, whereas off-line computational work can proceed for those subbasins requiring more complex models.

Now, suppose that at time k, storm ℓ is predicted as the storm in progress. The Master Problem is then solved using

$$O^{*i}(Q^{i}_{max}|\ell) = \sum_{k'=k+1}^{M} O^{*i}(k) \tag{16}$$

yielding optimal Q^{*i}_{max}, $i=1,\ldots,N$. This is only valid, however, if the actual states of the subbasins $\underline{S}^{i}(k)$ at time k can be brought to optimal states $\underline{S}^{*i}(k+1)$ at time $k+1$, for given Q^{*i}_{max}. This off-line/on-line framework is illustrated in Figure III-8.

This simplification results in the following: Given that current storm predictions have been carried out just prior to real-time t_k, $k < M$, on-line subbasin optimization (as discussed in the previous section) assures optimal policies from time t_k to t_M, assuming that the predictions are correct. Time period k is used to bring the subbasins to the optimal states. It may not be desirable, however, to bring the states of the subbasins to $\underline{S}^{*i}(k+1)$ exactly, if this is possible only at the expense of considerable overflow.

The critical factor in feedback control is the accuracy of the storm prediction. Considerable research must be carried out in this area, since *optimality* of control becomes virtually meaningless without accurate prediction. Again, prediction accuracy should increase as the storm progresses and more information is collected.

All of the discussion on the hierarchical approach has been related to the restricted subbasin configuration of Figure III-4. It

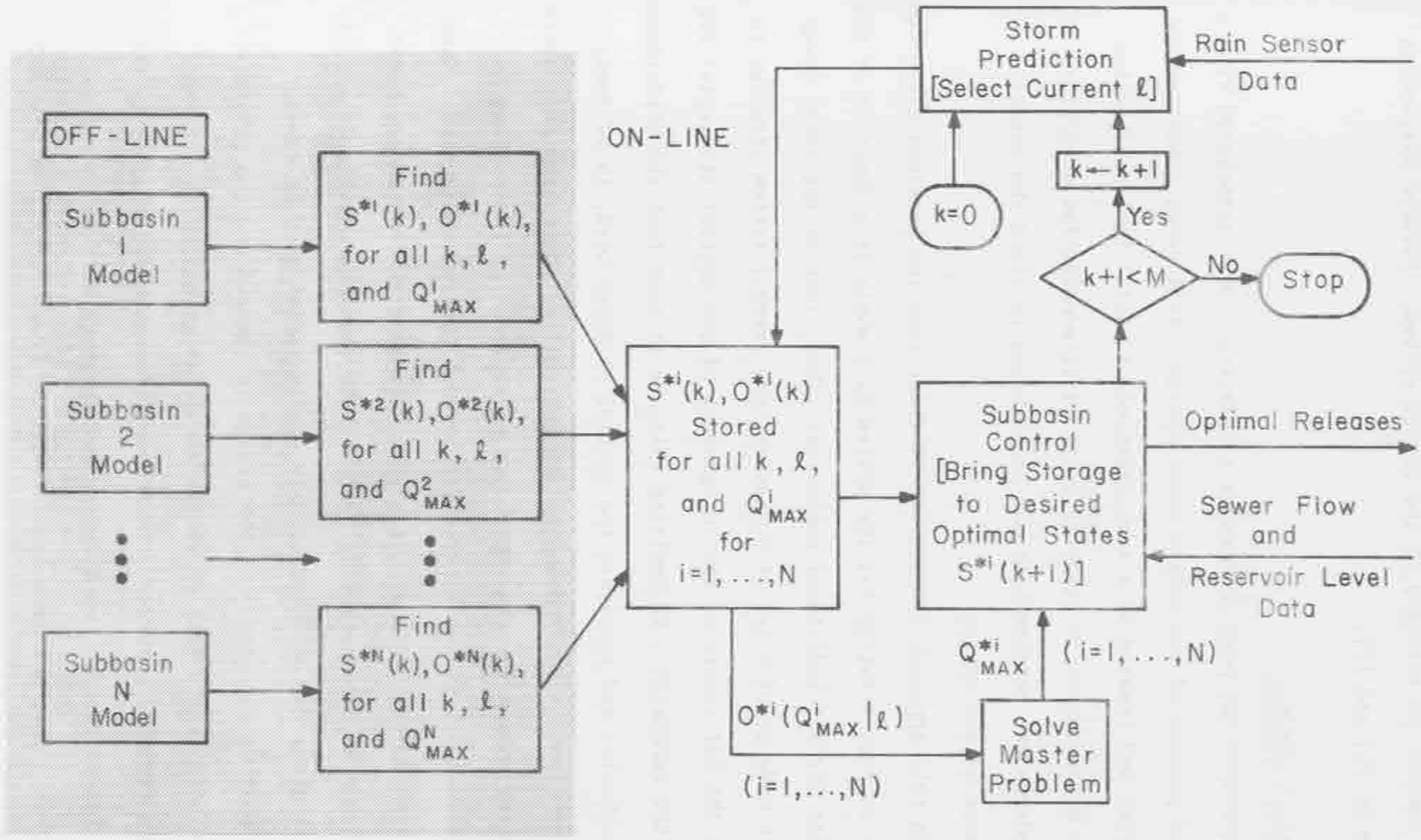


FIGURE III-8

FEEDBACK CONTROL USING OFF-LINE/ON-LINE OPTIMIZATION can, of course, be extended to the entire system. Further discussion is given in [6] and [7].

F. SUBBASIN CONTROL

A subbasin has been defined as a hydrologic unit associated with a specified portion of the entire sewer system. This sewer subsystem can be analyzed and treated as a semi-independent unit. The total system can then be conceived in terms of a set of interconnected subsystems whose independent performances can be combined to yield the overall performance of the system.

With this approach in mind, it is clear from the previous discussion that the control strategy for the system as a whole is a function of the strategies for the individual subbasins. Thus, just as the total sewer system is made up of a set of subbasins, the overall system strategy is actually the net result of the integration of the control strategies for each of the subbasins. Recognizing this, it is seen that the performance of the subbasin, and therefore the control strategy used, is of fundamental importance to understanding and controlling the system as a whole.

Another reason for the importance of approaching the problem at the subbasin level is the interaction between control and design. When dealing with a basic unit of the system this interaction becomes clear. If a subbasin is mathematically simulated so that the effects of a given control strategy can be observed, its configuration must be known, i.e., it must be designed. If the design is changed and the strategy fixed the performance will of course change as it will with a fixed design and variable strategy. Therefore, if system performance is the criteria for evaluation, both design and strategy must be considered as variables. This has been demonstrated by Crawford [2] in a study of the use of computer simulation to develop design criteria for urban flow storage systems. One of his major conclusions is:

> "The rule curve or release rule form the storage is of major importance. If its water is released from the storage within one hour of the time of rainfall, the storage will reduce peak flows in small tributaries but will not change peak flows in larger streams."

One can define three basic approaches to developing subbasin control strategy without the benefit of actual field experience.

1. Apply one's knowledge and experience in the area of hydrology, hydraulics and water quality to deduce a strategy.
2. Develop a simulation model and examine the performance using various strategies as parameters.
3. Combine formal optimization techniques with a simulation model or a more simplified model to directly give optimum strategies for prescribed criteria or objective functions.

The first approach alone may be useful for only the simplest of systems and therefore from a practical standpoint would be combined with the other approaches. The second approach is the principal topic of discussion in Chapter IV of this report, whereas the third can be found in Chapter V , and is more closely related to the discussion in Section D of this chapter.

CHAPTER III - NOTATION

L = total number of historical and synthetically generated storm events for off-line optimization

$\mathcal{L}$ = Lagrangian function

M = total number of discrete time periods

$O^i(k)$ = quantity of overflow from subbasin i, during time period k

$O^{*i}(k)$ = quantity of overflow during period k from subbasin i under an optimal control policy, for given Q^i_{max} and storm ℓ

$O^{*i}(Q^i_{max}|\ell)$ = total weighted overflow from subbasin i during storm event ℓ, for maximum outflow of Q^i_{max}

$Q^i(k)$ = flow contributed to interceptor from subbasin i during period k

Q_{Tmax} = maximum treatment plant capacity

Q^i_{max} = (i) physical upper bound on throughflow capacity from subbasin i (ii) used as a control variable for off-line optimization, where Q^i_{max} varies from zero to $\bar{Q}^i_{max}$, the actual upper bound

$\underline{S}^{*i}(k)$ = vector of optimal storage levels at the beginning of period k in all detention reservoirs contained in subbasin i, under an optimal control policy for a given Q^i_{max} and storm ℓ

$V^i(\underline{\alpha}^i)$ = total overflow during a particular storm event from subbasin i, based on a control policy $\underline{\alpha}^i$ decision vector

$\underline{\alpha}^i$ = a control policy vector for subbasin i

$\lambda(k)$ = Lagrange multiplier

$\omega^i(k)$ = weighting factor

CHAPTER III

REFERENCES

1. Adler, I. and A. Ülkücü, "On the Number of Iterations in Dantzig-Wolfe Decomposition," Decomposition of Large-Scale Problems, D. L. Himmelblau (ed.), North Holland, Amsterdam, pp. 181-7, 1973.

2. Crawford, N. H., "Computer Simulation for Design Criteria for Urban Flow Storage Systems," Hydrocomp International, Palo Alto, California, 1973.

3. Gilbert, J. B. and Associates, and Metcalf and Eddy, Inc., "Evaluation--San Francisco Wastewater Master Plan," March, 1973.

4. Haimes, Y. Y., "Decomposition and Multilevel Approach in the Modeling and Management of Water Resources Systems," Decomposition of Large-Scale Problems, D. L. Himmelblau (ed.), North Holland, Amsterdam, pp. 347-68, 1973.

5. Labadie, J. W. and N. S. Grigg, "Automated Control in Large-Scale Metropolitan Water Intelligence Systems," presented at the 43rd National Meeting of the Operations Research Society of America, Milwaukee, Wisconsin, May 9-11, 1973.

6. Labadie, J. W., N. S. Grigg, and B. H. Bradford, "Automatic Control of Large-Scale Combined Sewer Systems," Journal of the Environmental Engineering Division, ASCE, (to appear).

7. "Minicomputers that Run the Factory," Business Week, December 8, 1973.

8. Lasdon, L. S., Optimization Theory for Large Systems, MacMillan, 1970.

9. Mesarovic, M. D., D. Macko, and Y. Takahara, Theory of Hierarchical Multilevel Systems, Academic Press, 1970.

10. Wismer, D. A., Optimization Methods for Large-Scale Systems, McGraw-Hill, New York, 1971.

CHAPTER IV

SIMULATION FOR DESIGN AND CONTROL

A. APPLICATIONS OF SIMULATION TO URBAN WATER PROBLEMS

Although simulation is not new, its application to urban stormwater has been limited for the most part to the last ten years, being stimulated by increasing awareness of the importance of stormwater and combined sewer problems. Some early use of simulation in this area was done by sanitary engineers in analyzing the design aspect of combined sewer overflows. [1,12,16,17] In these studies the input model usually consisted of the rational formula to convert rainfall to runoff with a direct translation of peak flows without consideration of attenuation or lag. Frequency of overflow and overflow volumes were obtained and the quality of overflows was also considered. The *control* aspect was limited to a consideration of the effect of increasing the capacity of the interceptors on overflows. A more recent study by Chen and Saxton [4] employed a synthetic hydrograph to describe the volume and duration of combined sewer overflows. The model was used to demonstrate the effect of various intercepting capacities on overflows. The utility of simulation was recognized in the authors' discussion of practical applications:

> "This (method) will guide engineers in determining the optimum intercepting and wastewater treatment capacity and the capacities of stormwater holding tanks or overflow treatment facilities for various degrees of overflow reduction."

The use of simulation to develop control strategies is perhaps the most recent application of this tool in the urban water area. The Municipality of Metropolitan Seattle is a good example of this application.

In a report describing the development of the Seattle storage regulation system the value of a simulation model is pointed out [14].

> "A thorough engineering analysis of the system along with the flow calibration and weather analyses previously described can lead to a period of trial testing and program modification to develop refined rule curves for use in actual control of the system during precipitation events."

Another application was performed in the development of the Chicago flood and pollution control plan [7]. Both quality and quantity models were used to evaluate various aspects of design alternatives. Perhaps the most recent in-depth application of simulation to the control problem was reported by Crawford [5]. In this study the *Hydrocomp Simulation Program* was used to evaluate the effect on several urban flow systems of detention reservoirs placed at various locations, using various controlled outflow rates from the basins. This report points out the importance of describing the performance of the system in terms of the risk or probability of exceeding specified criteria and furthermore that continuous simulation will produce a reliable estimate of this risk.

A.1 Existing Models

In recent years model building has become a popular activity. This section is not intended to be an exhaustive review but merely to give recognition to a few of the current urban wastewater design or management models.

The *EPA Storm Water Management Model* was introduced in 1971 [13]. It is a comprehensive deterministic model capable of generating runoff hydrographs, and pollutographs for both dry and wet weather conditions. Both ambient and auxiliary storage facilities can be included as well as a variety of wastewater treatment options. The model also can compute capital and operation and maintenance costs so that cost effectiveness curves can be developed.

The *Hydrocomp Simulation Program* [9] is a general model applicable to both urban and rural watersheds. The deterministic model uses precipitation as input and generates continuous outflow hydrographs. Water quality parameters were not included in the original version but have been added to the most recent versions. Antecedent conditions are accounted for in determining runoff so that continuous simulation from historical rainfall is possible and probability statements can be derived from the output. The watershed can be broken up into a number of segments to reflect varying conditions and the model can be calibrated using historical records.

The *Battelle Urban Wastewater Management Model* [2] was developed to simulate major sewer system components. It is a lumped deterministic model which can be used for design as well as evaluation. The model considers water quality and quantity, determines optimal allocation of available storage and treatment capacities and generates optimal control strategy.

The Hydrologic Engineering Center in cooperation with Water Resources Engineers, Inc., has developed a deterministic urban model, designed primarily for use in planning, called *Storm* [10]. The model predicts the quantity and quality of urban runoff for the purpose of selecting stormwater storage and treatment rates required to meet prescribed standards. It is designed to be used as a rough continuous simulation model and therefore simplified relationships are employed.

The Metropolitan Sanitary District of Greater Chicago has developed a *Flow Simulation System* to simulate the hydrologic responses of watersheds in the Chicago area [11]. It is a distributed, deterministic quantity model which can include channel and sewer systems as well as ambient reservoirs. Peripheral programs have been developed to perform economic analyses based on model input.

B. DEVELOPMENT OF A COMBINED SEWER SYSTEM SIMULATION MODEL

Although several of the models discussed in the previous section could be adapted for use, they are somewhat lengthy. It was decided to develop a relatively simple quantity model for the specific purpose of developing control strategy. Although it is recognized that ultimately the quality rather than quantity of overflows must be considered, the two are highly correlated and it was judged that the advantages gained from this simplification were justified in this initial approach.

During Phase II of the MWIS project, a combined sewer system simulation model was developed and used to simulate that part of the Vicente Subbasin which is tributary to flow gage 125 [8]. This section presents a brief review of the components of this model and discusses the model developed during Phase III, which simulates the entire Vicente Subbasin. This subbasin was selected for analysis because it was also being studied by the SFDPW and because data was available in a reduced form. Also, it was believed that some operational experience in this watershed was essential for the further development of models.

B.1 Model Components

There are three basic components in the simulation model: 1) rainfall-runoff, 2) utilization of the proposed detention reservoirs and 3) transport.

The *rainfall-runoff* process is modeled in subroutine BASIN. The subroutine is based on the application of a *single linear reservoir* model which assumes that the volume of rainfall excess $V_e(t)$, stored in a drainage basin at any time t, is proportional to the flowrate out of the basin $Q(t)$. This is described by the following equation:

$$V_e(t) = K\,Q(t) \tag{1-a}$$

where K is a linear-reservoir routing constant having units of time. For an instantaneous inflow the continuity equation for the reservoir is

$$Q(t) = -K\,\frac{dQ(t)}{dt} \tag{1-b}$$

Integration of Equation 1-b yields the instantaneous unit hydrograph

$$Q(t) = \frac{V_e(0)}{K}\,e^{-t/K} \tag{2-a}$$

where $V_e(0)$ is the volume equivalent to one inch of rainfall excess initially present over the entire drainage basin. Using the convolution integral to expand Equation 2-a into a unit hydrograph of duration T yields

$$Q(t) = \frac{V_e(0)}{K}\,(1 - e^{-t/K}) \qquad 0 \leq t \leq T \tag{2-b}$$

$$Q(t) = \frac{V_e(0)}{K}\,e^{-t/K} \qquad t \leq T \tag{2-c}$$

The direct runoff hydrograph for a rainfall hyetograph containing M periods of rainfall excess $R_e(k)$ $k=1,\ldots,M$ (R_e in inches) is then obtained by multiplying the unit hydrograph ordinates by $R_e(k)$ and summing using a common time scale.

Subroutine BASIN requires five input parameters: 1) the basin drainage area A, 2) the runoff coefficient C, 3) the linear-reservoir routing constant K, 4) the time interval for calculation of points on the hydrograph Δt, and 5) the rainfall hyetograph $R(k)$ $k=1,\ldots,M$ ($M\Delta t$ is the total duration of the storm and $R(k)$ is the rainfall occurring between $t=(k-1)\Delta t$ and $t=k\Delta t$).

The output from subroutine BASIN is, of course, the basin hydrograph $Q(k)$ for each time point $t=k\Delta t$.

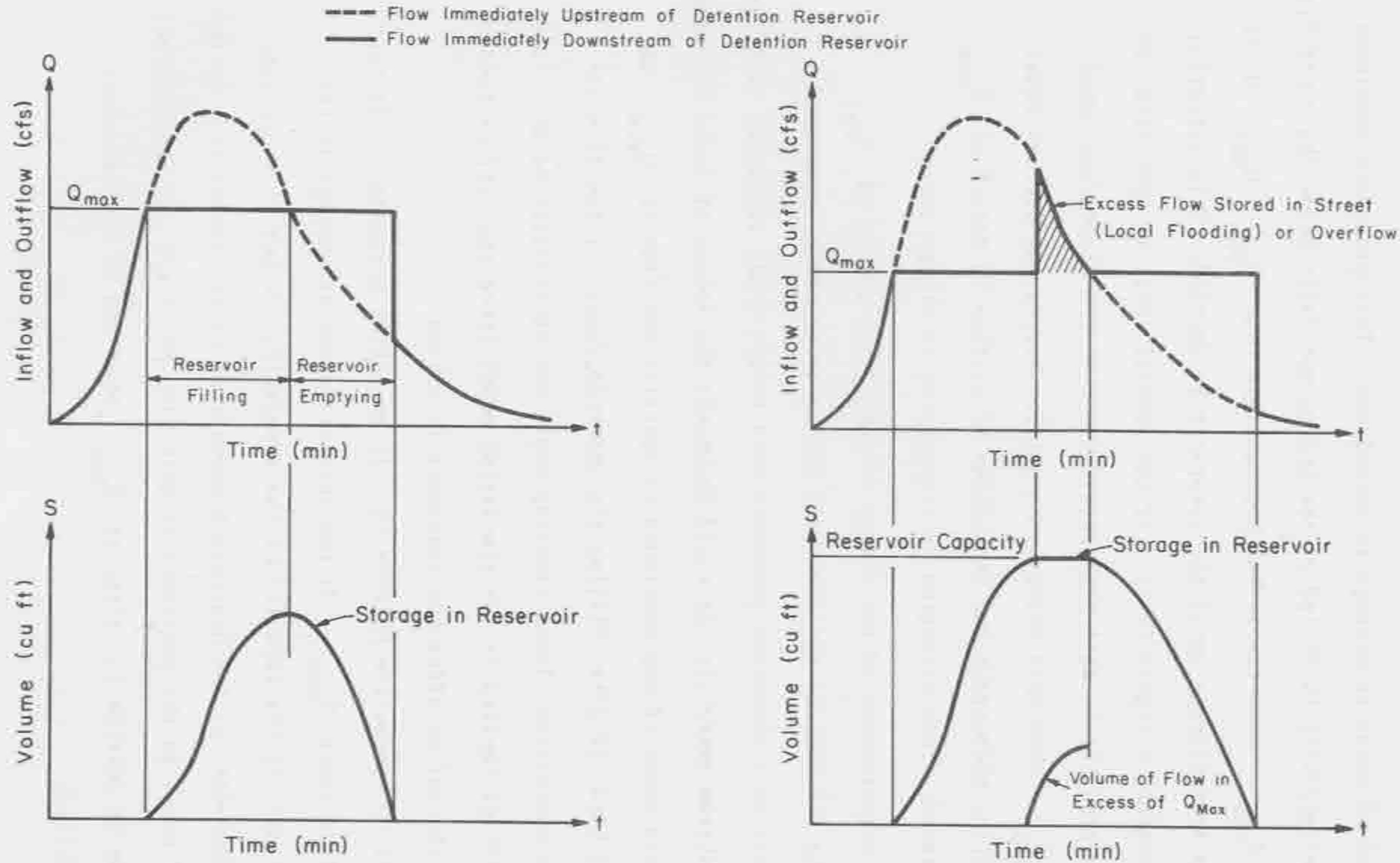
The value of C is the predicted ratio of storm runoff volume to rainfall volume. The parameter K is a time constant which corresponds roughly to the average travel time in the basin. The rainfall hyetograph is either read indirectly or calculated by reading rainfall depth and duration and assuming a uniform rainfall. The excess rainfall hyetograph is calculated by multiplying each element of the rainfall hyetograph C , or

$$R_e(k) = C\,R(k)$$

Utilization of the Proposed Detention Reservoirs is modeled in subroutine RETENTN. The control logic consists of specifying the maximum flowrate Q_{max} , to be allowed to proceed downstream of the detention reservoir. If the flowrate Q immediately upstream of the reservoir becomes greater than Q_{max} , $Q - Q_{max}$ is diverted into storage and the new volume of water in storage is calculated. This procedure continues until the reservoir is filled or the hydrograph falls below the controlled release Q_{max}. When the inflow becomes less than Q_{max}, $Q_{max} - Q$ is withdrawn from storage until the reservoir is emptied. This situation is illustrated in Figure IV-1a. If the control limit is specified too low it is possible to fill the reservoir before the hydrograph drops below Q_{max}. When this occurs, the outflow hydrograph becomes equal to the inflow hydrograph and the volume of outflow in excess of Q_{max} is calculated. This situation is illustrated in Figure IV-1b.

The significance of the volume of outflow in excess of Q_{max} depends on the control exercised and the location of the reservoir. In the case of a shoreline reservoir this volume would represent overflow. In an upstream reservoir, it would represent the volume of local flooding which would occur if one continued to restrict the flow to Q_{max}. On the other hand, if after filling the upstream reservoir the flow was no longer restricted, local flooding would not necessarily occur. This is the control implied in the simulation model since the outflow becomes equal to the inflow after the reservoir is filled.

Input to subroutine RETENTN is: 1) an inflow hydrograph; 2) the flow control limit Q_{max}; 3) the initial volume of storage in the reservoir and 4) the capacity of the reservoir. Output from the subroutine includes 1) the hydrograph downstream of the reservoir; 2) the volume of water in the reservoir at each time point and 3) the flowrate and volume of outflow in excess of Q_{max} (when and if the reservoir becomes filled).



(a) Reservoir Not Filled

(b) Reservoir Filled

FIGURE IV-1

RESERVOIR UTILIZATION

The *transport* component is modeled in subroutine REACH. The subroutine is based on the standard Muskingum routing method with a slight modification suggested by Cunge [6].

The modified Muskingum equation used in the subroutine is the following:

$$Q_{j+1}^{((k+1)\Delta t)} = \frac{2-\beta}{2+\beta} Q_j^{((k+1)\Delta t-\tau)} + \frac{\beta}{2+\beta} Q_j^{((k+1)\Delta t)} + \frac{\beta}{2+\beta} Q_{j+1}^{((k+1)\Delta t-\tau)} \tag{3}$$

where:

Q = flowrate

The subscripts j and $j+1$ refer to the upstream and downstream ends of the reach, respectively

The superscripts represent time

Δt = the chosen time interval

β = a weighting factor which must be between 0 and 2 so that the coefficients $\frac{2-\beta}{2+\beta}$ and $\frac{\beta}{2+\beta}$ will be non-negative

k = an index on time ($k=0$ at time $t=0$ and is incremented by one for each successive downstream flowrate $Q_{j+1}^{(k+1)\Delta t}$, which is calculated.

$\tau = \frac{\Delta X}{c}$ = the travel time for a flood wave to pass through the reach, where ΔX = the length of the reach and c = the celerity of the flood wave.

The celerity $c = \frac{dQ}{dA}$ is calculated assuming that the pipe is flowing one-half full and $\tau = \Delta X/c$ is found. The value of β suggested by Cunge (i.e., $\beta = \tau Q/S_o[\Delta X]^2 b$, where S_o = the slope of the reach and b = the stream width) is also calculated.

To begin the routing, values of Q_j and Q_{j+1} for times $t \leq 0$ are assumed equal to Q_j at time $t=0$. Equation 3 is then used to calculate successive values of downstream flowrates.

The required input to subrouting REACH is 1) the hydrograph at the upstream end of the reach and 2) the reach slope, length, diameter and Manning roughness coefficient. The output from REACH is 1) the hydrograph at the downstream end of the reach, and 2) the values of β and τ used.

Table IV-1 lists the input requirements and output from the various model components. The next section will describe how these components are assembled into a simulation model.

B.2 Simulation of a Drainage System

The drainage basin to modeled must now be approximated utilizing the three components described in section B.1. The user must decide on the degree of aggregation desired in the simulation. The drainage basin can be broken down into many subcatchments and connecting reaches, or it can be highly aggregated into only a few subcatchments and reaches. Obviously, a simulation composed of many small components more closely approximates the actual drainage system. On the other hand, the computer programming and execution time requirements are much less when the model is highly aggregated. Other considerations include the degree of sophistication of the model components and the purpose for which the simulation model is developed (e.g., planning studies, real-time storm prediction, analysis of control strategies, etc.)

TABLE IV-1

MODEL INPUT AND OUTPUT

MODEL COMPONENT	Input	Output
Rainfall-Runoff (Subroutine BASIN)	Rainfall Hyetograph Drainage Area Runoff Coefficient C Routing Constant K Time Interval Δt	Drainage Basin Hydrograph
Detention Reservoir (Subroutine RETENTN)	Inflow Hydrograph Flow Control Limit Q_{max} Initial Reservoir Storage Reservoir Capacity	Outflow Hydrograph Volume of Water in Storage Flowrate in Excess of Q_{max} Volume of Flow in Excess of Q_{max}
Transport (Subrouting REACH)	Upstream Hydrograph Slope S Length ΔC Diameter D Manning n	Downstream Hydrograph Calculated Weighting Factor β Wave Travel Time τ Volume

B.3 Calibration of the Simulation Model

The performance of a prediction model is, of course, no better than the user's estimate of required input parameters. In the transport model described the values of the weighting factor β and the wave travel time τ are calculated internally, but these are still estimates of average values for the reach. The rainfall-runoff model requires the user to estimate the subcatchment runoff coefficient C and the routing constant K. All of these values have a pronounced effect on the predicted hydrographs, so the need for accurate estimates of these values is obvious.

The usual means for obtaining these values is to study a basin which has known rainfall data and corresponding runoff data. The system parameters are then adjusted in an attempt to make the calculated output agree with the observed values. One tool for this is the rainfall-runoff parameter identification model explained in section B.5.

B.4 Modeling Water Quality

The simulation model in its present form will predict hydrographs and volumes of overflows. In order to predict the pollution resulting from these overflows it would be necessary to model quality as well as quantity.

It is possible to incorporate the computation of runoff quality into this model. A procedure similar to the one developed by Water Resources Engineers, Inc. could be used [10]. The amount of various pollutants on a watershed at the beginning of a storm would be calculated based on factors such as the land use, the number of days between street sweepings, the number of dry days since the last storm, etc. This amount would be used to calculate the rate at which pollutants are washed off the watershed and into the storm drains. In this way, *pollutographs* as well as hydrographs can be generated. These pollutographs would be routed through the reaches. Aging of the pollutants while in detention storage would also be calculated.

In this manner, the amount of various pollutants (such as BOD, suspended solids, coliforms) overflowing into the receiving waters could be predicted.

B.5 Rainfall-Runoff Parameter Identification Model

Modification and application of the hydrologic model developed in Phase II of the MWIS project was continued during Phase III. The purpose of this model is to use known rainfall and corresponding runoff measurements to determine *optimum* values of the input parameters.

The steps followed in this procedure are as follows:

1. Determine the direct runoff hydrograph by subtracting base flow from the observed hydrograph.
2. Calculate volume under the direct runoff hydrograph and the volume of rainfall. Let C equal the volume of direct runoff divided by the total volume of rainfall.
3. Multiply the rainfall hyetograph ordinates by C to get excess rainfall. (This is a linear assumption for the prediction of excess rainfall and it is admittedly oversimplified).
4. Assume a value of the routing constant K and calculate a hydrograph by one of the following methods:
   a. Single linear reservoir
   b. Linear reservoir-linear channel assuming a triangular time-area histogram.

c. Linear reservoir-linear channel utilizing a specified time-area histogram.

5. Determine the error between the calculated and observed hydrographs. The error is calculated from a specified function which will be termed the *fitting criteria.*

6. Adjust K and recalculate the hydrograph until the value of K which minimizes the *fitting criteria* is determined.

The computational procedures, and a flow diagram of the computer algorithm were presented in the Phase II report. Some of the more important features are reiterated below.

The Fitting Criteria

No value of K will make the calculated and observed hydrographs match exactly. Therefore, it is necessary to develop an expression for the error between the two. Two fitting criteria have been used in this model. The first is the standard error which is expressed as follows:

$$\text{Standard Error} = \left[\frac{\sum_{k=1}^{N} (Q_0^k - Q_c^k)^2}{N - 1} \right]^{1/2}$$

where

Q_0^k = the observed flowrate at the k^{th} time point

Q_c^k = the calculated flowrate at the k^{th} time point

N = the number of time points entered

The second expresses the error in terms of the peak flowrates and the times to peak as follows:

$$\text{Error} = \left[\left[\frac{Q_{P0} - Q_{PC}}{Q_{P0}} \right]^2 + \left[\frac{t_{P0} - t_{PC}}{t_{P0}} \right]^2 \right]^{1/2}$$

where

Q_{PO} = the peak flowrate of the observed hydrograph

Q_{PC} = the peak flowrate of the calculated hydrograph

t_{PO} = the time to peak of the observed hydrograph

t_{PC} = the time to peak of the calculated hydrograph

To begin the search for the value of K which minimizes the fitting criteria, two estimates of K are tried. The first is the time difference between the centroids of the excess precipitation hyetograph and the observed runoff hydrograph. This is the theoretical value of K for a single linear reservoir model. The second is the following regression equation developed at Purdue University [15].

$$K(hrs) = 0.887A_b^{0.490}\ (1 + U)^{-1.683}\ PE^{-0.24}\ T^{0.294} \tag{4}$$

where

A_b = the drainage area of the basin (mi^2)

U = the ratio of impervious area to total area

PE = total precipitation excess (in)

T = duration of precipitation excess (hrs)

The hydrographs and their corresponding values of the fitting criteria are calculated for these two values of K. The parameter K is then changed incrementally, starting with the value yielding the greater error and proceeding in the direction of the other value of K. With each new value of K, the hydrograph and corresponding fitting criteria are calculated. The iteration continues until the value of the fitting criteria increases. The previous value of K is then said to be the one which minimizes the fitting criteria.

Input-Output Information

The required storm information is the rainfall hyetograph and the excess precipitation hydrograph. The discrete time interval Δt is chosen and the rainfall hyetograph is read in units of inches per time interval. The excess rainfall hydrograph is found externally by converting depth measurements to discharge via a stage-discharge curve and subtracting the estimated base flow. The only drainage basin information required is its area, except when the basin *time-area histogram* is to be prespecified in a linear reservoir-linear channel routing.

Output includes the optimum value of K, the K calculated by the difference in hydrograph and hyetograph centroids and the value found by the regression equation. A listing and plot of the observed hydrograph and the calculated hydrographs using these three values of K are also generated.

C. DESIGN OF SIMULATION STUDY

Any simulation study is characterized by repeated application of the system model with an analysis of the output corresponding to certain objectives. When the input to the model is stochastic in nature it is logical to use a continuous simulation approach in which the performance of the system is statistically analyzed over a long period of time. This, however, requires a considerable length of input data which can either be purely historical or synthetically generated using variability parameters developed from the historical data.

C.1 An Alternative Approach to Continuous Simulation--Zero Overflow Curves

In the case of subbasin simulation for the purpose of developing optimum control strategy, a somewhat different approach was used. This was prompted in part by the fact that only three years of rainfall data from the raingage network were available for the Vicente subbasin. The long term records were available on an hourly basis only. Since the subbasin is sensitive to rainfall variations less than one hour in duration, the use of hourly data may not provide a realistic picture of performance. Therefore, an approach was developed in which a set of uniform intensity storms of various depths and durations could be used as input with various control strategies as parameters. *Control strategy* is defined here as a general operating rule for controlling detention reservoir outflow. Variations within a specific strategy can be described in terms of control parameters or *control levels*. The initial objective was to establish on a rainfall depth-duration plot the relationship between duration and the maximum rainfall depth which will not cause a detention reservoir overflow, using control level within a particular control strategy as a parameter. This line, which can be termed the zero overflow line, is useful because its location as a function of strategy gives a graphical picture of the relative benefit or advantage gained or lost by various strategies. Furthermore, the best simulated strategy can be defined as the one which yields a maximum zero overflow depth for any duration. In general, the best control level level within a strategy is not constant but a function of rainfall duration. This conclusion emphasizes the importance of accurate rainfall prediction if real-time control is to be used. The level of performance actually attained is directly related to the ability to predict rainfall depth and duration for the remainder of a storm, given the mass curve up to the current time. The simulation studies described in this report assume that the entire storm history is known, and thus are at least one step away from real-time control simulation. Nevertheless, this effort is useful since it provides an understanding of the nature of the performance of the system and a means by which improvement in performance can be estimated.

More information can be gained by superimposing a set of depth-duration-frequency curves for the subbasin. Figure IV-2 shows a schematic plot as described above. The solid lines show the zero overflow lines using the best control level for various simulated control strategies. The dashed lines are the rainfall frequency curves. If there is a rainfall frequency curve which is tangent to an overflow curve at some point (duration), and below it for all other duration then one could assign that frequency as a maximum overflow frequency for that strategy, regardless of rainfall duration. For example, in Figure IV-2 one could say that the maximum frequency of overflow for strategy C was one in 5 years on the average or a 20 percent probability of overflow in any year. However, if the shape of the overflow curves is such that no point of tangency exists with the frequency curves the maximum overflow frequency will be a function of rainfall duration.

The development of zero overflow curves is essentially a trial and error process. Sufficient storms of various depths and durations must be processed through the model in order to bracket the curve. Final determination as accomplished through linear interpolation of overflow volume as a function of rainfall depth at a constant duration.

C.2 <u>Application to Vicente Subbasin</u>

As an example, zero overflow curves were developed for Vicente Subbasin under design alternatives B and D (See Table IV-2) for

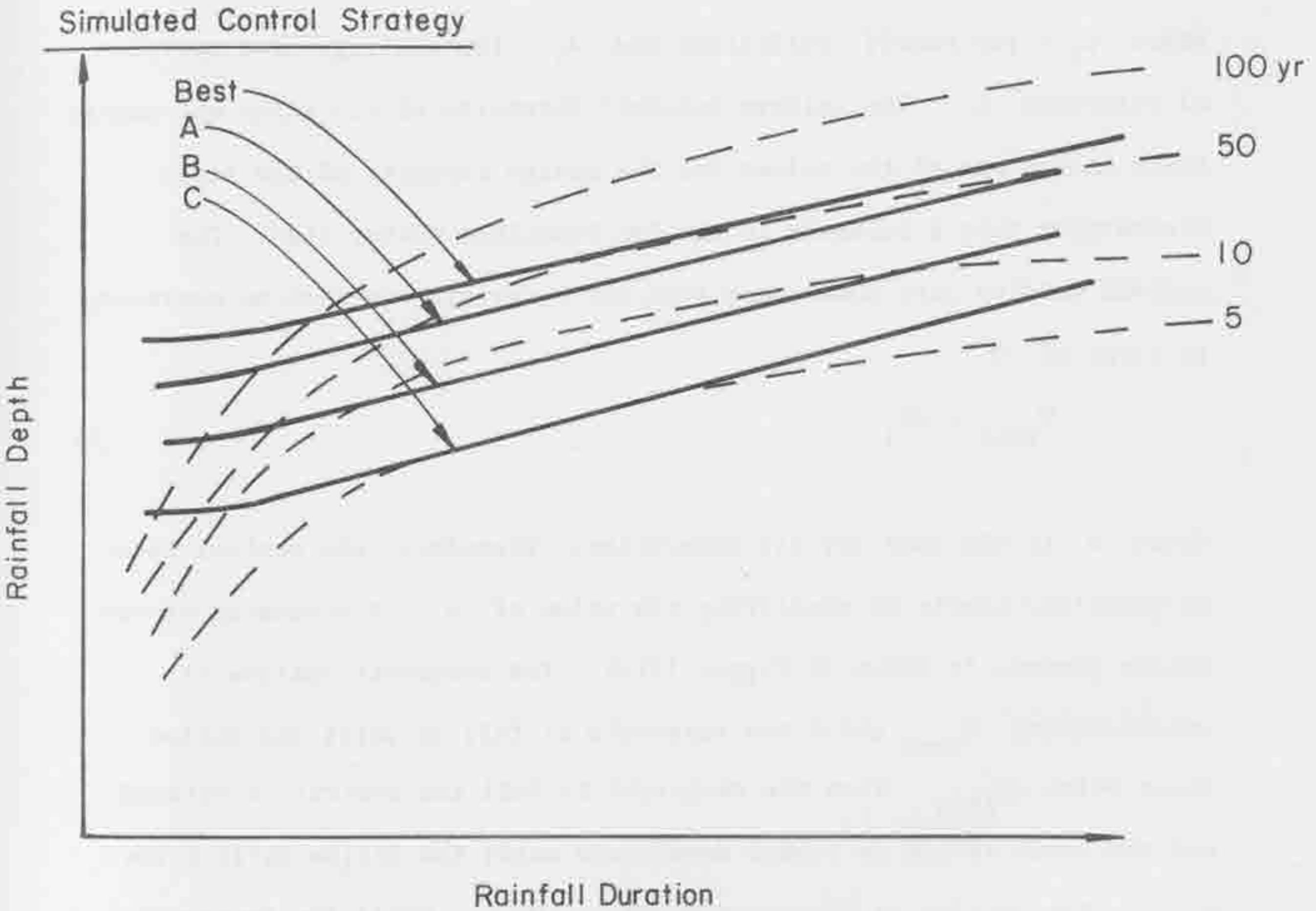


FIGURE IV-2

ZERO OVERFLOW CURVE SCHEMATIC

rainfall durations up to 4 hours. The simulated control strategy used was to regulate the maximum outflow from the three upstream detention reservoirs to various values, as described in section B.1. These were specified for each upstream reservoir in terms of reference value

$$\hat{Q}_i = C_i(0.3 \text{ in/hr})A_i \tag{5}$$

where C_i = the runoff coefficient and A_i = the drainage area upstream of reservoir i. The uniform rainfall intensity of 0.3 in/hr was chosen since it was one of the values for the design capacity of the lines discharging from a subbasin in the San Francisco Master Plan. The maximum outflow just downstream from any reservoir can then be expressed in terms of $\hat{Q}$.

$$Q_{imax} = \alpha\hat{Q}_i \tag{6}$$

where α is the same for all reservoirs. Therefore, the control level is described simply by specifying the value of α. A schematic of the entire process is shown in Figure III-2. The reservoir outflow is restricted to Q_{imax} until the reservoir is full or until the inflow falls below Q_{imax}. When the reservoir is full the control is relaxed and the total inflow is routed downstream until the inflow falls below Q_{imax}. The outflow is then maintained at Q_{imax} until the reservoir is empty. This drainage procedure is not realistic unless the reservoir is drained by pumping, which is not the case for planned upstream reservoirs in Vicente. However, because of the rapid response time of the sewer system, this was not regarded as important. For continuous simulation, the reservoir drainage aspects of the model should be revised. Mathematically the strategy can be described as

$$\begin{aligned}
Q_{out} &= Q_{in} && \text{for } Q_{in} < Q_{max} , S < S_{max} \\
Q_{out} &= Q_{max} && \text{for } Q_{in} \geq Q_{max} , S < S_{max} \\
Q_{out} &= Q_{in} && \text{for } Q_{in} \geq Q_{max} , S = S_{max} \\
Q_{out} &= Q_{max} && \text{for } Q_{in} < Q_{max} , S = S_{max}
\end{aligned} \tag{7}$$

where S = volume of storage in the reservoir, S_{max} = the reservoir capacity and Q_{in} and Q_{out} are the discharges immediately upstream and downstream respectively of a reservoir. Typical inflow-outflow hydrographs for this strategy are shown in Figures IV-1a and IV-1b.

It should be emphasized that this is not the only general strategy which could be investigated. Two other possibilities are:

1. Fill the reservoir to capacity during the initial part of the storm.
2. Restrict the outflow to a specified percent of the inflow for all inflows until the reservoir is filled.

Based on the results of the strategy tested, it does not appear that strategy (1) above will yield a favorable system performance. Strategy (2) is an interesting one which merits consideration, but perhaps would require more elaborate sensing and control mechanisms to be implemented.

The procedure followed to develop an overflow curve for a given control level (i.e., a specified value of α) was as follows:

a. A set of uniform intensity rainfall depths were chosen at a specified duration and the resulting overflows examined. The objective is to identify that rainfall depth at which one of the detention reservoirs just fills. This is done using a linear interpolation between a depth which does not quite fill the reservoir and one which causes a small overflow. The reservoir which fills first, which can be termed the critical reservoir, may be the downstream one, 12-2, or one of the three upstream reservoirs, depending on the value of α and rainfall duration.

b. Step (a) was repeated for various durations from 1 minute up to 4 hours.

c. The overflow depth for each duration forms an overflow curve as shown in Figure IV-2.

The most favorable simulated control level or value of α and the corresponding zero overflow curve can be obtained as follows:

d. Repeat steps (a) and (b) for various values of α.

e. For a specified duration plot curves of overflow depth vs. α for reservoir 12-2 and the upstream reservoir which has the lowest overflow depth. The depth at which these curves cross is the maximum depth which will not cause any overflow in the system and the corresponding value of α is the optimum.

f. Repeat step (e) for various durations.

g. The maximum overflow depth for each duration forms the optimum overflow curve and the corresponding values of α define the most favorable control level. In general, these values of α are a function of rainfall duration.

C.3 Continuous Simulation

The use of continuous simulation to develop control strategy or design parameters involves the following steps:

a. Obtain a continuous record in time of precipitation over the area. This could be in the form of precipitation during successive time increments. The data could be purely historical, purely synthetic or a combination of these.

b. Define a general control strategy which can be inserted in the simulation model in terms of specific control parameters.

c. For various values of the control parameters operate the model using the input data obtained in step (a).

d. Analyze statistically the model performance for each value of the control parameters.

e. By observing the change in model performance as a function of change in control parameters estimate an optimal control.

Continuous simulation has the advantage of incorporating all the variability which is implicit in the historical or synthetic input.

D. VICENTE SUBBASIN SIMULATION MODEL

The factors discussed in section B.2 were considered in developing the simulation model for the Vicente Subbasin. The Muskingum routing used in REACH and the single linear reservoir method used in BASIN are simplistic models of the physical processes which they represent. Therefore, a high degree of disaggregation is not justified. Furthermore, many combinations of storms, detention storage capacities and control strategies were to be investigated and the cost of these computer runs would be prohibitive on a highly disaggregated simulation.

It was desired, however, to explicitly consider the effects of each of the five proposed detention reservoirs in the Vicente Subbasin and to route the controlled flows from reservoirs 12-3, 12-4 and 12-5 into reservoir 12-2 (See Figure IV-4). The resulting simulation included nine subcatchments and twelve reaches. Figure IV-3 shows the delineation of the nine subcatchments superimposed on a USGS topographic map of the area. Figure IV-4 shows the physical arrangement of the five detention reservoirs, nine subcatchments and twelve reaches while Table IV-2 gives the input values used. Figure IV-5 is a schematic flow diagram of the simulation.

Note that overflow from detention reservoir 12-2 is not routed further even though this flow would be carried through existing lines which are tributary to detention reservoir 12-1. Although this situation could be modeled rather easily, it was not believed necessary as overflow from reservoir 12-2 would almost always contibute to additional overflow from reservoir 12-1.

E. RESULTS OF VICENTE SIMULATION

E.1 Zero Overflow Curves

Zero overflow curves for Vicente Subbasin were developed for the control strategy, using the procedure described in section C.2. The maximum outflow from the downstream reservoir 12-2 was equivalent to 0.3 in/hr runoff which implies a value of α of 1.0 for this reservoir. The results are shown in Figures IV-6 and IV-7 for design alternatives B

FIGURE IV-3

VICENTE ST. SUBBASIN - SUBCATCHMENT DELINEATION

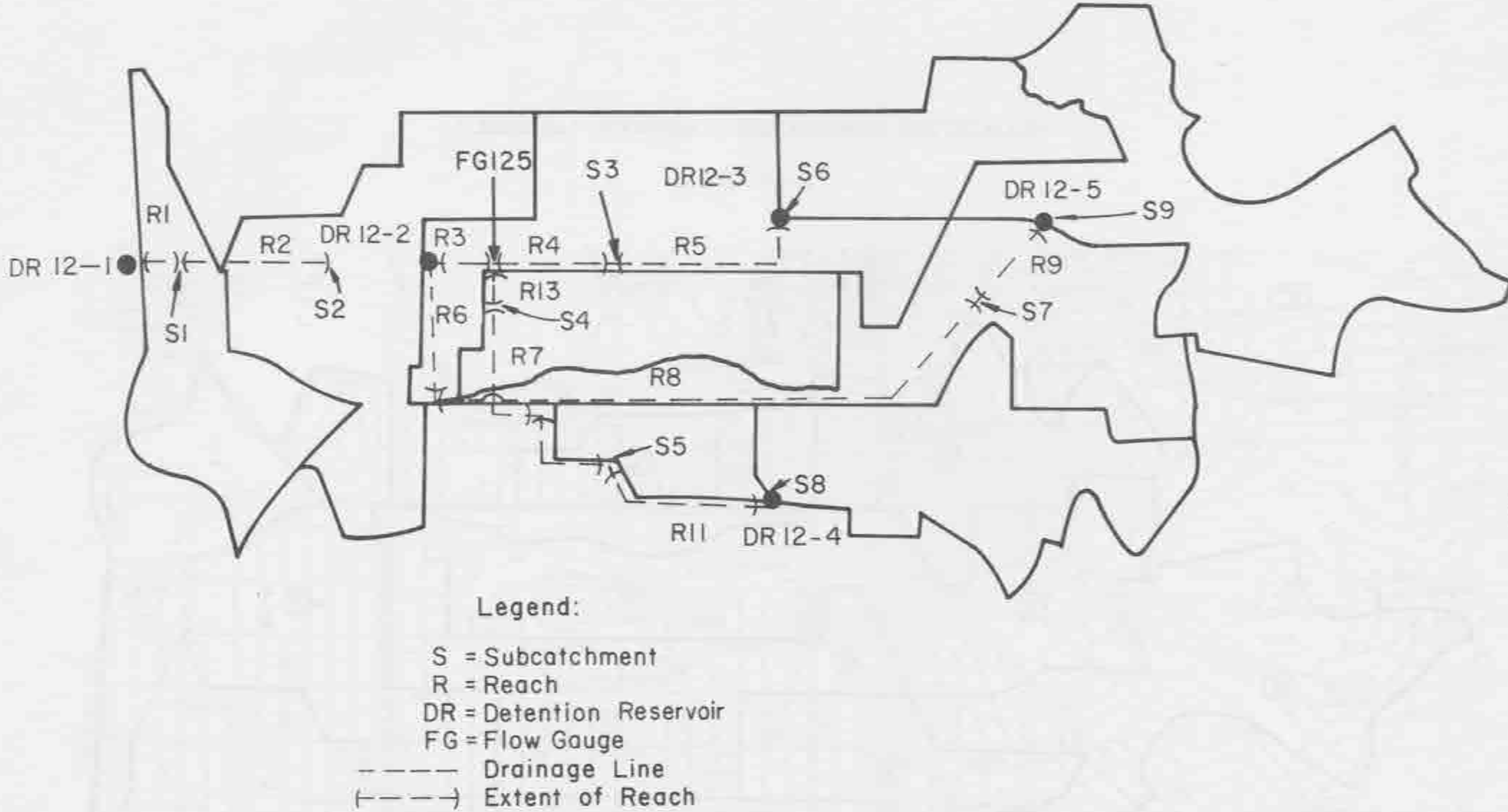


FIGURE IV-4

VICENTE ST. SUBBASIN SIMULATION MODEL

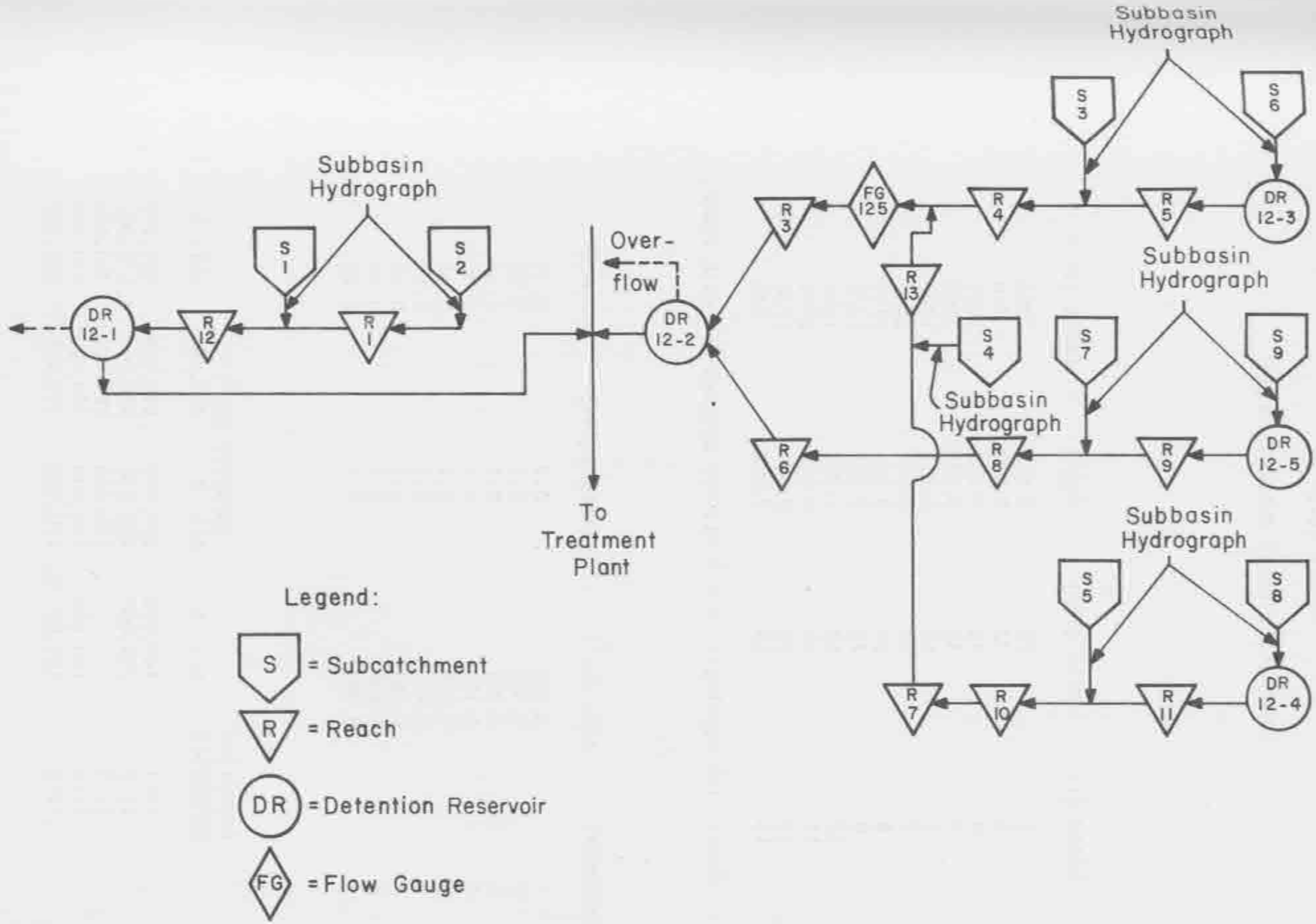


FIGURE IV-5

VICENTE ST. SUBBASIN - SCHEMATIC FLOW DIAGRAM

TABLE IV-2

INPUT FOR VICENTE SUBBASIN MODEL

REACH (1)	DIAM (in.)	SLOPE	LENGTH (ft.)
1	78	0.016	1300
3	72	0.020	740
4	42	0.042	1600
5	42	0.039	3500
6	60	0.015	1700
7	63	0.007	2600
8	60	0.026	7100
9	48	0.038	1400
10	60	0.005	1600
11	42	0.024	2300
12	78	0.029	1230
13	63	0.006	240

Note: (1) Manning's n = 0.012 was assumed for each reach.

SUBCATCHMENT	AREA (mi^2)	ASSUMED K(hrs)	ASSUMED C
1	0.242	0.2	0.35
2	0.336	0.1	0.65
3	0.291	0.1	0.65
4	0.156	0.2	0.35
5	0.116	0.1	0.65
6	0.202	0.1	0.65
7	0.461	0.1	0.65
8	0.431	0.1	0.65
9	0.347	0.1	0.65

DETENTION RESERVOIR	CAPACITY (ft^3) ALT. A	ALT. B	ALT. C	ALT. D
12-1	140,000	250,000	540,000	880,000
12-2	250,000	460,000	950,000	1,570,000
12-3	-	100,000	190,000	310,000
12-4	100,000	150,000	320,000	530,000
12-5	110,000	190,000	400,000	660,000

and D respectively. In each figure, the *no control* curve was the result of increasing the control level to the capacity of the pipes downstream of the detention reservoirs. The *reference* line is an idealized zero overflow curve whose intercept at zero duration is the total storage capacity of all five reservoirs in the subbasin converted to inches of rainfall and whose slope is 0.3 in/hr. It therefore has the equation

$$y_o = \frac{\sum S_{max}}{\overline{C}A} + 0.3t \tag{8}$$

where y_o = the overflow depth, $\sum S_{max}$ = the total storage volume, $\overline{C}$ = the average runoff coefficient for the subbasin and A = the subbasin area. The *most favorable control level* curve results from using the control level α which produces the highest rainfall depth which will not cause an overflow for each specific duration. Superimposed on these curves is a set of depth-duration-frequency curves prepared by the California Department of Water Resources [3] from the long term record of the San Francisco Federal Office Building raingage.

First of all, the shape of the two curves obtained from simulation should be noted. As the duration increases the slope of each approaches that of the reference line i.e., 0.3 in/hr. This implies that if the design capacity of the lines were reduced to some lower value, the zero overflow curves could be estimated by simply rotating the ones shown about their intersection with the depth axis until their new slope at high durations agreed with the new line capacity. At durations less than one hour the curves show a definite upward trend, particularly for the no control curves. This is because more of the storage capacity of the upstream reservoirs is being utilized as the rainfall duration decreases

The second important observation is the relative position of the three curves. The no control curve is below the others because little if any use is made of the upstream reservoir storage capacity at this control level. This is because the discharge from the subcatchments upstream of these reservoirs is lower than their discharge line capacities and so the flow is simply routed downstream to reservoir 12-2 without reservoir storage attenuation. The most favorable control level curve is the result of full utilization of the storage capacity of the smallest (in terms of storage volume per unit upstream drainage area) upstream reservoir as well as reservoir 12-2. Since all three upstream reservoirs did not have exactly the same storage capacity per unit drainage area the one with the smallest value was filled first. However, in all cases the other two were usually 95 percent full as well. This use of upstream storage is the difference between the two curves. This is shown graphically in Figures IV-6 and IV-7 by observing the rainfall depth which is equivalent to the upstream storage capacity. As the duration increases the two curves differ exactly by this amount.

These results confirm a conclusion which may be intuitive. That is the most favorable control level of a strategy makes full use of all reservoir storage capacity. Furthermore, for the particular control strategy studied, the reservoir capacities should be in proportion to their respective drainage areas. Otherwise, an improved curve would result from allowing α to vary from reservoir to reservoir.

Since the most favorable control level curve is the result of the use of various control levels, the manner in which the level must be set must be specified. The variation of α with rainfall duration to produce this curve is shown in Fig. IV-8 for the two alternatives.

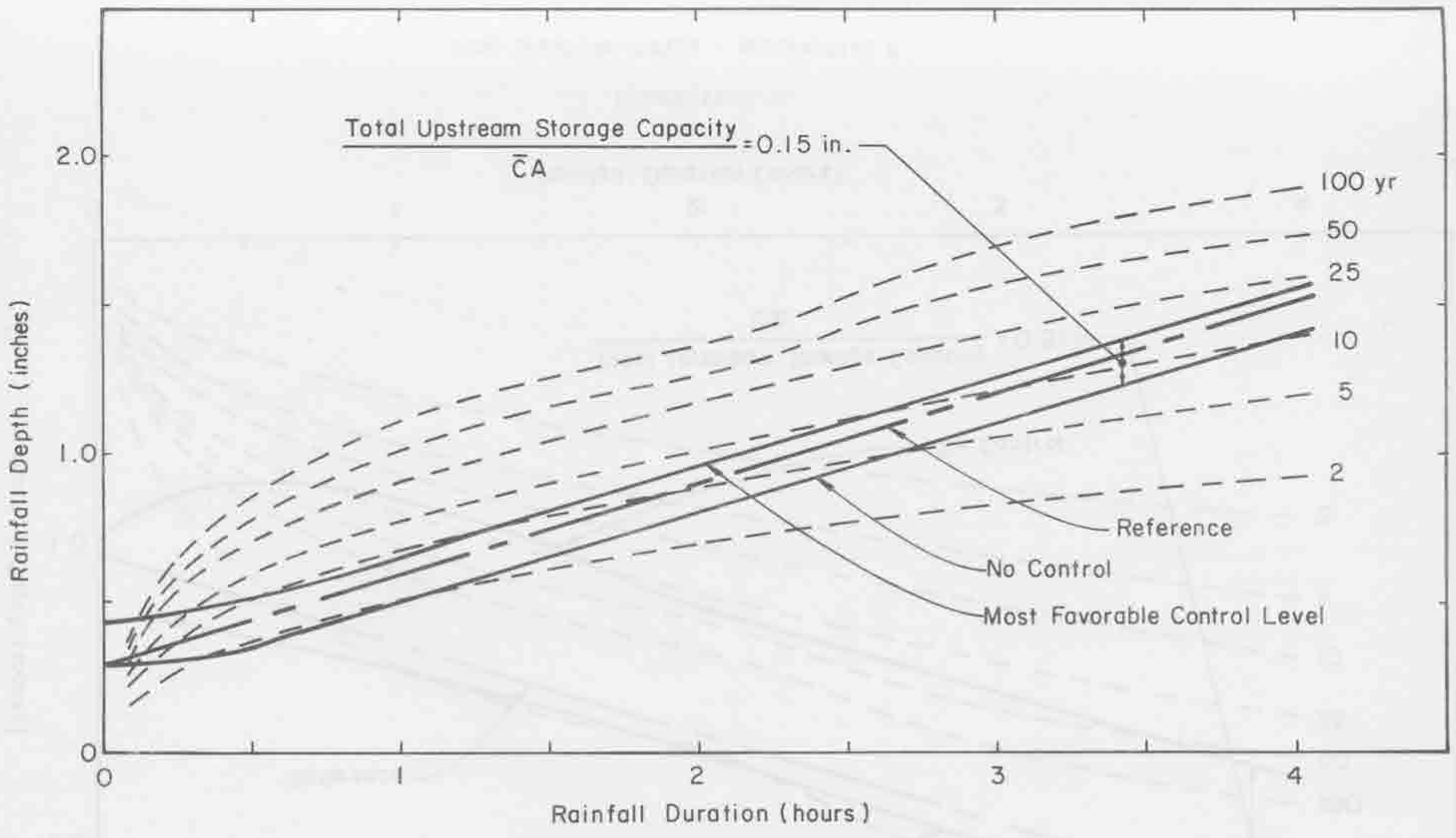


FIGURE IV-6

ZERO OVERFLOW CURVES - ALTERNATIVE B

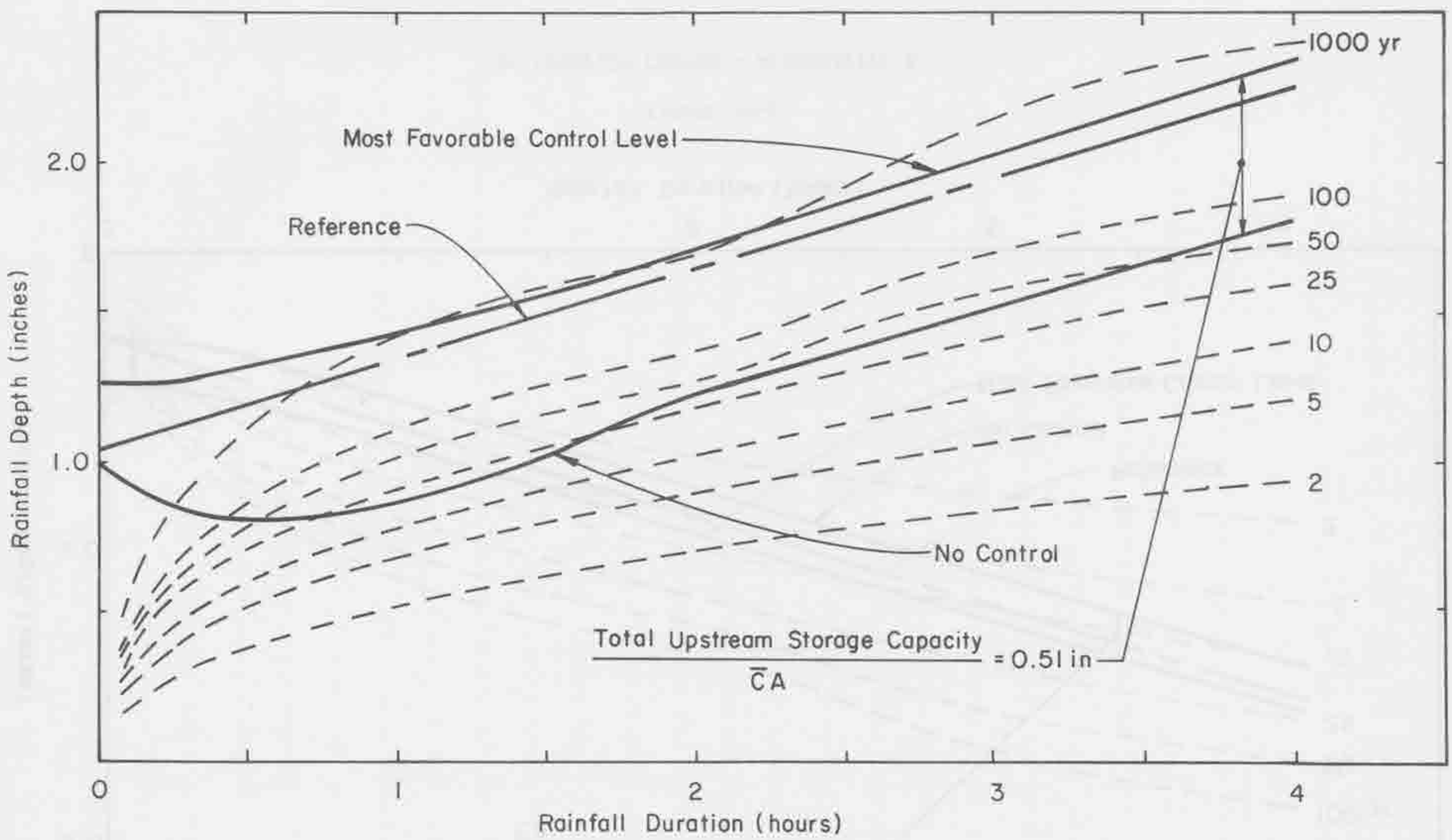


FIGURE IV-7

ZERO OVERFLOW CURVES - ALTERNATIVE D

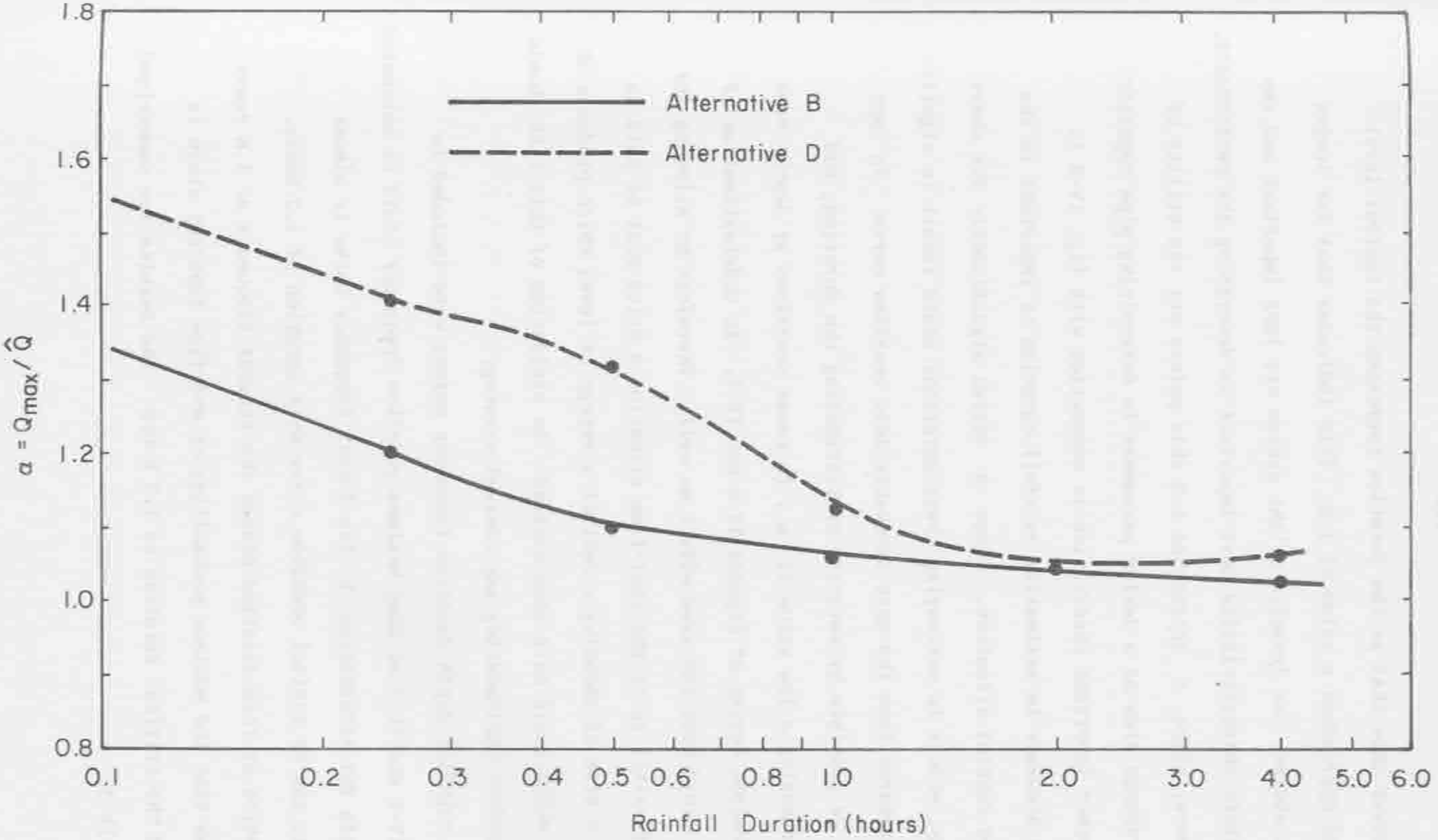


FIGURE IV-8

MOST FAVORABLE CONTROL LEVEL VS. DURATION

These curves show that as the duration increases the control level drops and approaches a value of 1.0. This indicates that for longer duration storms, the dynamics of the system are less important and the steady state characteristics more important in describing the performance. Furthermore, since α approaches 1.0 this points out the utility of using drainage area as a design parameter in determining pipe capacity.

Another important observation in connection with Fig. IV-8 is that the accuracy in estimating rainfall duration is important in the real-time control situation. Since α varies significantly for short durations, errors in estimating these durations could result in significant departures from the most favorable zero overflow curve. In fact it would be possible by severely overestimating the duration, and thereby choosing a low value of α, to cause overflows at depths *near* the *no control* curves of Figures IV-6 and IV-7. An underestimation of duration could have the same effect as well. Therefore by allowing for control flexibility in the real-time situation a price must be paid in terms of a risk of choosing a control strategy or level which produces an overflow which could have been avoided. An evaluation of this risk should be done before implementing any control strategy.

The rainfall depth-duration-frequency curves were included in Figures IV-6 and IV-7 so that maximum overflow frequency could be estimated. For example for Alternative B The 2-year frequency curve is almost tangent to the *no control* overflow curve at a duration of 1.2 hours. . Interpolation at this duration places the tangent frequency at 1.6 years. This means that the *maximum* probability of overflow from any storm is 0.625 and the critical duration as 1.2 hours. The results are summarized in Table IV-3.

TABLE IV-3

OVERFLOW PROBABILITY

	No Control Strategy		Most Favorable Control Strategy	
	Alter. B	Alter. D	Alter. B	Alter. D
Maximum overflow frequency	1.6 yrs.	18 yrs.	4.5 yrs.	1000 yrs.
Maximum overflow probability	0.625	0.056	0.222	0.001
Critical duration	1.2 hrs.	1.1 hrs.	1.6 hrs.	1.3 hrs.

The maximum overflow probability values provide simple means of comparing the results of strategies as well as different design alternatives. For example the table shows that for Alternative B the maximum probability of an overflow in any year using the most favorable control level is one-third the value using the no control strategy. This demonstrates that the use of proper control strategy can result in significant improvement in system performance. For Alternative D the relative improvement in maximum overflow probability is even greater. However, it should be recognized that there is a high level of uncertainty associated with the 1000-year frequency curve in Figure IV-7. On the other hand, it is reasonable to conclude that Alternative D could provide an extremely high level of protection against overflows, one which closely approaches satisfying the *no overflow* goal of the Federal Water Quality Act. One final observation is that the critical durations are all from 1.0 - 1.5 hours, which again emphasizes the importance of short-duration high-intensity storms in evaluating system performance.

E.2 Overflow Volume from Zero Overflow Curves

As described in section C.3 this approach can be used to generate overflow volume-probability curves if overflow volume can be predicted.

As study was made using the model for alternative B in which the total volume of overflow from reservoirs 12-2, 12-3, 12-4 and 12-5 was converted to equivalent rainfall depth over the drainage area for these reservoirs was compared to that computed from the difference between the rainfall depth at a given duration and the zero overflow depth for that duration. Figure IV-9 shows the result when using the most favorable control level curve of Figure IV-6. In Figure IV-9 $Y - Y_F$ is the difference between rainfall depth Y and the corresponding depth for the most favorable simulated control level overflow curve Y_F while Y_{OF} is the total volume of overflow computed by the model from the four reservoirs mentioned above converted to inches by dividing by their drainage area. It is interesting to note that the relationship of Figure IV-9 could be used in the generation of the overflow volume-probability curve using the most favorable control level control strategy.

This same procedure was followed using the *no control* strategy for purposes of comparison. The results are shown in Figure IV-10 where the variables are the same as the previous figure except that Y_{NC} represents the corresponding ordinate on the no control overflow curve and Y_{OF} was computed from the model output using the no control strategy. In this case the linear relationship still exists but the slope of the line is a function of rainfall duration for durations below one hour. This is not surprising since the control level α varied with duration for Figure IV-9 while it was constant (approximately 3.5 for the upstream reservoirs) for Figure IV-10.

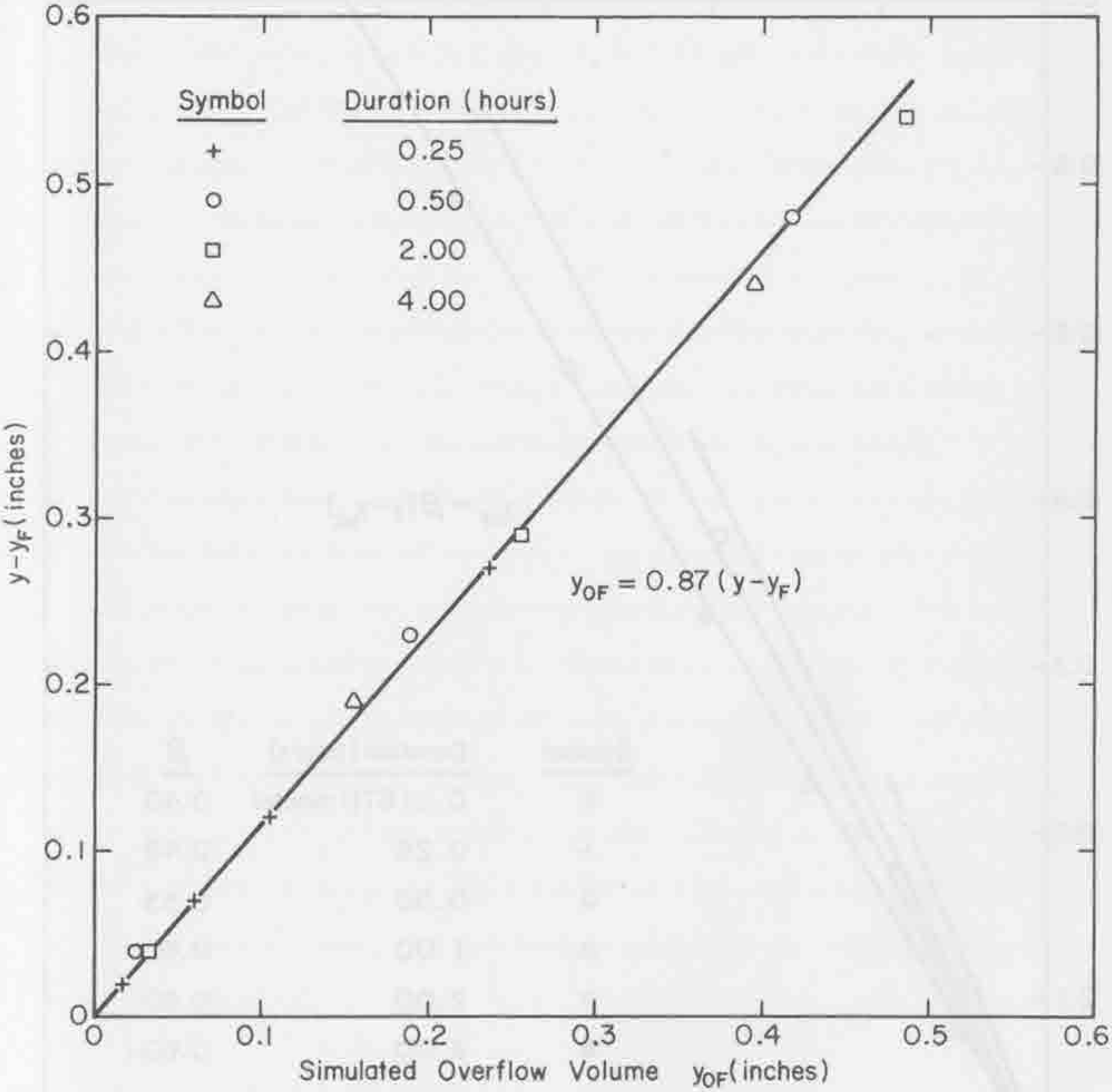


FIGURE IV-9

OVERFLOW VOLUME USING MOST FAVORABLE CONTROL LEVEL FOR ALTERNATIVE B

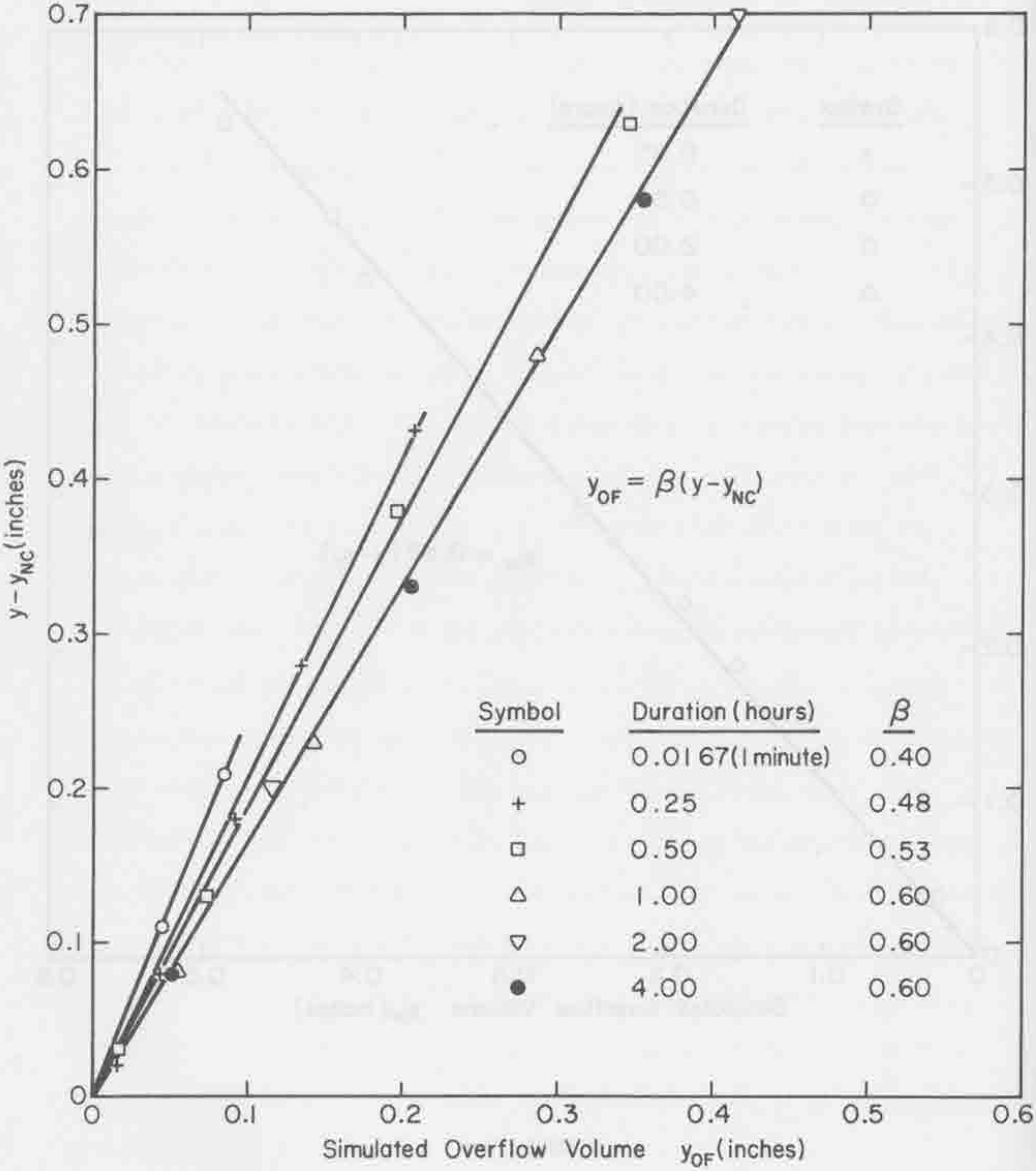


FIGURE IV-10

OVERFLOW VOLUME USING NO CONTROL
STRATEGY FOR ALTERNATIVE B

F. EXTENSION OF OVERFLOW CURVE METHOD TO OBTAIN OVERFLOW PROBABILITY

Although valuable information can be obtained from zero overflow curves, additional system performance description is needed. Recognizing that rainfall is a stochastic process, it is necessary to realize that the goal of complete elimination of all overflows is an idealization and is normally not practical for a real system such as San Francisco. Therefore, it is necessary to describe performance in terms of the probability of a given performance parameter being exceeded. Two parameters of interest are (i) average number of overflows in a given period of time and (ii) the volume of overflow in that period.

This procedure requires an analysis of the historical data together with the zero overflow curves. First consider the number of overflows in a given period as the performance parameter of interest. The rainfall data can be analyzed in terms of a depth-duration matrix. Any element in the matrix n_{ij} consists of the number of storms with depth within a specified interval $y_i < y \leq y_i + \Delta y$ and duration within an interval $t_j < t \leq t_j + \Delta t$. These intervals should be small enough so that the zero overflow curve can be approximated by a step function made up of these intervals. The joint probability mass function (PMF) ordinates are then given by

$$f_{Y,T}(y_i,t_j) = \frac{n_{ij}}{\sum_i^{\infty} \sum_j^{\infty} n_{ij}} \tag{9}$$

The sum of all joint PMF ordinates above the zero overflow curve is the *probability of any storm causing an overflow.*

$$P[\text{any storm causing overflow}] = \sum_{j=1}^{\infty} \sum_{i=g(j)}^{\infty} f_{Y,T}(y_i,t_j) \tag{10}$$

where $i = g(j)$ describes the relationship between i and j for the step function describing the overflow curve.

By including in the depth-duration matrix those periods in which no rain occurred, a joint PMF is generated whose sum above the zero overflow curve is the *probability of an overflow at any time*. These dry periods can be viewed as storms of zero depth and finite duration and would appear as entries in the first row of the matrix. The expression for the probability is the same as Equation 10, except for the interpretation of the probability.

The distribution of the *number* of overflows in a given period can be developed in two ways. Using the depth-duration matrix the average number of overflows in M periods is given by

$$\bar{n} = \frac{1}{M} \sum_{j=1}^{\infty} \sum_{i=g(j)}^{\infty} n_{ij} \tag{11}$$

This could be assumed to be the mean of an assumed distribution and the PMF thereby identified. A Poisson distribution would be a reasonable assumption in which case the probability of equalling or exceeding a certain number of overflows n_o in any period is given by

$$P[N \geq n_o] = 1 - \sum_{n=o}^{n_o - 1} \frac{\bar{n}^n e^{-\bar{n}}}{n!} \tag{12}$$

An alternative to assuming a distribution is to develop one from the historical data. This involves forming depth-duration matrices for each of the M periods and computing the number of occurences N_m above the zero overflow line for each period m, where $m=1,2,\ldots,M$. A PMF $f_N(n)$ is then developed for the number of occurrences of a particular value of N_m. The desired probability is then given by

$$P[N \geq n_o] = 1 - \sum_{n=0}^{n_o - 1} f_N(n) \tag{13}$$

In order to estimate the probability of exceeding a given overflow volume using the zero overflow curve, a relationship between rainfall depth and overflow volume is required. Such a relationship for the Vicente subbasin is described in the next section. A general relationship has the form

$$V = g(y - y_o) \tag{14}$$

where V = the volume of overflow, y = the rainfall depth and y_o = the zero overflow depth for some control strategy corresponding to the duration of the rainfall. In terms of the depth-duration matrix an overflow volume V_{ij} can be associated with each depth and duration, where $V_{ij} = 0$ for all points below the zero overflow curve. The average overflow volume for any storm is then given by

$$V = \frac{\sum_{j=1}^{\infty} \sum_{i=g(j)}^{\infty} V_{ij} n_{ij}}{\sum_{i=1}^{\infty} \sum_{j=1}^{\infty} n_{ij}} \tag{15}$$

where n_{ij} = the number of storms in the historical record with depth and duration associated with i and j respectively. The average overflow volume per overflow event would be given by Equation IV-15 with the lower limit on the summation in the denominator the same as that for the numerator.

The PMF for overflow volume can be developed using the depth-duration matrices developed for each period in the development of the PMF for the number of overflows in a given period. However, in this case the assumption of a Poisson distribution is not warranted. For each period m, the total overflow volume is computed as

$$V_m = \sum_{j=1} \sum_{i=g(j)} V_{ij} n_{ij} \tag{16}$$

A PMF is then developed whose ordinates are given by

$$f_V(v_k) = \frac{n_v}{M} \tag{17}$$

where n_v = the number of periods in which V_m lies within a specified interval around v. Since overflow volume is a continuous variable a probability density function could theoretically be approached if M increased and the interval around v decreased. However, from a practical viewpoint the PMF must be used. The probability of exceeding a given overflow volume in any period is then given by

$$P[V \geq v_o] = 1 - \sum_{o}^{v_o} f_V(v) \tag{18}$$

This approach has some advantages over continuous simulation and some disadvantages as well. The production of a zero overflow curve is very helpful in gaining an understanding of the performance of the system to various control strategies. In addition, since a long rainfall history is not processed through the model, less computer time may be involved. A disadvantage is that a uniform temporal distribution of rainfall was assumed as well as constant runoff coefficients in developing the zero overflow curves and in the subsequent statistical analyses. Since these assumptions are not actually satisfied there is some level of uncertainty associated with the results. This can be viewed in terms of an uncertainty associated with the zero overflow curve which in turn would be associated with the temporal variation of rainfall intensity. An estimate of this could be gained by developing zero overflow curves for various specified temporal rainfall patterns with fixed control stragegy and observing the variations in the location of the curve. Another disadvantage is that interaction between successive storms is not considered. That is, it is assumed that the reservoirs are empty at the beginning of each storm. The drainage time for the Vicente reservoirs is estimated at less than one hour for Alternate A to approximately four hours for Alternative D. It is very unlikely that successive large storms would be so closely spaced in time and so this problem is not regarded as serious.

G. APPLICATION OF VICENTE ZERO OVERFLOW CURVES

In order to demonstrate the procedure described in Section F the hourly precipitation data recorded by the gage on the Federal Office Building in San Francisco from 1907 to 1972 were used. The two overflow curves for Alternative B shown in Figure IV-6 were used to determine if a storm caused overflow. The criterion used was that if the mass curve for the storm fell above the overflow curve at any time that it was assumed that the storm caused an overflow. The volume of overflow was computed from Figures IV-9 and 10 where $Y - Y_{0F}$ was taken as the maximum value which occurred throughout the storm. This latter procedure was an attempt at accounting for the non-uniformity of the historical data.

The criteria for defining a storm is important. Unless continuous simulation is employed, some criteria for storm identification is necessary. In this case, a hydraulic analysis of the detention reservoirs in Vicente subbasin indicated that three hours would be sufficient emptying time if there were no inflow during this period. Therefore, the end of a storm was established if at least three successive hours of zero rainfall followed any hour of non-zero rainfall.

It was also observed that the historical record contained a significant number of hourly values less than or equal to 0.05 in. The effect of these data was to increase the duration of storms without significantly increasing the total depth, thereby making the storms less likely to exceed the zero overflow curve. Therefore, several criteria for rainfall depth were imposed. The resulting number of storms defined in the 66 years of record are summarized in Table IV-4.

Table IV-4

Number of Storms in Historical Record

Depth Criteria	Description	Number of Storms
A	All hourly values used	5188
B	Storms with total depth $\leq$ 0.05 in. ignored	2995
C	All hourly values $\leq$ 0.05 in. ignored	2893
D	All hourly values $\leq$ 0.05 in. ignored prior to beginning of storm but included within the storm	2342

The significant observation from this table is the large reduction in number of storms using criteria B, C, and D. This indicates that a large number of small storms occur which do not potentially generate overflows, since a storm of 0.05 in/hr falls within the capacity of the proposed treatment plant and a one hour storm with a total depth of 0.05 in. is within the storage capacity of the detention reservoirs.

In addition to the overflow curves of Figure IV-6, several other estimated overflow curves were used to simulate other situations of interest. Since the curvature for the curves of Figure IV-6 for durations greater than one hour was insignificant, they and all others used were approximated by straight lines. This would not be justified if the time increment on precipitation was less than one hour. A total of seven curves was used. Their equations are summarized in Table IV-5 and their significance discussed below.

Table IV-5

Zero Overflow Curves

Designation	Equation
AA*	$Y_{OF} = 0.3T + 0.35$**
AA	$Y_{OF} = 0.3T + 0.22$
BB*	$Y_{OF} = 0.1T + 0.43$
BB	$Y_{OF} = 0.1T + 0.30$
CC	$Y_{OF} = 0.1T$
DD	$Y_{OF} = 0.02T$
EE	$Y_{OF} = 0$

** T = storm duration in hours
Y_{OF} = overflow depth in inches

Curves AA* and AA are the linearized versions of the curves of Figure IV-6 using the most favorable and no control strategies, respectively. Curves BB* and BB are *estimates* of the corresponding curves assuming the maximum flow from reservoir 12-2 (termed reservoir 4) into the interceptor was limited to 0.1 in/hr rather than the 0.3 in/hr for curves AA* and AA, i.e., $\alpha_4 = 1/3$. This is a much more realistic estimate when considering the operation of Vicente Subbasin as a part of the entire city, since the proposed treatment plant capacity is 0.1 in/hr. Curves CC , DD and EE are estimates for the case of zero detention storage capacity and treatment capacity rates of 0.1, 0.2 and 0.0 in/hr respectively. These curves were included to demonstrate the dramatic improvement caused by the presence of detention storage capacity using even a no control operating strategy.

Table IV-6

Average Number of Overflows per Year

Depth Criteria	Overflow Curve						
	AA*	AA	BB*	BB	CC	DD	EE
A	0.05	0.06	0.91	1.88	10.12	45.70	78.61
B	0.05	0.06	0.91	1.88	10.12	41.77	45.38
C	0.09	0.15	1.88	3.45	21.94	43.83	43.83
C*	0.20	0.48	1.91	3.50	25.38	43.83	43.83
D	0.08	0.12	1.61	3.02	18.44	35.48	35.48

C* = Depth criterion C used. Overflow occurred if precipitation for any hour during a storm exceeded overflow curve for first hour.

Table IV-7

Average Volume of Overflow per Year (in.)

Depth Criteria	Overflow Curve						
	AA*	AA	BB*	BB	CC	DD	EE
A	0.006	0.009	0.216	0.257	1.82	12.81	20.36
B	0.006	0.009	0.216	0.257	1.82	12.75	19.64
C	0.011	0.018	0.386	0.468	3.56	11.93	14.62
C*	0.022	0.042	0.393	0.475	3.69	11.93	14.62
D	0.011	0.016	0.356	0.420	3.07	12.61	17.23

Table IV-8

Average Volume of Overflow per Overflow (in.)

Depth Criteria	Overflow Curve						
	AA*	AA	BB*	BB	CC	DD	EE
A	0.130	0.142	0.237	0.137	0.180	0.280	0.259
B	0.130	0.142	0.237	0.137	0.180	0.305	0.433
C	0.123	0.118	0.205	0.135	0.162	0.272	0.334
C*	0.114	0.087	0.206	0.136	0.145	0.272	0.334
D	0.143	0.128	0.222	0.139	0.167	0.355	0.486

The results of the procedure are summarized in Tables IV-6, 7, 8 and 9 in terms of average values. In addition, probability distributions for number of overflows and volume of overflow per year were computed.

Table IV-9

Average Duration of Storms Causing Overflow (hrs.)

Depth Criteria	Overflow Curves						
	AA*	AA	BB*	BB	CC	DD	EE
A	4.67	4.00	14.19	12.40	9.04	7.94	5.64
B	4.67	4.00	14.19	12.40	9.04	8.54	8.46
C*	4.30	3.72	8.13	6.86	3.88	3.10	3.10
C	5.68	6.89	8.07	6.85	4.09	3.10	3.10
D	8.50	7.00	13.67	11.17	8.04	7.20	7.20

Depth criteria C* was introduced to more realistically account for a storm having a high hourly intensity in the latter portion of its duration, representing a severe departure from the uniform case for which the overflow curves were developed.

Table IV-10 is a summary of the most extreme results from Tables IV-6, 7 and 8 so that comparisons can easily be made. The most dramatic result is the improvement gained by the availability of detention storage. However, of greater significance to this study is the improvement gained by the use of the most favorable control strategy over the no control strategy. These relative changes, which were determined using the data in Table IV-10, are shown in Table IV-11. This shows an 86 percent reduction in both average number and volume of overflows per year through the availability of Alternative B storage operated using a no control strategy. Furthermore a significant *additional* improvement in both variables, ranging from 17 to 58 percent, is obtained by using the most favorable control strategy. It should also be noted that the use of the favorable control strategy produces an *increase* in the average volume of overflow per overflow event over that for the no control strategy, even though the overflow volume per year is reduced. In other words, the favorable control strategy produced in this case larger average overflow volumes per overflow event but this was more than offset by the reduction in average number of overflows per year.

Table IV-10

Summary of Results Using Overflow Curves

Detention Storage	Most Favorable Control	No Control	Maximum Interceptor Flow (in/hr)	OF per Yr	Vol. of OF per yr(in)	Vol. of OF per OF(in)
Yes		X	0.30	0.48	0.042	0.087
Yes	X		0.30	0.20	0.022	0.114
Yes		X	0.10	3.50	0.475	0.136
Yes	X		0.10	1.91	0.393	0.206
No		X	0.10	25.38	3.690	0.145
No		X	0.02	43.83	11.930	0.272
No		X	0.00	43.83	14.620	0.334

OF = Overflow

Table IV-11

Relative Change in Average System Performance

Comparison Basis	Maximum Interceptor Flow (in/hr)	OF per yr	Percent Change Vol. of OF per yr	Vol. of OF per OF
No storage--Storage (no control)	0.1	-86.2	-87.1	-06.2
No control--Favorable control	0.1	-45.4	-17.3	+51.5
No control--Favorable control	0.3	-58.3	-47.6	+31.0

Although the use of average values conveys much information, a more complete picture of performance requires that the probability distribution be known. Figures IV-11 and 12 show the complimentary cumulative distribution functions for two of the variables summarized in Tables IV-6 and 7. Comparison of the shape of the curves for overflow curves BB and BB* ($\alpha_4 = 1/3$) shows that the distribution is not significantly changed through the use of the most favorable control strategy. A Poisson distribution (Equation 12) was fitted to the overflows per year data using the mean values in Table IV-6. The fit is particularly good for the two cases with storage.

H. Semi-Continuous Simulation for Vicente Subbasin

If computer costs were of no concern continuous simulation would be superior to the overflow curve approach. It has the distinct advantage of direct use of historical data without requiring any assumptions concerning the uniformity of the rainfall. In order to compare the Vicente subbasin drainage system performance generated using the overflow curve approach to that generated by continuous simulation, and still keep computer costs down, a modified or semi-continuous simulation approach was employed.

In this approach, individual storms were used as input to the Vicente simulation model described in Section IV-D. The output from the model was the total depth and duration of the overflow from each storm. The storms were defined according to criterion C of Table IV-4 and all reservoirs were assumed to be empty at the beginning of each storm. Furthermore all runoff coefficients were assumed constant. The approach is essentially the same as that of Section IV-D except that the Vicente

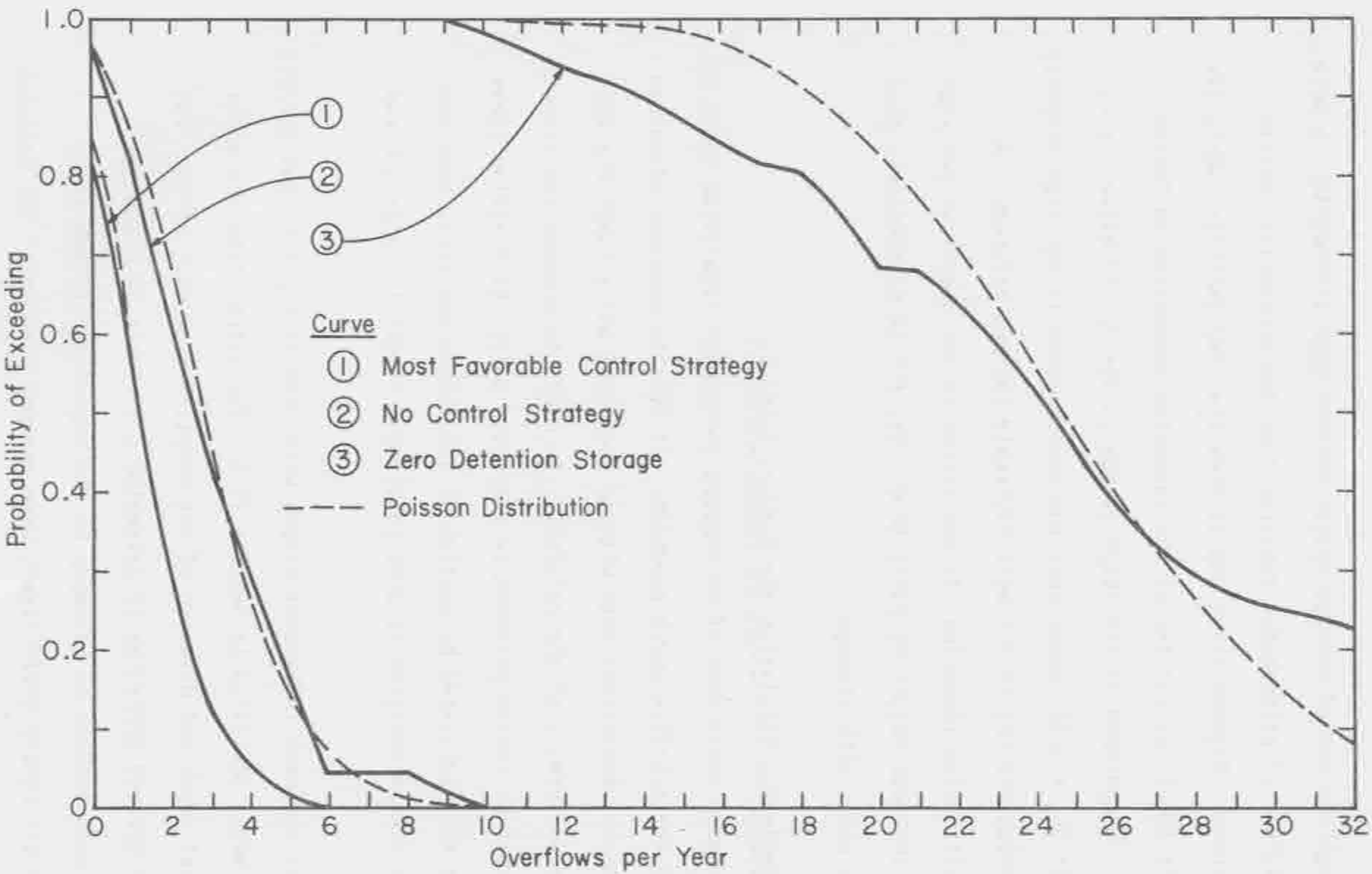


FIGURE IV-11

PROBABILITY DISTRIBUTIONS - OVERFLOWS PER YEAR

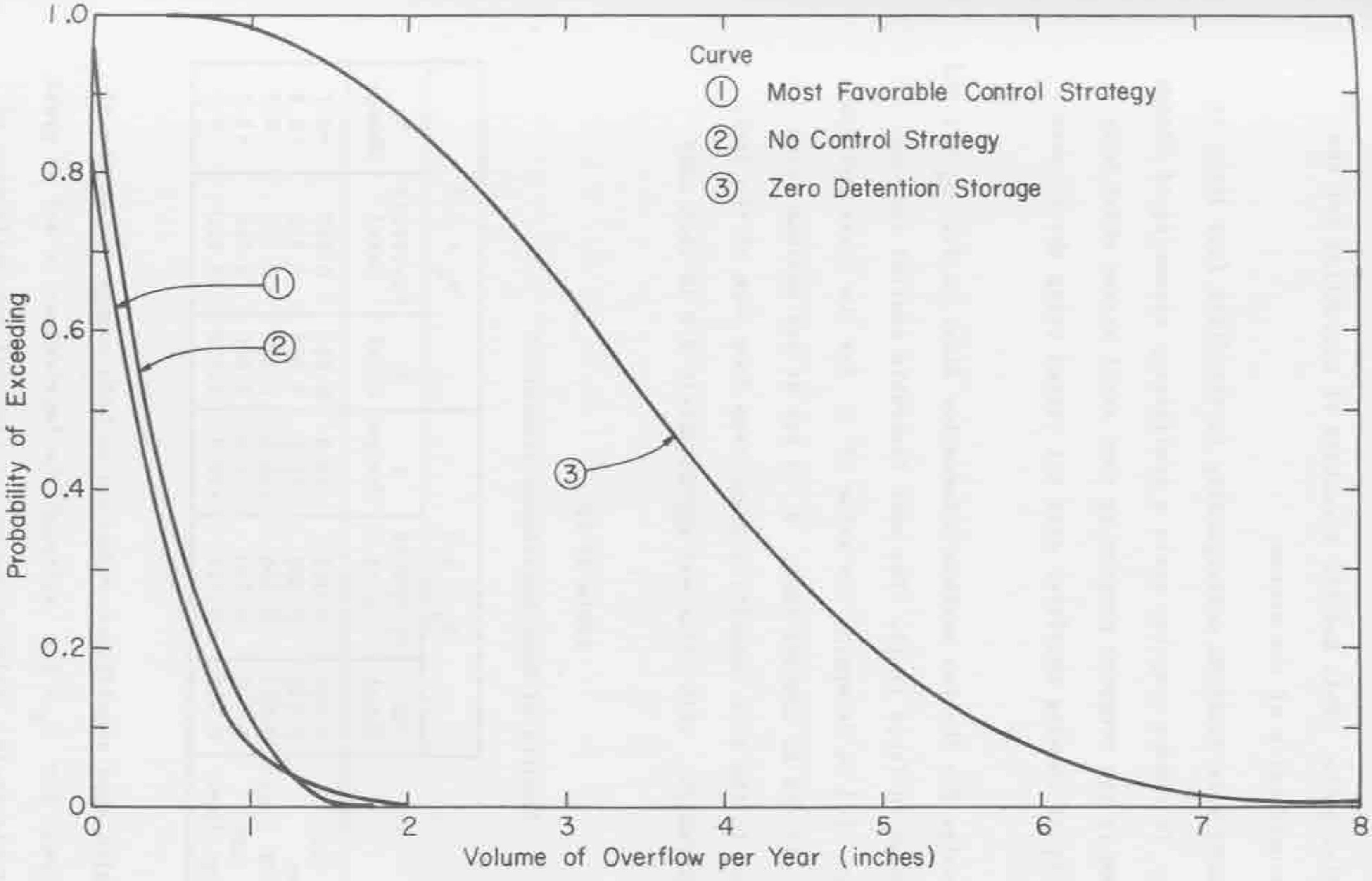


FIGURE IV-12

PROBABILITY DISTRIBUTIONS - VOLUME OF OVERFLOW PER YEAR simulation model is directly used to determine overflows rather than a zero overflow curve. This has the advantage of accounting for the temporal non-uniformity of the storms.

Four cases were examined corresponding to the first four lines in Table IV-10. To reduce computer costs a preliminary screening of storms was performed in the computer program so that small storms which have no possibility of causing overflows were not tested using the Vicente model.

The results for the two maximum interceptor flows, i.e., $\alpha_4 = 1$ and $1/3$, are shown in Table IV-12. The most favorable control case employed Figure IV-8 to determine the value of α for the three upstream reservoirs. In the no control case, α is set at its maximum value corresponding to the line capacities downstream from each of the three upstream reservoirs. This value was approximately 3.0 in each case.

Table IV-12

Results of Semi-Continuous Simulation

	$\alpha_4 = 1.0$			$\alpha_4 = 1/3$		
	No Cntrl	Favorable Cntrl	% Change	No Cntrl	Favorable Cntrl	% Change
Ave. vol. OF/yr (in)	0.058	0.032	-44.8	0.953	0.960	+0.7
Ave. number OF/yr	0.640	0.300	-53.1	7.390	7.450	+0.8
Ave. vol. of OF/OF (in)	0.091	0.106	+16.5	0.129	0.129	0.0
Ave. dur. of OF (hrs)	0.770	0.730	-5.2	2.000	2.060	+3.0
Ave. OF storm dur. (hrs)	6.950	7.710	+10.9	5.910	5.910	0.0

The results show significant reduction in both number and volume of overflow per year for $\alpha_4 = 1.0$ although the improvement is not as great as shown in Table IV-10. This is due to the temporal non-uniformity of the storms which was directly accounted for in this approach. However, the results for $\alpha_4 = 1/3$ show essentially no change from the no control to the favorable control strategy. This is explained by the fact that the favorable strategy was developed using $\alpha_4 = 1.0$ and is obviously not applicable if α_4 changes substantially. A strategy which does give improved performance for $\alpha_4 = 1.0$ and 1/3 is discussed in Chapter V.

I. CONCLUDING COMMENTS ON SIMULATION

The methods described in this chapter clearly show that simulation is a useful tool for both design and control strategy development. The relative advantage of one type of simulation approach over another is of secondary importance to the realization of the value of the concept itself. Although water quantity parameters were used here, quality parameters could be included in the model and used in evaluating the system in the same manner as described herein.

It is important to recognize the role that uncertainty plays in the performance of the system and therefore to describe it in probabilistic terms. Not only is the rainfall itself a stochastic variable but any attempt to predict the depth and duration of a particular storm, which is necessary when using real time optimal control, will contain uncertainty as well. Therefore, any performance description which does not recognize and account for uncertainty can be very misleading. The results shown here incorporate the stochastic nature of the historical rainfall record but do not account for uncertainty in storm prediction. The improvement in system performance, as shown in this chapter, through the use of real-time control strategy is significant and is an important conclusion. However, it must be recognized that the effect of uncertainty in storm prediction has not been evaluated. This evaluation is a necessary step in the final formulation of the control strategy for the total city system.

Finally, the importance of good data for model calibration must be emphasized. It is easy to accept the output of a simulation study without giving much thought to validity of the model, particularly when sufficient calibration data are not available, as is the case for many urban studies. The City of San Francisco is fortunate in having the most extensive rainfall-runoff monitoring system in the country. As more data are gathered this system will be the basis for the development of a design and control system which can be operated with a high level of confidence.

J. ADDITIONAL POSSIBLE SIMULATION STUDIES

Simulation is a very powerful tool and permits many different situations to be studied. The following studies, which are not ranked, will provide useful information in the development of the final control strategy to be implemented.

1. Investigate other reasonable general control strategies such as initial storage and storage which is proportional to reservoir inflow.
2. Perform a study of a subbasin having primarily a series system of detention reservoirs rather than the parallel system of Vicente.

3. If possible identify an elemental catchment in San Francisco having a flow gage at its outlet and apply the parameter identification model to obtain better estimates of the model parameters.
4. Develop guidelines for estimating the optimum level of aggregation in formulating a subbasin model.
5. Coordinate the use of simulation and formal optimization techniques for the development of design and control strategy.

CHAPTER IV - NOTATION

A = cross sectional area of flow

A_b = area of basin

b = width of free surface

c = celerity of a flood wave

C = runoff coefficient

$f_V(v)$ = probability mass function for overflow volume

$f_{Y,T}(y_i, t_j)$ = joint probability mass function for rainfall depth y_i and duration t_j

K = routing constant

M = number of periods in a record

n = Manning's roughness

n_{ij} = number of storms in depth interval i and duration interval j

N_m = number of storms above a zero overflow curve in period m

PE = precipitation excess

$P[N \geq n_o]$ = probability of equalling or exceeding n_0 overflows in any period

$Q(t)$ = discharge at a function of time

$\hat{Q}_i$ = reference discharge for detention reservoir i

Q_{in} = discharge immediately upstream of a detention reservoir

Q_{out} = discharge immediately downstream of a detention reservoir

Q_{PC} = peak discharge of calculated hydrograph

Q_{PO} = peak discharge of observed hydrograph

Q_{imax} = maximum controlled outflow from detention reservoir i

Q_c^k = calculated discharge at time k

Q_j^k = discharge at location j at time k

Q_o^k = observed discharge at time k

$R(k)$ = rainfall hyetograph ordinates

$R_e(k)$ = rainfall excess in inches

S = volume of reservoir storage

S_o = slope of pipe

S_{max} = reservoir storage capacity

t = time

t_{PC} = time to peak for calculated hydrograph

t_{PO} = time to peak for observed hydrograph

T = duration of precipitation excess

U = ratio of impervious basin area to total area

V = overflow volume

$V_e(t)$ = volume of rainfall excess

y = rainfall depth

y_F = overflow depth using most favorable control level

y_O = overflow depth for reference overflow line

Y_{OF} = overflow volume inverted to equivalent depth

Y_{NC} = overflow depth using no control strategy

α = control level parameter

β = Muskingum routing weighting factor

Δt = time increment

Δx = reach length

τ = travel time of flood wave through a reach

CHAPTER IV

REFERENCES

1. Benjes, H. H., et.al., "Storm-Water Overflows from Combined Sewers," Journal of the Water Pollution Control Federation, Vol. 33, 1961.

2. Brandstetter, A., R. L. Engel, and D. B. Dearlock, "A Mathematical Model for Optimum Design and Control of Metropolitan Wastewater Management Systems," Battelle Pacific Northwest Laboratories, Richland, Washington, April, 1973.

3. California Department of Water Resources, "Short-Duration Precipitation Frequency in California," 1972.

4. Chen, C. K. and W. W. Saxton, "Combined Wastewater Overflows," Journal of the Water Pollution Control Federation, Vol. 45, No. 3, March, 1973.

5. Crawford, N. H., "Computer Simulation for Design Criteria for Urban Flow Storage Systems," Hydrocomp International, Palo Alto, California, 1973.

6. Cunge, J. A., "On the Subject of a Flood Propagation Computation Method (Muskingum Method)," Journal of Hydraulic Research, No. 2, 1969.

7. Flood Control Coordinating Committee, "Development of a Flood and Pollution Control Plan for the Chicagoland Area Summary of Technical Reports," August, 1972.

8. Grigg, N. S., et.al., "Metropolitan Water Intelligence Systems Completion Report - Phase II," Department of Civil Engineering, Colorado State University, June, 1973.

9. Hydrocomp, Inc., "Hydrocomp Simulation Programming: Operations Manual," Palo Alto, California, 1969.

10. Hydrologic Engineering Center, Corps of Engineers, U. S. Army, "Urban Runoff: Storage, Treatment and Overflow Model 'Storm'," Computer Program 723-58-22520, September, 1973.

11. Lanyon, R. F. and Jackson, J. D., "A Streamflow Model for Metropolitan Planning and Design," ASCE Urban Water Resources Research Program Technical Memorandum No. 20, January, 1974.

12. McKee, J. E., "Loss of Sanitary Sewage Through Storm-Water Overflows," Journal of the Boston Society of Civil Engineers, Vol. 34, No. 2, 1947.

13. Metcalf and Eddy, University of Florida, Water Resources Engineers, Inc., "Storm Water Management Model, Vol. I--Final Report," EPA Report No. 11024 DOC 07/71, July, 1971.

14. Municipality of Metropolitan Seattle, "Maximizing Storage in Combined Sewer Systems," U. S. Environmental Protection Agency Water Pollution Control Research Series 11022 ELK, December, 1971.

15. Sarma, P. B. S., et.al., "A Program in Hydrology, Part II," Purdue University, October, 1969.

16. Shifrin, W. G. and W. W. Horner, "Effectiveness of the Interception of Sewage-Storm Water Mixtures," Journal of the Water Pollution Control Federation, Vol. 33, 1961.

17. Stanley, R. H., "How to Analyze Combined Sewage-Stormwater Collection Systems," Water and Wastewater Engineering, Vol. 3, No. 3 and 4, 1966.

CHAPTER V

OPTIMIZATION FOR REAL-TIME CONTROL

A. INTRODUCTION

In Chapter IV, the emphasis has been on development of a model simulating the flow and storage characteristics of Vicente Subbasin and analyzing the effects of various control strategies. A *most favorable* control policy was developed on the basis of zero overflow curves for uniform intensity storms, and found to significantly reduce overflows, as compared with the no control case for the specific allowable interceptor flow used in the development. In this chapter, reservoir control is again supported as giving distinct improvement over no control, but the control policies are determined in a different manner. Formal optimization is applied to finding optimal control policies for uniform intensity storm events, and these results then extrapolated in the form of operating rule curves that can be applied to all the existing historical data. These results were found to be comparably effective in comparison with the previous zero overflow curve studies, and in some cases, produced better results. There was also an increase in flexibility and generality of the operating policies, as well as a decrease in human time and effort involved in developing the policies.

It is interesting that heretofore simulation and optimization have been generally considered to be two distinct and basically incompatible methodologies in systems analysis. For this study, it was possible to blend simulation and optimization in such a way that restructuring of the simulation model was not necessary. The emphasis is on fitting an optimization technique to the realistic simulation model, rather than weakening model realism in order to apply a popular, well-developed optimization technique. For this study, the primary disadvantage of the former was restriction to rather simple control policies. For the latter, there is greater latitude for control, at the expense of requiring more simplified, less realistic models of the subbasin behavior.

The first part of this chapter will be concerned specifically with the aforementioned blending of simulation and optimization to Vicente Subbasin control development. The last half of the chapter will concentrate on more direct applications of optimization to a hypothetical subbasin, with emphasis on a new technique called *flow projection* which may open the way for determination of unbiased optimal control policies in conjunction with realistic models of basin behavior.

B. OPTIMIZATION AND SIMULATION

B.1 Development of Optimal Control Based On Uniform Intensity Storms

The approach here is to employ the basic control strategy described in Chapter IV, using optimization procedures to determine the optimum control level α^* for the three upstream reservoirs while considering the control level for the downstream reservoir α_4 as a parameter. The Vicente Subbasin model was used to determine the overflow volume.

The optimization algorithm used is shown in Figure V-1. It is a simple one-dimensional search starting at a low value of α and increasing in incremental steps until a local minimum overflow is found. The step size is then decreased and the local optimum, α^*, is more accurately determined. This procedure requires that the objective function be unimodal over the feasible range of α, which was found to be true for uniform intensity storms. Various combinations of storm depth and duration were investigated for the two values of α_4

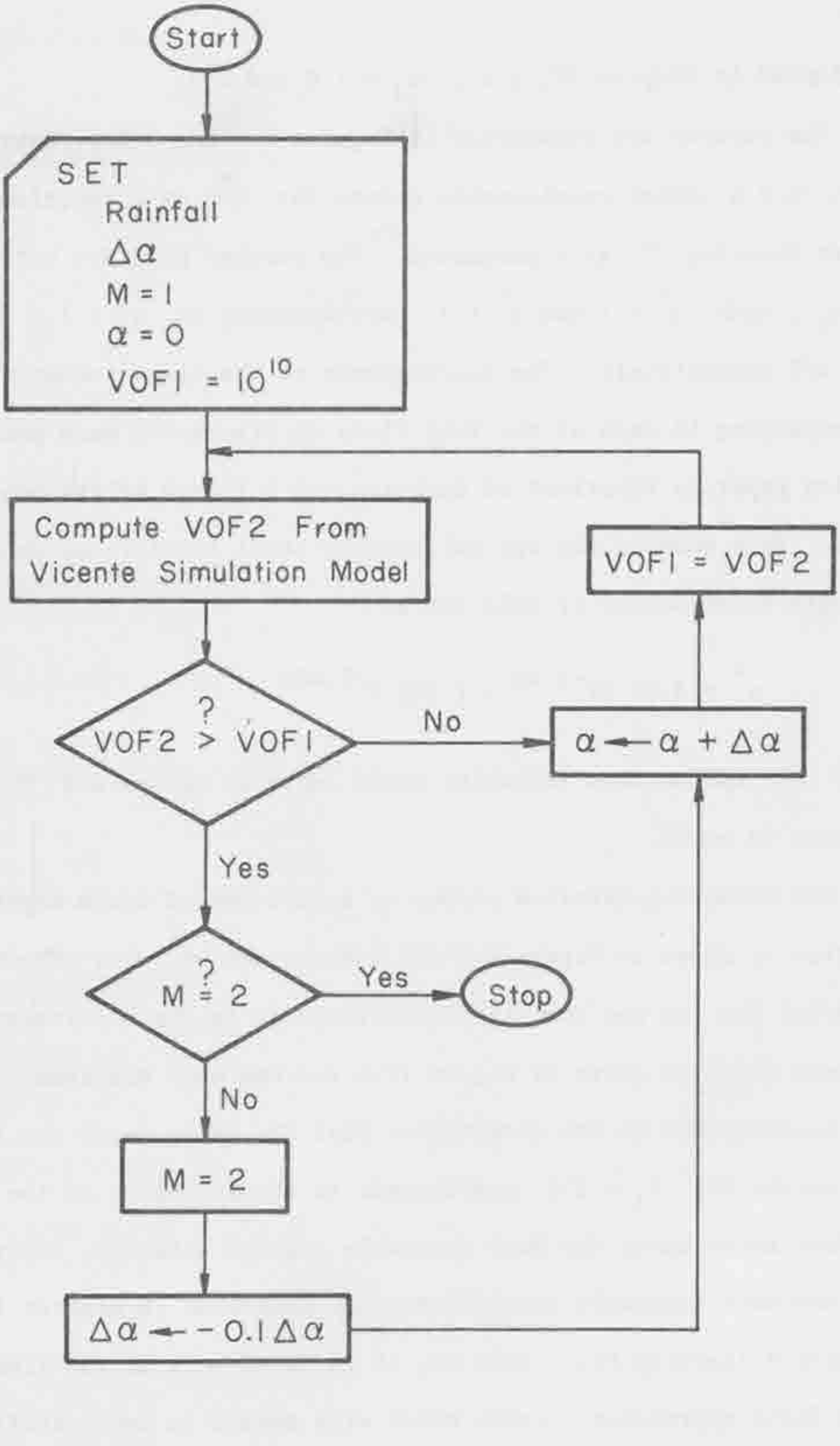


FIGURE V-1

FLOW CHART FOR OVERFLOW OPTIMIZATION

considered in Chapter IV, i.e., $\alpha_4 = 1.0$ and 1/3.

The results are summarized in Figures V-2 and V-3. Figure V-2 shows that a linear relationship exists for α^* as a function of depth D with duration T as a parameter. The results hold for both values of α_4, with $\alpha^* > 1$ and $\alpha^* \leq 1$ corresponding to $\alpha_4 = 1.0$ and $\alpha_4 = 1/3$ respectively. The coefficients of the linear equations corresponding to each of the four lines in Figure V-2 were plotted on log-log paper as functions of duration and a linear relationship was found. This enabled the optimal control level lines to be described by a single relationship or *rule curve*.

$$\alpha^* = 3.41\ DT^{-1.06} - 1.166\ T^{-1.405}$$

where D = the uniform intensity storm depth in inches and T = the duration in hours.

The resulting overflow volume as a function of storm depth and duration is shown in Figure V-3 for both values of α_4. The nonlinearity for the one hour storms corresponds to the nonlinearity of the zero overflow curve of Figure IV-6 for one hour and less. Of even more significance is the observation that the storm depth for zero overflow volume for $\alpha_4 = 1.0$ corresponds to the ordinates of the zero overflow curve using the most favorable control strategy. This means that the most favorable control strategy described in Chapter IV is actually a limiting one. That is, it is valid only in the limit as the storm depth approaches a value which will result in zero overflow volume. Comparison with Figure IV-9 shows that the overflow volume-depth relationship is approximately linear in both cases. However, the slopes of the lines for $\alpha_4 = 1.0$ in Figure V-3 vary from approximately

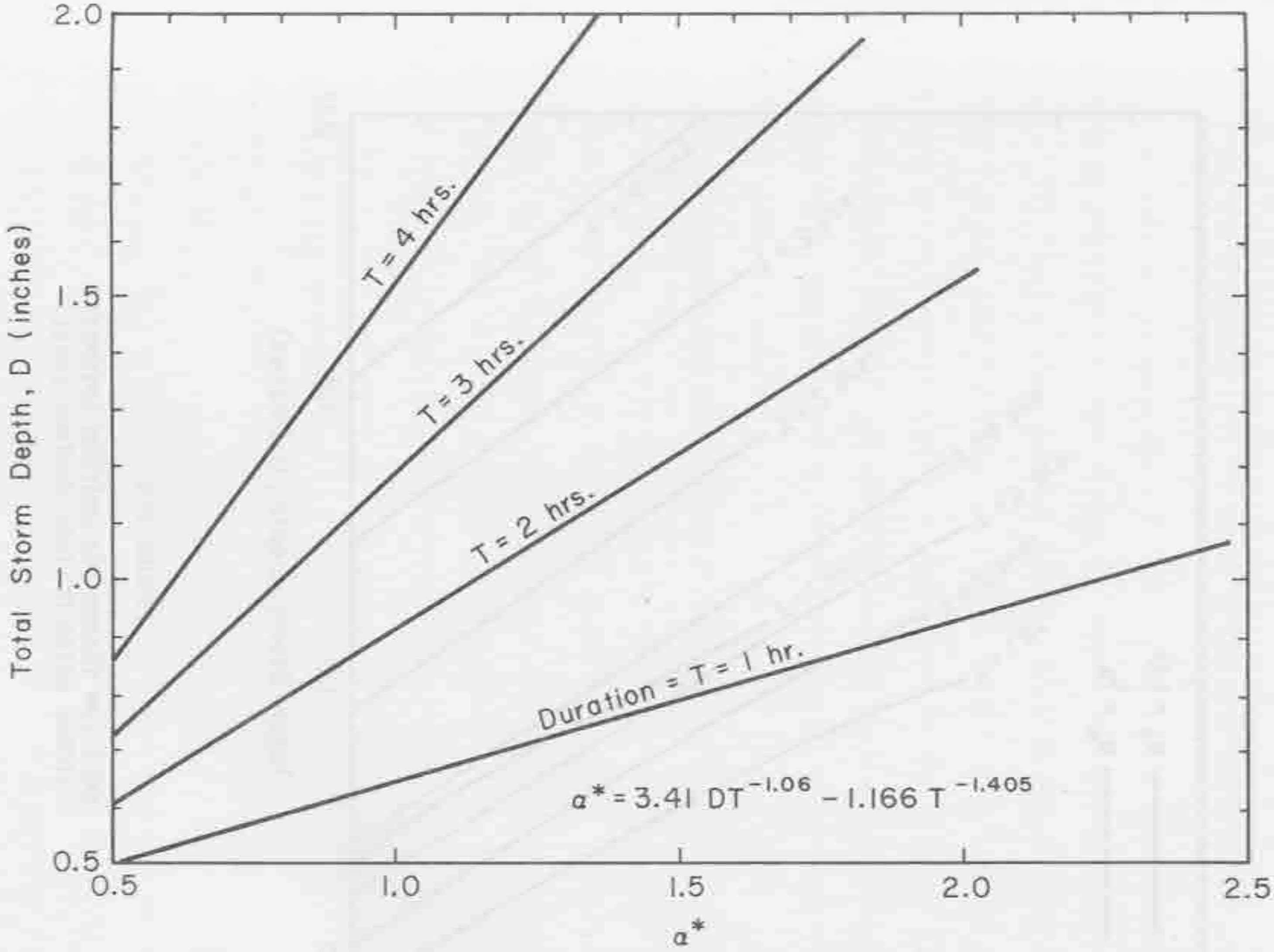


FIGURE V-2

OPTIMAL CONTROL LEVEL FOR
UNIFORM INTENSITY STORMS

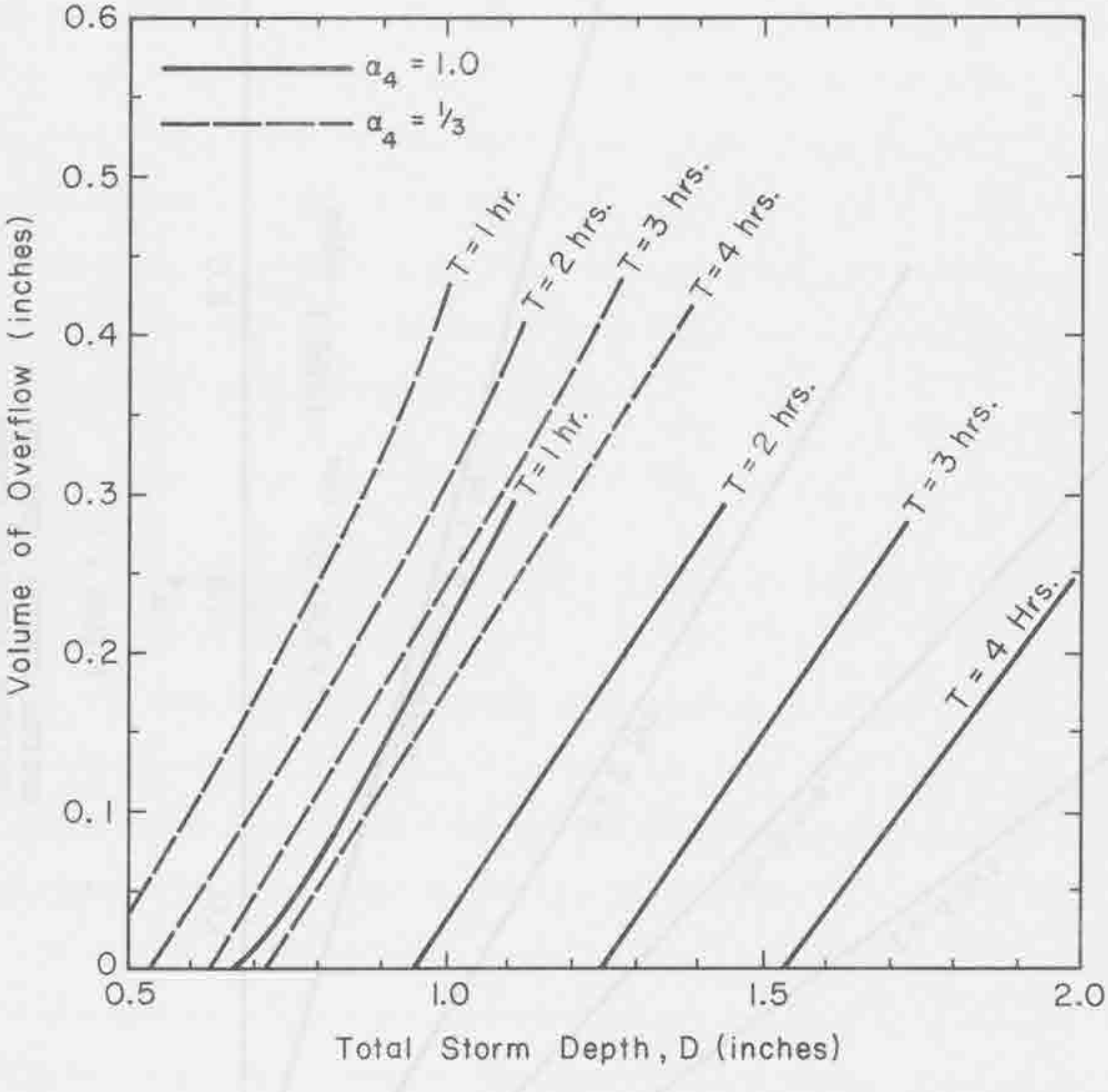


FIGURE V-3

OVERFLOW VOLUME FOR UNIFORM INTENSITY STORMS USING OPTIMAL CONTROL LEVEL

.65 to .55 as compared to the 0.87 slope of Figure IV-9, thereby corresponding more favorably with Figure IV-10.

B.2 Application of Uniform Intensity Storm Rule Curve to Historical Data

In order to evaluate the effect of optimal real time control on system performance the semi-continuous simulation approach described in Chapter IV, Section H was used in conjunction with the 66 years of hourly rainfall data for San Francisco. Instead of developing an α^* for each storm, however, the rule curve given in the previous section was adapted for use with non-uniform storms. This results in a control level which is sub-optimal itself but is based on optimal levels for uniform intensity storms.

The approach used was to define effective values of depth D_e and duration T_e for each storm and to substitute these values into the rule curve equation to determine an estimated α^*. The rule for defining D_e and T_e was developed by using the optimization scheme of Figure V-1 for a sample of non-uniform historical storms which were large enough to possibly cause overflows using storm definition C of Table IV-4. Both values of $\alpha_4 = 1.0$ and 1/3 were investigated. In most cases the objective function was unimodal, although this was not always true. An example is shown in Figure V-4 where the objective function for the same storm using $\alpha_4 = 1.0$ and 1/3 is shown. The curve for $\alpha_4 = 1/3$ is bimodal in the range of interest of α, while that for $\alpha_4 = 1.0$ is unimodal with a well defined minimum. It was observed that the degree of non-uniformity of storm intensity was of primary importance in determining the effective duration which would

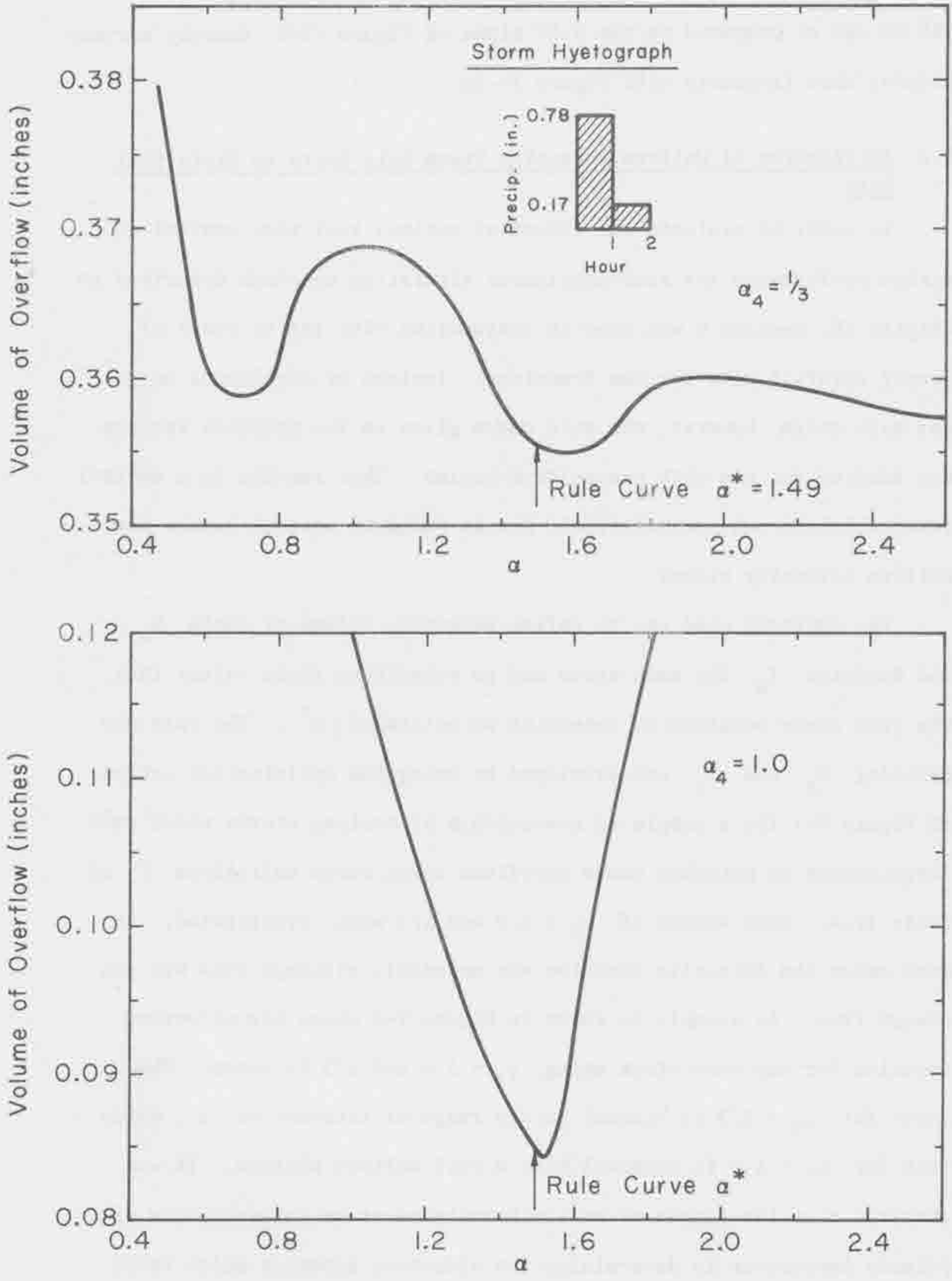


FIGURE V-4

OBJECTIVE FUNCTIONS FOR NON-UNIFORM STORM result in a value of α^* from the rule curve that would be close to the true value. A simple means of expressing the degree of rainfall non-uniformity is to use the ratio P_{max}/P_i , where P_{max} is the maximum hourly precipitation and P_i is any other hourly value during the storm. A critical value of $P_{max}/P^{i} = 2.0$ was adopted for purposes of defining T_e . That is, T_e was taken as the number of *consecutive* hours for which $P_{max}/P_i \leq 2.0$ for each storm. The effective depth D_e was then taken as the sum of the P_i values during the consecutive hours in which $P_{max}/P_i \leq 2.0$

One further modification in the use of Equation 1 was made. As α approaches zero the objective function must increase for storms whose total runoff exceeds the total storage volume of the reservoirs. Therefore, the true value of α^* is positive. Furthermore, observation of the objective function for the smaller storms in the sample showed that overflow volume became zero as α exceeded 0.45. Therefore, a lower bound, $\alpha^*_{min} = 0.5$, was used as well as an upper bound, $\alpha^*_{max} = 3.0$, corresponding to the approximate capacities of the lines draining the three upstream reservoirs. It should be emphasized that the value of the critical ratio and α^*_{min} were based on an analysis of a limited sample of storms. Further analysis might result in improved values, however the ones presented are acceptable for the purposes herein.

The results of the use of the rule curve as a control strategy in terms of average values resulting from semi-continuous simulation are shown in Table V-1. The no control results are obtained by fixing α for each of the upstream reservoirs at its maximum value, thereby allowing uncontrolled outflow at all times.

Table V-1

Comparison of Results of Control Strategies

	$\alpha_4 = 1.0$				$\alpha_4 = 1/3$			
Parameter	Zero OF	No Cntrl	Rule Curve	0/0 Change	Zero OF	No Cntrl	Rule Curve	% Change
Ave. Vol. of OF/Yr (in)	0.032	0.058	0.036	-37.9	0.960	0.953	0.693	-27.3
Ave. Number of OF/Yr	0.300	0.640	0.920	+43.7	7.450	7.390	5.610	-24.1
Ave. Vol. of OF/OF (in)	0.106	0.091	0.039	-57.1	0.129	0.129	0.124	-3.90
Ave. Dur. of OF (hrs)	0.730	0.770	1.070	+40.0	2.060	2.000	2.570	+28.5
Ave. OF Storm Dur. (hrs)	7.710	6.950	8.140	+17.1	5.910	5.910	6.670	+12.7

Note: OF = Overflow
Zero OF = Result of strategy of Figure IV-8
% Change is from no control to rule curve strategy

The most important result in this table is the reduction in overflow volume per year for both values of α_4 . Since this range of α_4 might be typical of that used in a city-wide strategy it could be concluded that a 25 percent reduction in average overflow volume per year could be achieved using real-time control over an uncontrolled system.

The average number of overflows per year is reduced for $\alpha_4 = 1/3$ but increased for $\alpha_4 = 1.0$. This latter situation is caused by a large number of small storms which produce a small overflow volume under the rule curve strategy. The no control strategy eliminates these overflows but offsets this gain by causing much larger overflows from the larger storms. This is clearly seen by observing the large reduction in average overflow volume per overflow event for $\alpha_4 = 1.0$.

It is interesting to compare the rule curve results to those produced by the use of Figure IV-8 which are shown in the *zero OF* column of Table V-1. For $\alpha_4 = 1.0$ the average overflow volume per year is about the same, but it is achieved in different ways. The rule curve strategy reduces substantially the average overflow volume from each overflow producing storm while increasing the average number of overflows. On the other hand, the Figure IV-8 strategy reduces the average number of overflows per year while increasing the average overflow volume per overflow producing storm. The real advantage of the rule curve strategy is shown in the data for $\alpha_4 = 1/3$. While there is essentially no difference between the Figure IV-8 strategy and the no control strategy, the rule curve produces significant reduction in both the average number and volume of overflow per year. This illustrates the greater generality and hence value of the optimization approach while at the same time the previous results demonstrate the improvements which can be gained by real-time system control.

One additional variable which bears comment is the average duration of overflow. As can be seen in Table V-1, the rule curve strategy results in a substantial increase in this variable. This is caused by two factors. The inflow hydrographs to the reservoirs are attenuated by the control strategy and the strategy time of the inflow hydrograph to a downstream reservoir with respect to the beginning of the storm is also somewhat increased over the no control strategy. The duration of overflow is defined as the time from the initiation of overflow from any reservoir to the time of cessation of overflow from any reservoir

Probability distributions for the first two variables in Table V-1 are shown in Figures V-5 and V-6. The effect of the use of the rule curve strategy is shown in the shift of the curves. Although not shown, a Poisson distribution using the mean values in Table V-1 agrees well with the curves of Figure V-6.

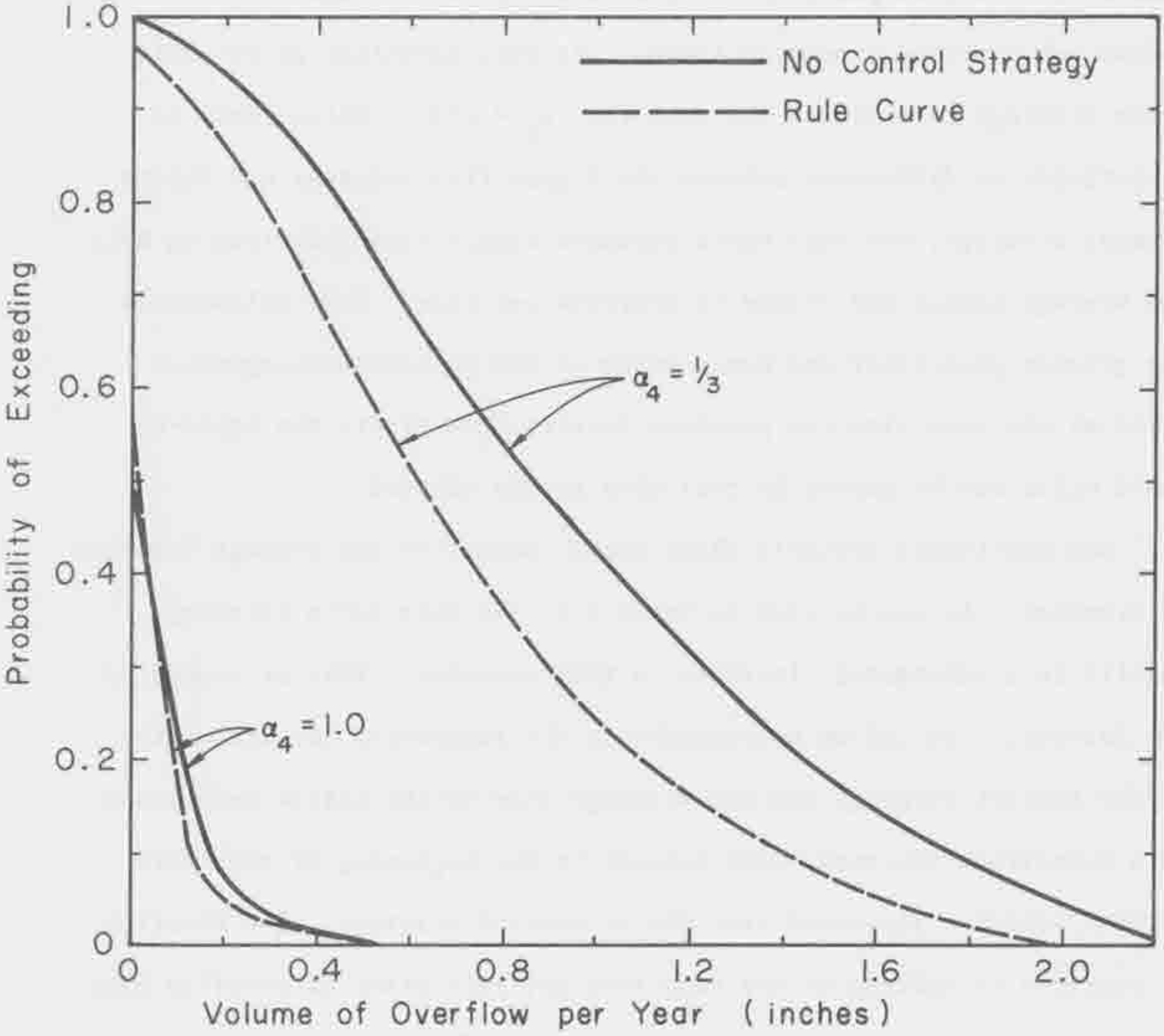


FIGURE V-5

PROBABILITY CURVES FOR
VOLUME OF OVERFLOW PER YEAR

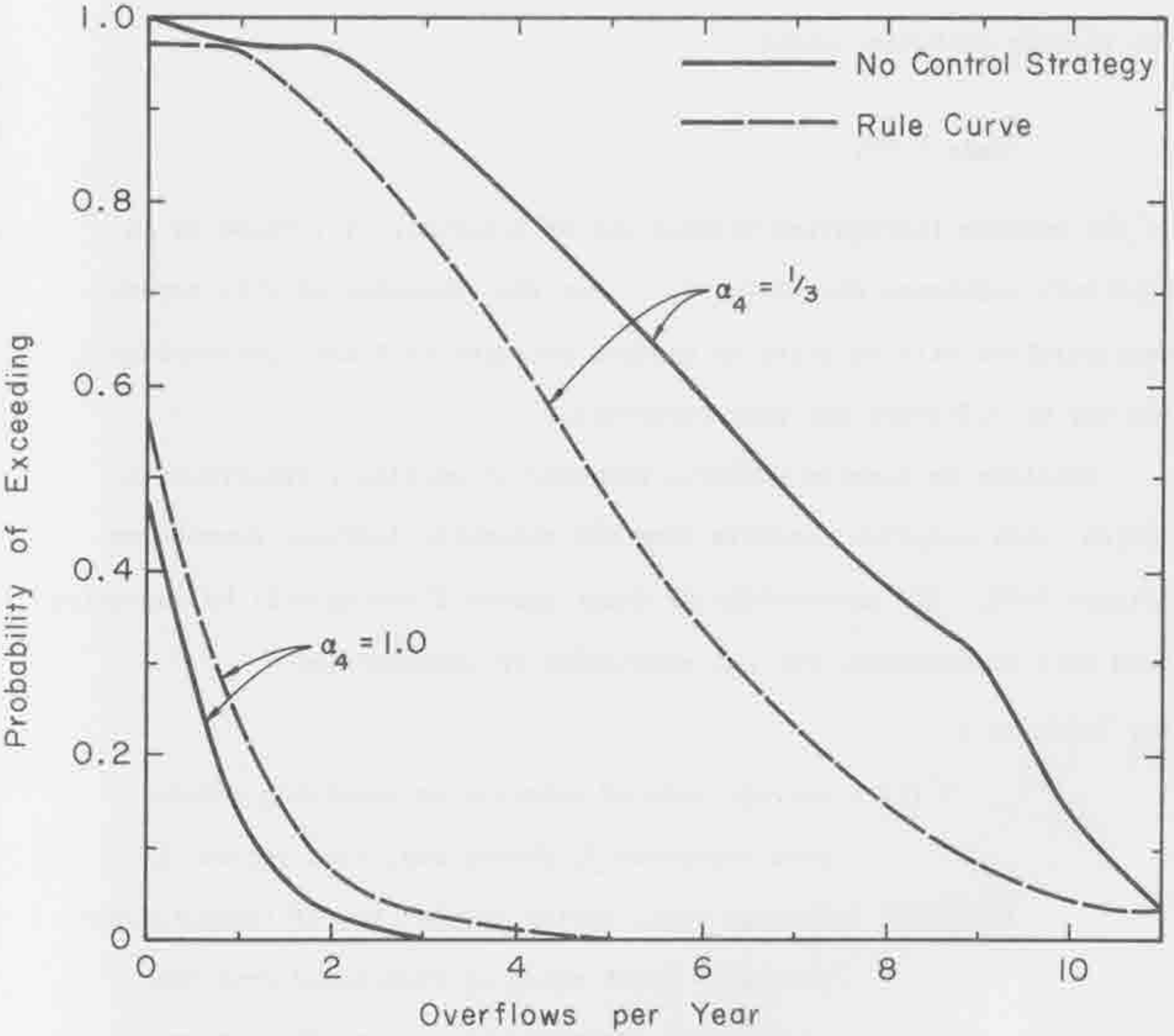


FIGURE V-6

PROBABILITY CURVES FOR
NUMBER OF OVERFLOWS PER YEAR

C. DIRECT APPLICATION OF OPTIMIZATION TECHNIQUES

C.1 Notation

Heretofore, subbasin control has been represented in terms of a time-invariant parameter α that is the same for all reservoirs in the Vicente Subbasin, where

$$Q_{i\max} = \alpha \hat{Q}_i$$

is the maximum throughflow allowed out of reservoir i, based on an arbitrary reference discharge $\hat{Q}_i$. For the remainder of this report, consideration will be given to control policies that are time-variant and may be different for each reservoir.

Consider an example subbasin composed of auxiliary reservoirs in series, with overflow possible from the reservoir farthest downstream (Figure V-7). The possibility of local street flooding will be neglected from this formulation, but can eventually be incorporated.

For subbasin i :

$O^i(k)$ = average rate of overflow to receiving waters from reservoir 3, during real-time period k.

$f_j^i(R_\ell^i(k))$ = average rate, during period k, of lumped direct stormflow input which is translated from the vicinity of reservoir j, given that storm ℓ is in progress. The notation $f_j^i(\cdot)$ represents an appropriate rainfall-runoff model used to compute the input, given $R_\ell^i(k)$.

$Q_j^i(k)$ = average rate of throughflow during period k, from reservoir j, with $Q_3^i(k)$ entering the interceptor sewer from reservoir 3 farthest downstream.

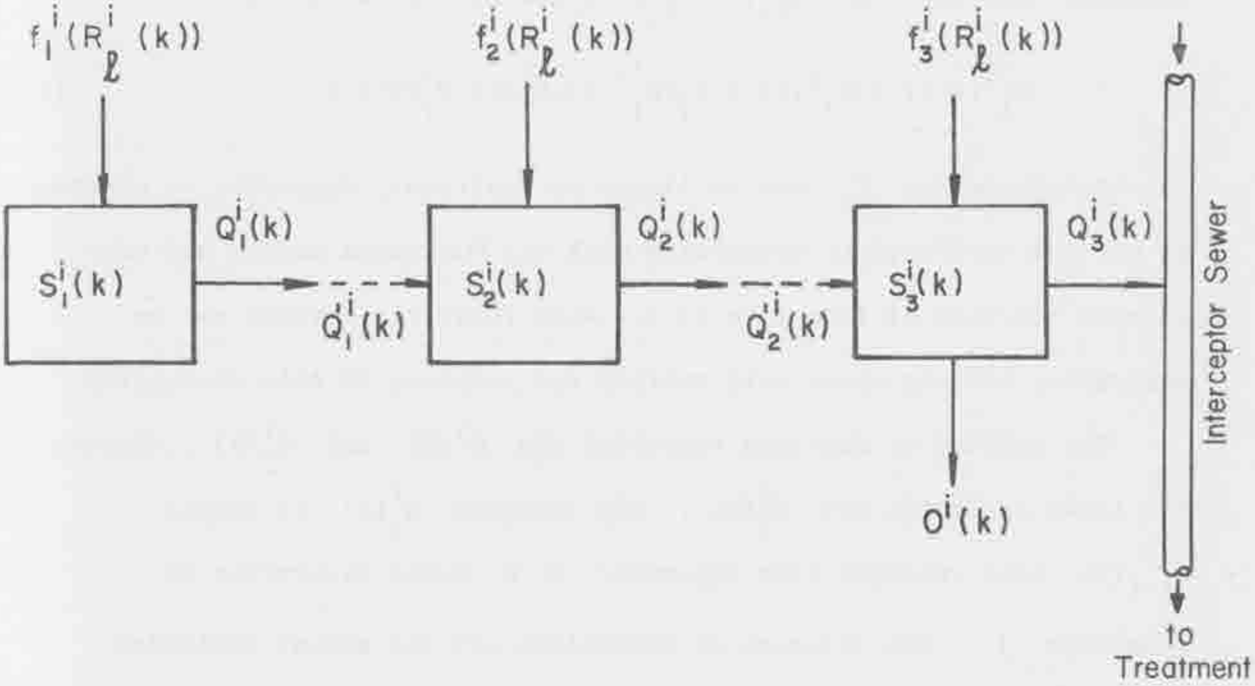


FIGURE V-7

SCHEMATIC OF LINKED DETENTION RESERVOIRS
IN SUBBASIN i

$Q_j^{'i}(k)$ = the routed or translated throughflow from reservoir j, entering reservoir $j+1$ (for $j=1,2$).

$S_j^i(k)$ = storage in reservoir j, at the beginning of period k.

The Muskingum method is one of the most common approaches to flow routing. Letting $\Delta t = t_{k+1} - t_k$, it can be represented by

$$Q_j^{'i}(k+1) = Q_j^{'i}(k) + T_j(Q_j^{'i}(k), Q_j^i(k), Q_j^i(k+1)) \tag{1}$$

The transformation T_j may be linear or nonlinear, depending on whether or not the coefficients associated with the Muskingum method are considered function of flow rate [6]. More realistic methods may be desirable, but the above will suffice for purposes of this discussion.

The *control* or decision variables are $O^i(k)$ and $Q_j^i(k)$, whereas the *state* variables are $S_j^i(k)$. The variable $Q_j^i(k)$ is simply $Q_{j-1}^{'i}(k)$ *plus* releases from reservoir j or *minus* diversions to reservoir j. The releases or diversions are the actual variables used for real-time control. They are, however, easily computed once $Q_j^i(k)$ and $Q_{j-1}^{'i}(k)$ are computed. The reason for using $Q_j^i(k)$ and $Q_{j-1}^{'i}(k)$ instead of actual releases and diversions in developing the optimal control problem is that it simplifies the formulation.

Notice that $Q_3^i(k)$ corresponds to $Q^i(k)$, as defined in Chapter III, for this particular example. In the following formulation the former will be replaced by the latter in order to be consistent with the notation used in Chapter III.

C.2 On-Line Subbasin Problem

There were two basic approaches to real-time control considered in Chapter III: on-line and off-line subbasin optimization. The On-Line Subbasin Problems were discussed in a general fashion in Section D.1 of Chapter III. A more detailed formulation follows.

For on-line subbasin optimization, the Lagrange multipliers $\lambda(k)$ are given by the Master Problem in order to influence subbasin control in such a way that Conditions 1 and 2 (Equations 10 and 11 in Chapter III) are eventually satisfied. If linear routing is used, the $\lambda(k)$ represent dual variables supplied by the Dantzig-Wolfe restricted Master Problem. Since optimization can commence at any time k', $k'=1,\ldots,M$. Current states of the system at time k', in terms of reservoir storage and sewer flow levels, will be known from current sensor data. Also, storm event ℓ has been predicted as the storm in progress.

On-Line Subbasin Problem:

$$\underset{\underline{S},\underline{Q},\underline{Q}',\underline{O}^i}{\text{minimize}} \quad \sum_{k=k'}^{M} \left[\omega^i(k)O^i(k) + \lambda(k)Q^i(k)\right] \tag{2}$$

subject to: [Note: Superscript i is deleted for convenience only]

$$\text{dynamic equations}\begin{cases} S_1(k+1) = S_1(k) + f_1^i(R_\ell^i(k)) - Q_1(k) & (3) \\ S_2(k+1) = S_2(k) + f_2^i(R_\ell^i(k)) + Q_1'(k) - Q_2(k) & (4) \\ S_3(k+1) = S_3(k) + f_3^i(R_\ell^i(k)) + Q_2'(k) - Q^i(k) & \\ \qquad - O^i(k) & (5) \\ Q_j'(k) = Q_j'(k-1) + T_j(Q_j'(k-1), Q_j(k-1), Q_j(k)), & (6) \\ \qquad (j=1,2;\ k=k',\ldots,M) & \end{cases}$$

$$\begin{array}{ll} \text{initial} \\ \text{conditions} \end{array} \left\{ \begin{array}{l} S_j(k'), Q_j(k'-1), Q_j(k'), Q_j'(k'-1)\ , \\ (j=1,2),\ \text{and } Q^i(k')\ \text{all known} \end{array} \right. \tag{7}$$

$$\begin{array}{l} \text{upper} \\ \text{and} \\ \text{lower} \\ \text{bounds} \end{array} \left\{ \begin{array}{lll} 0 \le S_j(k) \le S_{j\max} & j=1,2;\ k=k'+1,\ldots,M+1 & (8) \\ 0 \le Q_j(k) \le Q_{j\max} & j=1,2 & (9) \\ Q^i(k) \le Q^i_{\max} & k=k',\ldots,M & (10) \\ 0 \le Q_j'(k) \le Q_{j\max} & j=1,2 & (11) \\ O^i(k) \ge 0\ , & & (12) \end{array} \right.$$

$$Q_3(k) \triangleq Q^i(k) \tag{13}$$

where $\underline{S} = (S_j(k)\ ,\ j=1,2,3;\ k=k'+1,\ldots,M+1)$;

$\underline{Q} = (Q_j(k),\ j=1,2,3;\ k=k',\ldots,M)$; $\underline{Q}' = (Q_j'(k),\ j=1,2;\ k=k',\ldots,M)$;

$\underline{O}^i = (O^i(k),\ k=k',\ldots,M)$

C.3 Off-Line Subbasin Problem

As previously discussed in Chapter III, Section E, the off-line Subbasin Problems cannot be solved for all possible initial storage and flow conditions, for any real-time k'. The amount of computation required would be much too large. An alternate approach was to solve the previous on-line formulation off-line for initial time $k'=1$ only, and assume zero initial conditions in (7). In addition, $Q^i_{\max}$ is now considered as a control variable adjusted by the Master Problem and the term $\lambda(k)Q^i(k)$ in (2) can be deleted.

For off-line optimization, then, it is assumed that all reservoirs are initially empty, or that sufficient time elapses between successive storms to allow drainage. The addition of a term in the objective function (2) crediting throughflows would tend to strengthen this assumption, since it would then be suboptimal not to pass flow into the interceptor as much as allowed and detain it in the reservoirs. That is, add the term

$$- \mu(k)Q^{i}(k) \tag{14}$$

to (2), where the $\mu(k)$ are arbitrary weighting factors.

It should be noted that the Subbasin Problem is actually more complicated than suggested here since $Q_{j\max}$ may vary with storage $S_j(k)$, but this additional nonlinearity will not be ignored for now. For off-line work, then, the most important results are the optimal reservoir levels $\underline{S}^{*i}(k)$ under an optimal control policy, for $k=1,\ldots,M$, computed for all discrete levels of $Q^{i}_{\max}$ and all given storm events $\ell=1,\ldots,L$. These results can be stored in the on-line computer for use in real-time control.

C.4 Application of Linear Programming

If the transformation T_j is linear, then linear programming can be used to solve the Subbasin Problems as was done in Chapter VI, and a global solution assured if the problem is well-posed. Assuming a linear T_j may not, however, be consistent with reality. If a particular subbasin has a relatively flat average slope, then backwater effects may greatly inhibit use of linear models. The critical question is, what magnitude of error is introduced? If the error seems tolerable, then linear programming is still applicable. This question can be resolved by analyzing historical data for the basin and checking the ability of linear models to predict subbasin outflow, given the input. The relatively steep slopes in San Francisco suggest that simple routing may be adequate.

Linear programming is particularly compatible with direct on-line optimization. Well-documented, reliable linear programming (LP) codes are readily available commercially. In addition, considerable effort has been, and is continuing to be, expended on improving LP codes. One area that is pertinent to this control problem is revision of the simplex method to consider upper bounds on variables as effectively as nonnegativity bounds are currently considered [13]. Approximately two-thirds of the constraints associated with the control problem are simple upper bounds on the variables. If these could be eliminated as formal constraints and satisfied as a part of a modified *pricing-out* mechanism [13], considerable savings in required computer storage could be realized.

The A matrix associated with the LP formulation of the control problem is relatively *sparse*. That is, the great majority of the coefficients of the matrix are zero. *LU decomposition* [13] is a technique that exploits the special structure of sparse matrices to produce considerable computational savings.

Currently, this linear control problem has been solved by a revised simplex code using the explicit inverse of the basis. Numerical results are given in [8]. None of the above modifications have been employed as yet, but will be investigated in future studies. With regard to the application to Vicente Subbasin as considered in Chapter VI for 5 reservoirs and $M=7$ time periods, computation time was about 7 seconds on a CDC 6400 computer, requiring 100,000 words of storage in octal. Future studies will attempt to apply the above computation time and storage saving devices to the LP control formulation. The possible use of minicomputers on-line, as discussed in Chapter III, is a prime motivation for this effort.

C.5 Application of Dynamic Programming

Solution of the real-time control problem by dynamic programming is not possible until it is consistent with the general format for discrete-time control problems, since the present formulation has $k-1$ appearing on the right-hand side of Equation 6. This can be remedied by defining a new control variable $V_j(k)$, and replacing Equation 6 with

$$Q_j'(k) = Q_j'(k-1) + T_j(Q_j'(k-1), Q_j(k-1), V_j(k-1)) \tag{15}$$

$$Q_j(k) = Q_j(k-1) + [V_j(k-1) - Q_j(k-1)] \tag{16}$$

Note that $Q_j(k)$ $(j=1,2)$ are now regarded as dependent state variables.

In applying dynamic programming, no assumptions regarding T_j are required. Since there are, however, seven state variables at each stage k, conventional dynamic programming is not possible. Incremental or differential dynamic programming [12,9] can be utilized, but if T_j is nonlinear, only local solutions are guaranteed to result. Some reduction in the number of control variables is possible by replacing $O(k)$ in Equation 2 with

$$O(k) = 0 \text{ , for all } k \in K \tag{17}$$

$$O(k) = [S_3(k) - S_{3max}] \text{ , for all } k \notin K \tag{18}$$

where

$$K = \{k | S_3(k) - S_{3max} \leq 0\},\ k=1,\ldots,M \tag{19}$$

with constraint (10) deleted for $j=3$. In Equation 5, $Q^i(k) + O(k)$ is replaced by $U(k)$, and the number of control variables per stage has been reduced by one.

Computational experience with incremental dynamic programming as an alternative to the LP approach has been gained, but no results are available as yet for nonlinear problems. In comparing linear programming (LP) results with incremental dynamic programming (IDP), the following conclusions were noted: As M increases, computation time increases more rapidly for LP than for IDP. The reverse is true as the number of detention reservoirs increases.

C.6 Continuous-Time Optimal Control

For the practical problem of optimally controlling combined sewer overflows via storage regulation, it is safe to assume that controls will be carried out in discrete time intervals. This is due to the following factors associated with on-line, automated control:

1. There is a finite amount of time required to actually effect control. That is, a certain amount of time is required for passage of information, the opening and closing of valves and regulators, etc.
2. On-line control requires the processing of rainfall and sewer flow data, which is sampled at discrete-time (e.g., for the San Francisco system, data is collected every 15 seconds).
3. Sufficient data must be collected in order to make a reasonable prediction of future storm input so that the next control can be effected. There is an interesting trade-off here:

   (a) Large intervals between control would allow the processing of more data, resulting in more accurate prediction. Though the individual controls are more optimal in the sense that they are based on more accurate data, the system is less controllable due to the large intervals.

(b) Small intervals between control would result in less accurate storm prediction. Though the system is more controllable than in case (a), there is greater question as to the optimality of the controls.

Suppose it is decided that actual control of the system must occur between a discrete-time interval Δt . Then there are two basic ways of determining the optimal controls:

1. Finite-Dimensional Optimization: Solve a discrete-time optimal control problem and determine the optimal controls.
2. Infinite-Dimensional Optimization: Solve a continuous-time optimal control problem and determine the optimal controls to be carried out at discrete time intervals from these results.

The emphasis in this report is on use of finite-dimensional optimization as applied to discrete-time optimal control. As Canon, et.al. [5] have succinctly stated, the

> "...main reason for attaching so much importance to discrete optimal control is technical and stems from the constantly increasing use of digital computers in the control of dynamical systems. In any computation carried out on a digital computer, we can do no better than obtain a finite set of real numbers. Thus, in solving a continuous optimal control problem...we are forced to resort to some form of discretization."

The question, then, is whether to discretize prior to optimization (as in discrete-time optimal control) or during and subsequent to optimization (as in continuous time optimal control). Certain aspects of infinite-dimensional, continuous-time optimal control theory may be applicable to subbasin optimization. Some computational experience with this approach is reported in [2], [3], and [4]. Difficulties with application of the Maximum Principle to this problem are discussed in reference [10]. A new approach using linear regulator theory is presented in [14], and Bell [1] proposes an approach that utilizes some basic results derived from application of the Maximum Principle, but does not require solution of all the difficult necessary conditions associated with it. It appears that the main advantage of infinite-dimensional optimization over mathematical programming techniques is a considerable lessening of required core computer storage for carrying out on-line optimizations. At the present, the primary disadvantage is the difficulty of using realistic routing methods.

D. FLOW PROJECTION TECHNIQUE

D.1 Inducing Separability

All nonlinear programming computer codes, other than combinatorial grid-search methods, require that the constraint set be convex in order to insure convergence to global solutions. Any nonlinearity in T_j would immediately violate this condition, since nonlinear equality constraints always generate a nonconvex constraint region [10]. Various local solutions will result, in this case, depending on the particular initial starting point utilized. A technique has been proposed by Labadie, et.al. [11] that utilizes a combination of nonlinear and dynamic programming in an attempt to deal with the nonconvexity induced by realistic, nonlinear flow routing methods.

Suppose arbitrary functions $\phi(\underline{a},t)$, $\psi(\underline{b},t)$ are given, with associated parameter vectors $\underline{a},\underline{b}$ respectively, such that if $\underline{Q}^*$ is a global solution to the Subbasin Problem, then there exist optimal $\underline{a}^*,\underline{b}^*$ such that

$$\left.\begin{aligned} Q_1^*(k) &= \phi(\underline{a}^*,t_k) \qquad (20) \\ Q_2^*(k) &= \psi(\underline{b}^*,t_k) \qquad (21) \end{aligned}\right\} \quad k=1,\dots,M$$

Now, replace constraints (6) in the Subbasin Problem with

$$Q_1'(k) = Q_1'(k-1) + T_1(Q_1'(k-1),\phi(\underline{a},t_{k-1}),\phi(\underline{a},t_k)) \tag{22}$$

$$Q_2'(k) = Q_2'(k-1) + T_2(Q_2'(k-1),\psi(\underline{b},t_{k-1}),\psi(\underline{b},t_k)) \tag{23}$$

$$(k=1,\dots,M)$$

and add the additional constraints

$$\left.\begin{aligned} \phi(\underline{a},t_k) - Q_1(k) &= 0 \qquad (24) \\ \psi(\underline{b},t_k) - Q_2(k) &= 0 \qquad (25) \end{aligned}\right\} \quad k=1,\dots,M$$

The addition of (24) and (25) insure that this new problem is exactly equivalent to the original Subbasin Problem. In the new problem, $Q_1(k)$ and $Q_2(k)$ have been replaced by $\phi(\underline{a},t_k)$ and $\psi(\underline{b},t_k)$, respectively. This new problem, with (22) and (23) replacing (6), and the addition of (24) and (25) will in turn be modified as follows.

Considering that it will be generally impossible to find $\underline{a}^*,\underline{b}^*$ that will exactly satisfy (20) and (21), suppose it is then desired to satisfy (20) and (21) as closely as possible. This goal can be met by adding the terms

$$\sum_{k=1}^{M} \left\{ \rho(k)[\phi(\underline{a},t_k) - Q_1(k)]^2 + \nu(k)[\psi(\underline{b},t_k) - Q_2(k)]^2 \right\} \tag{26}$$

to the objective function (2), and removing (20) and (21) as constraints. These added terms will encourage the satisfaction of (20) and (21) as closely as possible in an indirect manner; otherwise a penalty is incurred by (26).

D.2 Decomposition into Subproblems

The advantage of this formulation is that once $\underline{a}$ and $\underline{b}$ are specified, the Subbasin Problem can be decomposed into three independent problems, one associated with each reservoir.

Subproblem 1:

$$v_1(\underline{a}) = \min_{\substack{S_1(k+1),Q_1(k)\\ k=1,\ldots,M}} \sum_{k=1}^{M} \rho(k)[\phi(\underline{a},t_k) - Q_1(k)]^2 \tag{27}$$

subject to: (all variables assumed nonnegative)

$$\left.\begin{array}{ll} S_1(k+1) = S_1(k) + f_1^i(R_\ell^i(k)) - Q_1(k) & (28) \\ S_1(1) \quad \text{given} & \\ S_1(k+1) \le S_{1\max}\ , & (29) \\ Q_1(k) \le Q_{1\max}\ , & (30) \end{array}\right\} \quad k=1,\ldots,M$$

which is easily solved as a one-dimensional dynamic programming problem ($S_1(k)$ as the state variable) with one decision variable at each stage k ($Q_1(k)$). Quadratic programming is also applicable here.

Subproblem 2:

$$v_2(\underline{a},\underline{b}) = \min_{\substack{S_2(k+1),Q_2(k)\\ k=1,\ldots,M}} \sum_{k=1}^{M} \nu_k[\psi(\underline{b},t_k) - Q_2(k)]^2 \tag{31}$$

subject to:

$$\left.\begin{aligned} &S_2(k+1) = S_2(k) + f_2^i(R_\ell^i(k)) + Q_1'(k) - Q_2(k) && (32)\\ &S_2(1) \quad \text{(given)} \\ &S_2(k+1) \le S_{2\max}\ , && (33)\\ &Q_2(k) \le Q_{2\max}\ , && (34) \end{aligned}\right\} k=1,\ldots,M$$

where $Q_1'(k)$ is determined on the basis of the given $\underline{a}$ from Equation 22. This subproblem is solved the same way as subproblem 1.

Subproblem 3:

$$v_3(\underline{b}) = \min_{\substack{S_3(k+1),Q^i(k),O^i(k)\\ k=1,\ldots,M}} \sum_{k=1}^{M} [\omega(k)O^i(k) - \mu(k)Q^i(k)] \tag{35}$$

subject to:

$$\left.\begin{aligned} &S_3(k+1) = S_3(k) + f^i(R_\ell^i(k)) + Q_2'(k) - Q^i(k) - O^i(k) && (36)\\ &S_3(1) \quad \text{(given)} \\ &S_3(k+1) \le S_{3\max}\ , && (37)\\ &Q^i(k) \le Q^i_{\max}\ , && (38) \end{aligned}\right\} k=1,\ldots,M$$

For this subproblem, let

$$\bar{Q}^i(k) = Q^i(k) + O^i(k) \tag{39}$$

and replace the overflow terms in the objective function with

$$O^i(k) = [S_3(k) - S_{3max}] \ , \quad \text{for all} \quad k \in K \tag{40}$$

where

$$K = \{k | S_3(k) - S_{3max} \geq 0\}$$

This results in only one control variable $\bar{Q}^i(k)$ for subproblem 3, at each stage k, and the upper bound S_{3max} is ignored. The term $Q_2'(k)$ is determined from (23) for the given $\underline{b}$.

The $\underline{a}$ and $\underline{b}$ must be properly adjusted until (2), with (26) added is minimized. That is, solve

$$\min_{\underline{a},\underline{b}} \ v_1(\underline{a}) + v_2(\underline{a},\underline{b}) + v_3(\underline{b}) \tag{41}$$

This is referred to as the *outer* problem. Basically, the flows in the subbasin have been *projected* into an outer problem that manipulates coefficients associated with the approximating functions until (20) and (21) are satisfied.

As the number of reservoirs increases, the dimensionality of the outer problem (41) increases proportionately, whereas the subproblems are relatively unaffected. In order to have enough degrees of freedom to allow accurate approximation of $\underline{Q}(k)$ by $(\underline{a},t_k)$ and $(\underline{b},t_k)$, the number of components of $\underline{a},\underline{b}$ would have to be relatively large. Since it would be difficult, if not impossible, to insure that (41) is generally a convex problem, global solution can only be assured through direct enumeration or combinatorial grid-search techniques. These methods, of course, are only feasible for problems of limited dimension (perhaps two or three variables).

D.3 Orthogonal Polynomials

A way out of this dilemma may be available through use of orthogonal polynomials. If a general orthogonal polynomial is represented as follows:

$$\phi(\underline{a},t) = \sum_{i=0}^{r} a_i g_i(t)$$

and assuming that $M \geq r + 1$, then $\phi(\underline{a},t)$ has the following properties [7]:

1. The orthogonality property

$$\sum_{k=1}^{M} g_i(t_k) g_j(t_k) = 0 , \qquad i \neq j$$

   guarantees that optimal values of the coefficients $\underline{a}$ that give the best fit are independent of the highest power of the polynomial, when fitted to a given set of data.

2. Computation of coefficients for orthogonal polynomials is generally faster than for nonorthogonal polynomials.
3. Chebyshev polynomials, the most common orthogonal polynomials in use, tend to give a reasonably consistent fitting error over the range of data.

Property 1 implies that the outer problem (41) can be solved using just a_0, b_0

$$\min_{a_0, b_0} \; v_1(a_0) + v_2(a_0, b_0) + v_3(b_0)$$

and obtain optimal a_0^*, b_0^* . Then solve

$$\min_{a_1, b_1} \; v_1(a_1) + v_2(a_1, b_1) + v_3(b_1)$$

using

$$\phi(\underline{a}, t) = a_0^* g_0(t) + a_1 g_1(t)$$

$$\psi(\underline{b}, t) = b_0^* g_0(t) + b_1 g_1(t)$$

and so on, until an accurate fit is assured. Therefore, the original large-dimensional outer problem is replaced by a sequence of 2-dimensional outer problems.

It must be pointed out that Property 1 is based on having a fixed set of data to fit to. For this problem, $\phi(\underline{a}, t_k)$ and $\psi(\underline{b}, t_k)$ are fitted to $\underline{Q}(k)$, with the latter tending to change as the above iterative sequence continues. The influence this might have on obtaining global solutions should be investigated through extensive computational experience.

D.4 Numerical Results

Some numerical results for the approximate-flow method were obtained, based on the following data and assumptions:

1. The number of time periods $M=7$. Therefore, Chebyshev polynomials up to order 6 were utilized.
2. A Muskingum routing was utilized, where

   $$Q_1'(k+1) = \beta_1 Q_1'(k) + \beta_2 \phi(\underline{a}, t_k) + \beta_3 \phi(\underline{a}, t_{k+1})$$

   $$Q_2'(k+1) = \beta_1 Q_2'(k) + \beta_2 \psi(\underline{b}, t_k) + \beta_3 \psi(\underline{b}, t_{k+1})$$

   where $\beta_1 = \beta_2 = \beta_3 = 1/3$.

3. For the objective function weighting factors,

$$\left.\begin{aligned} \mu(k) &= 0.2 \\ \rho(k) &= \nu(k) = 5000 \\ \omega(k) &= 1.0 \end{aligned}\right\} \qquad k=1,\ldots,7$$

4. Direct stormflow is (for j=1,2,3)

$$f_j(R(1)) = f_j(R(6)) = f_j(R(7)) = 0$$

$$f_j(R(2)) = 6$$

$$f_j(R(3)) = 2$$

$$f_j(R(4)) = 5$$

$$f_j(R(5)) = 4$$

5. Initial conditions and bounds are

$$\left.\begin{aligned} S_i(1) &= 7 \\ S_{i\max} &= 7 \end{aligned}\right\} \qquad i=1,2,3$$

$$Q_{1\max} = 4$$

$$Q_{2\max} = 8$$

$$Q_{3\max} = 6$$

6. $\Delta t = t_{k+1} - t_k = 1 \triangleq 10$ minutes in real-time.

The outer problem of adjusting $\underline{a},\underline{b}$ was carried out by Powell's method of unconstrained minimization (not requiring derivatives) in the iterative fashion discussed previously. That is, a_0,b_0 were adjusted first, resulting in a_0^*,b_0^*, then a_1,b_1 in a first order polynomial utilizing a_0^*,b_0^*, etc. Figures V-8 and V-9 depict the rapid convergence of the successively higher order polynomials to the actual global solution for throughflow. Numerical values associated with convergence in this example problem are given in Table V-2. Repetition of successive determination of $\underline{a},\underline{b}$, using $\underline{a}^*,\underline{b}^*$ computed previously, resulted in a close fit (average fitting error of 0.3%)

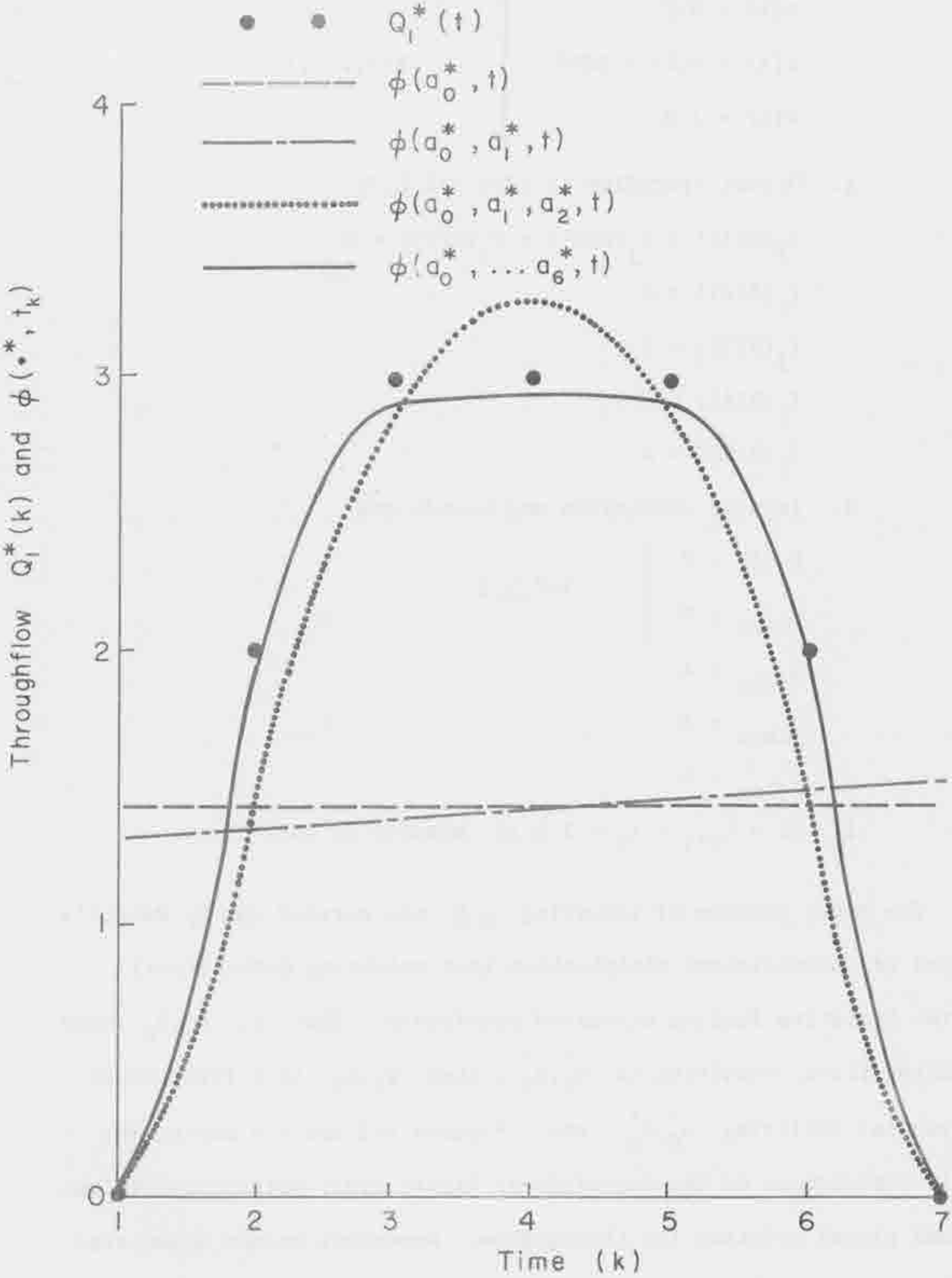


FIGURE V-8

CONVERGENCE OF $\phi(\cdot^*, t_k)$

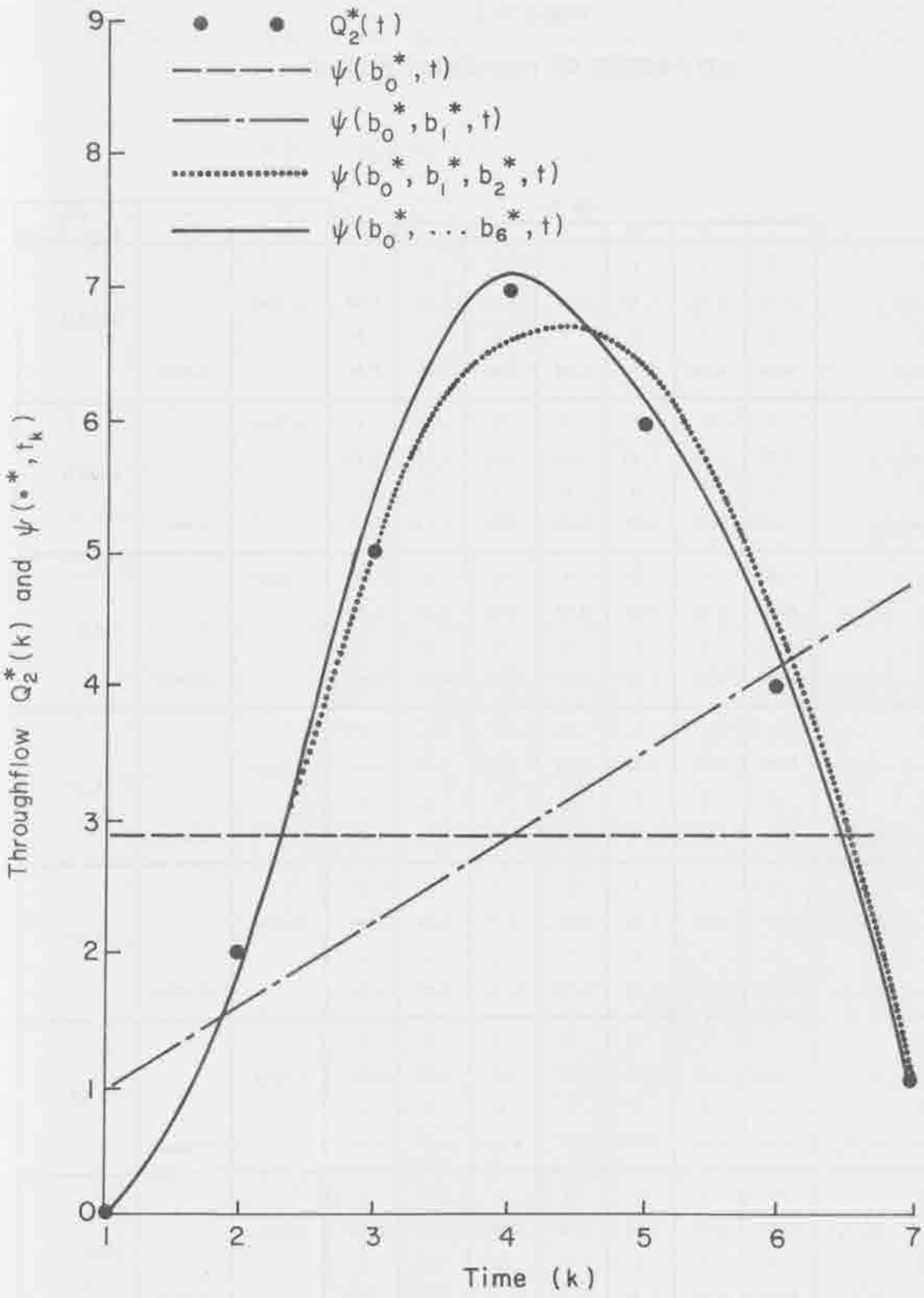


FIGURE V-9

CONVERGENCE OF $\psi(\cdot^*, t_k)$

TABLE V-2

CONVERGENCE OF POLYNOMIAL FITTING

ORDER OF POLYNOMIAL (i)		TIME (k) 1	2	3	4	5	6	7	a_i^*	b_i^*	Obj. Func. (v(·))
i=0	$Q_1^*(k)$	0	3	2	3	2	1	1			
	$\phi(a_0^*,t_k)$	1.43	1.43	1.43	1.43	1.43	1.43	1.43	1.4286		92026.0
	$Q_2^*(k)$	0	4	3	3	4	3	3			
	$\psi(b_0^*,t_k)$	2.86	2.86	2.86	2.86	2.86	2.86	2.86		2.8571	
i=1	$Q_2^*(k)$	0	2	2	3	3	1	2	0.1044		
	$\phi(a_0^*,a_1^*,t_k)$	1.33	1.36	1.39	1.43	1.46	1.50	1.53			61064.0
	$Q_2^*(k)$	0	2	3	4	5	4	5			
	$\psi(b_0^*,b_1^*,t_k)$	.94	1.58	2.22	2.86	3.50	4.13	4.77		1.966	
i=2	$Q_1^*(k)$	0	2	3	3	3	2	0	-1.8539		
	$\phi(a_0^*,\ldots,a_2^*,t_k)$	0.00	1.57	2.84	3.28	2.90	1.70	0.00			20.8
	$Q_2^*(k)$	0	2	5	7	6	5	1			
	$\psi(b_0^*,\ldots,b_2^*,t_k)$	0.00	2.00	5.15	6.62	6.42	4.55	1.01		-3.7625	
i=3	$Q_1^*(k)$	0	2	3	3	3	2	0			
	$\phi(a_0^*,\ldots,a_3^*,t_k)$	0.00	1.62	2.89	3.28	2.85	1.65	0.00	0.0606		14.49
	$Q_2^*(k)$	0	2	5	7	6	4	1			
	$\psi(b_0^*,\ldots,b_3^*,t_k)$	0.00	2.14	5.30	6.62	6.27	4.41	1.19		0.1793	
i=4	$Q_1^*(k)$	0	2	3	3	3	2	0			
	$\phi(a_0^*,\ldots,a_4^*,t_k)$	0.00	1.92	2.82	2.97	2.79	1.96	0.00	-0.3146		- 4.97
	$Q_2^*(k)$	0	2	5	7	6	4	1			
	$\psi(b_0^*,\ldots,b_4^*,t_k)$	0.00	1.99	5.33	6.77	6.30	4.26	1.34		0.1528	
i=5	$Q_1^*(k)$	0	2	3	3	3	2	0			
	$\phi(a_0^*,\ldots,a_5^*,t_k)$	0.00	1.93	2.80	2.97	2.81	1.95	0.00	0.0217		- 5.54
	$Q_2^*(k)$	0	2	5	7	6	4	1			
	$\psi(b_0^*,\ldots,b_5^*,t_k)$	0.00	1.97	5.38	6.77	6.25	4.29	1.29		-0.0545	
i=6	$Q_1^*(k)$	0	2	3	3	3	2	0			
	$\phi(a_0^*,\ldots,a_6^*,t_k)$	0.00	1.97	2.85	2.86	2.86	1.99	0.00	0.1126		-21.0
	$Q_2^*(k)$	0	2	5	7	6	4	1			
	$\psi(b_0^*,\ldots,b_6^*,t_k)$	0.00	1.86	5.23	7.11	6.10	4.18	.95		-0.3360	

after three such cycles, but did not change the final optimal $\underline{O}^*$ and $\underline{Q}^*$ resulting from the first cycle.

The dynamic programming subproblem calculations were carried out assuming increments of flow rate and storage of 1 unit. For example, the maximum flow rate possible between reservoirs 2 and 3 is 8 units. If each unit represents 50 cfs., then the maximum rate is 400 cfs. Actually, an increment of 50 cfs. is not unrealistic, based on the accuracy of typical flow sensing elements. Flow rates determined from the polynomials (given by the outer problem) were rounded off to whole numbers, for use in the dynamic programming subproblems. In solving the outer problem by an unconstrained minimization technique, penalty terms had to be added in order to keep flow rates given from the outer problem within the specified upper and lower bounds. Total computation time is quite sensitive to the sizes of flow and storage increments selected for the dynamic programming calculations. For this example, computation time was 23.5 seconds on the CDC 6400 computer at the University Computer Center at Colorado State University.

CHAPTER VI

REFERENCES

1. Bell, W., "Optimal Control of Flow in Combined Sewers," submitted for partial satisfaction of requirements for Ph.D., Department of Civil Engineering, Colorado State University, Fort Collins, Colorado, 1973.

2. Bell, W., G. Johnson, and C. B. Wynn, "Simulation and Control of Flow in Combined Sewers," Sixth Annual Simulation Symposium, Tampa, Florida, March, 1973.

3. Bell, W., and C. B. Wynn, "Minimization of Pollution from Combined Sewer Systems," paper presented at the International Symposium on Systems Engineering and Analysis, Purdue University, October, 1972.

4. Bell, W., C. B. Wynn, and G. L. Smith, "Model of Real-Time Automation and Control Systems," Technical Report #7, OWRR - Metropolitan Water Intelligence Systems Project, Department of Civil Engineering, Colorado State University, February, 1972.

5. Canon, M. D., C. D. Callum, and E. Polak, Theory of Optimal Control and Mathematical Programming, McGraw-Hill, 1970.

6. Cunge, J. A., "On the Subject of a Flood Propagation Computation Method," Journal of Hydraulic Research, Vol. 7, No. 2, pp. 205-30, 1969.

7. Graupe, D., Identification of Systems, Van Nostrand Reinhold, 1972.

8. Grigg, N. S. J. W. Labadie, G. L. Smith, D. W. Hill and B. H. Bradford, Completion Report - Phase II, MWIS Project, Colorado State University, 1973.

9. Heidari, M., V. T. Chow, P. V. Kokotovic, and D. D. Merideth, "Discrete Differential Dynamic Programming Approach to Water Systems Optimization," Water Resources Research, Vol. 7, No. 2, pp. 273-82, April, 1971.

10. Labadie, J. W., "Optimization Techniques for Minimization of Combined Sewer Overflow," Technical Report #11, MWIS Project, Colorado State University, 1973.

11. Labadie, J. W., N. S. Grigg, and P. D. Trotta, "Minimization of Combined Sewer Overflows by Large-Scale Mathematical Programming," presented at the International Symposium on Applications of Computers and Operations Research to Problems of World Concern, Washington, D. C., August 20-21, 1973.

12. Larson, R. E., State Increment Dynamic Programming, American Elsevier, 1968.

13. Lasdon, L. S., Optimization Theory for Large Systems, MacMillan, 1970.

14. Wynn, C. B. and J. B. Moore, "The Application of Optimal Linear Regulator Theory to a Problem in Water Pollution," IEEE Transactions on Systems, Man, and Cybernetics, September, 1973.

CHAPTER VI

OPTIMAL STORAGE SIZING AND PLACEMENT

A. INTRODUCTION

A simulation approach for developing design (with regard to detention reservoir sizing) and control criteria for combined sewer systems utilizing detention storage was presented in Chapter IV. The basic reason for using simulation was to gain operational experience and to have the flexibility to use whatever system models are needed. It was shown in Chapter V how the simulation model and formal optimization could be combined for development of real-time control strategies. These strategies, however, are based on a particular proposed configuration for the Master Plan detention reservoirs. A methodology is therefore proposed in this chapter for analyzing alternative configurations, as well as alternative storage sizings.

Various mathematical models have been developed which deal with the design of sewer systems. The Hydrocomp Simulation Program [2] has investigated the problem of "...determining the storage required for peak stormwater runoff volumes to achieve a fixed outflow from a watershed." Their approach uses flood routing with a restricted outflow capacity. All flows above this capacity are sent into a storage facility. The Hydrocomp model does not, however, directly deal with the distribution of storage facilities within the combined sewer system. Although the claim is made that this model finds the storage required, there is no optimization with respect to locations for this storage along the sewer pipe network. General guidelines as to the locations of storage facilities for maximum benefit are suggested. These guidelines, however, require analysis exterior to the simulation program.

The Battelle Urban Wastewater Management Model [1] deals only with major sewer system components. Under the category of major components, the model considers trunk and interceptor sewers, and storage and treatment facilities. Functions of the model consist of,

> "...determination of the optimum operation or automatic control of existing or planned systems during rainstorms; and determination of the most economically feasible combination of design alternatives for improving or expanding existing systems to meet specified performance criteria."

The optimum operation of the system under consideration is accomplished by means of dynamic programming. Details of its application to the control optimization is not presented. In the design stage of the Battelle model, optimization is mentioned but not elaborated upon. The design optimization is supposedly capable of selecting the least-cost combination of available sewer system modification by use of a gradient search technique. No connection is made between least-cost alternatives and the optimum allocation of detention reservoir sizes and spatial distributions.

If a combined sewer control system is planned using auxiliary storage, several basic design questions must be answered:

1. How shall the large-scale system be decomposed into subbasins?
2. For a given subbasin, what will be the total investment in storage facilities? This is roughly equivalent to asking what the total storage volume allocated to each subbasin will be.
3. How will the total storage be allocated spatially? In other words, how many detention basins will be built and where will they be located?

The first question has been discussed in Chapter III, as applied to the San Francisco Master Plan. The second question is difficult to answer due to social, political, and economic issues that have not as yet been resolved. One way to deal with this difficulty is to solve optimal design problems for a wide range of total storage investments that could possibly eventually be agreed upon. Such information would greatly aid decision makers in arriving at a solution, while effectively divorcing many, though not all, of the complex socio-political issues from the design problem formulation. In short, the goal is not to produce *the* optimal design, but rather a family of optimal design alternatives, one of which could eventually be decided upon through other analyses involving the political process. This final decision will ultimately come down to the following question: How much is the city willing to pay for pollution control measures, where the probability of meeting federal and state regulatory standards tends to be proportional to the economic investment in pollution control?

The following analysis is only a first step, since only a single design storm has been utilized at this point. The results of this study will be only one point on the depth-duration curves developed in Chapter IV. A wide range of historical and synthetically generated storms must eventually be used for producing optimal designs, and therefore completing these curves. The primary means of comparing designs will be based on their ability to reduce total overflows during the given design storm. There will be a concluding discussion, however, concerning the incorporation of cost-effectiveness analysis, where economies of scale in detention reservoir construction are taken into consideration.

In addition to the use of one design storm, other important limitations in this study include:

1. No consideration is given to the possibility of localized street flooding occuring within a subcatchment due to a particular reservoir placement.
2. Placement of reservoirs in order to facilitate gravity drainage is not a criterion.
3. Socio-political aspects of placement are not included.

Though these limitations appear serious, it is felt that they can be incorporated in future studies. This study is not meant to supersede the admirable work of the designers of the San Francisco Master Plan in placing and sizing the reservoirs. It is only offered as a tool in an infant stage of development that could possibly aid planners in the future.

B. GENERAL FORMULATION OF THE DESIGN PROBLEM

The basic design problem is to determine how various quantities of total storage for each subbasin should be allocated and distributed within each subbasin. In formulating the optimal design problem, it must be recognized that alternative designs cannot be compared unless they are optimally operated. Suppose, for example that two design alternatives are to be evaluated, with each having different reservoir locations and sizing. In order to properly compare the two alternatives, design storms must be applied to each and the given reservoir storage within each alternative utilized in the most effective way in order to minimize overflows. That alternative would then be chosen which produced the lowest total overflow (or *weighted* overflow) for the same given design storms. Thus, within every design problem there is always imbedded an operational problem.

Mathematically speaking, the problem can be posed as follows

$$\underset{\underline{d} \,\varepsilon\, \mathcal{D}}{\text{minimize}} \; Z(\underline{d}) \qquad (1)$$

where

$$Z(\underline{d}) = \underset{\underline{x} \,\varepsilon\, X(d)}{\text{minimum}} \; f(\underline{x}) \qquad (2)$$

$\underline{d}$ = the vector of design variables (i.e., spatial location coordinates for reservoir placement)

$\mathcal{D}$ = the set representing constraints on reservoir placement (i.e., spatial coordinates cannot be outside the boundaries of the subbasin).

$\underline{x}$ = the vector of operational variables (e.g., reservoir releases and necessary overflows at discrete points in time over the duration of the design storm).

$f(\underline{x})$ = the criterion function, which is basically total weighted overflows.

$X(\underline{d})$ = the set representing constraints on reservoir operation, for a given design $\underline{d}$ (e.g., line and storage capacity constraints).

The design or *outer* problem is defined by Equation (1), whereas Equation (2) is the operational or *inner* problem. The function $Z(\underline{d})$ simply represents the minimum total weighted overflow produced by design $\underline{d}$, as determined from optimal operation of the design. The goal of the outer problem is to find that design $\underline{d}^*$ that gives the lowest value of $Z(\underline{d})$, given that $\underline{d}$ is restricted by the set $\mathcal{D}$.

The set $\mathcal{D}$ should ultimately reflect various social and political factors that might influence the selection of possible detention reservoir locations. For now, there is no attempt to include these in this study. In addition, design cost-effectiveness, though not included in this formulation, can be considered through a sensitivity analysis. This will be elaborated on in a subsequent section.

C. APPLICATION TO VICENTE SUBBASIN

C.1 The *Inner* Operational Problem

The general design formulation has been applied to Vicente Subbasin in San Francisco, as delineated in Figure VI-1. The goal in formulating and solving this design problem is to open the way to deciding if the number, placement, and sizes of the detention reservoirs could be changed in order to more effectively minimize overflows. The general formulation should also be applicable to other subbasins, and perhaps even other cities. Vicente Subbasin then serves as a case study. A more detailed analysis of the Vicente Subbasin design problem can be found in [3]. The purpose here is to concisely summarize the formulation and important computational results.

Discussion will first be centered on the operational *inner* problem, where the number, placement, and sizes of the detention reservoirs is fixed with regard to a particular design alternative. The system must now be optimally operated for a given storm input so that its ability to control overflows can eventually be compared with other design alternatives. This design or *outer* problem will be discussed subsequently.

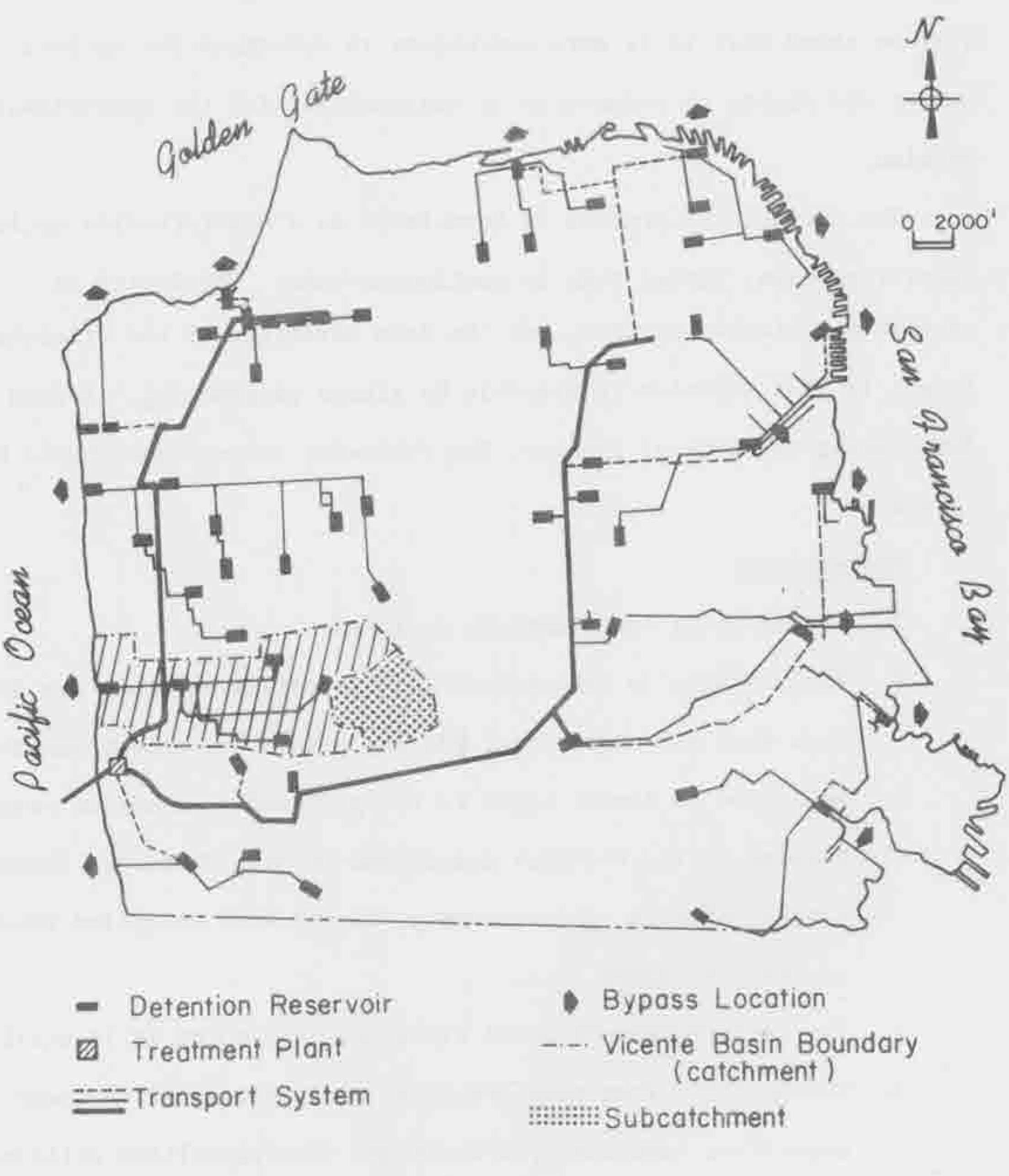


FIGURE VI-1

PROPOSED SAN FRANCISCO MASTER PLAN
FOR WASTEWATER MANAGEMENT [4]

Assume that the spatial placement of reservoirs as shown in Figure VI-1 is fixed. It would seem that the relative sizes of the reservoirs should also be specified as given design variables, but it will be shown that it is more convenient to determine the optimal number and sizing of reservoirs in conjunction with the operational problem.

The operational problem is formulated as a discrete-time optimal control problem, rather than in continuous-time. The latter is certainly a viable approach, but the main advantage of the discrete-time format is that solution is possible by linear programming. Before formulating the control problem, the following assumptions should be listed.

Assumptions:

1. The existing sewer network is fixed.
2. The subbasin is partitioned into subcatchments that are defined such that all storm input falling within the subcatchment can be lumped as direct input to one particular detention reservoir located at its farthest downstream point. There are therefore a total of five subcatchments; one for each detention reservoir, as seen in Figure VI-2.
3. The rainfall-runoff model discussed in Chapter IV is utilized.
4. Throughflows from each reservoir are constrained by sewer capacities immediately downstream. The capacities utilized are based on the pipe dimensions, roughness coefficients, and slope, but not on pressurized conduit flow. This is because pressurized flow is a function of the head in the detention reservoirs and would introduce considerable nonlinearity into the optimization problem.

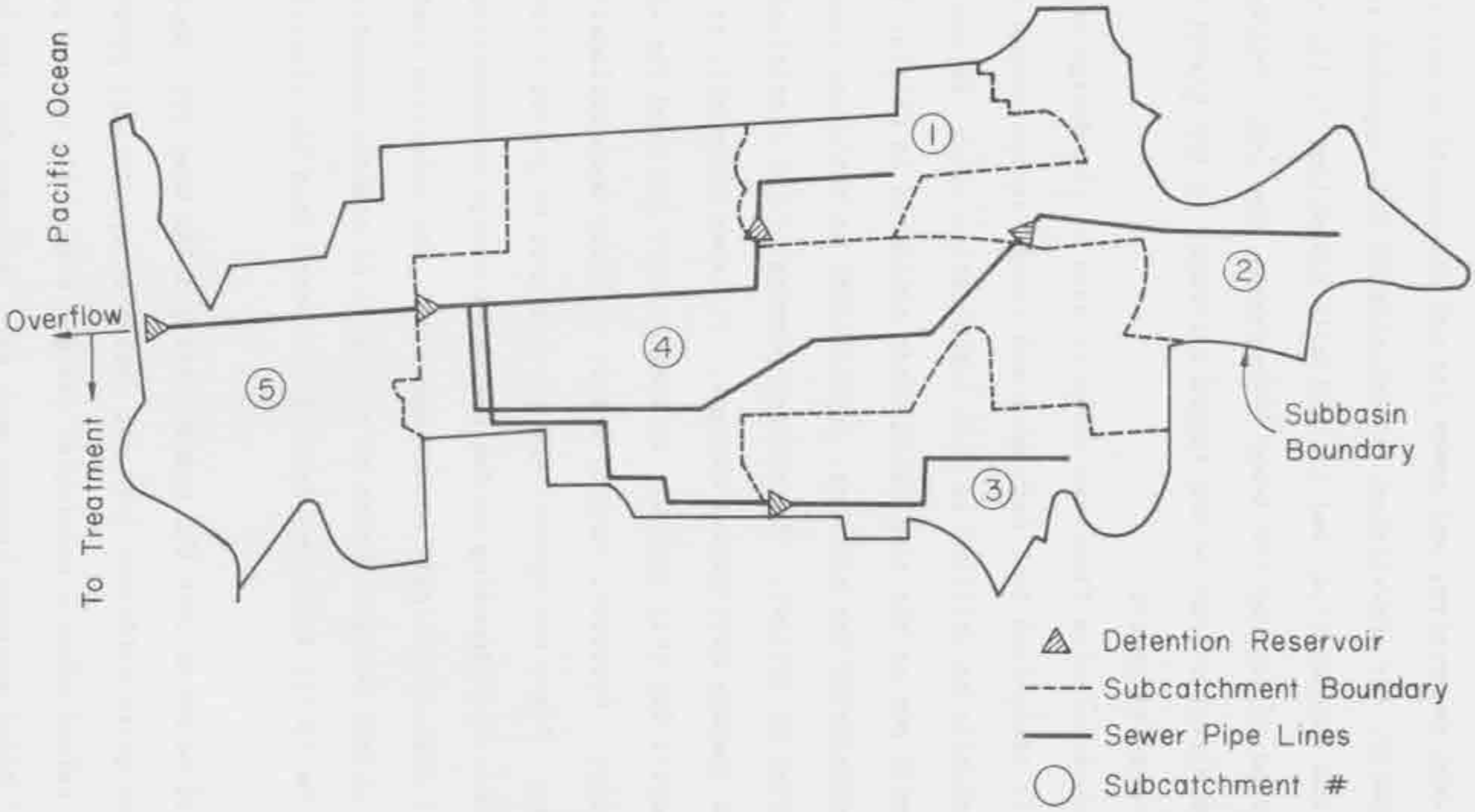


FIGURE VI-2

VICENTE SUBBASIN

5. Simple lag routing is used for the sewer system. That is, upstream hydrographs are lagged by estimated travel times between reservoirs, and peaks are not attenuated in any way. Actually, for convenience, a variation of this approach is used in the optimization, but is basically equivalent to lag routing. Instead of lagging the sewer hydrographs, the input hydrographs from direct stormflow are lagged according to the travel times between reservoirs.
6. A representative five-year storm is used as the design storm. It is recognized that many more and varied design storms must eventually be utilized in this optimization study, as was carried out on the simulation study presented in Chapter IV.

The simplicity of the modeling, particularly in the sewer routing, can be justified as follows. The primary purpose of the operational problem is to compare alternative designs. It seems reasonable to use simplified models for this purpose, as long as they are used for all of the alternatives. Accurate, complex models are more computationally time-consuming. Since the operational problem must be solved a large number of times, corresponding to the number of design alternatives, the computational load can quickly become unwieldy. The intuitive feeling is that the optimal designs chosen on the basis of complex operational models would be little different from those chosen from the simplified models.

It should be noted that the operational problem used for comparing designs can be quite different from the real-time operational problem that must be solved after a particular design is actually chosen and constructed. Model accuracy becomes much more important for the latter, so that more accurate models may be preferable. The real-time optimal control problem is discussed in Chapter V..

Therefore, given the reservoir placement as shown in Figure VI-2, the operational problem is formulated as follows

$$\underset{\underline{O},\ \underline{Q},\ \underline{S},\ \text{and}\ \underline{S}_{max}}{\text{minimize}} \quad \sum_{k=1}^{M} [\omega(k)O(k) - \mu(k)Q_5(k)] \tag{3}$$

Subject to:

$$S_j(k+1) = S_j(k) - Q_j(k) + f_j(R_\ell(k));\ j=1,2,3 \tag{4}$$

$$S_4(k+1) = S_4(k) + Q_1(k) + Q_2(k) + Q_3(k) - Q_4(k) + f_4(R_\ell(k)) \tag{5}$$

$$S_5(k+1) = S_5(k) + Q_4(k) - O(k) + f_5(R_\ell(k)) \tag{6}$$

(for $k=1,\ldots, M$)

$$\left.\begin{aligned} &0 \le Q_j(k) \le Q_{jmax} && (7)\\ &0 \le S_j(k+1) \le S_{jmax} && (8)\end{aligned}\right\} \quad j=1,\ldots, 5;\ k=1,\ldots, M$$

$$\sum_{j=1}^{5} S_{jmax} = S_{total} \tag{9}$$

where

M = total number of discrete time periods

$S_j(k)$ = storage in detention reservoir j (associated with subcatchment j) at the beginning of period k. The reservoir is initially assumed to be empty, or $S_j(0) = 0$. $\underline{S} = (S_j(k),\ j=1,\ldots, 5;\ k=2,\ldots, M+1)$.

S_{jmax} = maximum storage available in reservoir j. Though it is actually a design variable, it is considered as a variable in the operational problem.

$\underline{S}_{max} = (S_{1max}, \ldots, S_{5max})$.

$Q_j(k)$ = discharge from detention reservoir j during period k. Discharge $Q_5(k)$ goes directly into the interceptor. $\underline{Q} = (Q_j(k), j=1, \ldots, 5; k=1, \ldots, M)$.

Q_{jmax} = maximum discharge rates immediately downstream of reservoir j (given).

$O(k)$ = overflow to receiving waters released from reservoir 5, during period k. $\underline{O} = (O(1), \ldots, O(M))$.

$f_j(R_\ell(k))$ = lagged, lumped direct storm input to reservoir j, based on rainfall $R_\ell(k)$ over subcatchment j produced by design storm ℓ during period k, as transformed by the rainfall-runoff model (given).

S_{total} = total storage allocated to Vicente Subbasin (given).

$\omega(k)$, $\mu(k)$ = weighting factors (given).

The term $-\mu(k)Q_5(k)$ has been added in order to credit throughflows and therefore encourage as much flow to the interceptor as possible. The goal is to discourage flow being held in the reservoirs when it can be discharged to the interceptor. This insures that the reservoirs will be drained as much and as soon as possible, while minimizing overflow, in anticipation of another subsequent storm event.

The above problem can be readily solved by linear programming. There are a total of 11M variables, plus 10M slack variables that must be added in Equations (7) and (8). The constraints number 15M.

Notice, again, that the optimal reservoir sizing is accomplished by allowing the S_{jmax}, $j=1,\ldots,5$, to be variables in the operational problem. Notice also that the optimal number of reservoirs is determined at the same time. The maximum number is set at 5. If the optimal design should have less than five reservoirs, then one or more of the S_{jmax} would be zero in the optimal solution.

C.2 The *Outer* Design Problem

The outer design problem is actually solved for various values of total storage S_{total} allocated to Vicente Subbasin. Given S_{total}, the design problem is concerned with comparing total weighted overflows from spatially discretized alternate locations for any of the detention reservoirs. An increase in the number of possible detention reservoir locations within each subcatchment will increase the combinations of detention reservoir placements. The number of possible storage distributions is therefore equal to the number of combinations of all possible detention reservoir locations. As the number of locations increases, the amount of computer time needed to investigate all these distributions increases rather rapidly. Each configuration requires solution of the linear programming problem before the optimum design can be found.

A search technique employed in the design model was found to greatly reduce the total number of combinations that need to be investigated. Various infeasible and impractical locations can be removed through preliminary analysis of the subbasin and its sewer network. Such screening will greatly reduce the total computation time needed for selection of an optimum design.

The design problem generates the input data necessary for solution of the inner linear programming problem for each of the detention reservoir distributions. These input data are parameters describing real pipe network and physical constants associated with flow in the system. Each distribution resulting from a movement, either upstream or downstream, of any detention reservoir requires the redefining of these input parameters. Direction of movement, upstream or downstream, and determination of the subcatchment and associated detention reservoir requiring movement is stipulated by the design problem.

For every configuration specified by the design problem, the inner operational problem is called upon. After receiving the solution from the linear program, the design model tests the total weighted overflows returned and determines in which subcatchment and direction to move a detention reservoir. From this search routine decision a new inner operational problem is developed based on the new configuration of the subbasin. This process of defining a new set of equations and solving the resulting optimization problem continues until no further improvement in the value of the optimization objective function can be achieved through further manipulation of the detention reservoirs.

The computation time required is still too great, however, to permit consideration of every possible screened configuration of the subbasin. This large number of possible configurations necessitates the use of a more efficient search algorithm. Hopefully, only a small fraction of possible combinations need to be investigated in order to locate that optimal set of detention basin locations.

In essence, this search algorithm for the outer problem directs the movement of each detention reservoir and associated subcatchment to their best locations one at a time. The movement along the sewer pipe network either upstream or downstream of any one detention reservoir is referred to as a direction. This movement alters the size of several subcatchment drainage areas but only the location of one detention reservoir. The search algorithm is basically a cyclic or alternate one-dimensional search technique [6]. It performs an optimization on the locations for one reservoir at a time while holding all other reservoirs stationary. The algorithm is illustrated in Figure VI-3.

The number of optimization problem solutions needed to locate the best configuration is dependent upon the particular problem under consideration. A given drainage basin with a maximum of n subcatchments each with p possible sizes and therefore p possible detention reservoir locations would generate a total of p^n combinations. With n equal to 3 and p equal to 10, the result would be 1000 combinations. This cyclic search approach would require only $n(p-1) + 2$ optimization problems, or a total of 29 for the above example. For Vicente Basin, only three subcatchments took on five positions each and the other had four possible locations. Though a total of 100 possible combinations results, it took the cyclic search routine a maximum of 13 iterations to produce the optimal solution. Employment of this search routine to the outer problem can considerably reduce the amount of computation and therefore encourage consideration of a greater number of possible reservoir locations.

D. COMPUTATIONAL RESULTS

D.1 Behavior of the Search Algorithm

Contours of the function $Z(\underline{d})$ are plotted in Figures VI-4 and VI-5 to illustrate the behavior of the cyclic search algorithm. The number of possible locations for each reservoir (IMAX(J)) was set at five. Tables VI-1 and VI-2 show the pertinent data used for solving the design problem.

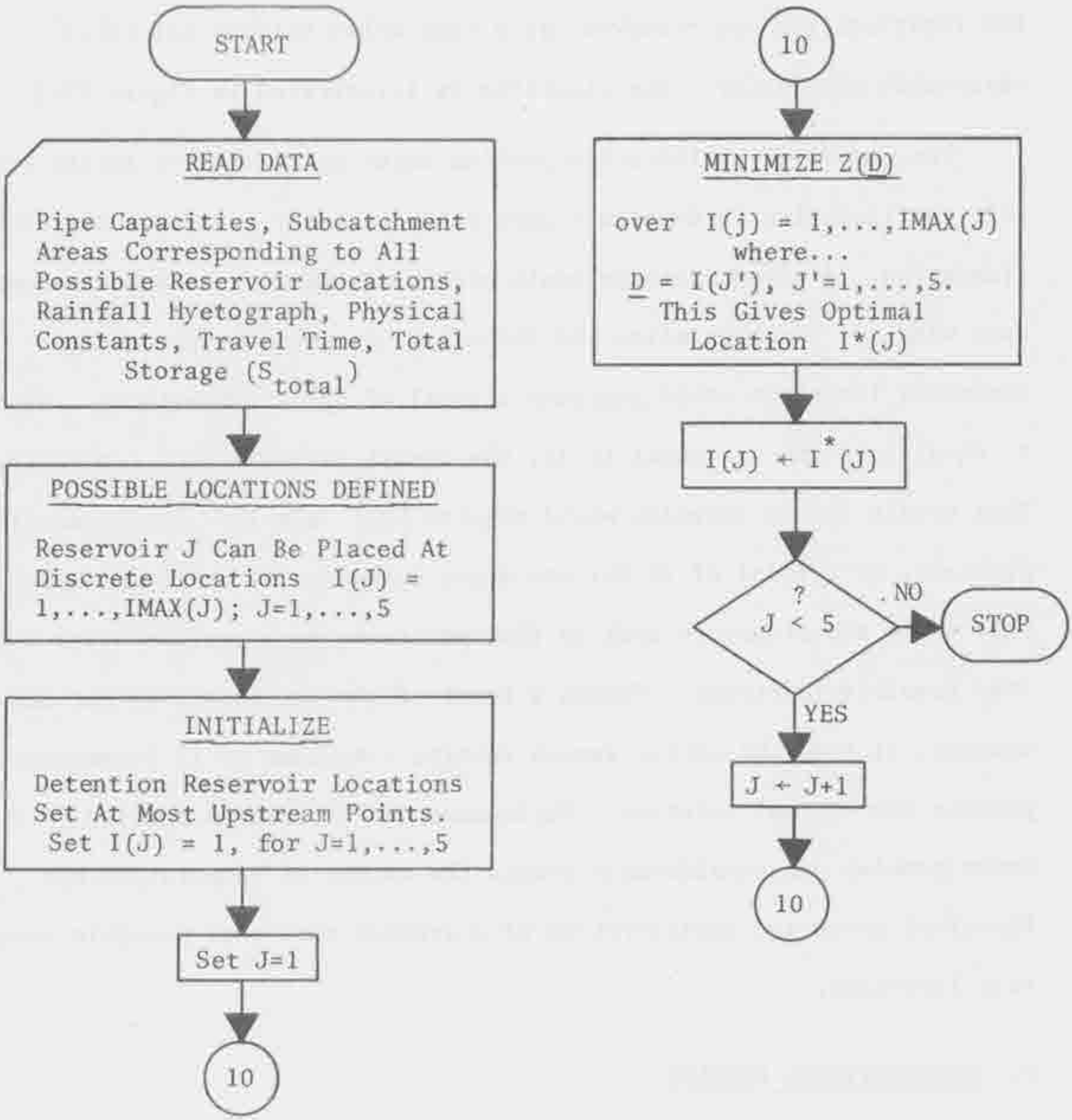
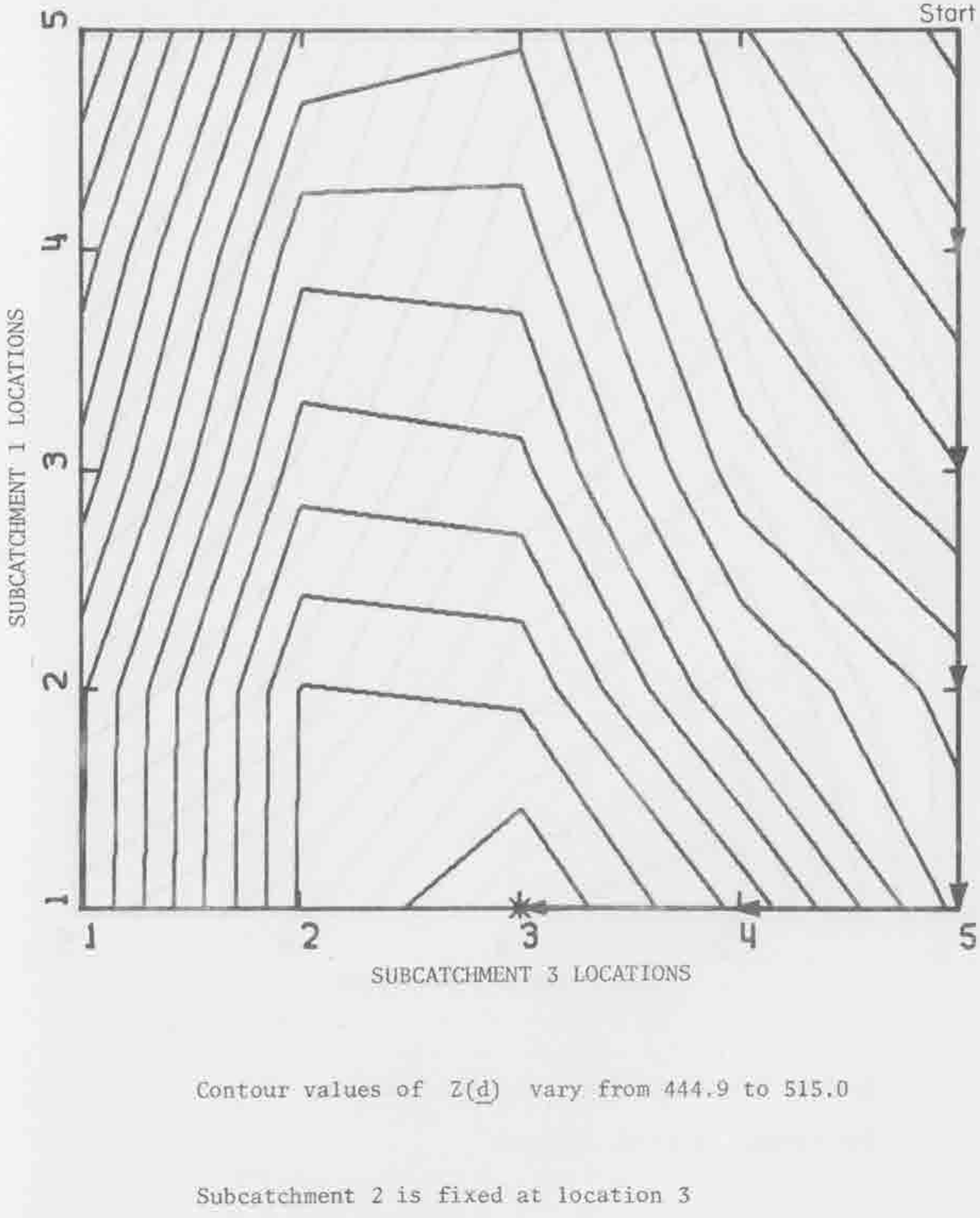


FIGURE VI-3

FLOW CHART FOR DESIGN PROBLEM
CYCLIC COORDINATE SEARCH
ALGORITHM



Contour values of $Z(\underline{d})$ vary from 444.9 to 515.0

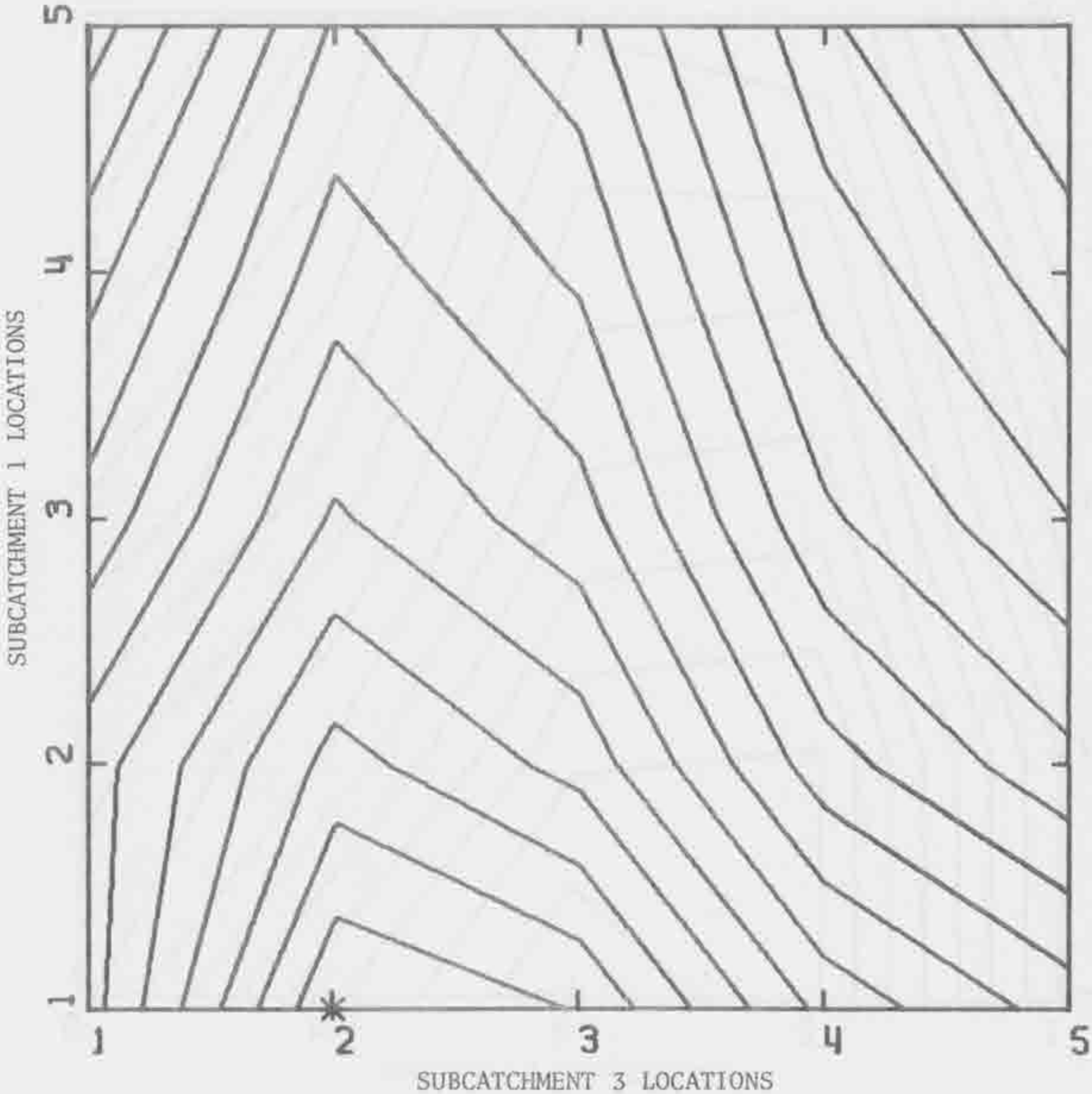
Subcatchment 2 is fixed at location 3

→ Designates the cyclic search path

* Denotes the calculated optimal configuration

FIGURE VI-4

CONTOUR MAP 1: CONTOURS OF $Z(\underline{d})$
SHOWING CALCULATED OPTIMUM



Contour values of $Z(\underline{d})$ vary from 443.9 to 523.3

The contour interval is 4.963

Subcatchment 2 is fixed at location 4

* Denotes the optimal configuration

FIGURE VI-5

CONTOUR MAP 2: CONTOURS OF $Z(\underline{d})$
SHOWING ACTUAL OPTIMUM

TABLE VI-1

PHYSICAL CONSTANTS USED FOR VICENTE SUBBASIN

Subcatchment No.	Location of Reservoir	Drainage Area Acres	Maximum Flow (cfs)	Travel Time (min.)	Routing Constant K (hr.)	Runoff Coefficient C	Base Flow cfs/mi.^{2}
1		0	0	0	0.1	0.65	1.0
	Taraval @ 22nd St.	95	82	10.01	"	"	"
	22nd St. @ Ulloa	129	82	8.54	"	"	"
	Vicente @ 22nd St.	160	110	7.51	"	"	"
	Vicente @ 27th St.	213	110	6.06	"	"	"
2		0	0	0	0.15	0.65	1.0
	Ulloa @ Kensington Way	147	105	14.76	"	"	"
	Wawona @ Ulloa	290	165	13.31	"	"	"
	Wawona @ 14th Ave.	419	170	12.18	"	"	"
3		0	0	0	0.1	0.65	1.0
	Monterey Blvd. @ Market St.	90	48	15.97	"	"	"
	Eucalyptus @ Junipero Serra	150	170	14.66	"	"	"
	Eucalyptus @ Melba	220	170	13.18	"	"	"
	Meadowbrook Dr. @ Ocean Ave.	261	170	10.70	"	"	"
4	Vicente @ 37th	1294*	520	3.46	0.1	0.6	1.0
5	Vicente @ Great Highway	370	320	0	0.15	0.5	1.0

* includes drainage areas contributing to subcatchments 1, 2, and 3

TABLE VI-2

5-YEAR DESIGN STORM FOR VICENTE SUBBASIN

TIME (MIN)	RAIN(IN)	EXCESS RAIN(IN)	FLOW(CFS)*
0.0	.04	.03	.3
5.0	.08	.05	39.8
10.0	.22	.14	96.5
15.0	.12	.08	259.3
20.0	.06	.04	231.4
25.0	.04	.03	160.0
30.0			109.2
35.0			47.7
40.0			20.9
45.0			9.3
50.0			4.2

* Example flow for subcatchment 1, with reservoir located at Vicente @ 22nd Street, found using the rainfall-runoff model described in Chapter IV. This hydrograph is then lagged by its travel time to the bypass point at the Great Highway and averaged over 10 minute time intervals. This averaged flow is then the input to reservoir 1. The same procedure is used for the other subcatchments.

An important question is whether or not $Z(\underline{d})$ is convex. Having a convex problem is a guarantee that the cyclic search will locate the global optimal design configuration. The incremental distance between alternate detention reservoir locations, however, directly affects the ability of the cyclic search to locate the optimal configuration. For an excessively large increment, the algorithm may terminate at a design other than the global optimum design. For Vicente Subbasin, an adequate interval between reservoir locations would be the length of a city block. The results that follow were not based on this distance, but rather on a coarser set of locations.

The contour plots are representative of a plane through the surface $Z(\underline{d})$ created by all combinations of detention reservoir locations and subcatchment sizes. Each plot was generated by calculating the optimal solutions of all possible combinations resulting from two varying subcatchments while holding the third subcatchment at a fixed position. In this manner, a cutting plane generated 25 values of overflow, one for each combination of the two varying subcatchments. Although a coarse grid was used, the shaping of a nearly convex surface is visible. A finer grid would further highlight the convex surface and the ability of the search to locate the optimal design.

The contour map shown in Figure VI-4 was produced from a fixed total storage capacity in Vicente Subbasin of 840,000 cubic feet. Subcatchment 2 was held fixed at its calculated optimal location. The coordinate numbers 1 through 5 correspond respectively to an infinitesimally small subcatchment area (furthest upstream location) to the largest sizes considered (most downstream), as listed in Table VI-3.

TABLE VI-3

LOCATION NO. AND CORRESPONDING PHYSICAL LOCATIONS

SUBCATCHMENT (see Figure 6)	LOCATION NO.	PHYSICAL DETENTION RESERVOIR LOCATION
1	1	Farthest upstream (area=0.0)
	2	Taraval St. @ 22 nd St.
	3	22 nd St. @ Ulloa St.
	4	Vicente St. @ 22 nd St.
	5	Vicente St. @ 27 th St.
2	1	Farthest upstream
	2	Ulloa St. @ Kensington Way
	3	Wawona St. @ Ulloa St.
	4	Wawona St. @ 14 th Ave.
3	1	Farthest upstream
	2	Monterey Blvd. @ Market St.
	3	Eucalyptus Blvd. @ Junipero Serra Dr.
	4	Eucalyptus Blvd. @ Melba Ave.
	5	Meadowbrook Drive @ Ocean Ave.

A somewhat better configuration than the one determined by the search routine was located, as shown in Figure VI-5. The deviation of this true optimum from the calculated optimum supports the need for a smaller discretization. Contour Map 2 shows the true optimum to be at location 2 on the abscissa for subcatchment 2 at location 4. The search routine terminated at point 3 on the abscissa of Contour Map 1. Thus, the search algorithm terminated at a point deviating from the true optimum by one incremental location in subcatchments 2 and 3. Again, reduction of the grid size used would hopefully reduce this error.

D.2 Total Storage Capacity Versus Overflow Volume

A graph of total storage capacity versus overflow volume was subsequently developed. Using the predetermined design storm, various values of total storage were used in the design model to obtain corresponding overflow values. The curve resulting from this plot of points turned out to be a straight line in the case of the Vicente Basin, as exhibited in Figure VI-6. Total storage capacity used in the plot varies from a minimum of 450,000 cubic feet. Below this value, the sewer pipes and detention basins become overburdened when trying to control local flooding resulting from the design storm. The maximum storage capacity used for the plot was 1,100,000 cubic feet. This value virtually eliminates all overflows to the receiving waters.

This straight line curve between storage and overflow exhibits a slope of exactly -1.0. The significance of this slope is that for each incremental increase in storage capacity, a corresponding incremental reduction in overflow can be obtained. Verification that the points on the graph are actually representative of the most efficient configuration and operation of the given storage capacities is supported by the slope of this curve. The best that can be done to curtail overflows is to utilize to capacity the storage available. Support of this fact is given by a mass balance of flow over the total system.

Entering Figure VI-6 with a limitation on overflow imposed by environmental constraints would immediately designate the required storage capacity needed to meet the overflow restrictions, based on the design storm used. It should be remembered that Figure VI-6 is based on an optimum allocation of storage within the drainage basin which has previously been determined. Though the curve itself may seem intuitively obvious, determination of the optimal allocation of these total storages to give the corresponding minimal overflow is not a trivial problem.

D.3 Vicente Subbasin Design

Vicente Subbasin was decomposed into a maximum of five subcatchments and therefore a maximum of five possible detention reservoirs were considered. Again, the outer problem implicitly optimizes over the number of subcatchments. An infinitesimal drainage area is associated with the elimination of a particular subcatchment, and therefore its downstream reservoir. In the outer problem's optimization over all the locations for the installation of detention reservoirs, the number of associated subcatchments is therefore implicitly determined.

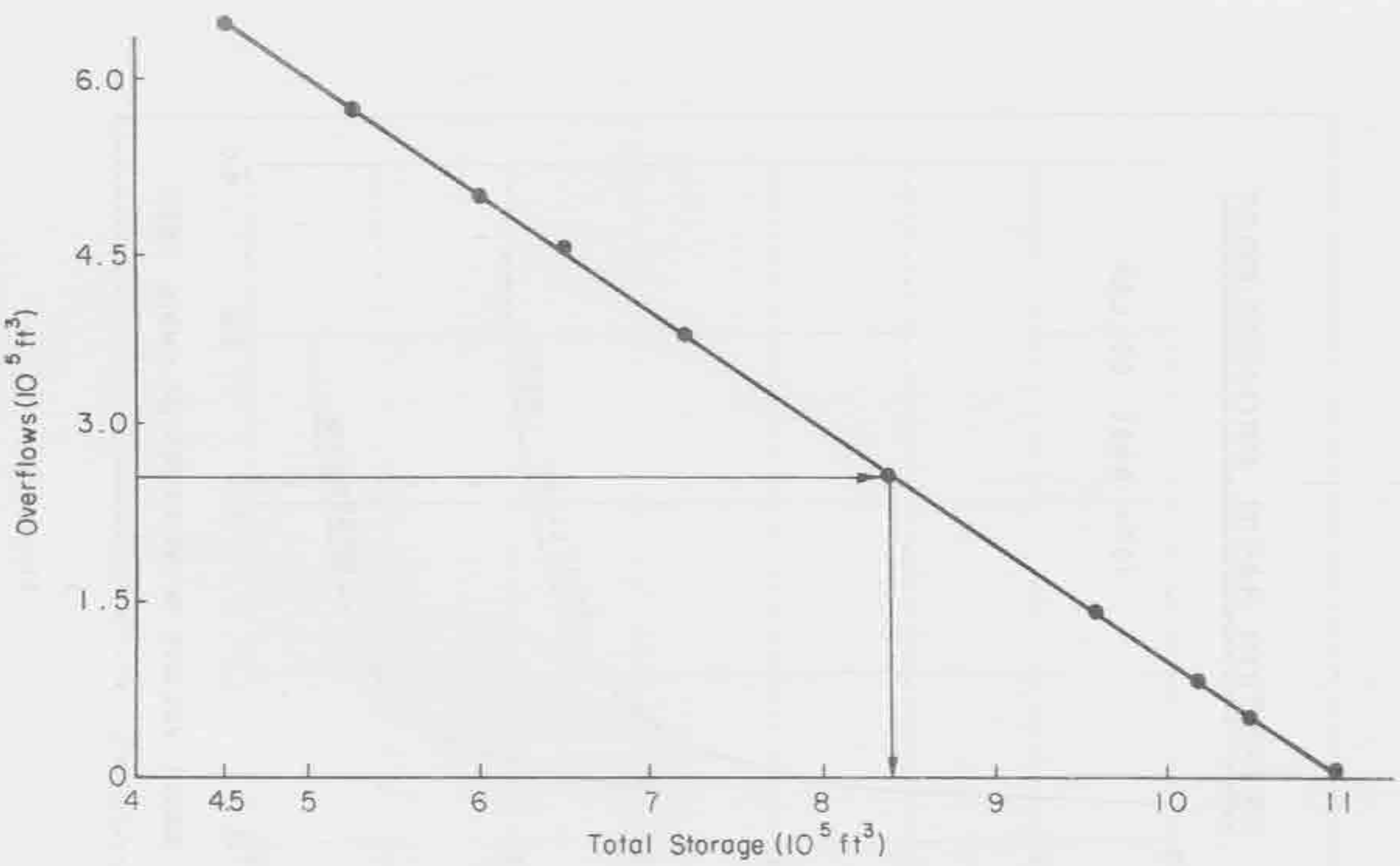


FIGURE VI-6

TOTAL STORAGE VS. OVERFLOW

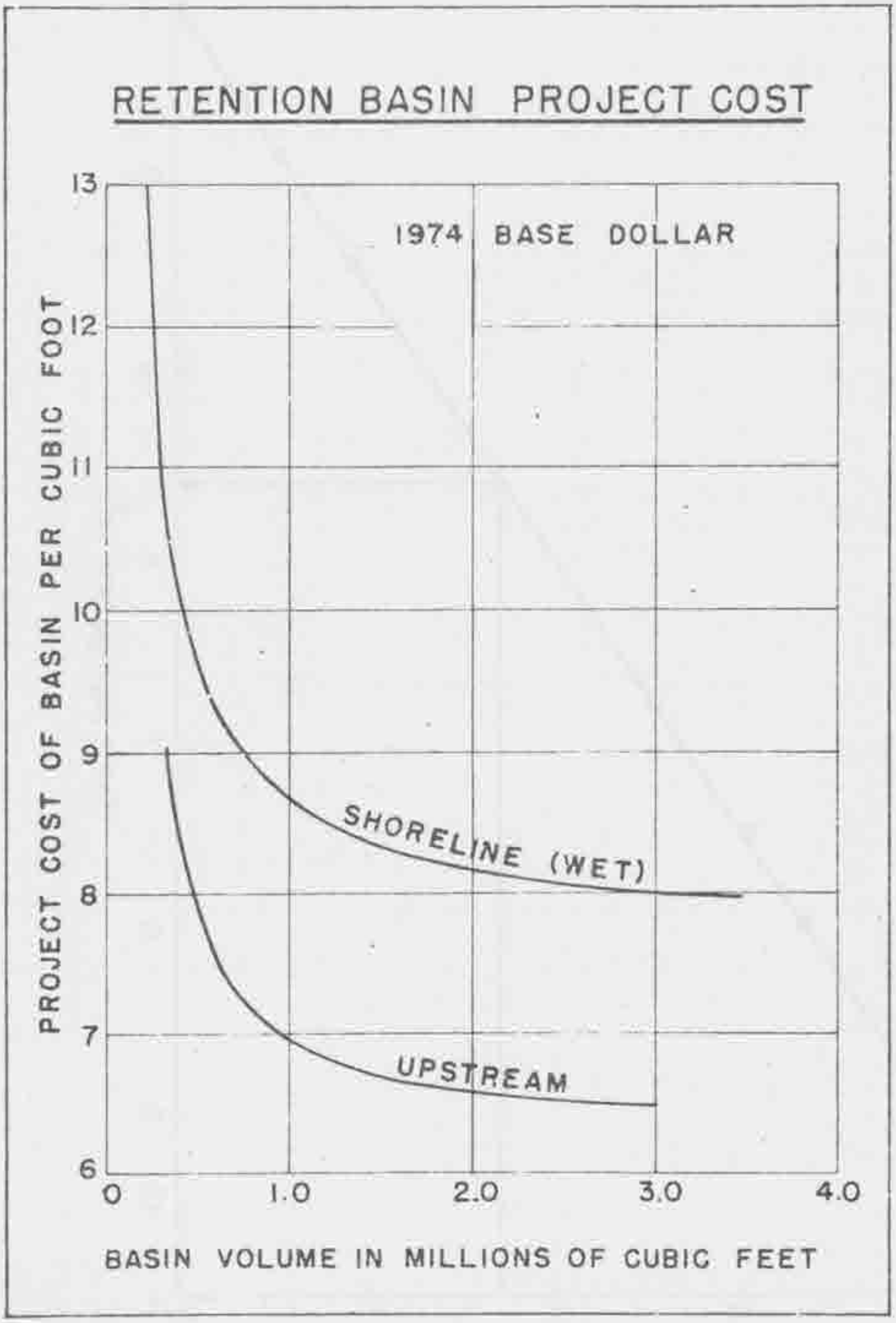


FIGURE VI-7

DETENTION BASIN CONSTRUCTION COST [5]

For all the given total storage capacities specified, the optimum number of subcatchments was always less than the original five. In every instance at least one detention reservoir was eliminated, and its storage put to better use elsewhere in the subbasin. Subcatchment number 1 in Figure VI-2 was always eliminated by the cyclic search in locating the optimum configuration. Detention reservoir number 5, on the other hand, located at the shoreline or point of overflow, was always eliminated by the inner optimization problem by assigning it zero storage. The locations of the other detention reservoirs did not vary by more than one increment. In view of this, all optimal configurations based on various total storage capacities encompass a rather small set of locations.

Due to the nature of the existing sewer pipe lines in Vicente Subbasin, a large reduction in overflows was not achieved by movement of the detention reservoirs. A difference of 25,000 cubic feet of overflow existed between the Master Plan reservoir locations (with a total storage of 840,000 cubic feet) under an optimal operating policy and the optimum locations obtained through use of this design model. This represents only about a 9% reduction, due to the fact that the pipe network in Vicente Subbasin is capable of transporting the sewage flow resulting from the design storm to most of the possible locations of the detention reservoirs. It should be noted, however, that there is possibility of street flooding due to removal of upstream reservoirs that is not considered in this current model. Future models should properly include this factor.

A revised simplex computer code using the explicit inverse method was used for solving the linear operational problem. This problem had to be solved for each designation of design variables. The advantage of the efficient search algorithm is clear when considering that each linear programming solution took about 7 seconds on the CDC 6400 computer at Colorado State University for M=7 time periods. Thirteen iterations on the design variables, as carried out by the search algorithm, therefore required about 95 seconds. About 110,000 words of storage (in octal) were required for the design problem, and were extremely sensitive to the number of time periods considered. A value for M exceeding seven would require more core storage than is currently available at CSU. Future research will concentrate on decomposition and storage reduction devices for this problem.

D.4 Extension to Least-Cost Alternatives

The cost curves in Figure VI-8 were developed from Figure VI-7. Since the least-cost alternative for construction of the detention reservoirs is desired, these curves are of use in minimizing total construction cost. As reservoir size becomes larger the cost per unit of storage capacity decreases. A reduction in total construction cost is realized by an increase in reservoir size along with a reduction in the number of storage reservoirs required. This curve demonstrates the economy of scale in auxiliary storage reservoirs for San Francisco. Because this curve is concave, linear programming techniques are ineffective in minimizing over an objective function described in terms of this cost curve.

The inner optimization problem minimizes overflows, as of now, without regard to costs of construction. It was found, however, that the inner problem could be minimized by several solutions, and not just

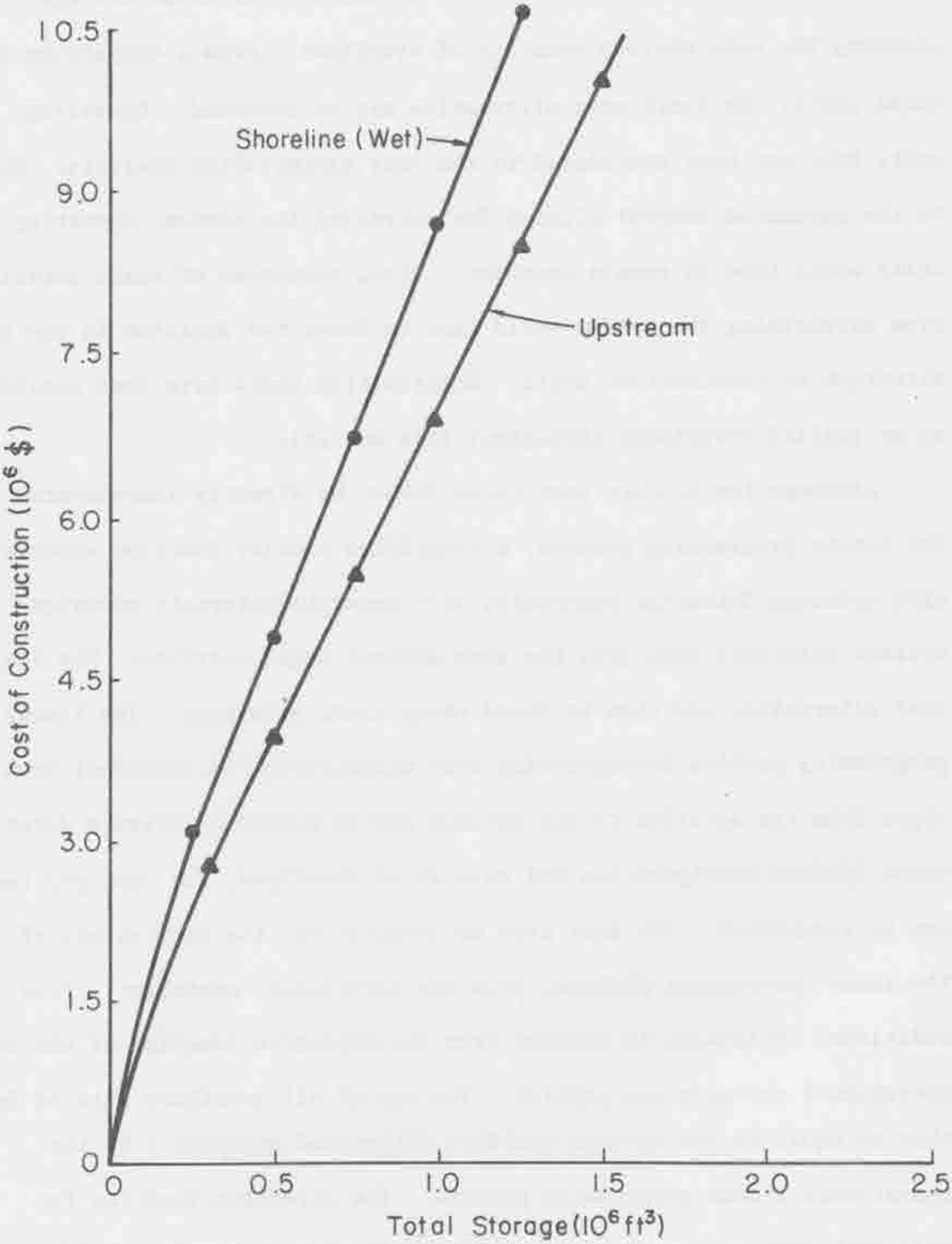


FIGURE VI-8

CONSTRUCTION COST VS. TOTAL STORAGE

one. By generating the various nonunique optimal solutions, a cost of construction can be found for various allocations of total storage yielding the *same* minimum quantity of overflows. From a comparison of these costs, the least cost alternative may be selected. Operating costs have not been considered in the cost minimization analysis. Due to the automated control planned for operating the system, operating costs would tend to remain constant. Also, economies of scale anticipated from maintaining the system would tend to force the decision in the same direction as construction costs. Construction costs have been considered as an initial investment throughout this analysis.

Although the concave cost curve cannot be directly incorporated into the linear programming problem, a simplified penalty function associated with selected detention reservoirs will generate alternate nonunique optimal solutions that give the same minimal total overflow. The least-cost alternative can then be found among these solutions. The linear programming problem incorporating cost minimization is dependent upon input from the solution of the optimum design problem. Given a determined optimum configuration and associated overflows, the cost problem can be formulated. The cost problem contains all the constraints of the inner operational problem, plus one additional constraint. This additional constraint is derived from the objective function of the inner operational optimization problem. The sum of all overflows must be less than or equal to the minimum overflow determined previously by the operational linear programming problem. The objective function for this cost problem, consists of weighting factors or penalty coefficients assigned to the maximum storage capacity in each subcatchment. Adjustment of these penalty coefficients are based on the available nonunique allocation of storage and the curves in Figure VI-8. By careful adjustment of these coefficients, an indirect search for the least-cost alternative is obtained.

The objective function for this modified problem is:

$$\underset{\substack{S_{jmax} \\ j=1,\ldots 5}}{\text{Minimize}} \quad \sum_{j=1}^{5} P_j S_{jmax}$$

Subject to: the constraints (4) - (9), plus the following constraint:

$$\sum_{k=1}^{M} O(k) \leq O^*_{min}$$

The values for P_j, $j=1,\ldots,5$ were respectively 1.0, 1.1, 1.15, 1.05, 1.20. The nonunique solution found by this set of weighting factors contained only *two* detention reservoirs, and yet still produced the minimum overflow O^*_{min}.

The cost of construction associated with this solution is 4.95 million dollars, based on a total storage of 600,000 cubic feet corresponding to Alternative A of the Master Plan. This corresponds to a cost of approximately 6.8 million dollars associated with the original Master Plan design, or about a 25% decrease in cost. Though the optimal design did not significantly reduce overflows, as compared to the original Master Plan design, it is clear that significant reduction in cost was achieved, at least for the given 5-year design storm.

CHAPTER V - NOTATION

$\underline{d}$ = the vector of design variables (e.g., spatial location corrdinates for reservoir placement)

$\mathcal{D}$ = the set representing constraints on reservoir placement (e.g., spatial coordinates cannot be outside the boundaries of the subbasin)

$f(\underline{x})$ = the criterion function, which is basically total weighted overflows

$f_j(R_\ell(k))$ = lagged, lumped direct storm input to reservoir j, based on rainfall $R_\ell(k)$ over subcatchment j produced by design storm ℓ during period k, as transformed by the rainfall-runoff model (given)

M = total number of discrete time periods

$O(k)$ = overflow to receiving waters released from reservoir 5, during period k. $\underline{O} = (O(1),\ldots,O(M))$

$Q_j(k)$ = discharge from detention reservoir j during period k. Discharge $Q_5(k)$ goes directly into the interceptor. $\underline{Q} = (Q_j(k),\ j=1,\ldots,5;\ k=1,\ldots,M)$

Q_{jmax} = maximum discharge rates immediately downstream of reservoir j (given)

$S_j(k)$ = storage in detention reservoir j (associated with subcatchment j) at the beginning of period k. The reservoir is initially assumed to be empty, or $S_j(0) = 0$. $\underline{S} = (S_j(k)$, $j=1,\ldots,5;\ k=2,\ldots,M+1)$

S_{jmax} = maximum storage available in reservoir j. Though it is actually a design variable, it is considered as a variable in the operational problem. $\underline{S}_{max} = (S_{1max},\ldots,S_{5max})$

S_{total} = total storage allocated to Vicente Subbasin (given)

$\underline{x}$ = the vector of operational variables (e.g., reservoir releases and necessary overflows at discrete points in time over the duration of the design storm)

$\mathcal{X}(\underline{d})$ = the set representing constraints on reservoir operation, for a *given* design $\underline{d}$ (e.g., line and storage capacity constraints)

$\omega(k),\mu(k)$ = weighting factors (given)

CHAPTER VI

REFERENCES

1. Brandstetter, A., R. L. Engel, and D. B. Dearlock, "A Mathematical Model for Optimum Design and Control of Metropolitan Wastewater Management Systems," Battelle Pacific Northwest Laboratories, Richland, Washington, April, 1973.

2. Crawford, N. H., "Computer Simulation for Design Criteria for Urban Flow Storage Systems," Hydrocomp International, Palo Alto, California, 1973.

3. Flug, Marshall, "Optimal Design of Stormwater Detention Storage," M.S. Thesis, Department of Civil Engineering, Colorado State University, 1974.

4. Gilbert, J. B. and Associates, and Metcalf and Eddy, Inc., "Evaluation--San Francisco Wastewater Master Plan," March, 1973.

5. San Francisco Department of Public Works, "San Francisco Master Plan for Wastewater Management," 3 volumes, 1971 and 1973 Supplement.

6. Zangwill, W. I., Nonlinear Programming, A Unified Approach, Prentice-Hall, New Jersey, 1969.

CHAPTER VII

FEASIBILITY OF REAL-TIME PREDICTION OF URBAN RAINSTORMS

A. IMPORTANCE OF STORM PREDICTION IN REAL-TIME ON SYSTEM OPERATION

The approaches to design and control strategy discussed in Chapters IV and V have assumed prior knowledge of the depth and duration of any storm for which the system is to be designed or operated. Real-time operation requires this knowledge or at least some estimate of it. This means that any *optimal* control strategy developed without considering uncertainty in storm parameters is incomplete. Although knowledge of best possible system performance is useful, it is not self-sufficient for purposes of design and evaluation and, at some point in the development of the system, a method of storm prediction must be incorporated in the control prediction model. Real-time optimal control requires that estimates of interior and overall storm parameters be made. These estimates may be updated as the storm progresses in time as actual data becomes available and their uncertainty will thus decrease. However, the uncertainty will never reach zero until the particular event is over thus the best level of performance obtained is directly related to storm prediction capability.

A.1 An Example--Vicente Subbasin

As an example of the effect of storm uncertainty, the zero overflow curve concept described in Chapter IV can be used. Specifically, the effect of an error in estimating storm duration on the most favorable control level overflow curve of Figure IV-6 can be examined. This curve was developed by varying the control level as a function of storm duration as shown in Figure IV-8. By assuming various errors in storm duration and using these incorrect durations to determine the control level from Figure IV-8, new overflow curves can be constructed with duration error as a parameter. Figure VII-1 shows the results of this effort for both overestimation (+ duration error) and underestimation (- duration error) of duration. The main objective of presenting this figure is to demonstrate that such errors can have a significant effect on system performance. The first observation of interest in this figure is that the effect of overestimating the duration is small compared to that of underestimating. Overestimation errors of 0.5, 1.0 and 2.0 hours all produced the same overflow curve which departs from the zero error curve only for durations less than about 0.25 hours. The curves for underestimated durations, however, show larger departures, particularly if the assumed duration was 15 minutes or less in which case the overflow depth approaches that resulting from a *no control* strategy. One could conclude from Figure VII-1 that it is much safer to overestimate the duration than to underestimate it for the particular control strategy of Figure IV-8.

B. ANALYTICAL APPROACH--MATHEMATICAL MODELS

The literature concerning prediction of time and space of storm rainfall, has shown an evolution in both the statistical approaches to this problem and the basic physical understanding of the phenomena. The combination of these trends has resulted in a rather complete stochastic modeling of the three major types of storms which exhibit the quickly varying properties which have been frustrating attempts to regulate the runoff from small urban basins.

Two major hurdles have stalled the purely statistical approaches to modeling small scale and short time increment rainfall. The first is

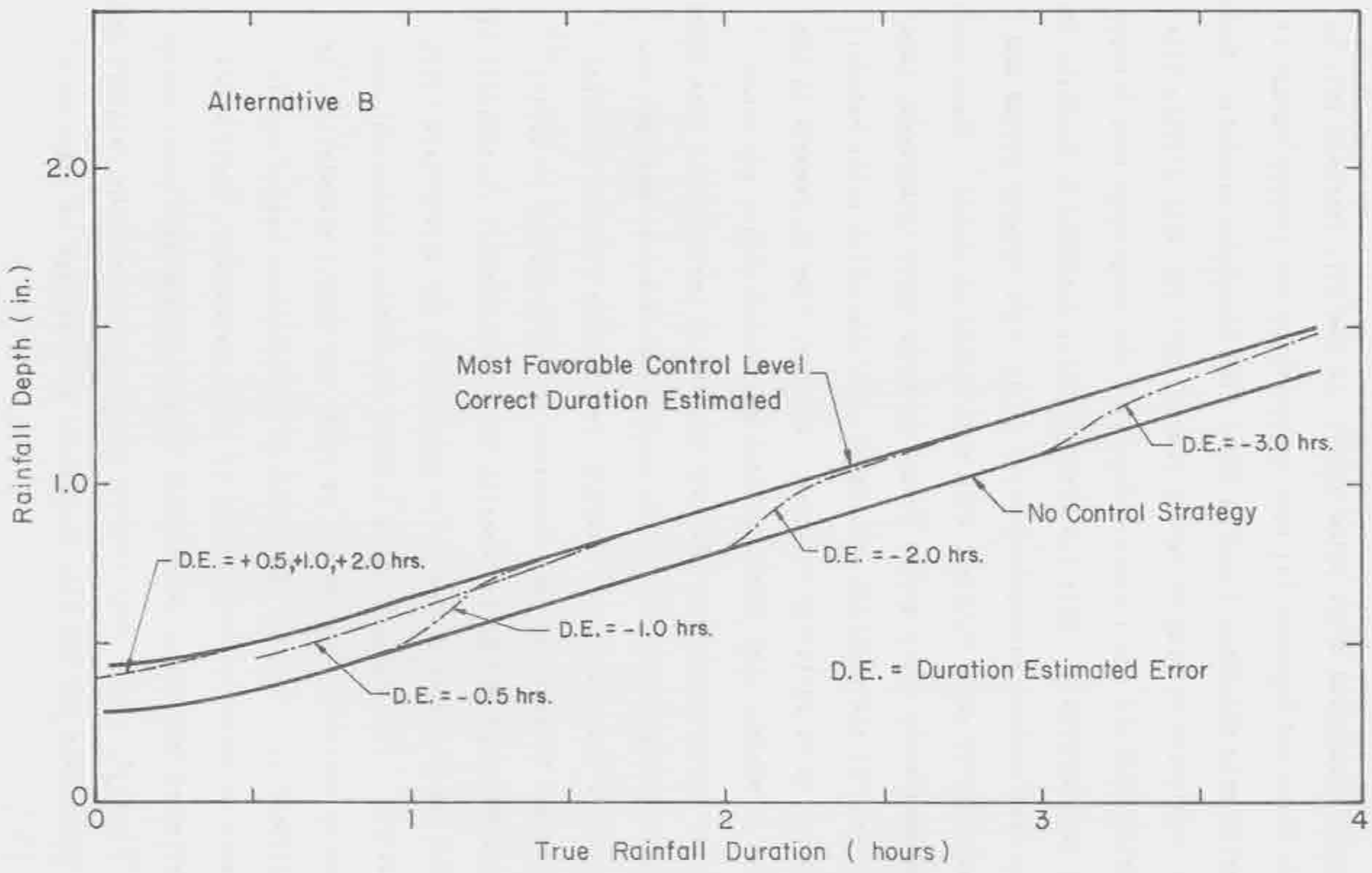


FIGURE VII-1

EFFECT OF DURATION ERROR ON MOST FAVORABLE CONTROL LEVEL OVERFLOW CURVES the dramatic change in the statistical properties which occurs if the time scale considered drops below hours. Le Cam [2], Pattison [6], as well as Grace and Eagleson [4] have demonstrated the complex nature of the persistence phenomena found in short time increment rainfall. Their models, basically relying on Markov chains, have run into difficulties simulating both the small-scale effects and the long-range time between events simultaneously. This has been partially overcome by breaking the problem into various *independent* parts (i.e., time between storms and the variation of point rainfall with time within an event). These parts, modeled separately using Markov chains or Monte Carlo techniques, have enabled many of the problems associated with simulating point rainfall variability to be partially overcome. However, time increments in the range of 10 minutes were found unsuited for Markov chains and other techniques were suggested to simulate the serial correlations (*Urn Models*).

The second major hurdle to the statistical modelers has been the simulation of the small-scale temporal variability of point rainfall. Wilkinson and Tavares [8] have described the difficulties of trying to use Markov-type models to describe the point rainfall variability of more than one point at a time while maintaining the appropriate cross correlations. They propose using a Monte Carlo-type simulation, constrained by suitable descriptions of cross and serial correlations of storm parameters. But, as the number of correlations needed equals the number of possible combinations of storm parameters, their model was limited to only three descriptors of a storm for each point chosen ($_3C_2 = 3$ but $_4C_2 = 6$). This limited description of a point rainfall pattern worked adequately for the time increments and spacings of gages on a river reservoir network with three gages but it is doubtful if it would suffice for the more densely spaced gages of an urban raingage network with short increments of time.

Within the past ten years, various researchers have begun to incorporate the growing body of knowledge concerning the physical makeup of rainstorm activity. Building on the work of Byers and Braham [1] a variety of researchers have identified a multitude of statistical descriptors for the variety of distributed rain cell parameters. Many of these endeavors have proved worthwhile for incorporation into models. The rain cell, the basic source of erratic rainfall patterns, can be simulated via its orientation, size, growth and decay cycle, and its relatively regular internal distribution of intensity along its axes. This type of rainfall simulation was attempted by two teams of researchers whose work, fortunately, is complementary.

Sorman and Wallace [7] using a model based on eight statistical-based descriptors of rain cell activity have created a model in which cells are generated, grow, decay, move (relative to the wind) and contribute definable distributed rain intensities. They use coordinate frames which move with each major cell (sequentially) and another, stationary, frame which is used to relate the meteorological activity to the stationary raingages on the ground. This model which has adequately simulated thunderstorm's internal spatial and temporal variability uses complete distributions of the values associated with the relevant parameters. Grayman and Eagleson [5] have adapted a slightly compromised approach to the complete distributions of cell parameter values. In designing a model consisting of different distinct discrete levels of activity, each represented by squares of unit size nested within the larger squares of encompassing levels, they have overcome some of the limitations inherent in Sorman and Wallace's model. Having single valued ratios to describe the intensity of activities in those particular squares which are activated by probabilistic switches, their model (though simplified) is capable of simulating the more extensive nature of fronts and squall lines which were not considered by Sorman and Wallace. Their model, also using a moving frame of reference, (moving with the entire storm rather than a particular cell) has the various levels of activities *switched* on and off within the constraints of the distributions and correlations thereby representing the passage of a cell (for example) as a square wave with internal uniform rainfall distribution.

Both of these simulation models appear to have application to some form of model which, given initial conditions, could predict the most probable outcome, but as they exist it would take major revision to accomplish this.

A bibliography of mathematical modeling of rainfall appears in Appendix A.

C. EXPERIMENTAL APPROACH--WEATHER RADAR

The primary function of a raingage network in the urban wastewater control system is to provide real-time data on rainfall that has occurred. This information may have some value in storm prediction, but the value is limited because of the practical limitations on the spatial extent and density of the network.

In order to overcome the deficiencies of raingage networks in predicting the intensities and volumes of precipitation delivered to the ground from showers, thunderstorms and other types of storms, researchers have developed weather radar systems to augment or replace raingage networks.

C.1 Basic Concepts

In a weather radar system, a pulse of electromagnetic energy is directed out in a conical beam from the radar antenna. Raindrops in the path of the beam are energized according to the properties of the raindrops and the electromagnetic intensity of the beam. The raindrops reradiate the energy into space. A portion of the reradiated energy is collected by the radar antenna. The signal from the antenna is amplified, processed and displayed, and stored for future use.

The electromagnetic energy reradiated from precipitation falling through a radar beam is proportional to the size and number of raindrops. Because the terminal velocities of raindrops are proportional to the size of the drop the reradiated energy is proportional to the rainfall intensity.

By adjusting the received signal strength to reflect the loss of electromagnetic intensity with distance between the storm and the radar, the signal received back from a storm can be related directly to the rainfall intensity.

The location of a rainstorm is determined by measuring the time taken for the radar signal to travel to storm and back (the energy travels at the speed of light) and by noting the direction in which the radar antenna is pointing. The rainstorms within the radar range can be located by rotating the antenna 360°.

The radar signal from a rainstorm is called an echo. The echo can be displayed as the planview appearance of the storm at ground level. An example of this kind of display is shown in Figure VII-2. In the figure, the location of the weather radar is represented by the center of the circles.

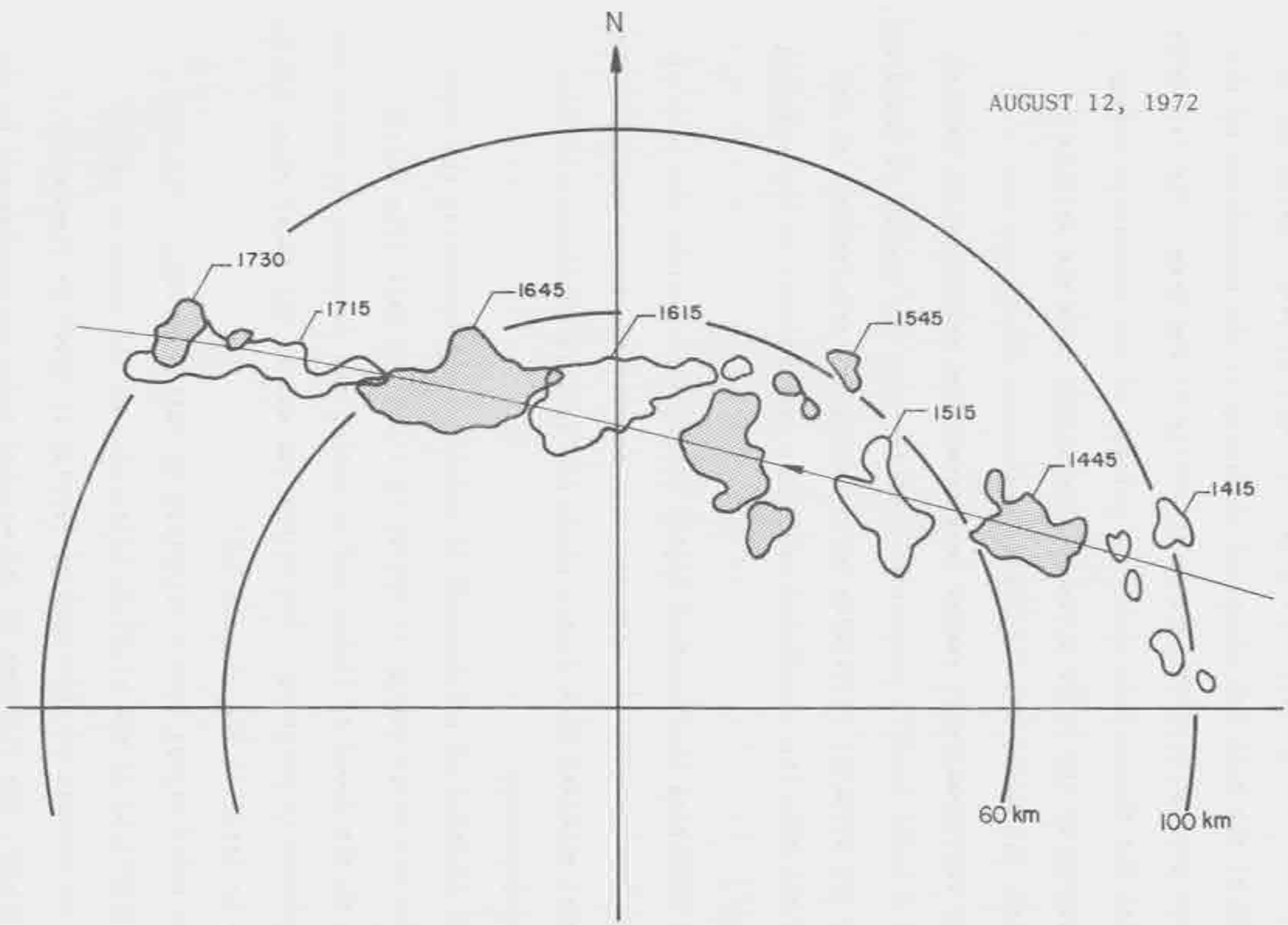


FIGURE VII-2

RADAR ECHOS - CARRIZAL, VENEZUELA

At 1400 hours on August 12, 1972, the radar scope was clear. There were no echos visible. At 1415, the five small echos appeared on the scope in the eastern region at a range of approximately 90 km. One-half hour later the five small echos had coalesced into one echo and had moved toward the west. One hour after the storm had appeared on the radar scope, its direction and speed of motion could be estimated.

The radar echo traces shown in Figure VII-2 illustrate that the weather radar can locate the regions of precipitation accurately and on a real-time basis. Echo images can be obtained once per antenna rotation if this is considered necessary. With some prior knowledge of storm track behavior, a good prediction of the track for the life of the storm can be made soon after the echos have appeared on the scope.

The outline of the echo represents a very low intensity rainfall. The value of this threshold intensity is related to the radar system's minimum detectable signal and on the distance between the radar and the storm. The minimum detectable signal is the smallest level of incoming electromagnetic energy that can be identified above the noise level in the radar system. The minimum detectable signal for a good weather radar would correspond to a rainfall of approximately 0.1 mm/hr at a range of 50 km.

The outline of the ground surface covered by the radar echos in their course of travel is called the storm composite. A storm composite for the Carrizal, Venezuela region on the afternoon and evening of August 8, 1972, is shown in Figure VII-3. The composite contained tracks of many storms which occurred that afternoon and evening.

The composite revealed the nonhomogeneous distribution of rainfall over the region. Thirteen of the 40 raingages in the regional network

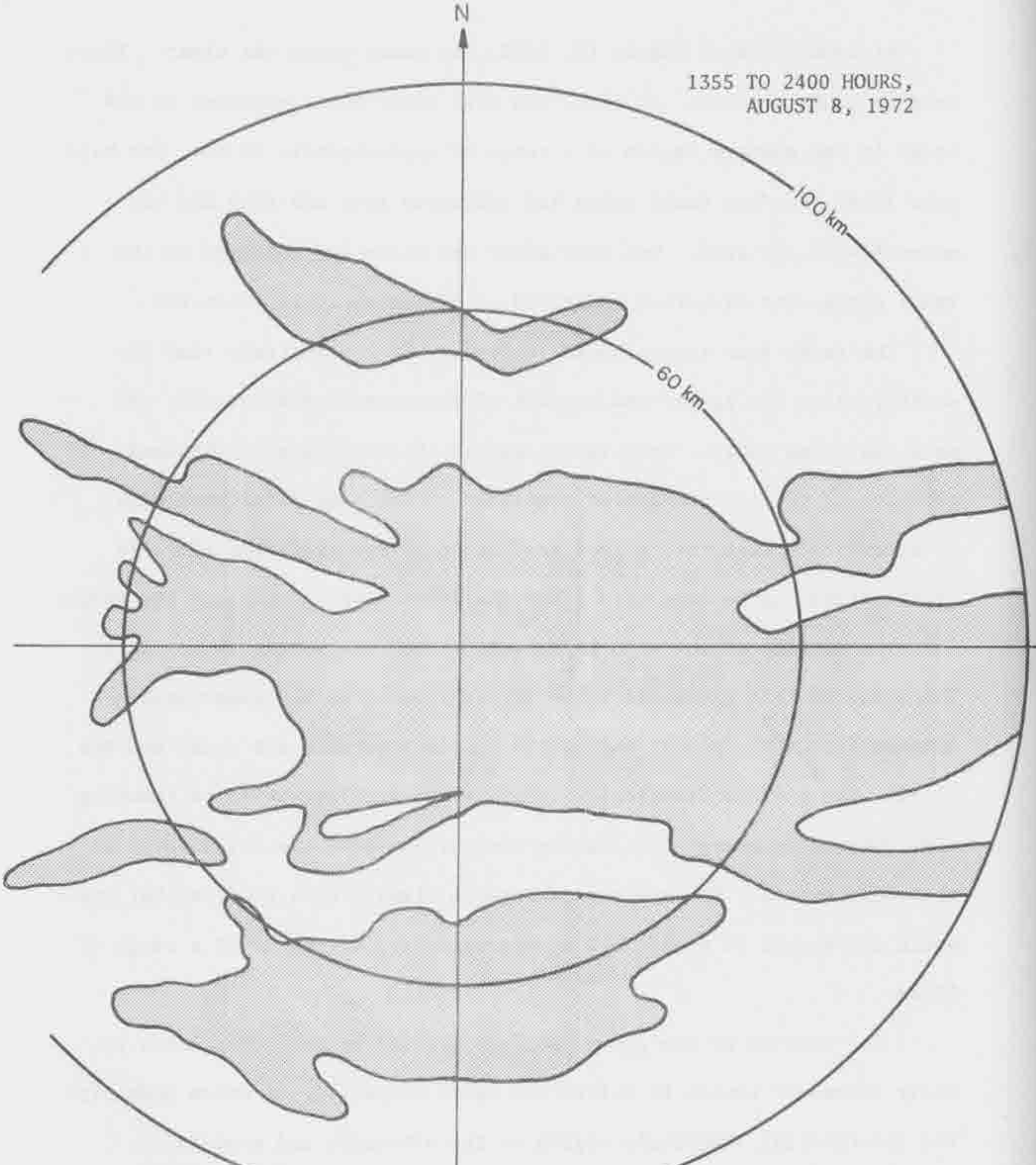


FIGURE VII-3

RADAR ECHO COMPOSITE - CARRIZAL, VENEZUELA inside the 60 km radius from the radar received no rain and eight of the 22 raingages in the line network were dry.

The range of weather radar can be varied. That is, the outer edge of the radar scope representing the 100 km radius circle in Figure VII-3 can be changed to represent 150 km or 50 km by turning a switch. The region of interest in the Venezuelan hydrometeorological experiment was the 60 km radius circle but storms were tracked into this region from as far away as 150 km by using the long range capability of the weather radar.

C.2 Intensities with Radar

The radar senses the number and size of the raindrops in the volume of space sampled by the electromagnetic beam. If it is assumed that the raindrops are falling at their terminal velocities, then the radar signal is proportional to the rainfall intensity.

The minimum volume of space sampled by a radar beam is illustrated in Figure VII-4. The radar beam is described by the horizontal beam width angle and the vertical beam width angle Θ . At all ranges R , the length of the volume sample along the beam axis is r . The value of r is one-half the product of the speed of light c and the electromagnetic pulse width τ or

$$r = \frac{1}{2} c\tau$$

The value of c is approximately 300,000,000 m/sec and for the Venezuelan radar, τ was 1 ms. Therefore, r was 150 km.

For horizontal and vertical beam width of 2°, the volume sampled at a range of 100 km (R=100 km) is approximately 1.5 cu km. The sample area normal to the direction of rainfall is approximately 0.5 sq km.

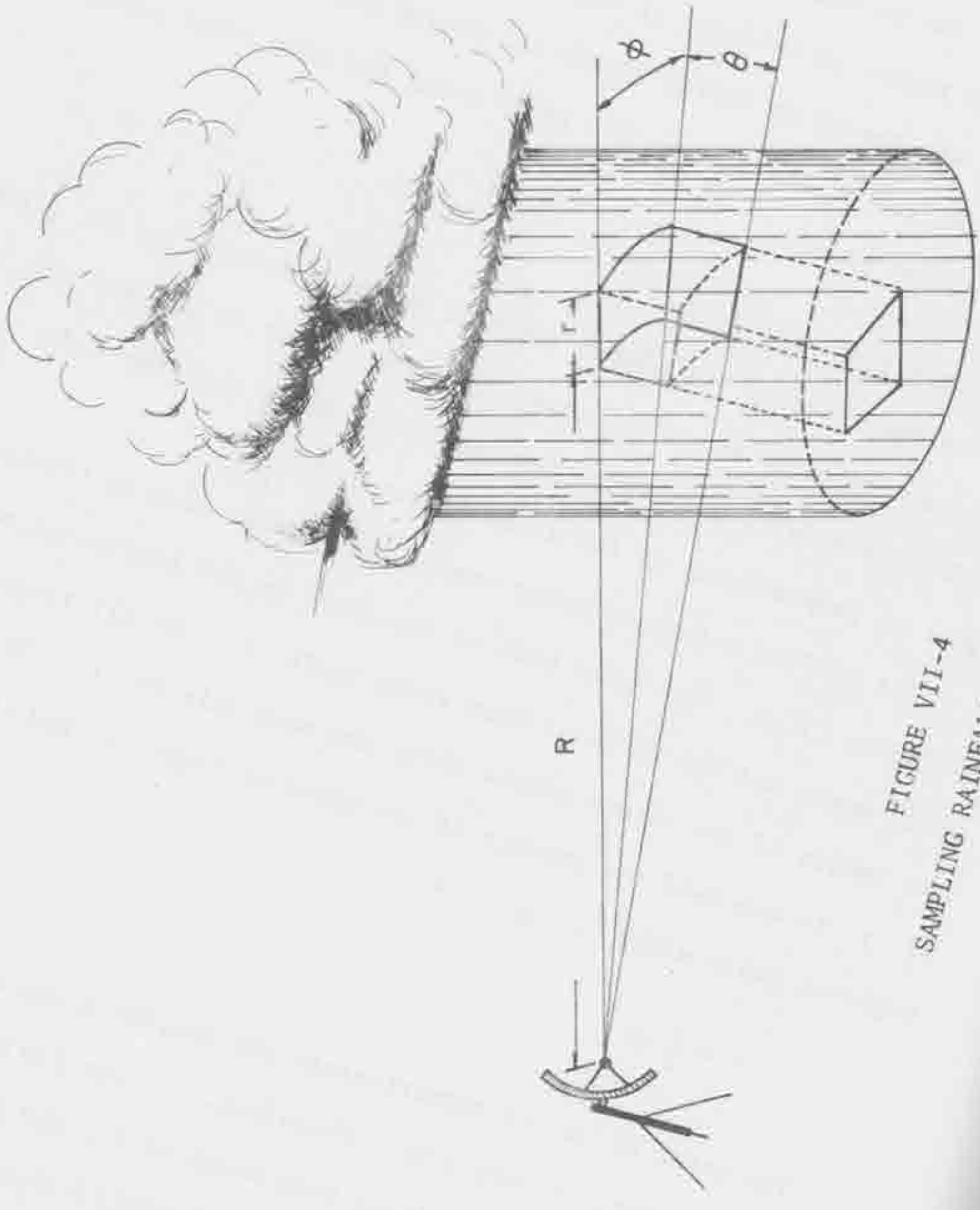


FIGURE VII-4
SAMPLING RAINFALL WITH RADAR

Thus the minimum area sample that can be obtained by radar is orders of magnitude greater than the area sampled by a single raingage.

The variation of signal strength (and rainfall intensity) with position can be obtained by recording the signal from various ranges through the storm. The intensity structure of a large storm is shown in Figure VII-5. The areas shaded dark were areas of intense rainfall (approximately 60 mm/hr). The lighter shaded areas had less intense rainfall.

The intensity pattern shown in Figure VII-5 illustrates the complex structure of this large storm at a moment in time. At 0700 hours on September 2 the storm consisted of a small intense precipitation core near the western (leading) edge, another very small but intense core further north and a large low-intensity area.

If the storm is intensity contoured as illustrated in Figure VII-5 and tracked as illustrated in Figure VII-2, the entire rainfall event is then mapped. The time variation of rainfall intensity for all 0.5 sq km regions within the range of the radar can be computed. If the radar signal is processed by an on-line computer, the complete real-time rainfall information for the region can be obtained.

The calibration of the radar with respect to rainfall intensity is one of the most difficult problems associated with weather radar. In the past, the calibration was done in two phases. The properties of the radar system were determined by electronic calibration and by using a radar target with known electromagnetic transmission properties. After the radar electronics were calibrated, the radar signal strengths from storms were compared with rainfall intensities measured by raingages. The radar versus raingage comparison has never been good.

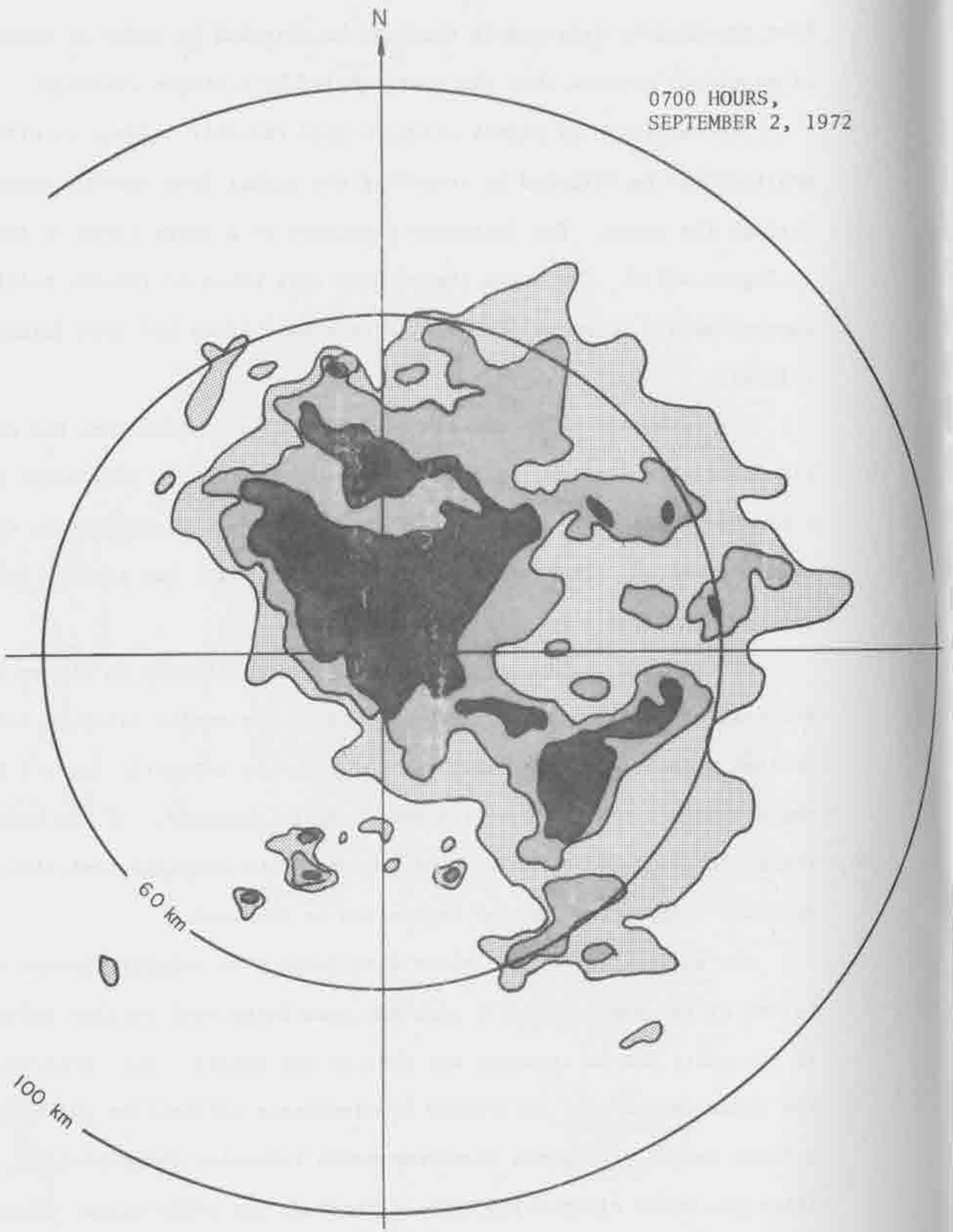


FIGURE VII-5

AREA RAINFALL INTENSITIES

In many cases the comparisons were not possible because of the poor time resolution of the raingage observation (for example, Woodley and Herndon, [10]). Even with good time resolution in the raingage record, a good radar versus rainfall correlation cannot be expected because the two systems are not sensing the same sample. The principle problem is that the radar samples as 1.5 cu km volume of raindrops over an area of 0.5 sq km whereas one raingage sampled an area of 3×10^{-8} sq km.

If weather radars are placed in urban areas, the opportunity exists to perform much more accurate calibration of radar intensities. This accurate calibration could be obtained by employing some impervious area of the city as an equivalent to a large raingage sampling an area of the same size as the radar beam. The calibration would be difficult and expensive to perform but only one calibration is needed.

C.3 Some Advantages

The weather radar meets all requirements for an urban rainfall data gathering and processing system. The weather radar samples the entire urban drainage area on a real-time basis. The weather radar can pick up and track storms moving into the drainage basin and can be used to predict storm track and travel times over the drainage basin accurately. In comparison with raingage networks, the weather radar is at least one generation advanced.

D. RAINGAGE NETWORKS

An adequate knowledge of the real-time spatial distribution of rainfall is a critical requirement in the design and operation of storm drainage systems in urban regions. A good urban rainfall data gathering system consists of these features:

1. Devices for sensing rainfall intensity must be accurate
2. The sampling time for rainfall intensities must be adequate. Five minute average samples would be sufficient.
3. The intensity samples must be collected at enough points in the drainage area to adequately define the spatial variation of intensity at any time.
4. The real-time production of rainfall information is necessary.
5. A reliable system for transmitting the rainfall data is required.
6. The capability of providing sufficient data for use in predicting rainfall in different regions of the drainage system is desirable.

The automated raingage system in San Francisco [3] is one of the most advanced systems being employed to collect urban rainfall data. This system consists of 30 remote raingages distributed over the San Francisco area. The raingages are connected by leased telephone lines to a minicomputer which logs the rainfall data. The software for the minicomputer includes a timer routine and a routine for calculating the five-minute intervals of maximum rainfall intensity for each hour of the day.

The San Francisco rainfall sensing system fulfills most of the requirements for a good rainfall system listed above. However, the density of the network may be insufficient to adequately define the spatial variation in intensity within the city and the limitation on overall extent of the system may reduce its value in predicting rainfall.

The spatial uniformity of rainfall intensity and the size of the storm determine what an adequate spatial sample is. For example, a raingage is an adequate sample of the rainfall into an evaporation pan located adjacent to the raingage. Each storm passing over the raingage almost certainly passes over the evaporation pan.

If the rainfall intensity is uniform over a wide area, then a single raingage is an adequate spatial sample. Usually low-intensity rainfall is more uniformly distributed than high-intensity rainfall. Low-intensity rainfall is produced by large frontal systems. Other types of weather systems produce extreme gradients in precipitation intensities.

D.1 Example Using Venezuela Network

Time and space variations in rainfall intensity are illustrated in Figure VII-6. The intensities shown in Figure VII-6 are station rainfall intensities obtained with the raingage and event recorder which gave the time to the nearest one-tenth second for 0.01 in. of rain to collect in the raingage. The storm was a squall line moving over the central plains of Venezuela at a speed of 18 m/sec. The storm required 35 minutes to pass over the raingage and produced 10.0 mm of rain in the raingage. The maximum recorded intensity was 70 mm/hr; that is, 0.1 in. of rain fell in 13.0 seconds.

If it is assumed that the squall line was stationary with respect to time for 35 minutes at least, then the time axis in Figure VII-6 can be replaced by a space axis with a length of 37.8 km corresponding to the time scale of 35 minutes. Then the intensities in Figure VII-6 represent the intensities over 37.8 km of space in the direction the storm was

JULY 24, 1972

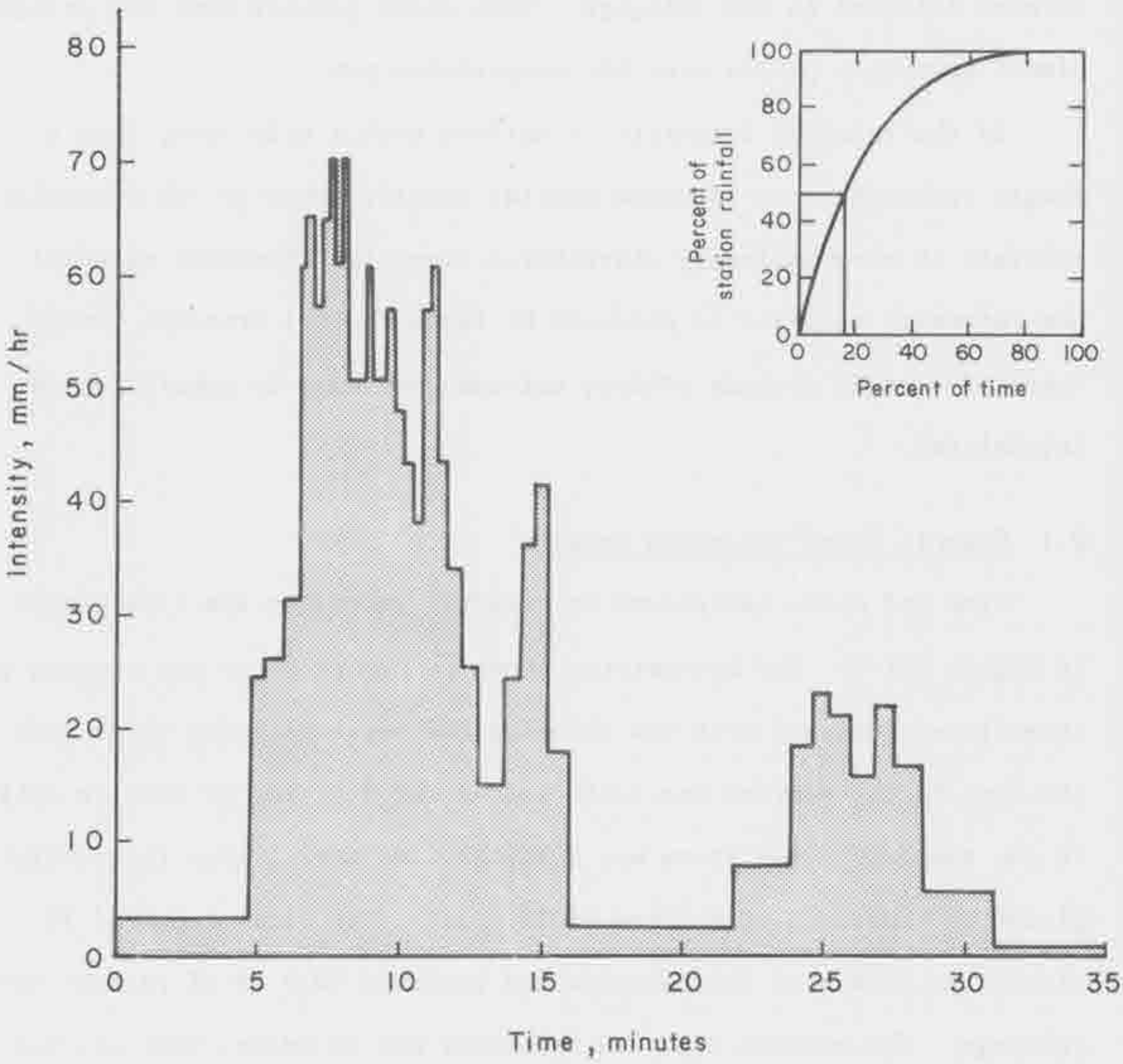


FIGURE VII-6

RAINFALL INTENSITIES - CARRIZAL, VENEZUELA moving. We see that the storm consisted of a major cell of high intensity precipitation approximately 8 km long and a smaller cell approximately 4 km long following behind.

At the raingage, 50 percent of the rain from the passing squall line occurred in 16 percent of the time and 90 percent fell in 50 percent of the time. That is, most of the rainfall from this storm fell in a very short period of time.

The changing of the time record at a raingage to a space record in the direction of storm movement can be accomplished only if the storm velocity is known. Knowledge of the storm structure is required if the raingage record at a station is to be used to infer rainfall amounts normal to the direction of storm movement.

Three sets of daily records from 22 raingages each spaced approximately 5 km apart and aligned in a row nearly normal to the direction of storm travel are shown in Figure VII-7. This row of raingages was a part of the regional network designed for a hydrometeorological experiment conducted in Venezuela in 1972. The location of the raingage network is shown in Figure VII-8. The row of 22 gages is the north-south row through the center of the 60 km radius circle.

On September 1, 1972, a large storm developed over the study area and moved slowly normal to the raingages. The storm grew to a maximum size of more than 11,000 sq km and then dissipated. The arithmatic average of the rainfall collected in the 22 gages for the storm was 69.6 mm. The storm rainfall at each of the 22 raingages is shown in Figure VII-7a. The bars represent rainfall amounts and are spaced according to position of the raingages on the ground. The most northerly raingages are on the left hand side of the figure and the most southerly raingages are on the right hand side.

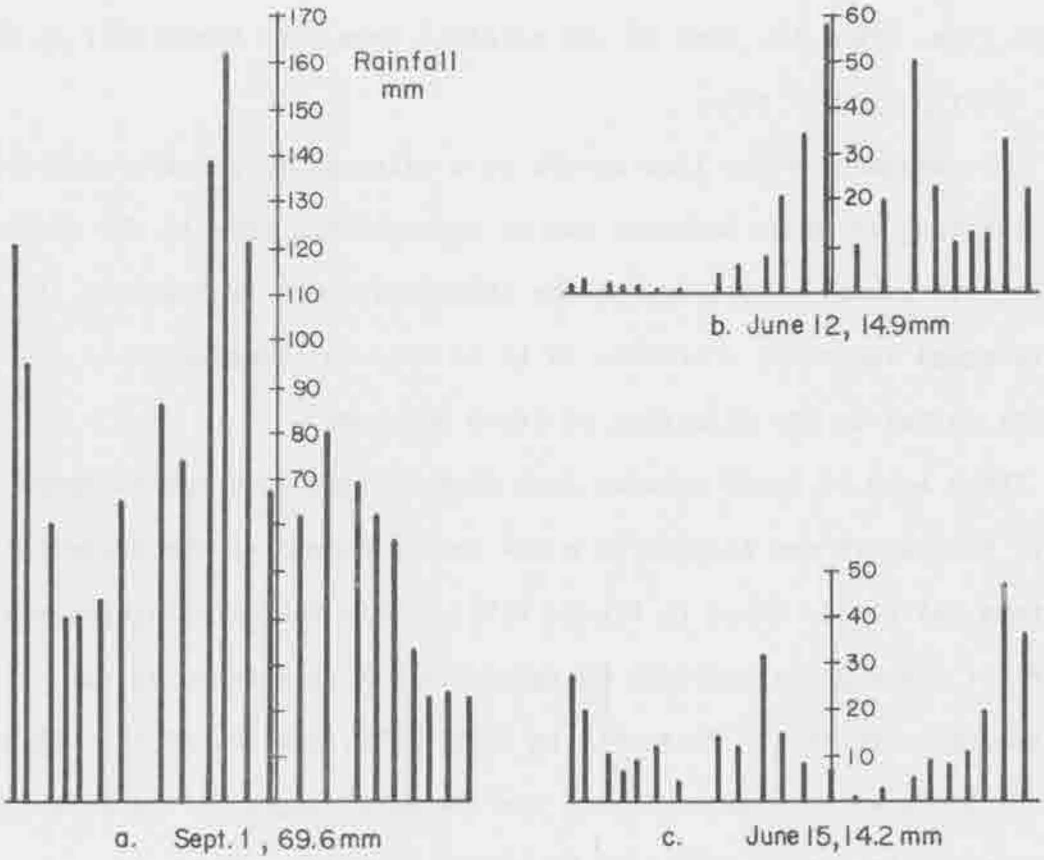
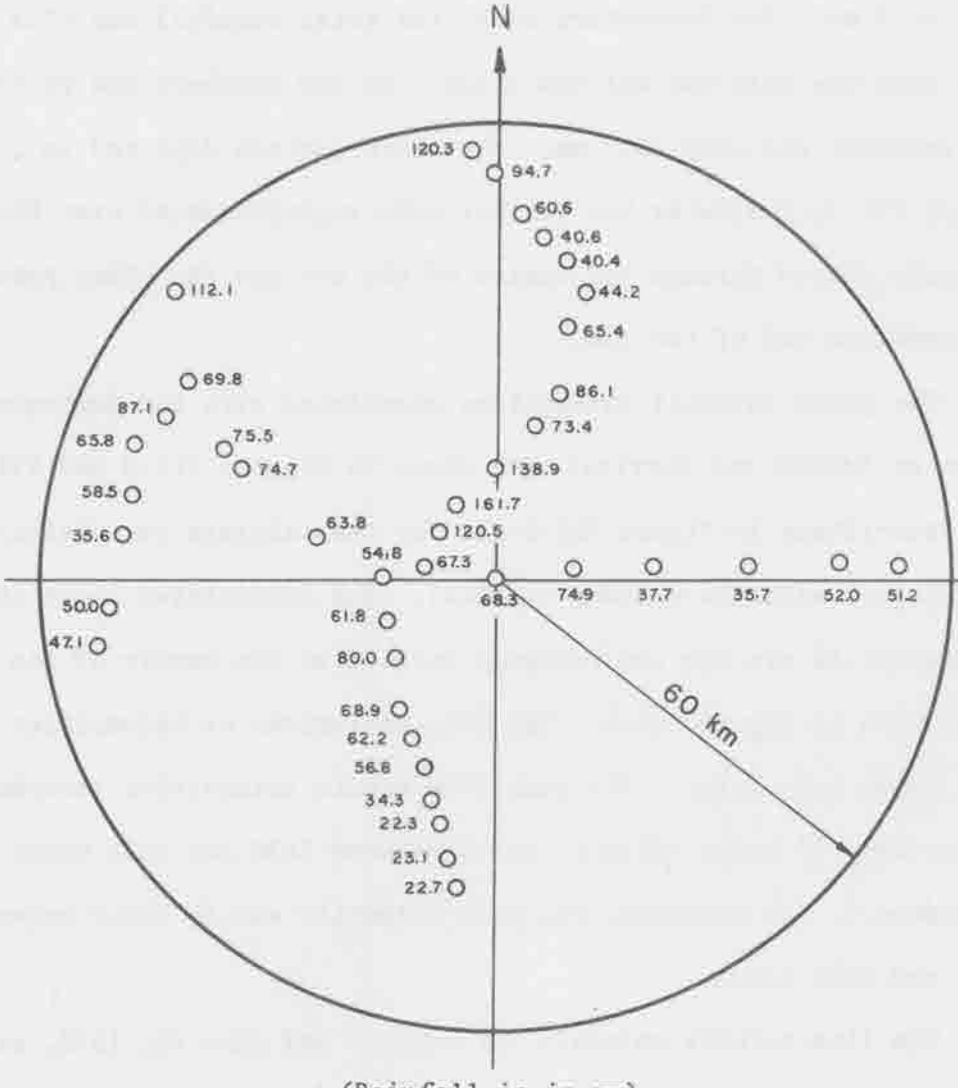


FIGURE VII-7

LINE NETWORK STATION RAINFALL



(Rainfall is in mm)

FIGURE VII-8

RAINGAGE NETWORK SHOWING RAINFALL BETWEEN
0800 HOURS SEPTEMBER 1 AND 1400 HOURS SEPTEMBER 2, 1972

The variation of the point rainfall along the north-south cross-section of the storm was large. The maximum storm rainfall recorded was 161.7 mm. Ten kilometers away, the total rainfall was 67.3 mm, less than one-half the maximum value. On the southern end of the row, the rainfall was only 22.7 mm. The storm pattern depicted in Figure VII-7a indicates two central core regions passed over the row. One core passed through the center of the row and the other passed over the northern end of the row.

The point rainfall intensities associated with the September 1 storm at Oscuro and Carrizal are shown in Figures VII-9 and VII-10. The intensities in Figure VII-8 are for the raingage immediately north of the gage with the maximum rainfall. The intensities shown in Figure VII-10 are for the raingage located at the center of the study area shown as Figure VII-8. The time variations of intensities at both gages were large. The peak five-minute intensities recorded at Oscuro was 120 mm/hr which occurred between 0630 and 0635 hours on September 2. At Carrizal, the peak intensity was 85 mm/hr between 0800 and 0805 hours.

The line network rainfall for June 12 and June 15, 1972, are also shown in Figure VII-7. The line averages on these two days were 14.9 mm and 14.2 mm--nearly the same. However, the lateral distribution of rainfall was different. On June 12, the southern one-half of the line network received nearly all the rain, whereas on the 15th, the rain was distributed more equally over the line. Still on June 15 the lateral precipitation gradients in daily precipitation were large.

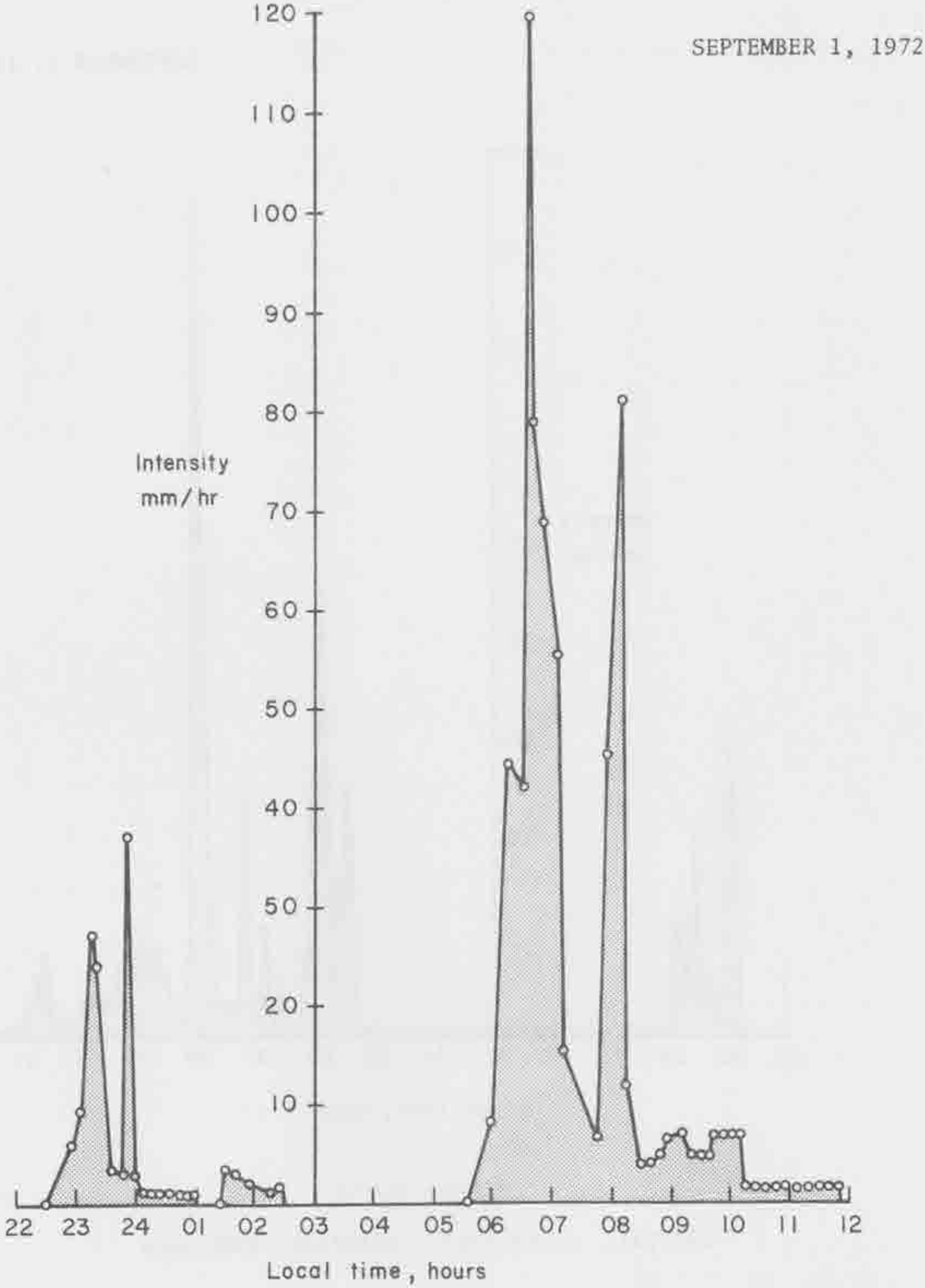


FIGURE VII-9

RAINFALL INTENSITIES - OSCURO, VENEZUELA

SEPTEMBER 1, 1972

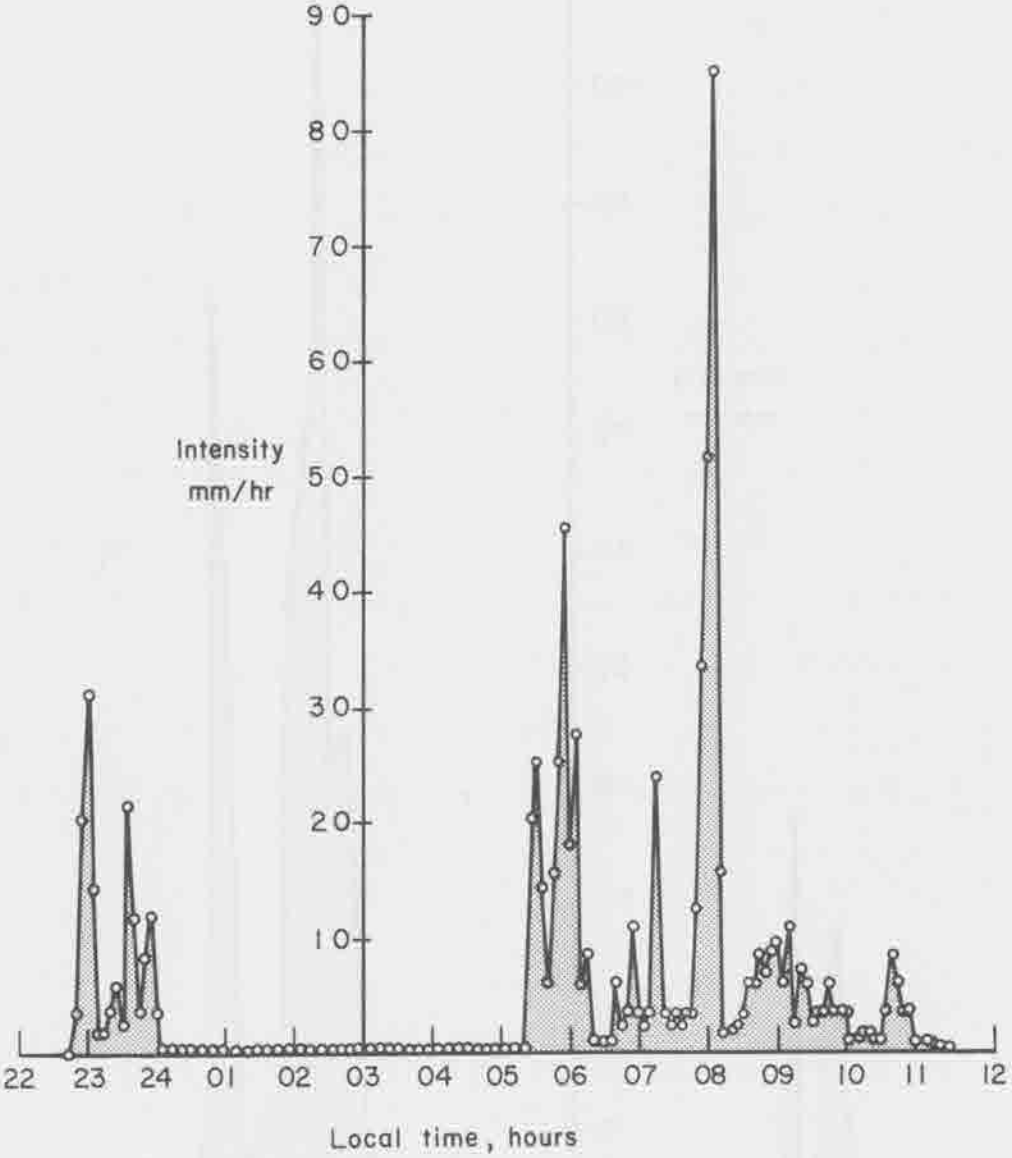


FIGURE VII-10

RAINFALL INTENSITIES - CARRIZAL, VENEZUELA

D.2 Some Limitations

Overall raingage networks have severe limitations in sampling rainfall over a drainage area. Most recording raingages fail to record the very high intensity rainfall rates accurately. Even the most dense raingage networks yield a poor spatial sample of the intensities that occur at any time. Studies of Florida showers and thunderstorms by Woodley, Norwood and Sancho [9] show that on the average only about 10 percent of the area precipitation (the inner-most intense core) accounts for approximately 50 percent of the total rainfall. They noted

> "...that the minimum mean distance between the precipitation core and the edge of the shower was only 2.8 nautical miles--a very small space compared to the distance between raingages in southern Florida. As a consequence, it is physically impossible to make volumetric precipitation estimates from isolated showers using raingages."

In the San Francisco storm drainage area of 28,500 acres, the catch area of the 30 raingages is approximately 9,000 sq. cm. The ratio of catch area to the total area is approximately 1 to 100 billion. The even distribution of these raingages over an area results in a network which samples on the average, only the low intensity region of convective-type storms. The probability is that the high-intensity core of a convective storm will not pass over a raingage.

E. NEED TO EVALUATE EFFECTS OF STORM PREDICTION UNCERTAINTY ON SYSTEM PERFORMANCE

Because of the additional difficulties in optimal control development presented by storm uncertainty a logical approach is to assume zero uncertainty in the initial stages of the process. Once a model or procedure is developed to generage optimum strategy, a sensitivity analysis can be performed in which the effect of various degrees of storm uncertainty on system performance is evaluated. The specific manner in which the uncertainty is specified may depend on the nature of the strategy prediction model or method. Various depth-duration probability distributions or specific errors in depth and/or duration could be assumed and their effect on system performance observed. With this information, a quantitative judgment can be made as to the degree of importance and nature of the storm prediction model to be developed for inclusion in the final control software package. This model should be optimized in the sense that it should be designed for the specific purpose of providing necessary input to the control model rather than being the best general purpose storm prediction model.

F. CONCLUDING COMMENTS

It can be safely concluded that the entire question of storm prediction for application to real-time urban wastewater system control is a topic for much-needed research. As discussed in the previous section, it is first necessary to determine the detail and accuracy required. When this is accomplished, the prediction method can be developed with these requirements as objectives using perhaps the advantages of both the analytical and experimental approach. The use of long-range radar data as input to a mathematical prediction model with continuous update capability could result in predictions with much lower uncertainty than present methods.

The use of weather radar as alternatives to urban raingage networks should be seriously considered. Raingage networks have a number of built-in disadvantages as pointed out earlier.

Real-time continuous time and space prediction of urban storms may have many spillover benefits, as well. For example, police patrols could be deployed in real-time as determined from an accident simulation model, with rainfall as an input. In our emerging cybernetic society, there will be many uses for reliable, real-time weather information.

CHAPTER VII

REFERENCES

1. Byers, H. R., and R. R. Braham, Jr., "The Thunderstorm Structure and Circulation," J. of Meteorology, Vol. 5, No. 3, 1948, pp. 71-86.

2. Cam, L. Le, "A Stochastic Description of Precipitation," Proceedings of the 4th Berkeley Symposium on Mathematics, Statistics and Probability, Vol. III, June 20-July 30, 1960, University of Califronia, pp. 165.

3. Giessner, W. R., F. H. Moss, and R. T. Cockburn, "Minicomputer Points the Way to Sewer-System Improvements," Electronics, May 24, 1973, pp. 114-118.

4. Grace, R. A. and P. S. Eagleson, "The Synthesis of Short-Time-Increment Rainfall Sequences," Report No. 91, Hydrodynamics Laboratory, MIT, Cambridge, MA, May, 1966.

5. Grayman, W. M., and P. S. Eagleson, "Streamflow Record Length for Modelling Catchment Dynamics," Hydrodynamics Lab Rep. No. 114, MIT, Cambridge, MA, 1969.

6. Pattison, A., "Synthesis of Hourly Rainfall Data," Water Resources Research, Vol. 1, No. 4, 1966, pp. 489-498.

7. Sorman, U. A. and J. R. Wallace, "Digital Simulation of Thunderstorm Rainfall," Environmental Resources Center, Georgia Institute of Technology, Atlanta, GA, August, 1972.

8. Wilkinson, J. C. and L. V. Tavares, "A Methodology for the Synthesis of Spatially Distributed Short Time Increment Storm Sequences," J. of Hydrology, 16(1972), 307-315.

9. Woodley, W. L., J. Norwood and B. Sancho, "Some Precipitation Aspects of Florida Showers and Thunderstorms," Weatherwise, Vol. 24, No. 3, June 1971.

10. Woodley, W. and A. Herndon, "A Raingage Evaluation of the Miami Reflectivity--Rainfall Rate Relation," J. of Applied Meteorology, Vol. 9, No. 2, April, 1970.

APPENDIX

BIBLIOGRAPHY ON URBAN RAINFALL VARIATION

Ackerman, W. C., "Systematic Study and Development of Long Range Programs of Urban Water Resources Research," A Study by the Am. Soc. Civil Eng., New York, 1968.

Allis, J. A., B. Harris, and A. L. Sharp, "A Comparison of Performance of Five Rain-Gage Installations," J. Geophysical Research, Vol. 68, No. 16, 1963, pp. 4723-4729.

Amorocho, J., "The Effects of Density of Recording Rain Gauge Network on the Description of Precip. Patterns," Paper presented at IUGC Meeting, Berne, 1967.

Amorocho, J., and J. R. Slack, "Simulation of Cyclonic Storm Field for Hydrologic Modeling," presented at the A. G. U. Annual Meeting, Washington, D. C. 1970.

Amorocho, J., and Morgan, D., "Convective Storm Field Simulation for Distributed Catchment Models," paper presented for International Symposium on Math. Models in Hydrology, July 26-31, 1971.

Austin, P. M., "Application of Weather Radar to Intensity of Surface Precipitation," Weather Radar Research, M. I. T., Cambridge, Mass., 1965.

Austin, P. M., "Application of Radar to Measurement of Surface Precipitation," Final Report, Department of Meteorology, M. I. T., Cambridge, Mass., 1969.

Battan, L. J., "Duration of Convective Radar Cloud Units," The Bulletin of Amer. Meteorological Soc., Vol. 34, 1953, pp. 227-228.

Battan, L. J., "Radar Meteorology," The University of Chicago Press, Chapter 10 and 11, 1959.

Benjamin, J. R., and Cornell, C. A., "Probability, Statistics, and Decision for Civil Engineers," McGraw-Hill Book Company, New York, N. Y., 1970.

Bernard, E. A., "Sur Les Erreurs de Divers Types de Pluriometres dans les Conditions Climatologiques du Congo Belge," Instit. Royal Colonial Belge, Bulletin des Seances, 25(2), 1954, pp. 896-912.

Bigler, S. G., R. G. McGrew, and M. St. Clair, "An Experiment in Digitizing Weather Radar Data From a Four Station Network," Presented at the 14th Radar Meteorology Conf., A. M. S., Tucson, Arizona, 1970.

Bhavnagri, V. S. and G. Buglarello, "Mathematical Representation of an Urban Flood Plain," J. of the Hydraulics Division, ASCE, Vol. 91, No. HY2, March 1965, pp. 149-173.

Biswas, A. K., "Development of Rain Gages," J. of the Irrigation and Drainage Division, ASCE, Vol. 93, No. IR3, Sept. 1967.

Blackmer, R. H., "The Lifetime of Small Precipitation Echoes," Proceedings of the 5th Weather Radar Conf., pp. 103-108.

Bleasdale, A., "The Measurement of Rainfall," Weather, Vol. 14, 1959, pp. 12-18.

Body, D. N., "The Design of Hydrometeorological Networks," Presented at the ECAFE - 7th Regional Conf. on Water Resources Development, Sept. 1966.

Braham, R. R., Jr., "The Water and Energy Budgets of the Thunderstorm and Their Relation to Thunderstorm Development," J. of Meteorology, Vol. 9, No. 4, 1952, pp. 227.

Brancato, G. N., "The Meteorological Behavior and Characteristics of Thunderstorms," U. S. Weather Bureau, Hydrometeorological Section, Washington, D. C., 1942.

Brooks, H. B., "A Summary of Some Radar Observations," The Bulletin of Amer. Meteorological Soc., Vol. 27, No. 10, 1946, pp. 557.

Browne, I. C., Palmer, H. P., and Wormell, T. W., "The Physics of Raincloud," Quar, J. of Royal Meteorological Soc., Vol. 80, No. 345, 1954.

Browning, K. A., and T. W. Harrold, "Air Motion and Precipitation Growth in a Wave Depression," Quar. J. of Royal Meteorological Soc., 95, 1969, pp. 288-309.

Byers, H. R., and colloborators, "The Use of Radar in Determining the Amount of Rain Falling Over a Small Area," Transactions of American Geophysical Union, Vol. 29, No. 2, 1948, pp. 187-196.

Byers, H. R., and R. R. Braham, Jr., "The Thunderstorm Structure and Circulation," J. of Meteorology, Vol. 5, No. 3, 1948, pp. 71-86.

Byers, H. R., and R. R. Braham, Jr., The Thunderstorm, U. S. Government Printing Office, 1949.

Byers, H. R., "Thunderstorms," Compendium of Meteorology, T. F. Malone, editor, 1951, pp. 681-693.

Cam, L. Le, "A Stochastic Description of Precipitation," Proceedings of the 4th Berkeley Symposium on Mathematics, Statistics and Probability, Vol. III, June 20 - July 30, 1960, University of California, pp. 165.

Chandrashekhar Aiya, S. V., Sonde, B. S., "Number of Cells Developed During the Life Time of a Thunderstorm," Nature, Vol. 200, No. 4906, Nov. 1963, pp. 562-563.

Changnor, S. A., Jr., and Wilaon, J. W., "Heavy Rain, Hail, and Tornadoes on 15 May 1968," Illinois State Water Survey, Urbana, Report of Investigation 66, 1971.

Charba, J., and Y. Sasaki, "Structure and Movement of Severe Thunderstorm of 3 April 1964, as revealed from Radar and Surface Mesonetwork Data Analysis," ERLTM-NSSL-41, 1968.

Chow, V. T., Handbook of Applied Hydrology, McGraw-Hill Book Company, New York, N. Y., 1964.

Clark, R. A., "A Study of Convective Precipitation as revealed by Radar Observation," J. of Meteorology, Vol. 17, pp. 415.

Cochran, H. B., "A Numerical Description of New England Squall Lines," S. M. Thesis, Dept. of Meteorology, M. I. T., Cambridge, Mass., 1961.

Conte, J. J., "An Evaluation of the Precipitation Capabilities of the WSR-57 Radar," Prog. Report No. 12, U. S. Weather Bureau, 1963.

Court, A., "Aread-Depth Rainfall Formulas," J. Geophysics, Res. 66:6, 1961, pp. 1823-1831.

Crawford, N. M. and R. K. Linsley, "Digital Simulation in Hydrology: Stanford Watershed Model IV," Dept. of Civil Engineering, Tech. Rep. No. 39, Stanford University, Stanford, California, July 1966.

Crovelli,PhD Dissertation, "Stochastic Models for Precipitation," Colorado State University, Department of Statistics, 1968.

Curry, R. G., R. A. Clark, R. C. Runnels, "Hydrograph Synthesis from Digitized Radar Data by Streamflow Routing," Presented at the 14th Radar Meteorology Conference, AMS, Tucson, Arizona, 1970.

Dawdy, D. R., H. E. Kubik, L. R. Beard, E. R. Close, "Worth of Streamflow Data for Project Design--A Case Study," Presented at the 49th Annual Meeting of the A. G. U., Washington, D. C., April 1968.

Day, H. J., "Flood Warning Benefit Evaluation--Susquehanna River Basin (Urban Residences)," ESSA Technical Memorandum, WVTM HYDRO 10, U. S. Dept. of Commerce, Environmental Science Services Administration, Silver Springs, Maryland, March 1970.

Drake, A. W., Fundamentals of Applied Probability Theory, McGraw-Hill Book Company, New York, N. Y., 1967.

Eagleson, P. S., R. Mejia and F. March, "Computation of Optimum Realizable Unit Hydrographs," Water Resources Research, Vol. 2, No. 4, 1966, pp. 755-764.

Eagleson, P. S., "Optimum Density of Rainfall Networks," Water Resources Research, Vol. 3, No. 4, 1967, pp. 1021-1033.

Eagleson, P. S., "Modeling Surface Runoff in Urban Hydrology," Memo to OWRR Methods of Analysis Committee, 1968.

Eagleson, P. S., Dynamic Hydrology, McGraw-Hill Book Company, New York, N. Y., 1970.

Eckstein, O., Water-Resource Development, Harvard University Press, Cambridge, Massachusetts, 1961.

Fankhauser, J. C., "On the Motion and Predictability of Convective Systems as Related to the Upper Winds in Case of Small Turning of Wind with Height," NSSP Report, No. 21, Washington, D. C., 1964.

Fiering, Myron B., "Streamflow Synthesis," Harvard University Press, Cambridge, Massachusetts, 1967.

Forrester, J. W., Principles of Systems, Wright-Allen Press, Cambridge, Massachusetts, 1968.

Franz, D. D., "Hourly Rainfall Synthesis for a Network of Stations," Dept. of Civil Engineering, Stanford University, Tech. Rep. No. 126, Stanford, California, 1970.

Freeny, A. E. and J. D. Gabbe, "A Statistical Description of Intense Rainfall," The Bell System Technical Journal, Vol. 48, No. 6, 1969, pp. 1789-1851.

Gandin, L. S. and R. L. Kagan, "The Economic Approach to the Planning of a Network of Meteorological Stations," Soviet Hydrology: Selected Papers, Issue No. 6, 1967.

Geotis, S. E., "Some Radar Measurements of Hailstorms," J. of Applied Meteorology, Vol. 2, 1963, pp. 270-275.

Goodspeed, M. S. and J. V. Savage, "Event Recorder-Technical Description," Tech. Mem. 66/2, Division of Land Research, CSIRO, Canberra, Australia, 1966.

Gordon, Geoffrey, "System Simulation," Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1969.

Grace, R. A., and P. S. Eagleson, "The Synthesis of Short-Time-Increment Rainfall Sequences," Report No. 91, Hydrodynamics Laboratory, M.I.T., May 1966.

Grace, R. A., and P. S. Eagleson, "A Model for Generating Synthetic Sequences of Short-Time-Interval Rainfall Depths," Proceedings International Hydrology Symposium, Fort Collins, Colorado, 1967, pp. 268-276.

Grant, E. L., Principles of Engineering Economy, The Ronald Press Co., N.Y., N.Y., 1950.

Grayman, W. M., and P. S. Eagleson, "Streamflow Record Length for Modelling Catchment Dynamics," Hydrodynamics Lab. Rep. No. 114, M.I.T., Cambridge, Massachusetts, 1969.

Grayman, W. M., and P. S. Eagleson, "A Review of the Accuracy of Radar and Raingages for Precipitation Measurement," Hydrodynamics Lab. Rep. No. 119, M.I.T., Cambridge, Massachusetts, 1970.

Grayman, W. M., and P. S. Eagleson, "Evaluation of Radar and Raingage Systems for Flood Forecasting," Report No. 138, R. M. Parsons Laboratory for Water Resources and Hydrodynamics, M.I.T., August 1971.

Gunn, R. and G. D. Kinzer, "The Terminal Velocity of Fall for Water Droplets in Stagnant Air," J. Meteorology, Vol. 6, 1949, pp. 243-248.

Handcock, D. E., "Routine Measurement of Precipitation," Presented at the Regional Seminar of Agrometeorology, W.M.O., Melbourne, Australia, 1966.

Harington, E. L., "Observations on the Appearance and Growth of Tropical Cumuli," J. of Meteorology, Vol. 15, 1959, pp. 127-130.

Harley, B. M., F. E. Perkins and P. S. Eagleson, "A Modular Distrubuted Model of Catchment Dynamics," Hydrodynamics Lab., Rep. No. 133, M.I.T., Cambridge, Massachusetts, December 1970.

Herschfield, D. M., "Agricultural Research Service Precipitation Facilities and Related Studies," Chapter 8, ARS 41-176, June 1971, pp. 21-24.

Hershfield, D. M., "On the Spacing of Raingages," I.A.S.H., Pub. No. 67, Symposium Design of Hydrological Networks, pp. 72-79.

Hilst, G. R., and G. P. MacDowell, "Radar Measurements of the Initial Growth of Thunderstorm Precipitation Cells," Bulletin of Am. Meteorological Soc., Vol. 31, No. 3, March 1950, pp. 95-99.

Hirshleifer, J., J. DeHaven and J. Milliman, Water Supply: Economics, Technology and Policy, University of Chicago Press, Chicago, Illinois, 1960.

Hopkins, C. D., Jr., and D. O. Hackett, "Average Antecedent Temperatures as a Factor in Predicting Runoff from Storm Rainfall," J. of Geophysical Research, Vol. 66, July-December, 1961, pp. 3313-3318.

Houze, R. A., "Characteristics of Mesoscale Precipitation Areas," S. M. Thesis, Dept. of Meteorology, M.I.T., Cambridge, Massachusetts, 1969.

Huff, F. A., and J. C. Neill, "Rainfall Relations on Small Areas in Illinois," Illinois State Water Survey, Bull. 44, Urbana, Illinois, 1957.

Huff, F. A., "The Adjustment of Radar Estimates of Storm Mean Rainfall with Raingage Data," Proceedings 12th Conf. on Radar Meteorology, A.M.S., Norman, Oklahoma, 1966, pp. 168-203.

Huff, F. A., "Time Distribution of Rainfall in Heavy Storms," Water Resources Research, 3(4), 1967, pp. 1007-1019.

Huff, F. A., "Spatial Distribution of Heavy Storm Rainfalls in Illinois," Water Resources Research, 4(1), 1968, pp. 47-54.

Huff, F. A., "Spatial Distribution of Rainfall Rates," Water Resources Research, Vol. 6, No. 1, 1970, pp. 254-260.

Hudson, H. E., Jr., G. E. Stout, and F. A. Huff, "Rainfall Studies Using Rain Gage Networks and Radar," Proceedings ASCE, September, 178, March 1953.

Imai, I., E. Suzuki, et al., "Statistical Properties of Rainfall and the Radiowave Attenuation Due to Rain," Journal of Meteorological Research (Kishocho Kenkyu Jiho), Tokyo, 10(3) 137-166, March 1964. English Summary, pp. 137-139.

Israelsen, C. E., "Reliability of Can-Type Precipitation Gage Measurements," Utah Water Research Laboratory, Utah State University, Tech. Rep. No. 2, Logan, Utah, 1967.

James, L. G. and R. R. Lee, Economics of Water Resources Planning, McGraw-Hill Book Co., New York, N.Y., 1971.

Jens, S. W. and D. E. Jones, "Water and Metropolitan Man," An Engineering Foundation Research Conference, co-sponsored by ASCE Urban Hydrology Research Council, 1969.

Jones, D. M. A., "The Correlation of Raingage - Network and Radar-Detected Rainfall," Proceedings 12th Conf. on Radar Meteorology, A.M.S., Norman, Oklahoma, 1966, pp. 204-207.

Kates, R., "Industrial Flood Losses: Damage Estimation in the Lehigh Valley," Dept. of Geography Research Paper No. 98, University of Chicago, Chicago, Illinois, 1965.

Kessler, E., and K. E. Wilk, "The Radar Measurement of Precipitation for Hydrological Purposes," Report No. 5, Reports on WMO/IHD, Secretariat of the World Meteorological Organization, Geneva, Switzerland, 1968.

Koschmeider, H., "Methods and Results of Definite Rain Measurements," Monthly Weather Review, 62, January 1934, pp. 5-7.

Leclerc, G. and J. C. Schaake, Jr., "Methodology for Assessing the Potential Input of Urban Development on Urban Runoff and the Relative Efficiency of Runoff Control Alternatives," Hydrodynamics Lab. Rep. No. 167, MIT, Cambridge, 1973.

Ligda, M. G. H., "The Horizontal Motion of Small Precipitation Areas Observed by Radar," Tech. Rep. 21, M.I.T., Cambridge, 1953, pp. 60.

Lighthill, M. J. and G. B. Whitham, "On Kinematic Waves - I. Flood Movement in Long Rivers," Proceedings Royal Society, London, Vol. 229, A., May 1955, pp. 281-316.

Maass, A., "Benefit-Cost Analysis: Its Relevance to Public Investment Decisions," Quar. J. of Economics, Vol. LXXX, May 1966.

Maass, A., et al., Design of Water-Resource Systems, Harvard University Press, Cambridge, Massachusetts, 1966.

Major, D. C., "Benefit-Cost Ratios for Projects in Multiple Objective Investment Programs," Water Resources Research, Vol. 5, No. 6, December 1969.

Marglin, S. A., Public Investment Criteria, M.I.T. Press, Cambridge, Massachusetts, 1967.

Marshall, J. S. and W. Hitschfeld, "The Interpretation of the Fluctuating Echo for Randomly Distributed Scatterers," Part. I, Can. J. Phys., Vol. 31, 1953, pp. 962-994.

Martin, F. F., "Computer Modeling and Simulation," John Wiley and Sons, Inc., 1968.

Mather, J. R., "An Investigation of the Dimensions of Precipitation Echos by Radar," The Bulletin of Am. Meteorological Soc., Vol. 30, No. 8, 1949, pp. 271-277.

McCallister, J., J. L. Teague, and C. E. Vicroy, "Operational Radar Rainfall Measurements," Proceedings 12th Conf. on Radar Meteorology, A.M.S., Norman, Oklahoma, 1966.

McGuinness, J. L., and G. W. Gaughan, "Seasonal Variation in Rain Gage Catch," Water Resources Research, Vol. 5, No. 5, October 1969, pp. 1142-1146.

McGuinness, J. L., "A Comparison of Lysimeter Catch and Raingage Catch," ARS, USDA, ARS. 4-124, October 1966.

McPherson, M. B., "A Basic Information Need in Urban Hydrology," A study made.by ASCE, sponsored by Geological Survey, April 1969.

Meinzer, O. E. (Ed.), Hydrology, McGraw-Hill Book Co., Inc., New York, N.Y., 1942.

Morgan, D. L., and F. J. Lourence, "Comparison Between Raingage and Lysimeter Measurements," Water Resources Research, Vol. 5, No. 3, June 1969, pp. 724-728.

Muchnik, V. M., M. L. Markovich, and L. M. Volynetz, "On the Evaluation of Accuracy of the Ukrainian Hydrometeorological Research Institute," Proceedings 12th Conf. on Radar Meteorology, A.M.S. Norman, Oklahoma, 1966, pp. 216-219.

Nason, C. K., "Mesoscale Precipitation Patterns in Central New England," S.M. Thesis, Dept. of Meteorology, M.I.T., Cambridge, Massachusetts, 1965.

Naylor, T. H., J. L. Balintfy, D. S. Burdick, K. Chu, "Computer Simulation Techniques," John Wiley and Sons, Inc., 1968.

Neff, E. L. and R. Hamon, "Evaluation of Precipitation Gage Performance," Boise 1965 Reynolds Creek Watershed Annual Report, Idaho - Bo - 102.6, 1965.

Newton, C. W., and J. C. Fankhauser, "On the Movements of Convective Storms, with Emphasis on Size Discrimination in Relation to Water-Budget Requirements," J. Applied Meteorology, Vol. 3, No. 6, 1961, pp. 651-668.

Newton, C. W., and Fankhauser, J. C., "Movement and Development Patterns of Convective Storms and Forecasting the Probability of Storm Passage at a Given Location," NSSP Report No. 22, Washington, D.C., 1964.

Osborn, H. B., and L. D. Lane, "Stochastic Models of Spatial and Temporal Distribution of Thunderstorm Rainfall," The Symposium on Statistical Hydrology, Tucson, Arizona, August 31-September 2, 1971.

Omotoso, E. A., "Development of the More Intense Storms in a Well Organized Squall Line," S.M. Thesis, Dept. of Meteorology, M.I.T., Cambridge, Massachusetts, 1967.

Pattison, A., "Synthesis of Hourly Rainfall Data," Tech. Rep. No. 40, Dept. of C. E., Stanford University, 1964.

Pattison, A., "Synthesis of Hourly Rainfall Data, Water Resources Research, Vol. 1, No. 4, 1966, pp. 489-498.

Petterssen, S., Introduction to Meteorology, McGraw-Hill Book Co., N.Y., N.Y., 1969.

The President's Water Resources Council, "Policies, Standards, and Procedures in the Formulation, Evaluation, and Review for Use and Development of Water and Related Land Resources, U. S. Congress 87:2, Sen. Doc. 97, 1962.

Ramaseshan, S., "Stochastic Modelling of Storm Precipitation," Nordic Hydrology (1971), II, 109-129.

Richards, M. M. and J. A. Strahl, "Elements of River Forecasting (Revised)," ESSA Technical Memorandum WBTM HYDRO 9, U. S. Dept. of Commerce, Environmental Science Services Administration, Silver Springs, Maryland, March 1969.

Rosenblueth, A., and N. Wiener, "The Role of Models in Science," Philosophy of Science, XII, No. 4, 1945, 316-321.

Satyam, M., "Cyclic Variation of the Rate of Flashing in Thunderstorms," J. of Atmos. Sci. 19, 346, 1962.

Schaake, J. C., "Response Characteristics of Urban Water Resource Data Systems," ASCE Urban Water Resources Research Program, Tech. Mem. No. 3, 1968.

Scott, H. A., "Types of Heavy Rain-Producing Storms in Georgia," Monthly Weather Review, 61, 299, 1933.

Semplak, R. A., "A Gauge for Continuously Measuring Rate of Rainfall," Review of Scientific Instruments, 37, 1966, pp. 1554-1558.

Semplak, R. A. and H. E. Keller, "A Dense Network for Rapid Measurement of Rainfall Rate," The Bell System Technical Journal, Vol. 48, No. 6, 1969, pp. 1745-1756.

Shubik, Martin, "Simulation of the Industry and the Firm," American Economic Review, No. 5, 1960, 908-919.

Sibley, A. K., "Flood Control in New England," J. of the Waterways and Harbors Division, ASCE, WW1, Paper 1518, January 1958.

Sorman, Unal A. and James R. Wallace, "Digital Simulation of Thunderstorm Rainfall," Environmental Resources Center, Georgia Institute of Technology, Atlanta, Georgia, August 1972.

Stout, G. E. and E. A. Mueller, "Survey of Relationships Between Rainfall Rate and Radar Reflectivity in the Measurement of Precipitation," J. of Applied Meteorology, Vol. 7, June 1968, pp, 465-474.

Thomasell, A., Jr., "Rainfall Variability for Urban Drainage Problems," Travelers Research Corporation, Proposal on Urban Drainage Problem to OWRR, 1968.

Thomasell, Albert, "Considerations for Characterizing the Time and Space Distributions of Metropolitan Storm Rainfall," The Travelers Research Center, September 1968.

Tracton, M. S., "The Role of Cellular Convection Within an Extratropical Cyclone," Proceedings 13th Radar Meteorology Conf., A.M.S., Montreal, Canada, 1968, pp. 216-221.

Trewartha, G. T., An Introduction to Climate, McGraw-Hill Book Co., N.Y., N.Y., 1968.

United Nations, "Standards for Methods and Records of Hydrologic Measurements," United Nations, Bangkok, 1954.

U. S. Water Resources Council, "Principles for Planning Water and Land Resources," Report to the Water Resources Council by the Special Task Force, Washington, D.C., July, 1970.

U. S. Weather Bureau, "Thunderstorm Rainfall," Hydrometeorological Report 5, 1947, pp. 262-263.

U. S. Weather Bureau, "Rainfall Intensity-Frequency Regime," Tech. Paper 29, Part 1-5, Washington, D.C., 1957-60.

U. S. Weather Bureau, "Climatological Survey, Climatography of the United States," U. S. Dept. of Commerce, Weather Bureau, 1951-1970.

U. S. Weather Bureau, "Local Climatological Data, Boston, Massachusetts," U. S. Dept. of Commerce, Weather Bureau, 1951-1970.

Wexler, R., "Radar Echoes from a Growing Thunderstorm," J. of Meteorology, Vol. 10, No. 4, 1953, pp. 285-290.

Wexler, R., and Atlas, D., "Precipitation Generating Cell," J. of Meteorology, Vol. 16, 1959, pp. 327-332.

White, G. F., "Choice of Adjustment to Floods," Dept. of Geography Research Paper No. 93, University of Chicago, Chicago, Illinois, 1964.

Wilkinson, J. C. and L. V. Tavares, "A Methodology for the Synthesis of Spatially Distributed Short Time Increment Storm Sequences," J. of Hydrology, 16(1972), 307-315.

Wilson, J. W., "Evaluation of Precipitation Measurements with the WSR-57 Weather Radar," J. of Applied Meteorology, Vol. 3, No. 2, 1964, pp. 164-174.

Wilson, J. W., "Radar Rainfall Measurements in Thunderstorms," Report 7476-248, The Travelers Research Center, Inc., Hartford, Connecticut, 1967.

Wilson, J. W., "Factors Affecting the Accuracy of Radar Rainfall Measurements," Report 7488-331, The Travelers Research Center, Inc., Hartford, Connecticut, 1968.

Wilson, J. W., "Radar Measurements of Rainfall for Operational Purposes," Report 7488-360, The Travelers Research Corp., Hartford, Connecticut, 1969.

Wilson, J. W., "Operational Measurement of Rainfall with the WSR-57: State-of-Art and Recommendations," The Center for the Environment and Man, Inc., Hartford, Connecticut, July 1970.

Wooding, R. A., "A Hydraulic Model for the Catchment-Stream Problem, I. Kinematic Wave Theory," J. of Hydrology, Vol. 3, Nos. 3/4, 1965, pp. 254-267.

Wooding, R. A., "A Hydraulic Model for the Catchment-Stream Problem, II. Numerical Solutions," J. of Hydrology, Vol. 3, Nos. 3/4, 1965, pp. 268-282.

Wooding, R. A., "A Hydraulic Model for the Catchment-Stream Problem. III. Comparison with runoff observations," J. of Hydrology, Vol. 4, 1966, pp. 21-37.

Woolhiser, D. A. and J. A. Liggett, "Unsteady One-Dimensional Flow over a Plane - The Rising Hydrograph," Water Resources Research, Vol. 3, No. 3, 1967, pp. 753-771.

Workman, E. J., R. E. Holzer, and G. T. Pelsor, " The Electrical Structure of Thunderstorm," NACA Technical Note, No. 864, Washington, D.C., 1942, pp. 26.

Workman, E. J., and S. E. Reynolds, "Electrical Activity as Related to Thunderstorm Cell Growth," The Bulletin of Am Meteorological Soc., Vol. 30, 1949, pp. 142-144.

Yevjevich, Vujica, "Probability and Statistics in Hydrology," Water Resources Publications, Fort Collins, Colorado, 1972.