WATER RESOURCE SYSTEMS PROGRAM

Department of Civil Engineering
Colorado State University
Fort Collins, Colorado

ONTROL STRATEGY DEVELOPMENT STUDY

an F[illegible]o Wastewater Master Plan

y Harry G. Wenzel and Bruce H. Bradford

MWIS PROJECT
JULY 1974

CER74-75BHB18

CONTROL STRATEGY DEVELOPMENT STUDY

San Francisco Wastewater Master Plan

prepared by

Harry G. Wenzel

and

Bruce H. Bradford

Metropolitan Water Intelligence Systems

August 1974

Department of Civil Engineering
Colorado State University
Fort Collins, Colorado

COLORADO STATE UNIVERSITY

FORT COLLINS, COLORADO 80521

ENGINEERING RESEARCH CENTER, FOOTHILLS CAMPUS

August , 1974

Mr. A. O. Friedland
Department of Public Works
City and County of San Francisco
San Francisco, CA 94102

Dear Mr. Friedland:

This report contains the latest results from our control strategy studies in Vicente Basin. It is intended to be the completion report for the simulation studies initiated in 1972 as a joint effort by your staff and CSU. An interim report addressed to the same problem was submitted on April 20, 1973. We subsequently discussed with Messrs. Giessner, Moss and Coffee the continued Vicente Basin simulation work reported herein. For full understanding, this report should be used in conjunction with our report "Metropolitan Water Intelligence Systems, Completion Report, Phase III," 1974.

In order to be useful this work should be presented to your technical staff members responsible for implementing the wet weather portion of the Master Plan. Toward that objective, I can arrange such a presentation at your convenience. For the information of persons not familiar with the SFDPW-CSU cooperative studies, I have described the background leading to the presentation of the report in a "Foreword" section of this report.

Speaking for myself and the others at CSU who have worked on this project, we feel that the concepts presented for wet weather control in the Master Plan offer substantial promise for a cost-effective solution to the wet weather problem. We feel that the technical work contained in this and in related reports has a great deal to offer those who will implement the automatic control system necessary to best utilize this innovative system. We therefore hope that our work will continue to be of use in this effort. With best regards,

Yours very truly,

Neil S. Grigg
Associate Professor

NSG:kv

FOREWORD

By Neil S. Grigg

This report represents the partial results of a two year cooperative research effort between Colorado State University and the San Francisco Department of Public Works. The focus of the investigation has been on the development of techniques for computer control of the wet weather portion of the 1973 "Master Plan for Wastewater Management."

The work reported herein has been completed with financial support from the U. S. Department of the Interior, Office of Water Resources Research. The title of the three-year project was "Metropolitan Water Intelligence Systems" (MWIS). A number of other reports have been issued. They are listed at the end of this section.

This cooperative work was initiated through the efforts of Murray B. McPherson, Director of the ASCE Urban Water Resources Research Program. The work was essentially started in summer of 1972. Professor McPherson described the work plan for CSU as follows (from his July 5, 1972 memorandum to Mr. William Giessner).

Telephone: 617-631-7137

AMERICAN SOCIETY OF CIVIL ENGINEERS

URBAN WATER RESOURCES RESEARCH PROGRAM

ADDRESS REPLY TO:
M. B. McPherson, Director
ASCE Urban Water
Resources Research Program
23 Watson Street
Marblehead, Massachusetts 01945

MEMORANDUM

To: Mr. William R. Giessner, Planning and Studies Head, Division of Sanitary Engineering, DPW, San Francisco

From: M. B. McPherson, program director

Subject: Preliminary work plan for Colorado State University project for a supplementary input to the first step of the DPW/SF Plan for Research, Development and Demonstration Period of Wet-Weather Upstream Control Program

Date: July 5, 1972

Under a memorandum of June 20, 1972, I sent to Mr. A. O. Friedland my interpretation of "City and County of San Francisco, Plan for Research, Development and Demonstration Period of Wet-Weather Upstream Control Program, June, 1972-February, 1977," 34 pages. Please see pages 29 and 30, particularly the latter, for my general summary of CSU's intentions. The following is the preliminary CSU work plan for "Demonstration of Control Development Capability" as prepared last week by Prof. Neil Grigg and subsequently modified by me:

1. Obtain physical definition from DPW of Vicente catchment, including catchment boundary, sewer layout, and related factors that the DPW have been using for analysis of rainfall-runoff data on the catchment.

2. Obtain one-minute interval rainfalls (or shorter interval, if available) and associated stages or flows for the about one dozen storms of record applicable to the Vicente catchment. Presumably, the DPW is currently using this same data in analysis of the Vicente catchment.

3. Using a simple model, determine the runoff coefficient, or similar parameter, and other necessary catchment response characteristics, for the storms of record at the flow gage sites.

4. Apply these calibrations to the Vicente catchment as changed by the addition of the new storage basins as sized and located for the preliminary Master Plan of 1971 and make the following tests:

a. Simulate operation for the same storms of record for a range of withdrawal-to-treatment rates and possibly a range of storage-use.

b. Explore effects of potential equipment malfunctions on capabilities for meeting the preceding ranges of control objectives.

5. Subsequent to a review of findings on "4" with DPW personnel, reach agreement with them on design storms or series of storms to be applied in subsequent tests. For example, large-volume and small-volume extremes could be used or a series of large-volume storms based on the U.S. Weather Service gage record might be elected instead. Also, review with DPW personnel overall City storm movement-pattern characteristics as they might affect individual catchment responses.

6. Using the storm series selected:

   a. Simulate operation for the same range of operating criteria as before except for different storage volumes and/or locations.

   b. Study potential equipment malfunctions as before, plus the effects of changes in control criteria during a storm or deviation of rainfall from predicted behavior during a storm.

7. Prepare summary of findings for the DPW and review these with DPW for the purpose of insuring their maximum utility to the DPW.

8. Proceed to expand the Vicente case study into a generalized application. (This is Phase III of the CSU study). Whether or not this phase would commence with analysis of other SF catchments would be up to the DPW.

Any and all reports prepared by CSU dealing with SF, directly or by implication, would be reviewed by the DPW before distribution by CSU, and revised as required by the DPW with regard to interpretation. For example, there should be no suggestion included that operating criteria studied are necessarily among those the DPW will ultimately adopt, inasmuch as DPW work will have proceeded independently of the CSU project.

Your early reaction to this preliminary plan for CSU is earnestly requested.

cc: Dr. Neil Grigg, CSU
Mr. A. O. Friedland, SF/DPW
Dr. G. F. Mangan, OWRR

Following the early coordination necessary to initiate the project, a number of visits were exchanged between SFDPW and CSU personnel. A great deal of data and technical guidance was furnished by SFDPW to CSU. Extremely helpful initially were W. R. Giessner and Frank Moss and later, Harold Coffee.

Since the work described by Professor McPherson was initially concerned with a catchment study, the Vicente Basin, an effort was mounted at CSU to launch a city-wide study concerned with control strategy. A grant from NSF-RANN was approved to begin this study on July 1, 1973. The principal result of this so far has been one Ph.D. dissertation entitled, "Real Time Control of a Large Scale Combined Sewer System" by Bruce H. Bradford, and one paper scheduled for publication by ASCE entitled, "Automatic Control of Large-Scale Combined Sewer Systems" by John W. Labadie, N. S. Grigg and B. H. Bradford.

A proposal to OWRR entitled "Implementation of Optimal Computer Control for Combined Sewer Systems" was submitted in January, 1974. The objective of that planned work is to continue and assist in the implementation of the work described in this report. The proposal has not been acted upon at the Time of this writing.

Other MWIS Reports that have been issued are as follows:

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 Minneapolid-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 Systmes" 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)

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

The project personnel have enjoyed the excellent cooperation of the Department of Public Works of the City of San Francisco. The assistance of W. R. Giessner, H. C. Coffee and F. H. Moss, Jr., in providing information and data has been of particular value and the cooperative arrangements have been greatly facilitated by the support of A. O. Friedland.

The senior author was involved in this work while on leave from the Department of Civil Engineering, University of Illinois. He is grateful for the opportunity of working with the project staff at Colorado State University and the San Francisco DPW.

The junior author has been involved in the project since its incpetion. The aggregation technique discussed in Chapter IV was the subject of his doctoral dissertation. He will soon join the faculty of the Civil Engineering Department, Georgia Institute of Technology.

LIST OF SYMBOLS

A = drainage area

C = runoff coefficient

D = storm depth

f = function

k = statistical parameter

K = routing constant

n = Manning's roughness

$\tilde{n}$ = number of overflows per year

p = hourly precipitation

P = cumulative probability

Q = discharge

R = risk

T = storm duration

x = volume of overflow per year

α = control level

$\Gamma[\ ]$ = Gamma function

λ = statistical parameter

σ = standard deviation

μ = mean

LIST OF FIGURES

LIST OF TABLES

TABLE OF CONTENTS

CHAPTER I

INTRODUCTION

A. Background for the Report

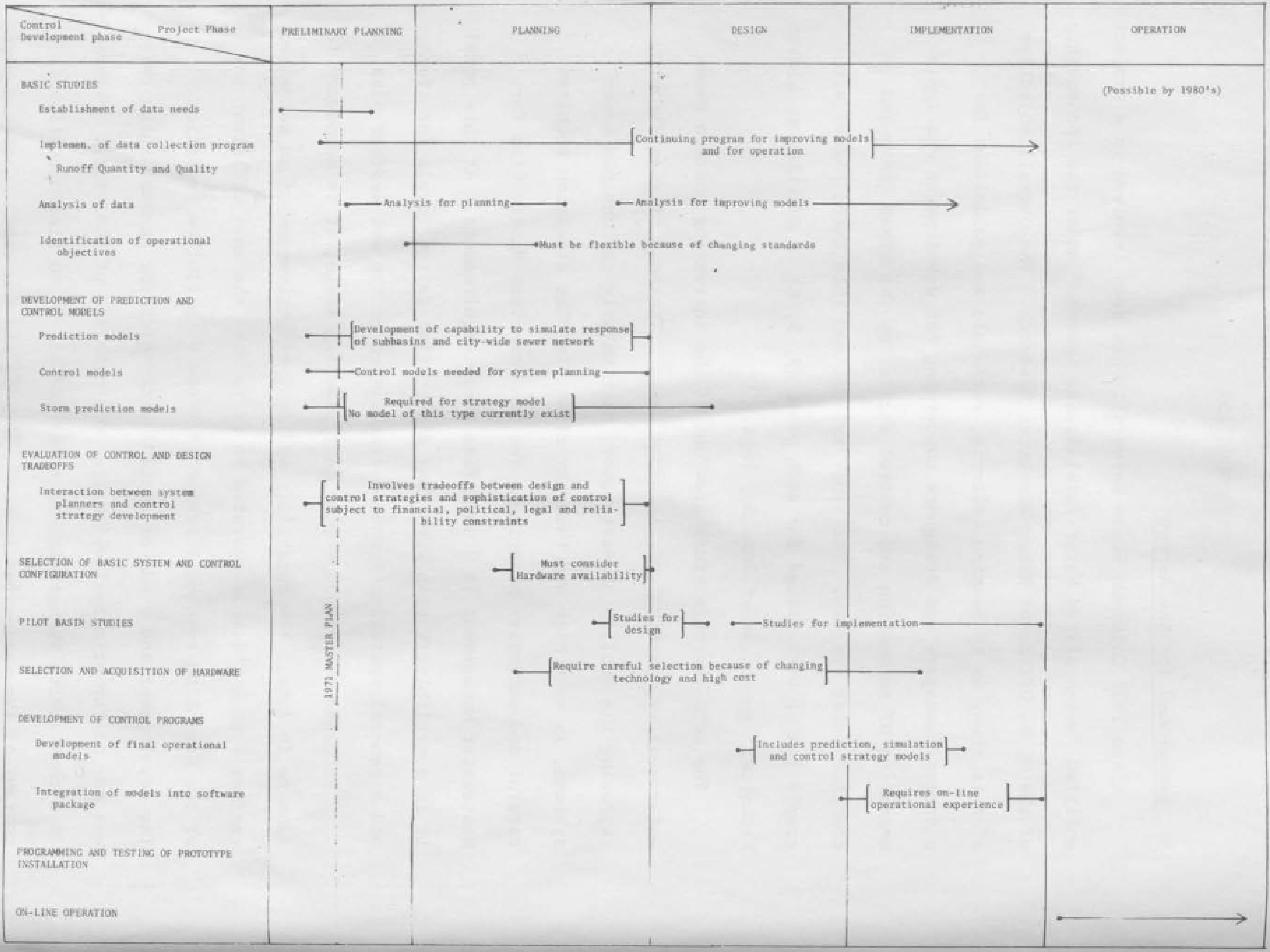
Since 1971 Colorado State University has been involved in a project entitled "Metropolitan Water Intelligence Systems" under the sponsorship of the U. S. Office of Water Resources Research. The overall objective of this study is to examine criteria, rationale and guidelines for planners, managers and designers concerning the development and implementation of automation and control facilities for urban storm and combined sewer systems. The study was divided into three phases with completion reports issued for each phase [9, 5, 6] in addition to eleven Technical Reports as of June 30, 1974.

The need for this study grew out of the increasing concern about water quality, particularly in urban areas. The cost of significantly improving the quality of wastewater, particularly in combined sewer systems, is very high and thus any effort in this direction requires careful and extensive planning. The San Francisco Master Plan for Wastewater Management [4] satisfies these requirements. At this point it is a preliminary plan with four alternative design levels for storage and a general operational scheme established as the wet weather plan.

In order to proceed to the next planning stage, it is necessary to examine in detail the capability and cost-effectiveness of an automated control system which is operated so as to make the most efficient use of the detention reservoir system in terms of pollution reduction. The development and study of control strategies for accomplishing this was one of the principal objectives of Phase III of the study. Figure I-1 is a chart which summarizes the steps leading to on-line operational control of the wastewater system. The efforts described in this report

FIGURE I-1

IMPLEMENTATION OF AUTOMATIC CONTROL FOR SAN FRANCISCO WASTEWATER MASTER PLAN



begin in the preliminary planning stage and carry over into subsequent stages.

B. Objectives of This Report

This report was prepared specifically for the City of San Francisco as a supplement to the MWIS Phase III Completion Report. Its objective is to concentrate on the results and techniques for control strategy development. In addition, certain data and analyses which were gathered or performed during the course of the study and not presented in the Phase III report appear here.

Although considerable reference to the Phase III report is made, the material concerning control strategy development is organized differently. Primary emphasis is placed upon results and theoretical development is minimized. This report should be regarded as a supplement to the Phase III report. It focuses on the control strategy aspect and is written for the planner and engineer rather than the researcher.

The material that is not in the Phase III report is contained primarily in Chapter IV and the appendices. A quantitative summary of the thesis results of the second author concerning a specific city-wide control strategy technique are presented in Chapter IV. Detailed rainfall data for large storms in San Francisco, analysis of storm parameters and a summary of subcatchment data are presented in the appendices. This information is presented as reference material for possible future use.

CHAPTER II

RUNOFF HYDROGRAPH PARAMETER IDENTIFICATION MODEL

A. Model Description

Of basic importance in any simulation model is the evaluation of the parameters or coefficients which are used, i.e., the calibration of the model.

The rainfall-runoff model used in the Vicente Subbasin simulation model is based on the instantaneous unit hydrograph. This is a relatively simple model requiring two parameters: a runoff coefficient, C , and a routing constant, K .

Because rainfall data are available from the San Francisco raingage network and runoff data are available from the flow gage system it is possible to calibrate the rainfall-runoff model using actual field data. To do this a parameter identification model was developed to determine C and K using actual data for Vicente Subbasin.

The rainfall data was supplied by the City of San Francisco in the form of an average mass curve for each storm with values at 15 minute increments. This data was formulated from the raw raingage data using the City's SYMAP computer program. The runoff data was in the form of sewer level readings at 15 second intervals for Flow Gage 125 located in a 6.0 ft. diameter sewer at Vicente St. and 34th Avenue. The rating curve used by the City to convert level readings to discharge was based on the application of Manning's equation with n=0.013.

The objective of the parameter identification model was to determine the values of C and K which produced the best agreement between the predicted outflow hydrograph from the rainfall-runoff model and the actual hydrograph as measured by F.G. 125. The runoff coefficient is easily computed as the ratio of runoff volume to rainfall volume for any storm. However, because K is primarily a mathematical parameter rather than a physical one it is the parameter which can be adjusted to maximize hydrograph agreement.

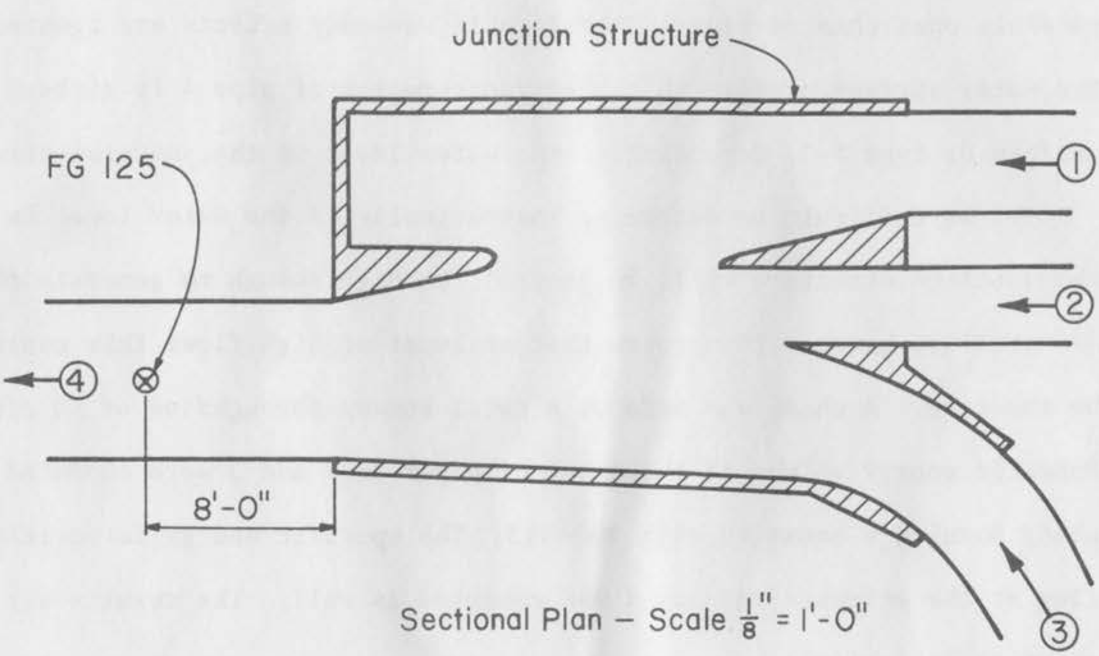
The model was set up to determine the outflow hydrograph using one of two methods, both based on the instantaneous unit hydrograph. The first method treats the watershed as a single linear reservoir. The second employs a linear reservoir-linear channel or Clark [2] model using either a triangular or specified time-area histogram. Two types of hydrograph fitting criteria were used. One identified the K value which minimized the standard error between the entire observed and calculated hydrographs. The other identified the K which minimized the sum of the relative error for the peaks and time to peaks. Further details are explained in the Phase III MWIS completion report.

A listing of the FORTRAN program for the model is given in Appendix A.

B. Flow Gage 125 Rating Curve

Initial results from the parameter identification model indicated runoff coefficients greater than unity for some storms. Since it was unlikely that the precipitation data caused this problem a hydraulic analysis of F.G. 125 was performed to check the validity of the assumption of uniform flow in developing a rating curve. The analysis is described herein because it points to potential problems at other flow gage sites as well.

Flow Gage 125 is located 8 ft. downstream from the outlet of a junction structure which combines the flow from three inflow lines. A schematic with the pertinent information is shown in Figure II-1. The



Pipe	Size	Slope
1	45 in. dia. Circ.	0.042
2	36 in. × 54 in. Semi-eliptical	0.050
3	63 in. dia. Circ.	0.0068
4	72 in. dia. Circ.	0.0161

FIGURE II-1

FLOW GAGE 125 SITE

slope of all four pipes is hydraulically steep throughout the range of possible open channel flow. Therefore if unsteady effects are ignored the water surface profile in the entrance region of pipe 4 is either uniform or type S-2, depending on the water level in the junction structure.

It is difficult to determine theoretically if the water level in the junction structure will, in general, be high enough to generate the S-2 profile, however it appears that at least at high flows this would be the case. A check was made at a total steady throughflow of 50 cfs. Specific energy values at the exits of pipes 1, 2 and 3 were computed using Manning's equation with n=0.013. The specific energy for critical flow at the entrance to pipe 4 was computed as well. The results are shown in Table II-1.

Table II-1

Specific Energy for Q=50 cfs

Pipe	Assumed Q (cfs)	Depth (ft.)	Specific Energy (ft.)
1	12.4	0.56	2.77
2	18.6	0.63	3.82
3	19.0	0.97	1.71
4	50.0	1.92	2.56 (critical flow)

Table II-1 shows that sufficient energy is available to cause some pooling in the junction structure at this relatively low flow. Pooling would also be encouraged by the inflows from the three upstream lines colliding in the structure.

It therefore seems possible that the depth at F.G. 125 is above normal and possibly near critical depth. It is difficult to be more precise without field measurements because of the complex flow pattern in the entrance region of pipe 4. With this in mind there are two approaches to developing a new rating curve. The first and most simple is to assume that critical depth occurs at F.G. 125. This will produce the lowest flows for a given level reading commensurate with the hydraulic conditions. The second is to assume that critical depth occurs at the entrance to pipe 4 and to construct appropriate water surface profiles for various flows to determine the corresponding depth at F.G. 125.

The results of the first approach are shown in Figure II-2 together with the uniform flow curve. For flows below 200 cfs both curves are linear on the log-log plot, thereby facilitating their mathematical description. Letting Q_n and Q_c represent the discharge assuming normal and critical depth respectively at F.G. 125 the relationships shown on Figure II-2 apply. They can be combined to yield

$$Q_c = 0.522\ Q_n^{0.951} \qquad Q_n, Q_c \leq 200 \text{ cfs} \qquad (1)$$

which shows that the discharge is approximately 50 percent of the value obtained using the uniform flow rating curve.

The second approach was used for three discharges. The resulting depths at F.G. 125 were applied to the critical flow rating curve to obtain corresponding discharges. The ratio of the actual discharge to the value obtained from the rating curve is the correction factor which should be applied if the critical flow rating curve is used. The results are shown in Table II-2.

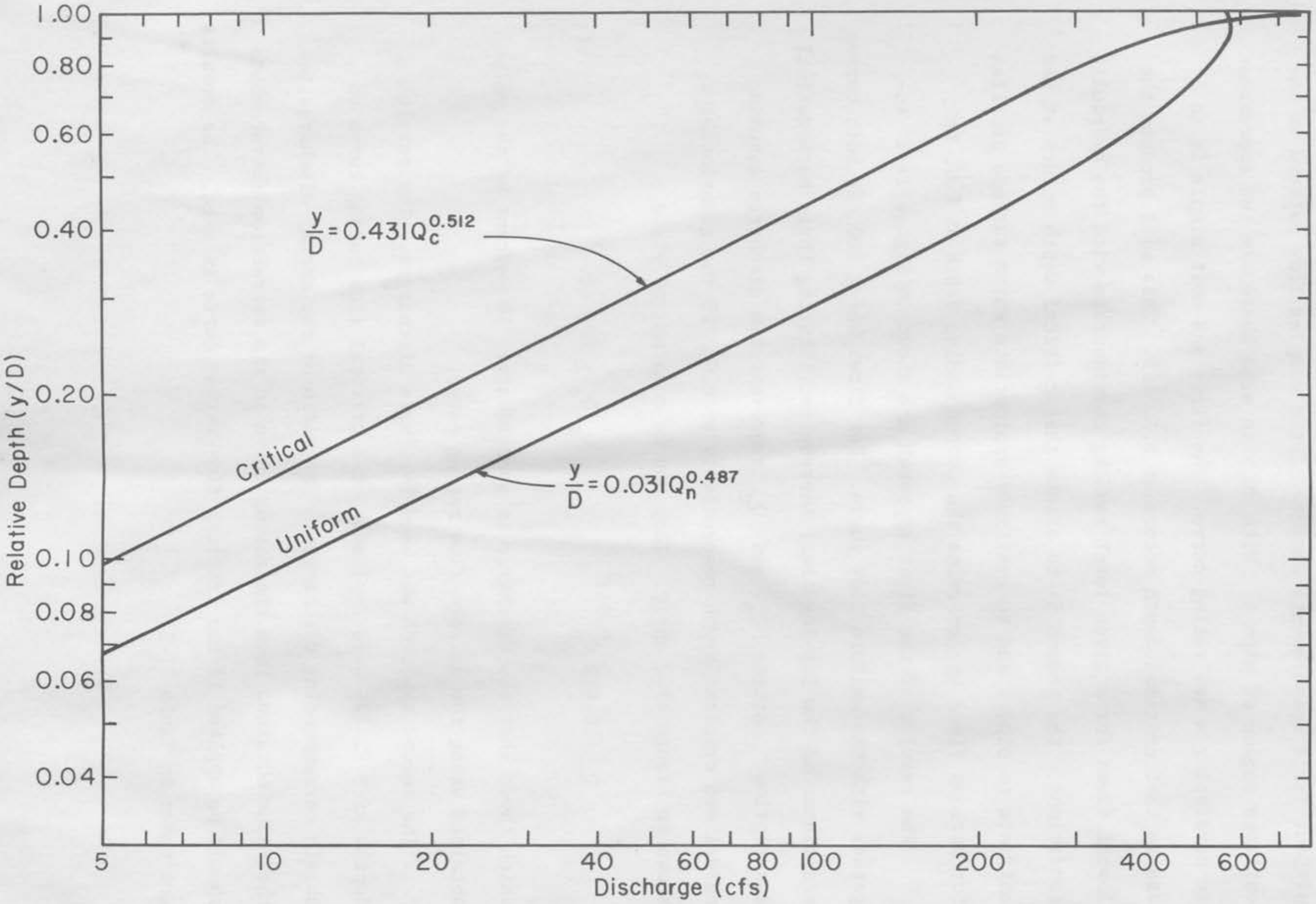


FIGURE II-2

FLOW GAGE 125 RATING CURVES

Table II-2

Rating Curve Correction Factors

Actual Q (cfs)	Q from Rating Curve (cfs)	Correction Factor
50	39.7	1.26
100	78.1	1.28
300	250.0	1.20

In this case the flow is approximately 25 percent higher than if critical depth occurred at F.G. 125.

As a result of this analysis the critical depth rating curve shown in Figure II-2 was used in the parameter identification model. This results in reasonable runoff coefficients and the results are discussed in the next section.

It can be concluded that if the data from any flow gage is to be used for model calibration or verification a hydraulic analysis of that particular gage is necessary.

C. Results of Hydrograph Parameter Identification Model

A total of 19 storms were processed through the model. The results are summarized in Table II-3. In this table the Time Increment is the increment associated with the rainfall hyetograph and $\overline{T}$ is the time difference between the centroids of the hyetograph and the outflow hydrograph. For the Clark routing a symmetrical time-area graph was used with a total base time of $\overline{T}$.

The resulting runoff coefficients are reasonable with the exception of the final value in the table. The average of these values is 0.633 which is close to the 0.66 value commonly used for preliminary design

Table II-3

Results of Hydrograph Parameter Identification Model

Storm Date		Duration (min)	Time increment (min)	Total Precip.(in)	Precip. Excess (in)	Runoff Coef.	Optimum Peak Fit Criterion		K (hrs) Overall Fit Criterion		$\overline{T}$ (hrs)
Year	Day						Clark	SLR	Clark	SLR	
1971	342	375	15	0.28	0.17	0.600	0.51	0.51	0.41	0.61	0.84
1971	343	600	30	0.31	0.16	0.518	0.54	0.04	0.04	0.44	1.28
1971	345	705	15	0.44	.31	0.697	0.65	0.74	0.95	0.85	0.86
1971	346	525	15	0.46	0.29	0.625	0.97	0.97	0.67	0.67	0.57
1971	348	120	15	0.07	0.04	0.581	0.19	0.09	0.29	0.69	0.83
1971	348	330	15	0.12	0.11	0.905	1.22	1.22	0.52	1.02	1.17
1971	358	600	60	0.54	0.30	0.547	1.01	0.01	0.01	0.01	1.33
1971	358	400	10	0.56	0.28	0.507	0.87	0.87	0.07	0.57	0.83
1971	359	600	20	0.48	0.29	0.610	0.92	0.92	0.32	0.42	1.23
1972	22	150	5	0.26	0.12	0.453	0.37	0.47	0.37	0.57	0.62
1972	22	300	60	0.22	0.17	0.776	0.72	0.02	0.02	0.02	1.19
1972	26	330	30	0.29	0.20	0.683	0.41	0.01	0.21	0.21	0.66
1972	27	150	5	0.26	0.15	0.591	--	--	0.20	0.40	0.47
1972	35	480	30	0.49	0.34	0.697	1.29	1.19	1.38	1.19	1.39
1972	52	135	15	0.32	0.19	0.596	0.25	0.35	0.35	0.55	0.71
1972	52	300	60	0.19	0.13	0.676	0.61	0.91	0.71	0.71	1.22
1972	52	100	10	0.12	0.11	0.904	0.26	0.36	0.36	0.76	0.77
1972	81	165	15	0.14	0.06	0.427	0.03	0.03	0.13	0.43	0.62
1972	102	150	10	0.09	0.13	1.442	0.25	0.65	0.35	0.65	0.87

$\overline{c}=.633$

$\sigma=.133$

of the system. The high values are usually associated with a low total precipitation which is reasonable since in this case errors in base flow estimation seriously affect the precipitation excess value. If storms with total precipitation less than 0.20 in. are excluded from the analysis this problem will be considerably reduced.

The optimum K values vary considerably and it is difficult to correlate this variation with any of the storm parameters. Some of this variability is caused by the optimization scheme which simply seeks the value of K which minimizes the error criterion and does not consider variation in the criterion around the optimum value. In other words, the relative reduction in the fit error may be small over a range of K values but a minimum is achieved at an extreme value. This could explain some of the very low values shown. The K values for the Clark method are generally lower than those for the single linear reservoir method, particularly when using the overall fit criterion which could be expected. There is some correlation between $\overline{T}$, which is a measure of travel time, and K. It can be concluded that the uncertainties in the data together with the approximations inherent in the model do not justify the use of an optimization scheme for choosing K which ignores these uncertainties. It would be better to exercise some judgment based on experience gained from processing more storms on various sizes of subcatchments through the hydrograph model.

The data in Table II-3 are all for one size of subcatchment. The subcatchment sizes used in the Vicente simulation model were much smaller and thus these results are of little value in assigning K values in that case. Therefore an analysis of rainfall-runoff data from small subcatchments would be quite useful and would probably result in a better fit between observed and predicted hydrographs. Time did not permit this to be done in the context of this study. In fact the general question of the optimum level of aggregation to be used in the simulation model remains to be investigated.

CHAPTER III

SIMULATION APPROACH FOR CONTROL STRATEGY DEVELOPMENT FOR VICENTE SUBBASIN

A. Vicente Simulation Model

The Vicente model is described in some detail in the MWIS Phase III report. It was developed to investigate the response of the Vicente Subbasin detention reservoir drainage system to various reservoir control strategies. The model is general in that it will accept as input storms with temporal and spatial variation. The manner in which these storms are obtained is arbitrary. It is a distributed deterministic model using the instantaneous unit hydrograph concept to generate runoff. Level pool routing is used for the detention reservoirs and a modified Muskingum routing scheme is used in the sewer lines.

The Cunge-Muskingum routing method described in the MWIS Phase III report was modified somewhat from the original approach as proposed by Cunge [3]. The Cunge method treats the travel time through the reach as a variable based on the wave celerity which is computed at each step in the routing process. It was found that this process resulted in a loss of water volume under the downstream hydrograph. In other words, conservation of mass was being violated. This was an unacceptable situation because of the importance of reservoir overflow volume as a performance parameter for evaluation. Therefore a constant value of wave celerity was used for each reach, regardless of the actual flow. This value was computed assuming that the pipe was flowing half full. Although this may have produced some minor changes in the resulting hydrographs, they no longer violated the continuity equation and therefore this modification was adopted.

The input data for the Vicente model in addition to the rainfall data is summarized in Table IV-2 of the MWIS Phase III report. Pipe geometry and slope data were average values estimated from the detailed information in the San Francisco Department of Public Works Master Plan. The runoff coefficients for all but two subcatchments are for similar residential areas and a value of 0.65 is reasonable based on Table II-3. The two subcatchments with C=0.35 are park areas with a larger proportion of unpaved area than the others. The K values are estimates based on approximate travel times for the subcatchments. Since the data in Table II-3 are for the group of subcatchments upstream from F.G. 125 the values shown there are not applicable. The two subcatchments with K=0.2 are for the park areas which have a lower sewer density, hence the higher value of K. The dry weather flow values used in the model are based roughly on an average value of 1cfs/sq.mi. The results are not sensitive to this value since the flow during storms is usually much higher. The data from F.G. 125 as well as other flow gages could be used to estimate the dry weather flow more accurately.

It is clear that the model calibration as described above is not precise. Good calibration of F.G. 125 as well as flow measurements at other points in the system are needed. However, the purpose of the model was to compare control strategies, and for this purpose it is adequate.

Appendix B contains a FORTRAN listing of the model which is included as a subroutine in the statistical analysis program discussed in Section D.3.

B. Effect of Control Strategy on Vicente Subbasin System Performance

B.1 Possible Approaches

These are two basic approaches which can be employed to develop control strategy. One is to assume various strategies and to test them using the model. The second is to determine the optimum strategy for a series of storms and to attempt to generalize the results. The criterion for optimality using the Vicente Subbasin model is to minimize the volume of overflow from the detention reservoirs. The second approach has the advantage of directly yielding the desired results. However, the resulting strategy may be quite complex and difficult to specify as a function of individual storm event characteristics. The first approach has the advantage of the prior knowledge of the general form of the control strategy. The strategy parameters can then be manipulated to produce the best results within the context of that particular form of strategy. However, there is no guarantee that some other general strategy would not produce still better results.

The problem of developing an optimal control strategy for the entire city system is indeed a challenging one. This study is just a first step in solving that problem. Hopefully, by examining a particular subbasin in some detail some idea of the relative improvement in system performance gained by real time reservoir control can be achieved. With this in mind a single general strategy was chosen for investigation. It is a logical one, easily described and could be readily implemented. It thus could serve as a basis for evaluating the possible improvement which might be expected for the city-wide system performance as a result of real time control.

B.2 The Control Strategy Selected for Study

The general strategy is described in Chapter IV, Section C.2 and illustrated in Figure IV-1 in the MWIS Phase III report and is summarized here. The outflow from each of the three upstream reservoirs in Vicente Subbasin is uncontrolled until the inflow exceeds a value Q_{imax} . If and when this occurs the outflow is controlled at Q_{imax} and the excess inflow is stored in the reservoir. If the reservoir becomes filled the excess inflow becomes overflow. This may take the form of street flooding in the case of the upstream reservoirs, or would be discharged into the receiving waters in the case of the downstream reservoir (reservoir 12-2). The control is maintained until the inflow drops below Q_{imax} in which case the outflow is uncontrolled again. The maximum outflow is described in terms of a reference flow, $\hat{Q}_i$.

$$Q_{imax} = \alpha\hat{Q}_i \tag{2}$$

where

$$\hat{Q}_i = C_i(0.3 \text{ in./hr.})A_i \tag{3}$$

where C_i and A_i are the runoff coefficient and drainage area upstream of reservoir i . A value of 0.3 in./hr. was used in Equation (3) since this rainfall intensity was one of the values for the design capacity of the lines discharging from a subbasin in the San Francisco Master Plan. This procedure proportions the controlled outflows according to drainage area yet permits Q_{imax} to be specified for all reservoirs simply by specifying the value of α . Therefore α , which can be termed the *control level*, is the parameter which completely describes the specific control strategy for the upstream reservoirs. The maximum outflow from the downstream reservoir is governed by the overall operating strategy for the city-wide system. It can be viewed as the link between the various subbasins. It has an upper limit established by the capacity of the proposed line leading to the interceptor. This corresponds to $\alpha_4 = 1.0$ since the line has a design flow equivalent to 0.3 in./hr. The subscript on α refers to the fourth reservoir (12-2) in the subbasin system. The case where outflow is limited to the treatment plant capacity (0.1 in./hr. over the entire city) is represented by $\alpha_4 = 1/3$.

B.3 Results of Control Strategy Application

The control strategy described in the previous section was developed and applied for Alternate B storage using the techniques described in Sections C and D. The important results are summarized here for emphasis rather than at the end of the chapter.

The evaluation of a strategy must be done on a statistical basis to be meaningful. To use a few individual storms for this purpose could be very misleading. Therefore, the average values and probability distributions of performance parameters which result from the application of a long term historical rainfall record to the Vicente simulation model are meaningful and serve as a valid means of evaluation.

Within the general control strategy under study a number of variations or *control level strategies* were investigated using the semi-continuous simulation technique. In order to facilitate discussion they are numbered as follows:

Control Level Strategy Number	Description
1	No control. Maximum upstream reservoir outflow = outflow line capacity. $\alpha \cong 3.0$.
2	Zero overflow rule curve, Figure III-4
3	Optimization rule curve [$\alpha = \alpha^*$ from Equation 8] Effective duration defined by $p_{max} p_i = 1.6$ Minimum $\alpha = 0.4$.
4	Optimization rule curve, [Equation 8] Effective duration defined by $p_{max} p_i = 2.0$ Minimum $\alpha = 0.5$
5	Constant $\overline{\alpha}$ (weighted average) for all storms $\overline{\alpha} = 1.416$ for $\alpha_4 = 1.0$ from strategy 4 results $\overline{\alpha} = 0.735$ for $\alpha_4 = 1/3$ from strategy 3 results
6	Constant $\overline{\alpha}$ (mean value) for all storms $\overline{\alpha} = 0.829$ for $\alpha_4 = 1.0$ from strategy 4 results.

Strategy 1 is the *do nothing* strategy and serves as a common basis for comparison. Strategy 2 was developed using the zero overflow curves with $\alpha_4 = 1.0$ as described in Section C.1. Strategies 3 and 4 are based on the rule curve developed from the optimization technique discussed in Section C.2. The adaptation of this rule curve to non-uniform intensity storms is discussed in Section D.3 and these strategies represent different adaptation criteria. Strategies 5 and 6 were included to show the results of using a constant value of α for all storms. This implies that no storm forecasting procedures are employed. The value of $\overline{\alpha}$ used was computed in two ways. In strategy 5, $\overline{\alpha}$ was computed as the mean of the α values for each overflow producing storm from strategies 3 and 4 weighted according to the overflow volume from each storm. In strategy 6, the simple unweighted mean of the α

values from strategy 4 for $\alpha_4 = 1.0$ was used. The corresponding case for $\alpha_4 = 1/3$ was not studied.

The average values of four system performance parameters resulting from the 66 year historical rainfall record for San Francisco are given in Tables III-1 and 2 for $\alpha_4 = 1$ and 1/3 respectively.

Table III-1

Average Results of Control Level Strategies for $\alpha_4 = 1.0$

	Control Level Strategy					
Parameter	1	2	3	4	5	6
Ave. Vol. of OF*/yr. [in.]	0.058	0.032	0.072	0.036	0.036	0.046
Ave. Number of OF/yr.	0.641	0.300	1.760	0.920	0.500	0.580
Ave. Vol. of OF/OF [in.]	0.091	0.106	0.041	0.039	0.072	0.079
Ave. Dur. of OF [hrs.]	0.770	0.730	1.430	1.070	0.870	0.800

* OF = overflow

Table III-2

Average Results of Control Level Strategies for $\alpha_4 = 1/3$

	Control Level Strategy					
Parameter	1	2	3	4	5	6
Ave. Vol. of OF/yr. [in.]	0.953	0.960	0.635	0.693	0.908	-
Ave. Number of OF/yr.	7.390	7.450	4.200	5.610	7.180	-
Ave. Vol. of OF/OF [in.]	0.129	0.129	0.151	0.124	0.126	-
Ave. Dur. of OF [hrs.]	2.000	2.060	2.790	2.570	2.260	-

The initial conclusion which can be drawn from these results is that substantial improvement in system performance can be achieved by utilizing some type of control strategy over a no control policy. Reduction in average overflow volume per year of up to 38 percent and in overflow events per year of over 50 percent were achieved. Although only one subbasin of the entire system was considered it is believed that equal or better performance than shown in Table III-2 is possible for the entire system since advantage can be taken of the spatial variation in rainfall intensity as well as the variation in travel time from the subbasins to the treatment plant. This can be done by individual control of the outflow from each subbasin, i.e., adjusting the value of the equivalent of α_4 for each subbasin.

There is, however, one important qualification which must be placed on the above conclusion. The results for strategies 2, 3 and 4 were obtained using historical rather than predicted hourly rainfall values. The question of prediction capability and its effect on system performance should be regarded as a high priority research topic which must be undertaken before an intelligent decision regarding control system design can be made. It appears at this point that storm prediction is the weakest link in the system control process and therefore merits attention.

A second conclusion from the results is that the most important single parameter in determining subbasin system performance is not the control level strategy but α_4 , i.e., the maximum allowable outflow from the subbasin into the interceptor. Comparison of the figures in Tables III-1 and 2 shows that the variation of performance parameters within either table is insignificant compared to order of magnitude change from $\alpha_4 = 1.0$ to 1/3 . This observation leads to a recognition of the importance of the city-wide control strategy relative to the subbasin strategy. A total system strategy which maximizes the allowable flow to the treatment plant subject to treatment rate limitations can be much more effective than a sophisticated subbasin strategy alone.

The above conclusions are of major importance from a practical viewpoint. Some comments concerning specific control level strategies follow.

In Table III-1, strategy 2 gave the best results while strategy 3 was best in Table III-2. Strategy 2 was developed specifically for the case of $\alpha_4 = 1.0$. Its use in the case of $\alpha_4 = 1/3$ gave results even poorer than the no control strategy. This is because all of the α values from strategy 2 are above 1.0 since for $\alpha_4 = 1.0$ only the larger storms will cause overflows. The use of $\alpha > 1.0$ for small storms will increase the overflow volume they may cause. Since restriction of α_4 to 1/3 greatly increases the overflow producing potential of small storms, the use of strategy 2 in this case produced poor results.

If a single strategy regardless of α_4 is used, then strategy 4 is best. It resulted from the use of optimization techniques for both $\alpha_4 = 1.0$ and 1/3. The results generated from the rule curve thereby produced were a function of the definition of effective storm duration and depth used in Equation 1 as described in Section D.3. Strategy 3 produced somewhat better results for $\alpha_4 = 1.0$. This is primarily because a minimum value of $\alpha = 0.4$ produced overflows from some smaller storms in the case of $\alpha_4 = 1.0$ which would be eliminated using a minimum $\alpha = 0.5$. In other words, strategy 4 achieves a better trade off between reduction of overflow from large storms and elimination of overflow from smaller storms than does strategy 3.

Strategies 5 and 6 were studied with the idea that no storm prediction methods would be used. In this case a constant value of α would be used, based on the historical rainfall record. It is clear at the outset that such a strategy will produce the best results when the variability in depth and duration of overflow producing storms is minimum. Since this variability is much less for $\alpha_4 = 1.0$ than $\alpha_4 = 1/3$ one might expect better results in the former case. Two methods of computing the value of α to be used were employed. For strategy 5, a weighted average was used, with the overflow volume from each overflow producing storm from strategy 3 or 4 serving as the weighting factor. In strategy 6 the unweighted mean α from strategy 4 was used. Strategy 5 produced fairly good results for $\alpha_4 = 1.0$ but very little improvement over strategy 1 for $\alpha_4 = 1/3$. This is not surprising because of the difference in storm variability as a function of α_4 as discussed above. For the case of $\alpha_4 = 1/3$ the large number of small overflow producing storms resulted in a low $\overline{\alpha}$ which in turn caused larger overflow volumes from the larger storms. It is likely that an optimum $\overline{\alpha}$ could be found, but it would be a function of α_4 . A value of $\overline{\alpha}$ independent of α_4 would probably result in little if any improvement over strategy 1.

The average duration of overflow for constant α_4 was generally higher for strategies 2 through 6 than for strategy 1. This is due to the increased attenuation of the hydrograph caused by the additional control as it passes through the reservoirs. The overflow duration also increases as α_4 decreases which is caused by the reduction of allowable inflow to the interceptor.

Although much information is conveyed by the average values in Table III-1 and III-2, a more complete picture of the variation in system performance parameters is provided by their probability distributions. Another advantage of the simulation technique applied to long term data is that a good estimate of the probability distributions is obtained. The probability distribution is particularly useful in conveying the idea that since rainfall is a natural event, no practical design will eliminate overflows and that the effect of different design alternatives is to change the probabilities associated with the performance variables. The design decisions are then in terms of acceptable levels of probability that certain variables will be exceeded.

Cumulative probability distributions for the first two variables in Tables III-1 and III-2 for strategies 1 and 4 are shown in Figure III-1 and III-2. The average values in the tables are equal to the areas under the respective probability curves. It may be useful to fit theoretical distributions to these curves. This information would be useful in estimating probability distributions associated with other mean values of these parameters.

The Poisson distribution, which requires only the mean value of the variable, was found to describe the number of overflows per year very well. The probability density function for this distribution is given by

$$f(\tilde{n}) = \frac{\mu^{\tilde{n}} e^{-\mu}}{\tilde{n}!} \tag{4}$$

where $\tilde{n}$ = the number of overflows per year and μ = the mean value of n. This is discrete distribution and the resulting cumulative distribution function is computed as

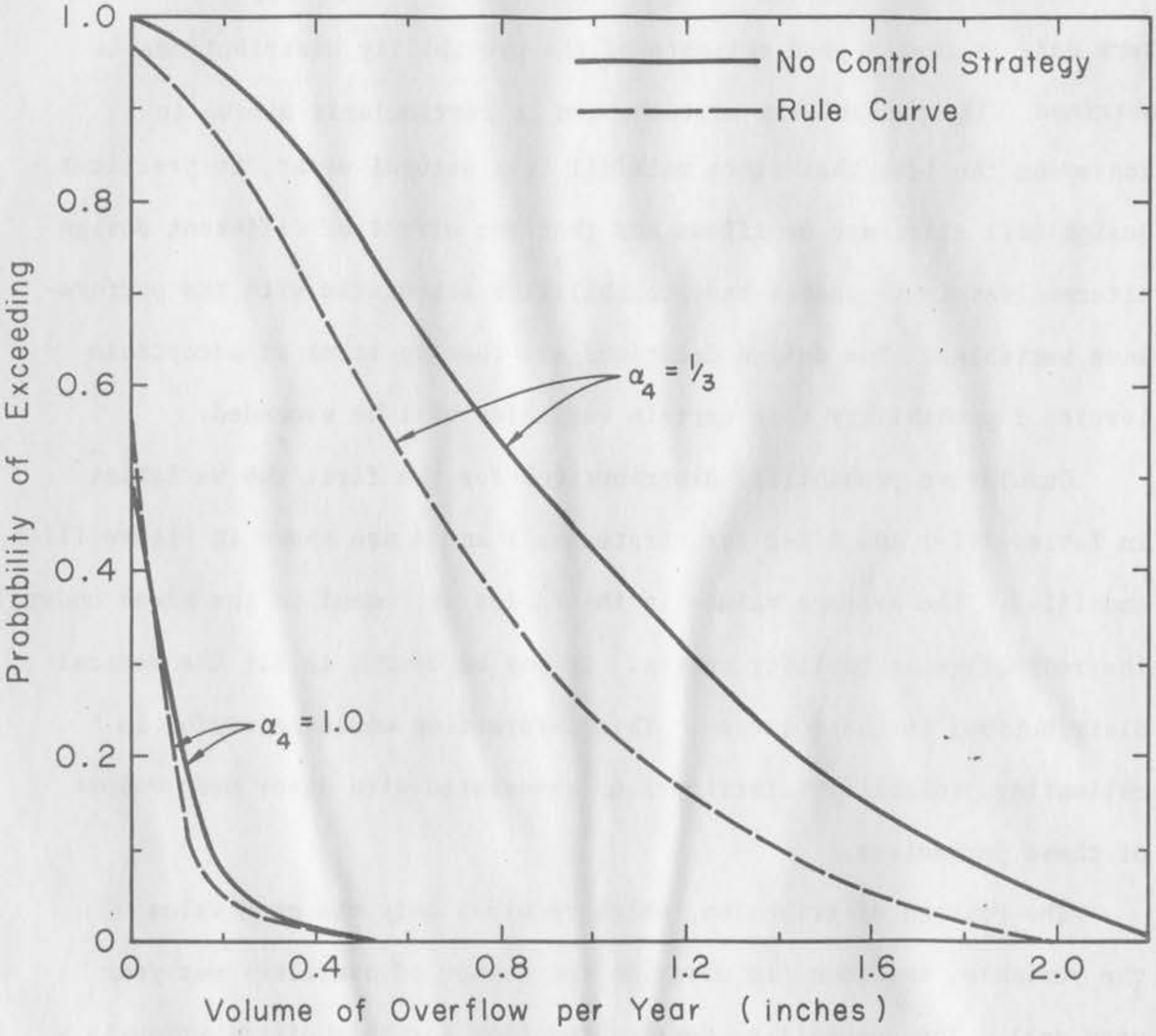


FIGURE III-1

PROBABILITY CURVES FOR
VOLUME OF OVERFLOW PER YEAR

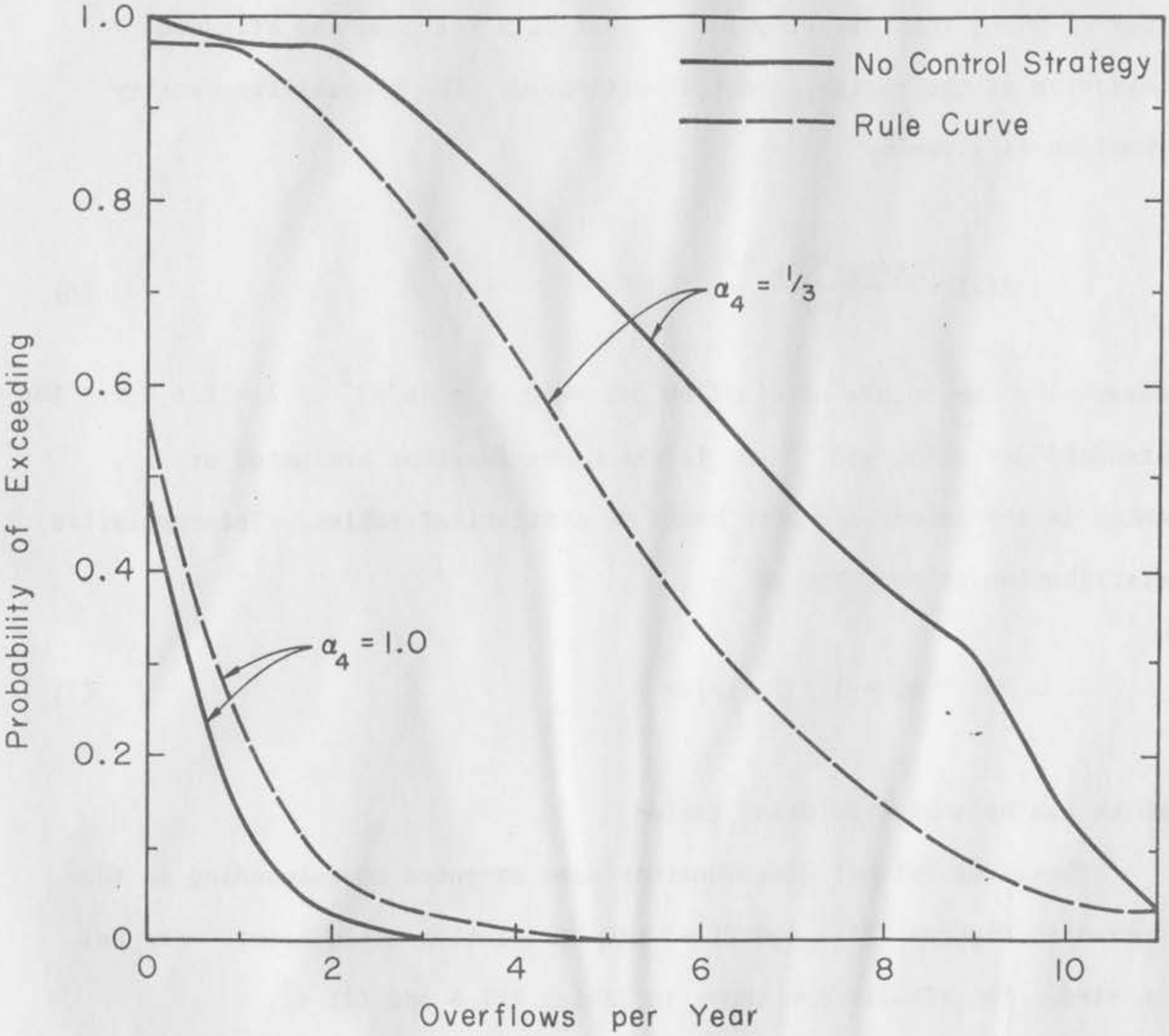


FIGURE III-2

PROBABILITY CURVES FOR
NUMBER OF OVERFLOWS PER YEAR

$$P[N > \tilde{n}] = 1 - \sum_{\tilde{n}=0}^{\tilde{n}} f(\tilde{n}) \qquad (5)$$

The volume of overflow per year is a continuous variable and the Gamma distribution was found to fit the data rather well. The disadvantage of using this distribution is that both the mean and standard deviation of the variable must be estimated. The probability density function is given by

$$f(x) = \frac{\lambda(\lambda x)^{k-1} e^{-\lambda x}}{\Gamma(k)} \qquad (6)$$

where x = the volume of overflow per year, $k = [\mu/\sigma]^2$, $\lambda = k/\mu$, σ = the standard deviation and $\Gamma(k)$ is the Gamma function evaluated at k , which is tabulated in common books of statistical tables. The cumulative distribution is computed as

$$P[X > x] = 1 - \int_o^x f(x)dx \qquad (7)$$

which can be evaluated using tables.

These theoretical distributions were computed corresponding to the curves in Figures III-1 and III-2 and correlation coefficients were calculated. The results are shown in Tables III-3 and III-4.

Table III-3

Correlation of Overflow Volume per Year
with Gamma Distribution

Strategy	α_4	μ [in/yr]	σ [in/yr]	k	λ [in^{-1}]	$\Gamma(k)$	correlation coefficient
1	1.0	0.058	0.102	0.323	5.572	2.769	0.976
4	1.0	0.036	0.074	0.240	6.675	3.786	0.957
1	1/3	0.953	0.558	2.912	3.056	1.847	0.996
4	1/3	0.693	0.469	2.188	3.152	1.095	0.997

Table III-4

Correlation of Number of Overflows per Year
with Poisson Distribution

Strategy	α_4	μ	Correlation Coefficient
1	1.0	0.64	0.999
4	1.0	0.92	0.994
1	1/3	7.39	0.996
4	1/3	5.61	0.998

The correlation coefficients are all above 0.95 indicating that the distributions fit the data well.

C. Techniques for Developing Control Levels for the General Strategy Under Study

Once the general control strategy is selected, which in this case is described in Section B.2, the problem then becomes one of deciding specific values for the strategy parameters. For the strategy under consideration the choice of control level, i.e., values of α , must be made. Two techniques were used. The first was an empirical approach which involved the evaluation of limiting depths which would just cause overflows for storms of various durations. This is termed the zero overflow curve approach. The second technique involved the application of an optimization scheme and in retrospect was clearly the better approach. Both techniques were based on uniform intensity storms and then adapted for use with non-uniform historical storms.

C.1 Zero Overflow Curve Technique

Of fundamental importance in developing control strategy is a method of determining if a particular storm will cause an overflow and if so the volume of that overflow. The Vicente simulation model can provide that information. However, to avoid the necessity of using the model for each storm of interest and to gain insight into the nature of overflow producing storms the concept of a graphical representation on a depth-duration plot of the boundary between storms which would and would not produce overflows is useful. This boundary is called a zero overflow curve. If historical storms were used to determine this curve, it would not be unique because of the temporal non-uniformity of the storms. Therefore, in order to establish a unique zero overflow depth for each duration only uniform intensity storms were considered.

The procedure followed was to select a set of storm depths at each of a number of durations and for each of these determine the overflow volume for a set of α values using the Vicente simulation model. In all cases the maximum allowable outflow from reservoir 12-2 was 0.3 in./hr., i.e., $\alpha_4 = 1.0$. For each duration, each value of α , and each reservoir a plot of overflow volume vs. storm depth was made and a curve drawn from which the storm depth at which the overflow vanished could be obtained. These curves were linear so interpolation was easy. Then a plot of α vs. overflow volume at constant duration was made for all four reservoirs. The value of α corresponding to the minimum overflow from any reservoir was then chosen as a *most favorable value*, as shown in Figure III-3. The value of the overflow volume expressed in inches is the ordinate on the zero overflow curve for that duration. The resulting curves for alternates B and D are shown in Figure III-4 and the zero overflow curve for alternate B is shown in Figure III-5.

The most favorable values of α as a function of storm duration then form a control level policy for uniform storms. The application is discussed in Section D.

It should be emphasized that this approach represents an initial attack on the problem. The development of curves such as shown in Figure III-1 required considerable effort. Although the results are useful in providing insight into the problem, the desired control level policy could have been achieved much more easily using the optimization technique described in the following section.

C.2 Optimization Technique

The Vicente simulation model can be viewed as a basic tool in directly obtaining the optimum control level policy. A simple search scheme was used to determine α^* for the three upstream reservoirs as a function of total depth for uniform intensity storms. The use of the same value for α^* for each upstream reservoir is reasonable since the reservoir volumes are approximately proportional to the respective drainage areas and the inflow hydrographs into the reservoirs are all of similar shape. A flow chart describing this scheme is shown in Figure III-6. The overflow volume for a given storm was computed for increasing values of α until a minimum was reached. A typical curve

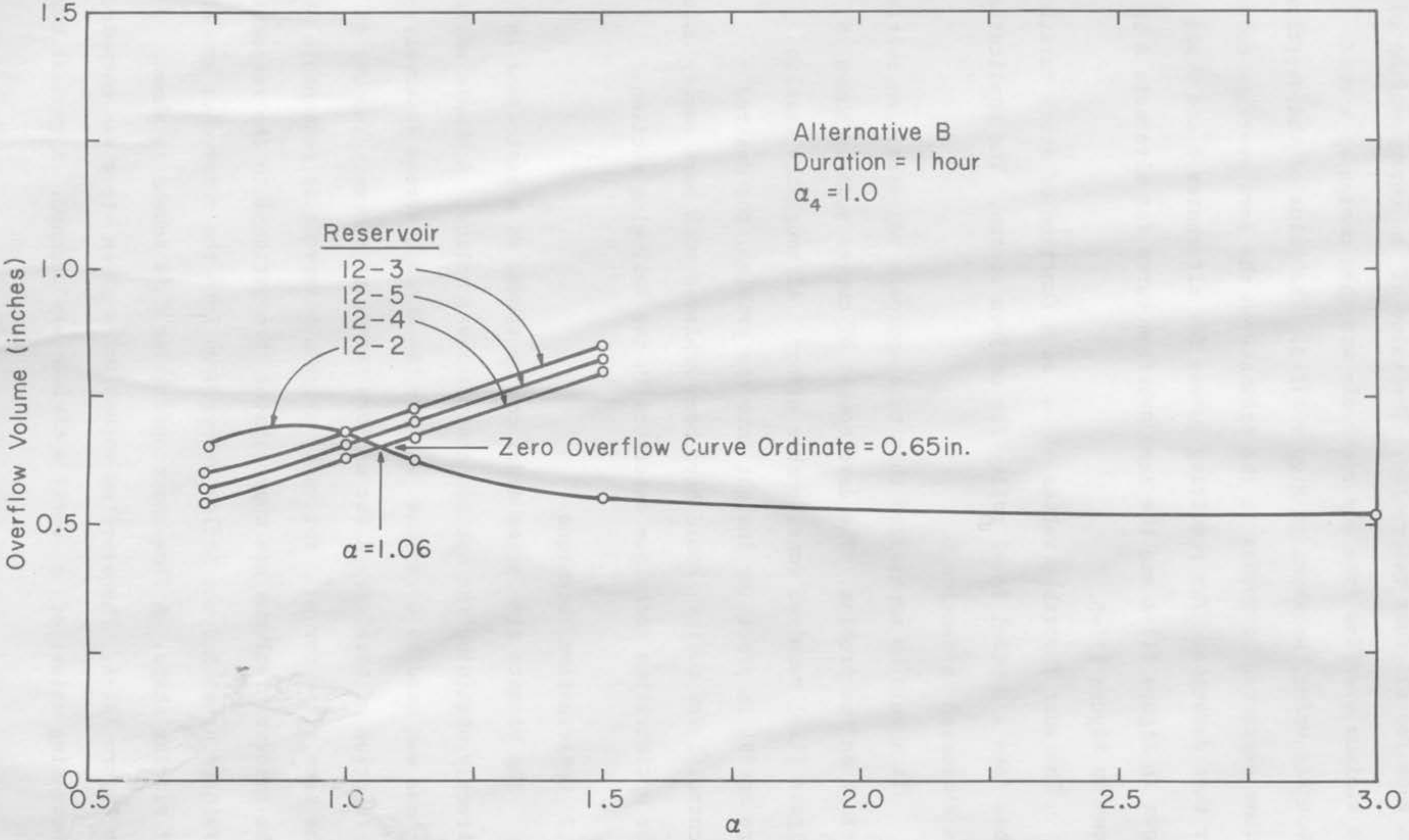


FIGURE III-3

CONTROL LEVEL VS. OVERFLOW VOLUME

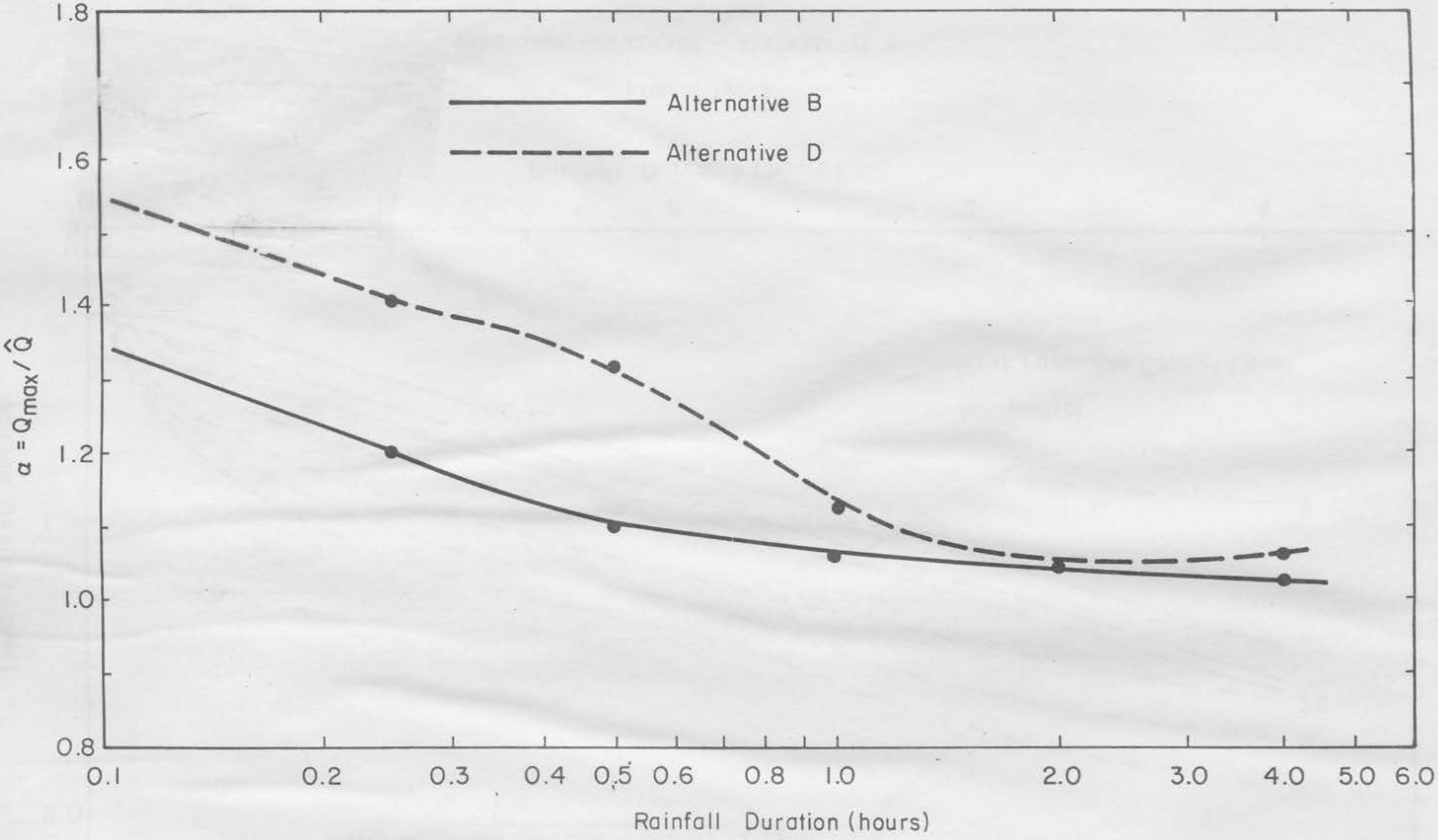


FIGURE III-4

MOST FAVORABLE CONTROL LEVEL VS. DURATION

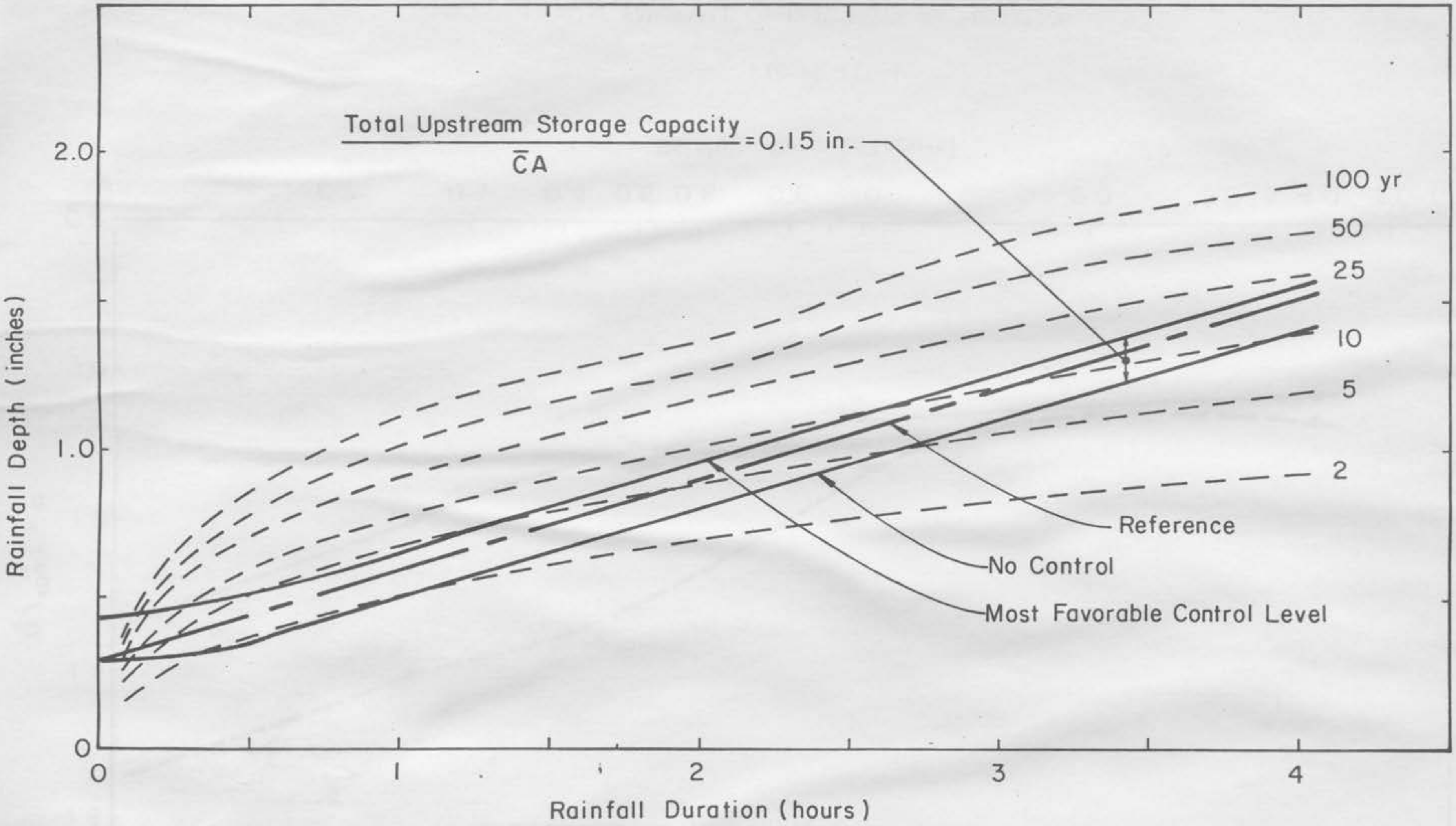


FIGURE III-5

ZERO OVERFLOW CURVES - ALTERNATIVE B

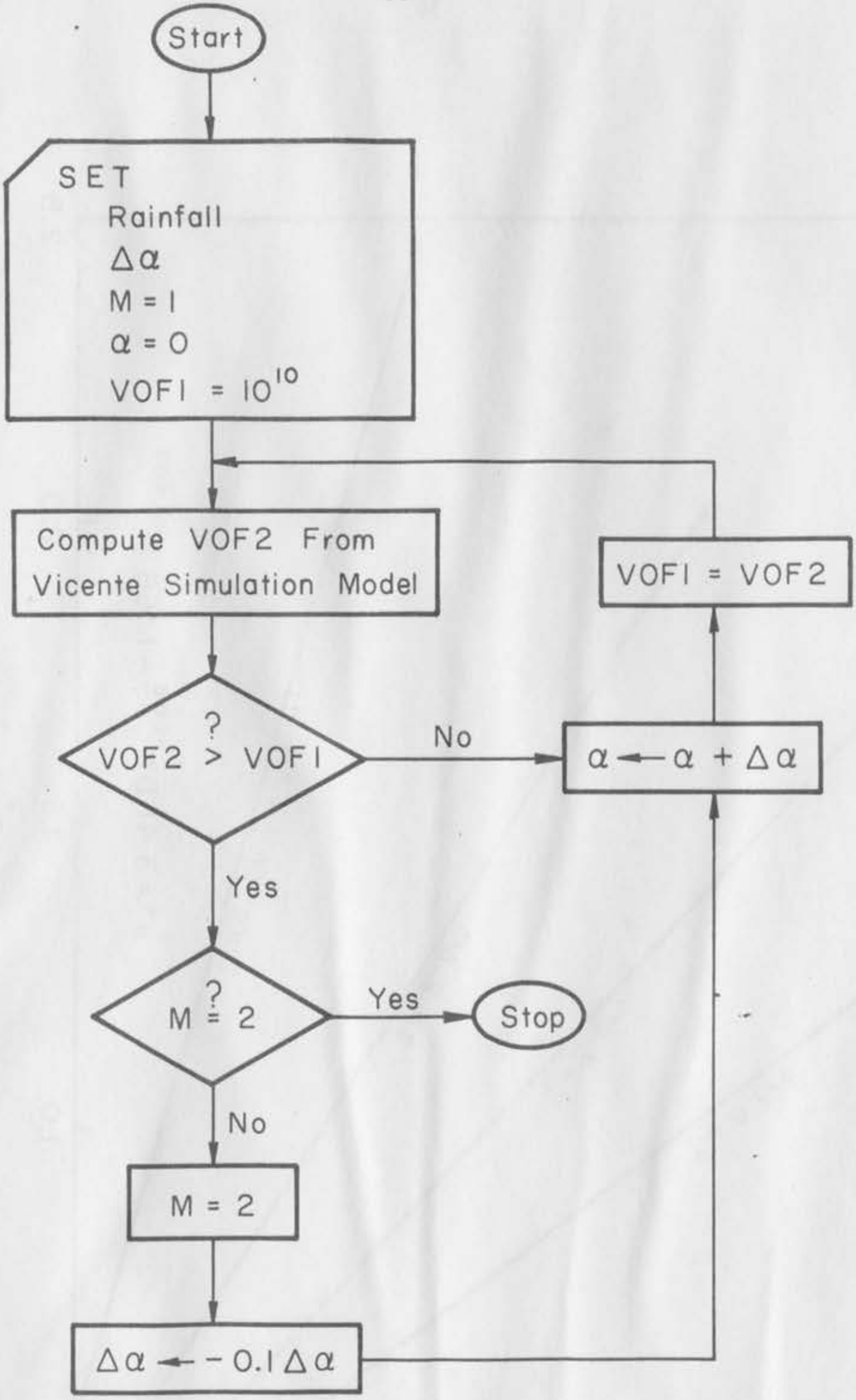


FIGURE III-6

FLOW CHART FOR OVERFLOW OPTIMIZATION

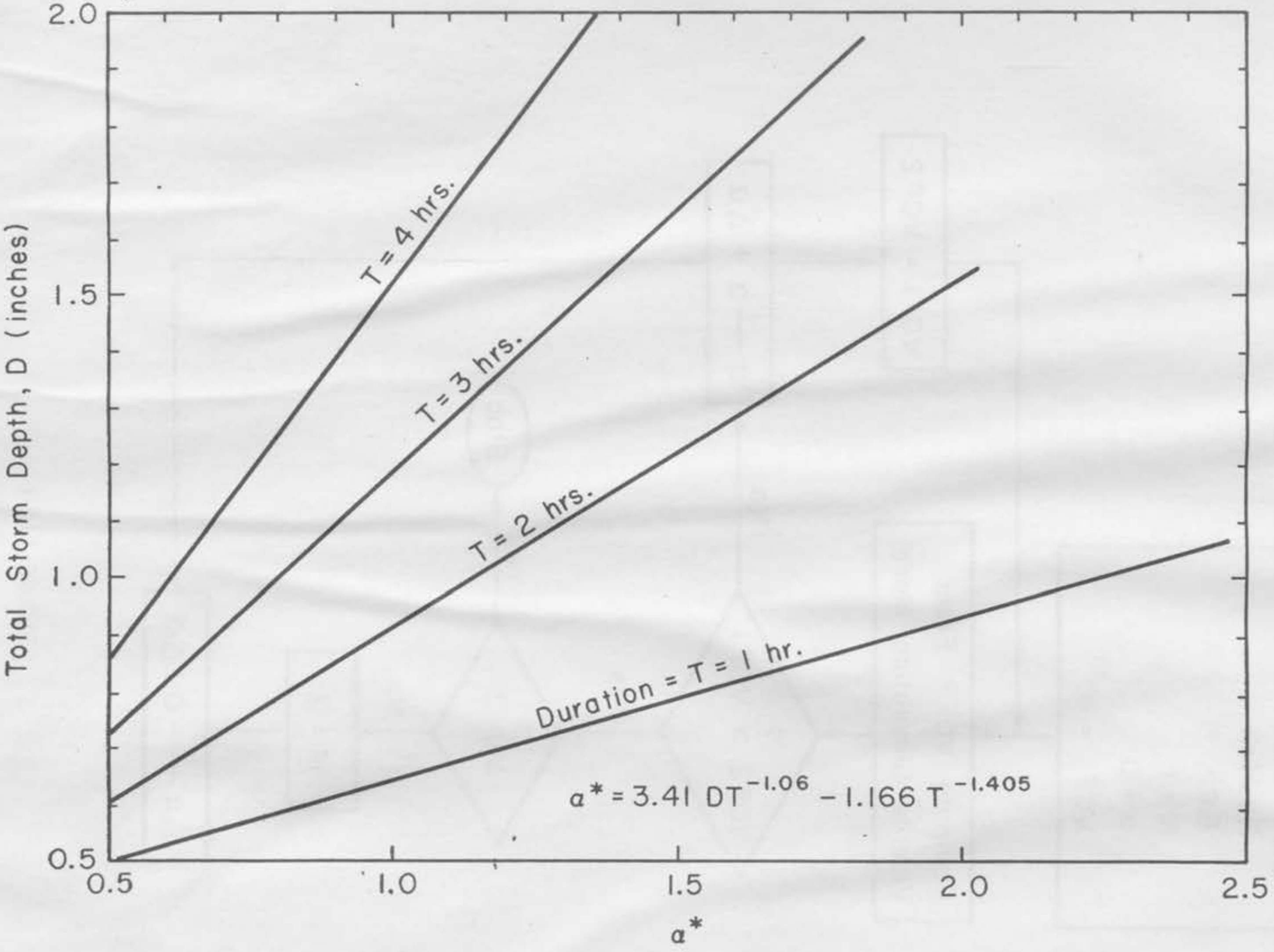


FIGURE III-7

OPTIMAL CONTROL LEVEL FOR
UNIFORM INTENSITY STORMS

of overflow volume vs. α for uniform storms has a single, well-defined minimum, unless the overflow volume reaches zero, within the feasible range of α between zero and 3.0. Therefore the search scheme worked well and curves of the optimum control level, α^*, as a function of storm depth for various durations could be obtained. The results for both $\alpha_4 = 1.0$ and 1/3 plotted as straight lines as shown in Figure III-7. These lines can be expressed by a single equation.

$$\alpha^* = 3.41 \, D \, T^{-1.06} - 1.166 \, T^{-1.405} \tag{8}$$

where D = total storm depth in inches and T = storm duration in hours.

A plot of overflow volume vs, storm depth using the optimal control levels is shown in Figure III-8 for $\alpha_4 = 1.0$ and 1/3. The intercepts of these curves on the depth axis are the ordinates for the optimum zero overflow curves for these values of α_4. This can be seen by comparing the values on the most favorable control level curve of Figure III-4. However, in this case these results are a by-product of the technique rather than the first objective as was the case in the previous section.

A comparison of the two techniques shows that the optimization approach is far superior. It produced more general results, i.e., a control level policy for both $\alpha_4 = 1.0$ and 1/3, with less effort than the overflow curve approach.

D. Techniques for Evaluation of Effect of Control Strategy on Vicente System Performance

D.1 Rainfall Data

Any technique for system performance evaluation requires rainfall data of some type. Since a long term rainfall record (66 years) from

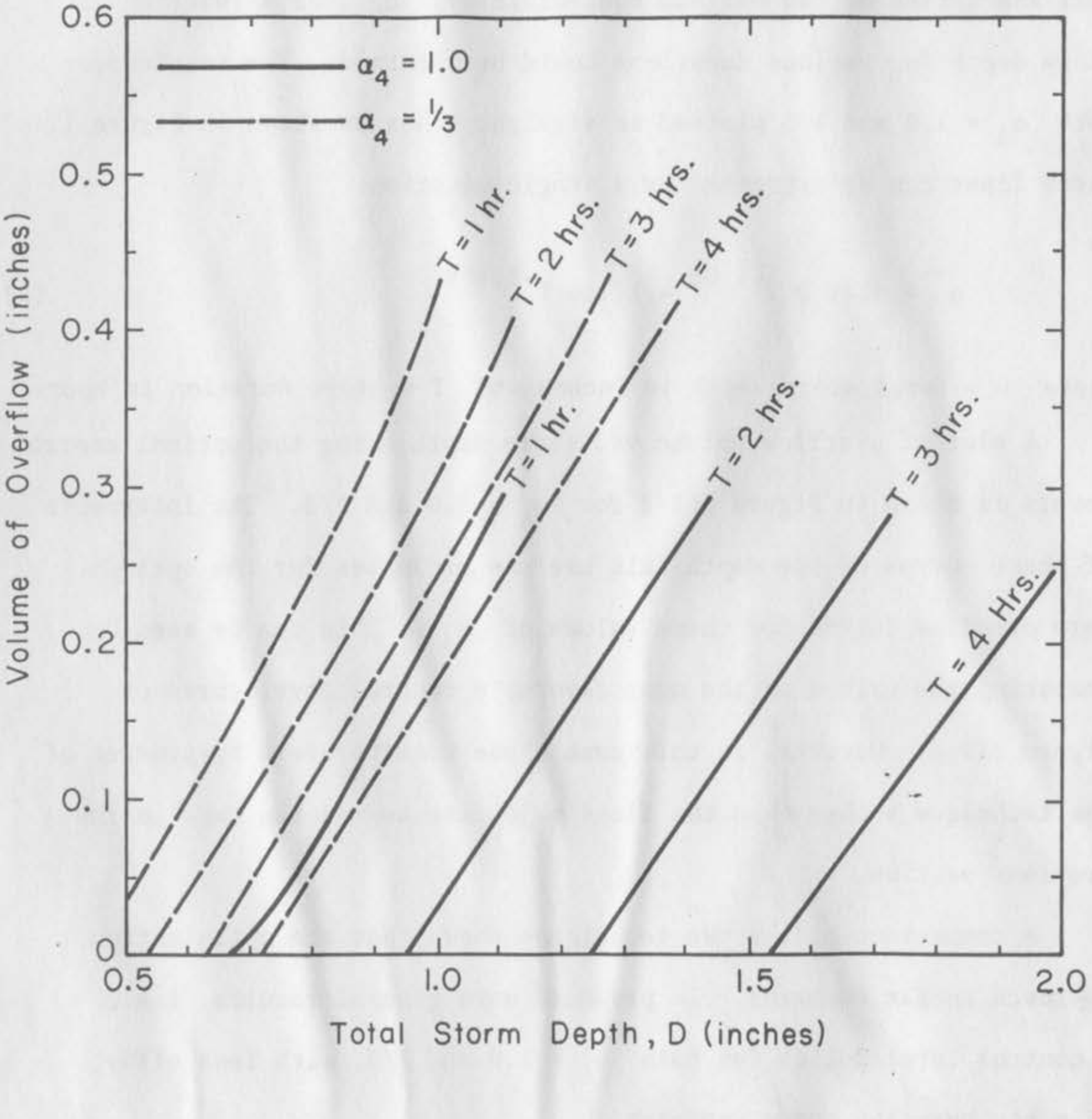


FIGURE III-8

OVERFLOW VOLUME FOR UNIFORM INTENSITY STORMS USING OPTIMAL CONTROL LEVEL the gage on the Federal Office Building in San Francisco was available it was decided to use this data even though it was on an hourly basis only. As data from the new raingage system accumulates it will be of greater value since storms with durations less than one hour will be well defined. Comparison on the basis of individual storms could be misleading and therefore the average values and probability distributions which result from the use of a long term record outweigh the advantage of analyzing a small number of well-defined storms.

The rainfall record is essentially a continuous one. However, a continuous simulation over this period of time would be prohibitive because of computer time costs. Therefore it was decided to run the simulation only during storm periods. Since the model used constant runoff coefficients the only potential problem this created was the case of storms so close in time that the reservoirs would not have an opportunity to drain before the next storm began. A rough hydraulic analysis indicated that 3 hours would be sufficient drainage time. Therefore a storm was considered as terminated if three successive hours of zero rainfall occurred following any non-zero hour.

In order to investigate the statistical effects of various assumptions concerning precipitation data several criteria were investigated. The results are summarized in the MWIS Phase III report, Table IV-4. The principal conclusion is that the assumption that all hourly precipitation values less than or equal to 0.05 inches can be safely ignored without significantly affecting the results of the simulation. This means that the number of storms in the 66 year record was reduced by 44 percent and that none of the storms thereby eliminated would generate overflows. The only statistical parameter that this would affect is the probability of overflow from any storm which would be increased by the above percentage. Therefore, all semi-continuous simulation work was done using the above storm definition criterion.

D.2 Zero Overflow Curves

In order to evaluate overflows for the historical rainfall record using this approach it was necessary to develop an overflow criteria for non-uniform intensity storms and then to estimate the overflow volumes. Since the overflow curves were developed for uniform intensity storms any such criteria will result in some error. The criteria adopted was that if the mass curve for the storm rose above the zero overflow curve at any time or if the rainfall during any hour was greater than the overflow ordinate at the first hour then an overflow was assumed to occur. In that case the volume of overflow was computed using the maximum difference between the mass curve and the zero overflow curve at any time. This difference was assumed to be proportional to the overflow volume using curves generated from the analysis of uniform intensity storms. The curves are shown in Figure IV-9 and IV-10 of the MWIS Phase III report.

This technique has several disadvantages in comparison to the semi-continuous simulation technique. First, a zero overflow curve must be developed for each design alternative, control strategy and allowable interceptor flow. This makes the method prohibitive for evaluation of a large number of such cases because considerable effort is required to develop the overflow curves. Furthermore some error is introduced because of the adaptation of the overflow curves to non-uniform intensity storms. This error was not evaluated numerically but it could be significant, particularly in regard to overflow volume evaluation.

The use of zero overflow curves was an initial approach to the evaluation problem. It provided some insight but was definitely inferior to the semi-continuous simulation approach.

D.3 Semi-Continuous Simulation

This technique proved to be very useful in performance evaluation. It is termed semi-continuous because of the time gap between storms as discussed in Section D.1. It consists of three basic steps:

(a) Definition of storms from historical data and determination of specific control level for each.

(b) Evaluation of overflow volume using Vicente simulation model.

(c) After all storms have been processed a statistical analysis of the results is performed including determination of probability distributions for number and volume of overflow per year.

The program for step (a) is given in Appendix D using the control level strategy described by Equation (8). However, this equation was developed for uniform intensity storms and required some modification for use with the non-uniform historical storms. This was done by defining an *effective* duration and depth. These definitions were developed by selecting a series of historical storms and determining α for each using an optimization procedure similar to that of Figure III-2. However, since for some of the storms the curve of overflow volume (objective function) vs. α had more than one local minimum in the feasible range of α , this simple search procedure did not always yield the optimal value for α, and the entire objective function over the feasible range of α had to be examined. A set of the 16 largest overflow producing storms plus a set of 19 smaller overflow producing storms were selected as a basis for establishing a definition of effective duration and depth. It was found that the most intense period of continuous rainfall during a storm was the important portion of the storm in correlating the actual α^* to the value obtained from Equation (8). Therefore the following definitions were adopted:

1. The effective duration is the number of consecutive hours in any storm where $p_{max}/p_i \leq r$, where p_{max} is the maximum and p_i is any hourly rainfall during a storm and r is a constant. Values of r of 1.6 and 2.0 were used.
2. The effective depth is the total rainfall which occurred during the effective duration.

These definitions, when applied to Equation (8), produced excellent estimates of α^* except for small storms which resulted in $\alpha^* < 0.5$. The objective function for these small storms usually was minimum at $\alpha \cong 0.5$ and therefore a minimum value of $\alpha^* = 0.5$ was used in cases where Equation (8) resulted in a lower estimate.

It must be pointed out that this adaptation of the rule curve to non-uniform storms means that the resulting control level is suboptimal in the strictest sense. However, for practical purposes the results are very close to optimal, particularly for the large storms. Modification of the minimum value for α^* would result in the elimination of overflow from some of the smaller storms which, when using $\alpha^* = 0.5$, produce very small overflows. This adjustment was not done, however, because the results of the same rule curve strategy for $\alpha_4 = 1.0$ and 1/3 was desired so that the variation with α_4 could be seen.

The results for the *most favorable control level strategy* curve of Figure III-4 were obtained by fitting an equation to the curve of the form

$$\alpha = 1.0 + 0.147e^{-.1226D} \qquad (9)$$

where D = the total storm duration in hours regardless of how non-uniform the rainfall intensity was. Equation (9) was then used in step (a) to determine the specific control level for that storm.

The statistical analysis consisted of determining the number and volume of overflows for each year from 1907 to 1972 inclusive, computing average values over this period for a number of variables, and determining cumulative probability distributions as described in step (c) above. It should be pointed out that the number of overflow events per storm is limited to one even though it is possible for overflow to start and stop again during a storm. A FORTRAN listing of the statistical analysis program which includes the Vicente model as a subroutine is given in Appendix B. This listing includes the logic needed to implement Equation (5) as the control level strategy.

It is concluded from the experience gained in using this technique that it is greatly superior to that described in the previous section. The question of whether a particular storm produces overflow and the value of that overflow is determined directly by the Vicente model. Furthermore the rule curve is applicable to both $\alpha_4 = 1.0$ and 1/3 and presumably to values within this range as well, thereby making it of more general value. Finally, it is relatively easy to investigate different strategies simply by changing the logic in step (a), the other steps remaining unchanged.

CHAPTER IV

LARGE-SCALE SYSTEM CONTROL

The development of a control strategy for the entire reservoir system is the long term goal of this line of study. It would be optimistic indeed to expect this goal to be achieved by this initial project. However, considerable effort has been devoted to the large-scale problem and this is discussed qualitatively in Chapter III of the MWIS Phase III report.

Because of the size and complexity of the total reservoir system, some type of formal approach to the control problem is necessary. Because the control strategy should, in some sense, make the best use of the storage capability, it is logical to consider optimization techniques as useful tools. However, the direct application of such techniques to a system of the size of the San Francisco Master Plan would be infeasible because of the computer requirements. Therefore, special methods developed specifically for large-scale system optimization must be employed. There are many such methods. All of them break the total system down in some way and consider the total problem as a group or series of smaller system problems which are connected or related. The smaller problems are then solved while maintaining their relationship to the total problem. Two such techniques are discussed below.

A. Decomposition

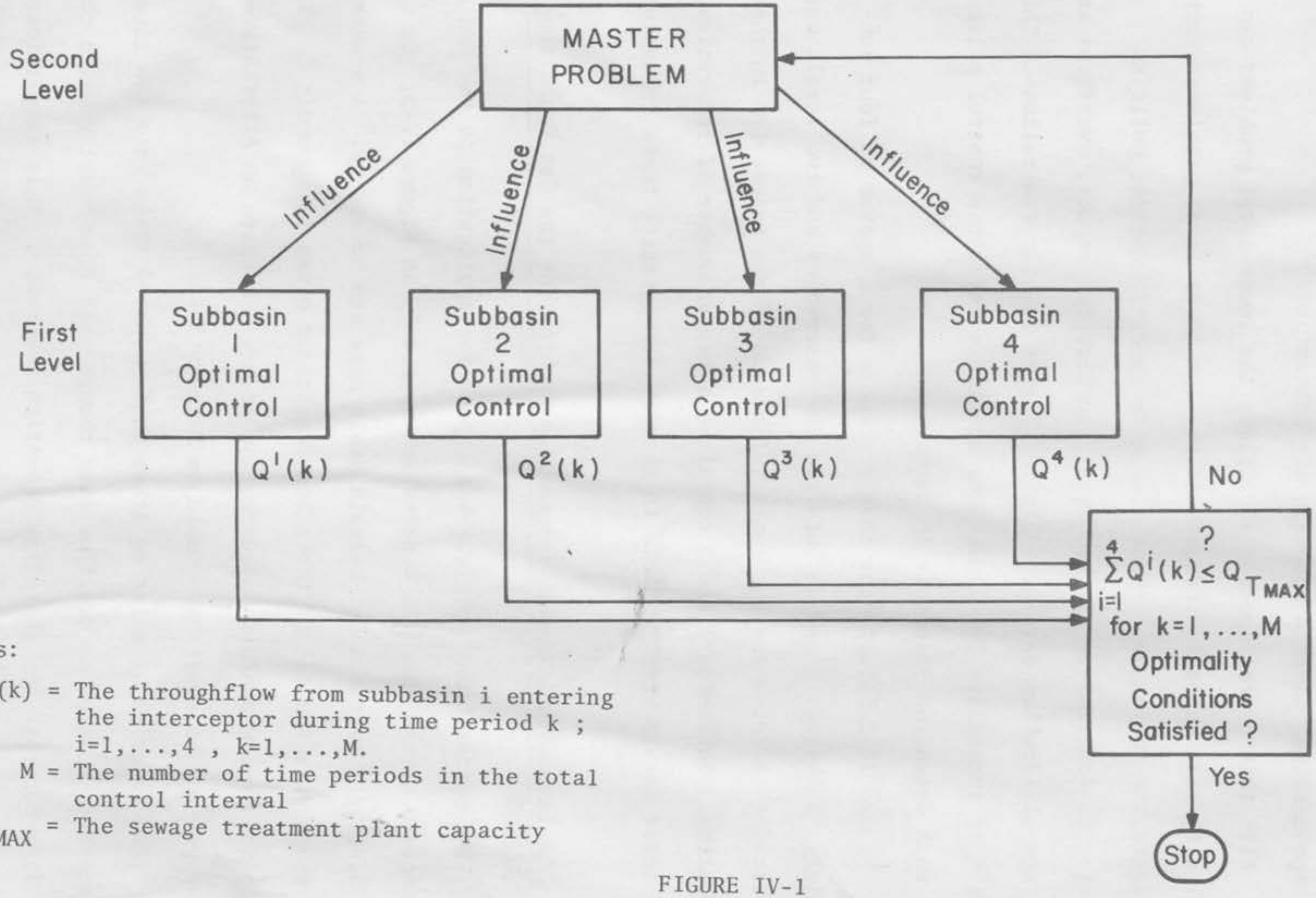
Decomposition is a methodology whereby a large system is decomposed into several subsystems which are *mildly interlinked*. The subsystems are treated independently then recombined by a master program in such a way as to achieve an overall optimum strategy.

In the case of a combined sewer system the subsystems are called *subbasins*. A subbasin is defined here as an area which is tributary to a particular trunk sewer which flows into the interceptor sewer. An interceptor sewer delivers sewage directly to the treatment plant (i.e., sewage in the interceptor sewer cannot be diverted into a detention reservoir). The only interlinking between subbasins, therefore, is the treatment plant and the interceptor sewers. This minimal degree of interlinking between subsystems makes decomposition a feasible method for analysis of a combined sewer system.

Decomposition applied to a sewer system of this type would involve separate determinations of optimal control for each subbasin. The master problem would then check to see if the interlinking constraints (interceptor and treatment plant capacities) and optimality conditions are satisfied. If they are not, another iteration or cycle would take place in which the master problem would adjust influences on the subbasin problems and the subbasin problems would be solved again. Iterations would continue until an optimal solution for the entire system was determined. Figure IV-1 illustrates this two-level approach for a system which has been decomposed into four subbasins. A more detailed account of decomposition is contained in reference [7].

B. Aggregation

Another multi-level approach which is applicable to the combined sewer control problem is aggregation [10]. Here, the highest level problem, where the individual reservoirs in each subbasin are aggregated together so as to represent one large reservoir, determines the overall policy for each subbasin. At this level, only the overflow from each



Notes:

$Q^i(k)$ = The throughflow from subbasin i entering the interceptor during time period k ; $i=1,\ldots,4$, $k=1,\ldots,M$.

M = The number of time periods in the total control interval

Q_{TMAX} = The sewage treatment plant capacity

FIGURE IV-1

DECOMPOSITION APPLIED TO A LARGE-SCALE COMBINED SEWER SYSTEM subbasin, the throughflows into the interceptor, and the total detention storage utilized in each subbasin are determined.

With this overall policy specified, the lower-level problems can deal with each subbasin independently. The lower level problems further disaggregate the subbasins and find more specific control policies within the constraints of the overall interceptor inputs, overflows and storage utilization determined in the higher level optimizations. This series of successive level problems continues until the control policy for each detention reservoir is determined.

Figure IV-2 illustrates this procedure for a system of four subbasins. Subbasins 2 and 4 contained few reservoirs and two levels were sufficient to determine the control for each of the reservoirs in these subbasins. Subbasins 1 and 3 contained a larger number of reservoirs, and three levels were required to totally disaggregate these subbasins.

C. The Large-Scale Linear Programming Problem for the San Francisco System

The aggregation technique was chosen for application to the San Francisco system which is modeled schematically in Figure IV-3. In order to develop a system model certain basic data are necessary. A summary of these data for all of the subcatchments is given in Appendix C. The proposed system contains 58 detention reservoirs based on Alternative C storages and 56 reservoirs based on Alternative B.

Flow carried by existing lines past proposed lines into the interceptors is modeled as overflow even though this flow usually has a chance to be intercepted by shoreline detention reservoir. This simplification is believed to be justified since the shoreline reservoirs and pumps and lines leading from these reservoirs to the interceptor are sized only

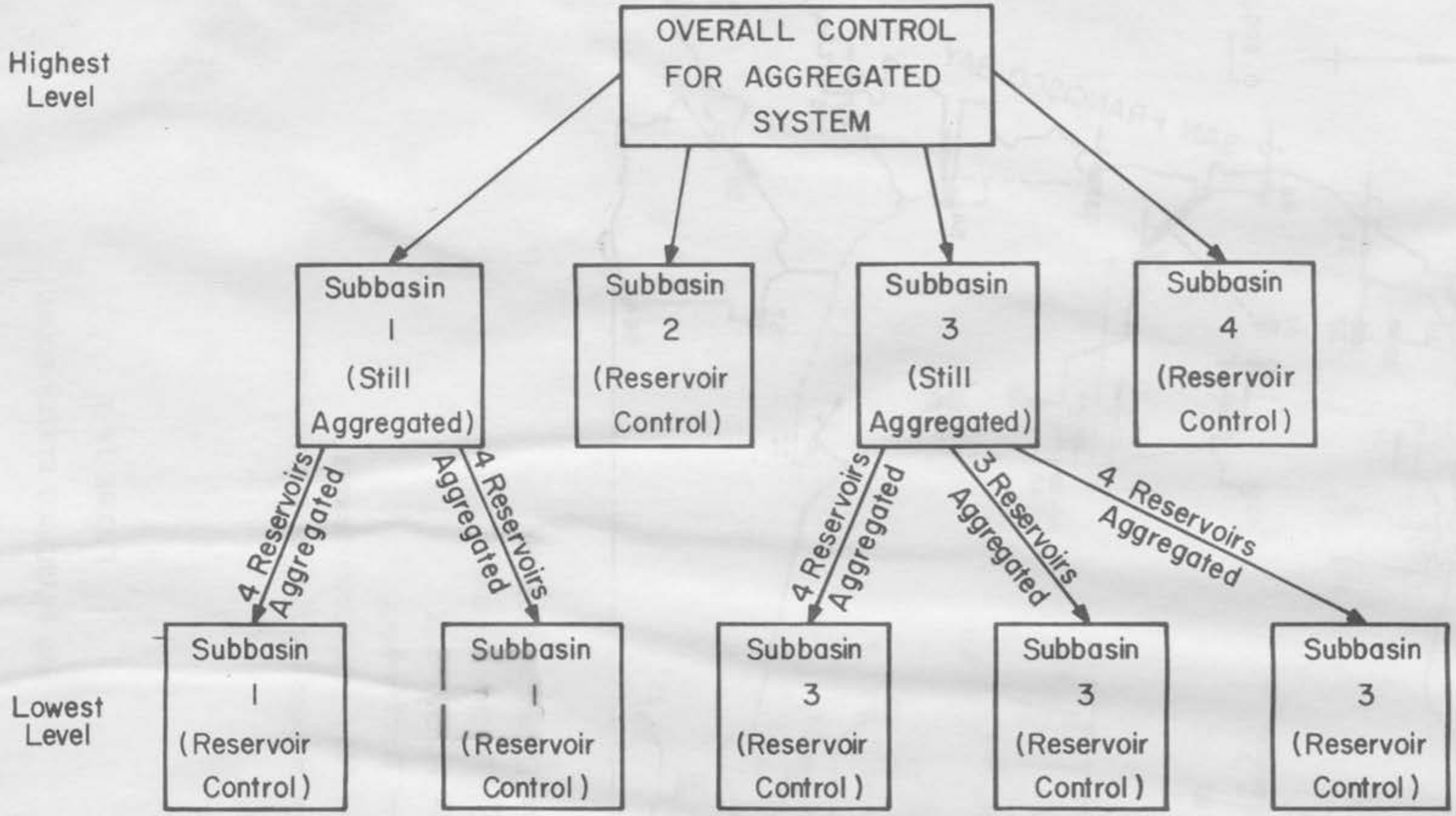


FIGURE IV-2

AGGREGATION TECHNIQUE APPLIED TO
A LARGE-SCALE COMBINED SEWER SYSTEM

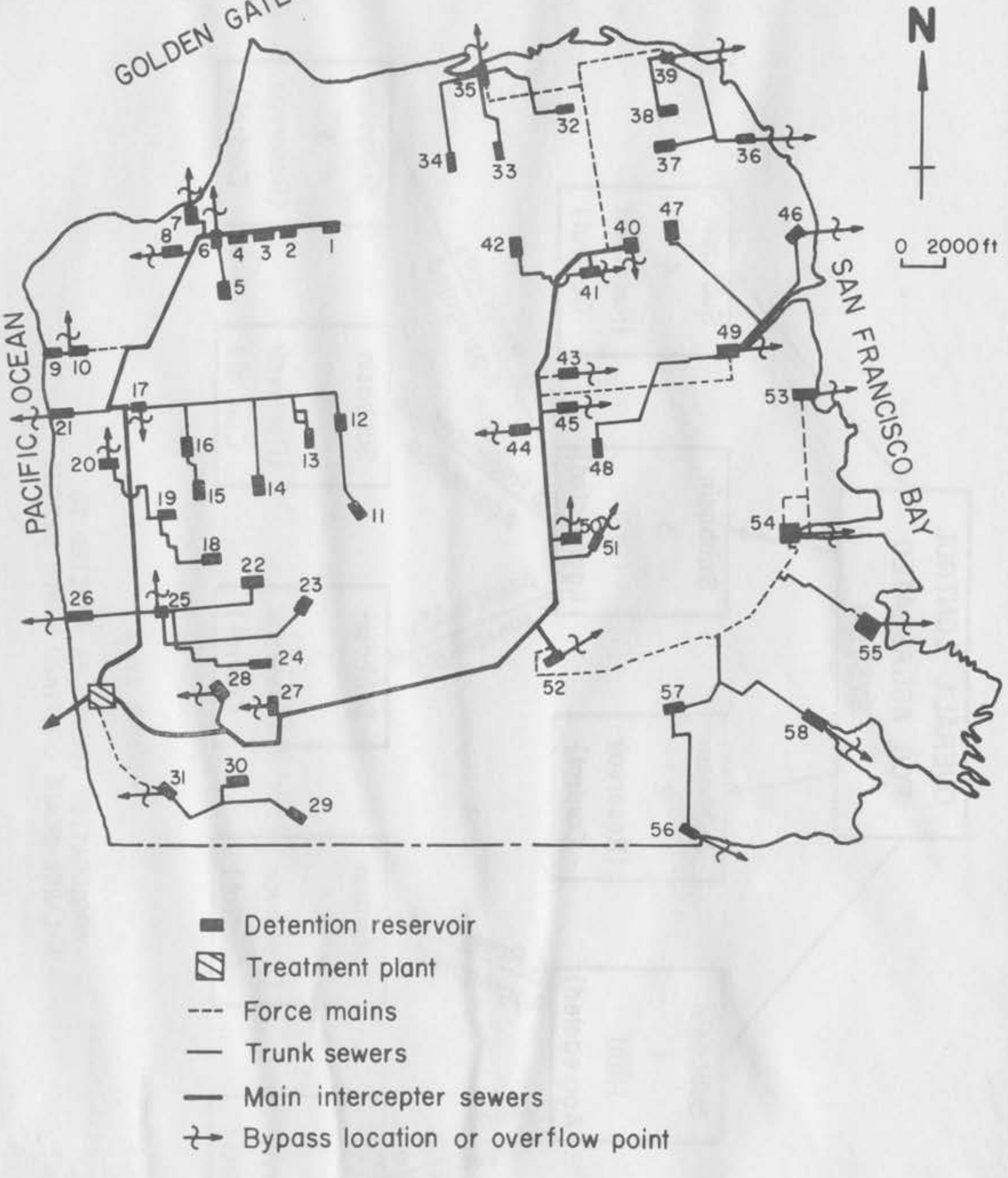


FIGURE IV-3

SAN FRANCISCO SYSTEM MODEL

for the downstream tributary area.

It is now necessary to choose the number of time increments to be considered and the total time period of analyses. From Appendix C it is seen that travel times throughout the system range from about 10 minutes to 150 minutes. All inflow hydrographs are lagged by their travel time. Therefore, after the input hydrographs have been lagged the decisions regarding the most upstream reservoir do not begin until time t=150. Of course, one would wish to consider a period of predicted input from this subcatchment. This means that the total time period of analysis must begin by time t=10 and end after time t=150. This is in terms of the time as viewed from the treatment plant (i.e., actual time plus travel time). The time interval chosen was from t=10 minutes to t=190. This time period is discretized into 9, 20 minute periods for formulation as a linear programming problem.

These values and the system model define the entire large-scale linear programming problem. Those constraints which are redundant are ignored. For instance, constraints on flowrates in the interceptors are not considered since either the constraints on subbasin flowrates into the interceptor or the treatment plant capacity constraint is more restrictive. The resulting large-scale linear programming problem is one of approximately 2000 variables and 1000 constraints.

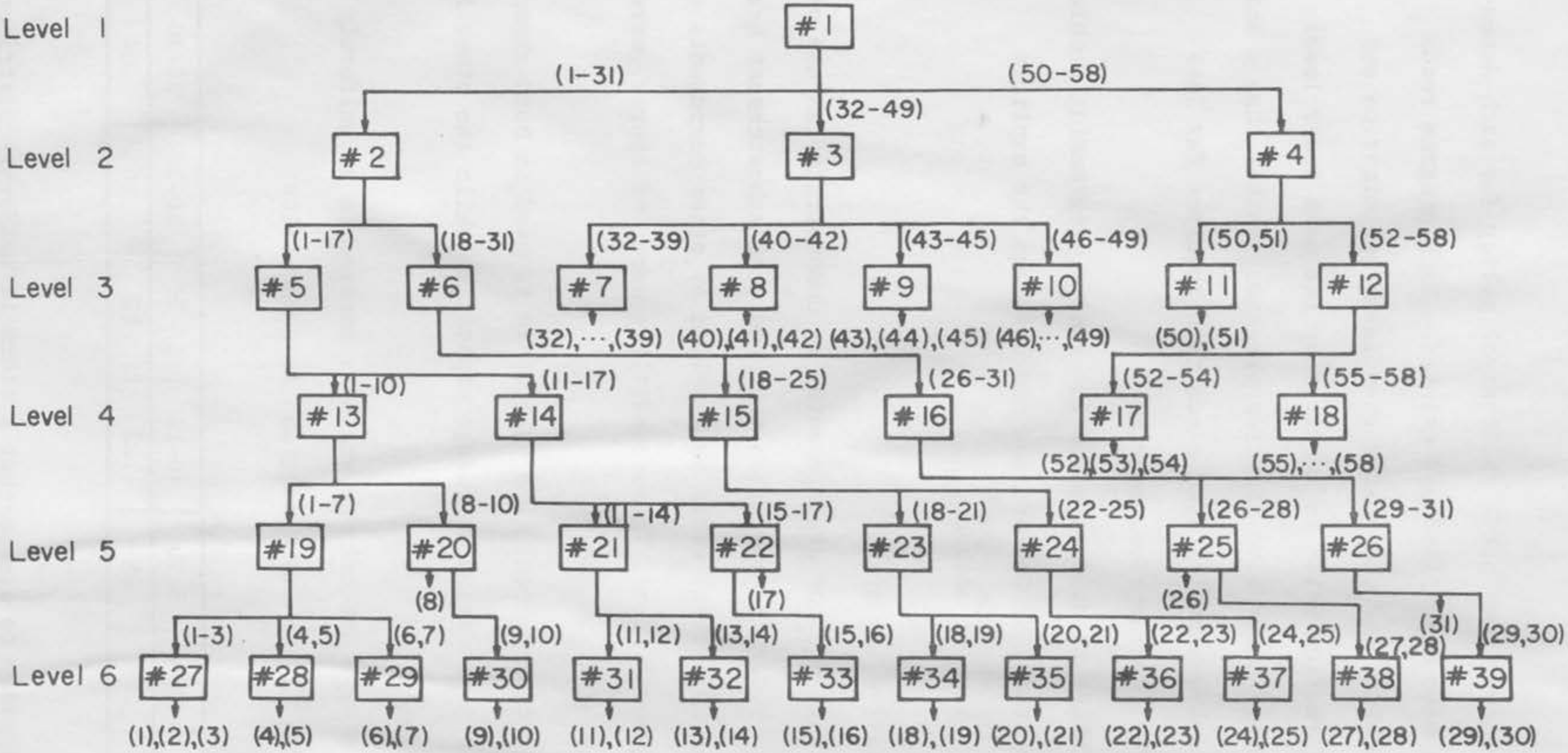
D. Multi-Level Aggregation of the San Francisco System Model

The large-scale problem is seen to be of enormous size. Therefore, it will be necessary to go through many successive levels of disaggregation in order to determine the control at each reservoir. The highest level problem divides the city into three sections corresponding to detention reservoirs 1-31, 32-49, and 50-58. The next lower level consists of three problems which further disaggregate these sections. In all, six levels and 39 linear programming problems are required.

Figure IV-4 describes the various levels and l.p.'s involved in this application of the aggregation technique to the San Francisco system. Each problem was formulated from the original large-scale problem.

FORTRAN IV programming language was used to develop the computer model for execution on the Colorado State University CDC 6400 computer system. The model consists of seven programs, AGREGAT, LEVEL 1, LEVEL 2, LEVEL 3, LEVEL 4, LEVEL 5, and LEVEL 6. AGREGAT reads in data which describe the system model and the initial state of the system (i.e., flow and storage constraints, travel times, initial storages, predicted hyetographs at the raingages, etc.). It then generates the lagged, discretized subcatchment hydrographs. All of the necessary information is then transmitted to temporary disc storage for use in the six remaining programs.

Program LEVEL 1 is executed next. It reads the information obtained from AGREGAT. In addition, it reads information which is particular to the highest (first) level optimization problem (i.e., number of variables, number of constraints, penalty coefficients for aggregated reservoirs, etc.). The objective function, A-matrix and B-vector of the first level optimization are then defined via FORTRAN programming, and a linear programming subroutine is called to solve the problem. The results of the problem are printed and the information required for the next lower (second) level problems is transmitted to temporary disc storage.



Notes: (1) Boxes refer to a particular linear programming problem.
(2) Numbers in parenthesis refer to the reservoir aggregation after that l.p. is completed.

FIGURE IV-4

APPLICATION OF MULTI-LEVEL AGGREGATION TO THE PROPOSED SAN FRANCISCO COMBINED SEWER SYSTEM

This procedure continues until the control policy for each detention reservoir has been calculated and printed. Each program reads information from AGREGAT and the next higher level optimization and transmits that information which is required by the next lower level problems. A series of successive computer programs, rather than a main program with many subroutines is used since this requires far less computer storage for the compiled program.

Subroutine SIMPLEX is used to solve the linear programming problems. It was developed by the RAND Corporation and utilizes the explicit inverse form of the simplex method.

D.1 Use of Aggregation Technique

The examples which follow are presented to demonstrate the use of this technique. In real-time operation the predicted subcatchment hyetographs and system conditions would be supplied by other components of the water intelligence system. These conditions were, of course, merely read in for these runs.

Two examples are presented. The same storm is used in both examples, but one is based on Alternative C storage capacities while the other is based on Alternative B.

The storm is a hypothetical one which is assumed to be uniformly distributed over the City. Its hyetograph is shown below:

Time (minutes)	0-5	5-10	10-15	15-20	20-25	25-30
Rainfall (inches)	.04	.08	.22	.12	.06	.04

It is not necessary to assume that a storm is uniformly distributed. The computer model starts with a separate predicted hyetograph at each of the City's 30 raingages. A separate hyetograph for each subcatchment is then determined as a weighted average of the hyetographs of the six closest raingages.

The complete output from each example run consists of the inflow hydrographs for each subcatchment and the results of each linear programming problem. This, of course, includes the control and storage policy for each reservoir from its beginning time interval through the final time interval. This output is quite lengthy and difficult to interpret. Therefore, efforts have been made to present these results in a condensed, interpreted form.

In these examples, the penalty coefficients on overflows, $P^i(k)$, and credit coefficients on throughflows entering the interceptor, $C^i(k)$, decrease as k increases so that no overflow will occur until the corresponding reservoir is full. These coefficients were not varied with respect to the location of the outfall for simplicity in analyzing results.

Example 1: Alternative C Storage

The storm used represents an intense rainfall (roughly a 5-year recurrence interval). Examination of the subcatchment inflow hydrographs indicates that significant local flooding and overflows would occur if system storage were not utilized. However, control of Alternative C storage was sufficient to completely eliminate overflows and street flooding.

The effects of real time control are shown below in units of inches of water over the entire drainage area:

Total Runoff = .378 in.

Total Overflow = .000 in.

Delivered to Treatment = .169 in.

Diverted to Storage = .209 in.

The control strategy determined was one which allowed zero overflows and maximized the delivery of sewage to the treatment plant. This can be seen from the results of the first level optimization shown in Table IV-1. The values listed under the columns labeled "B" are deliveries to the treatment plant which were already in the interceptor at the beginning of the storm. The columns labeled "A" represent results of the first level optimization. The values listed in columns labeled "B" represent flows that were already in the system at the beginning of the storm. These were assumed to be dry weather flows since they were released before the beginning of the storm.

Only the level 1 results are shown in Table IV-1. At this level the system is aggregated into three sections. Section 1 is the west side of the San Francisco and contains reservoirs 1-31. Section 2 is the northeast side and contains the subcatchments tributary to reservoirs 32-49. Section 3 contains reservoirs 50-58 and is located in the southeast section of the City. These sections correspond to the areas which are tributary to the existing Richmond-Sunset, North Point and Southeast treatment plants.

The results are presented at the level 1 degree of aggregation since showing the complete control policy would require 58 columns similar to the three columns of Table IV-1.

In periods 3 through 9 the total delivery to the treatment plant was 1550 cfs which is the plant's capacity. In the first two periods

TABLE IV-1

Example 1 - Level 1 Mass Balance

k		Section 1 Reservoirs 1-31		Section 2 Reservoirs 32-49		Section 3 Reservoirs 50-58		Total Delivery to Treatment
		A	B	A	B	A	B	
1	S F Q O	0.000 782. 462. 0.	36.		53.		67.	618.
2	S F Q O	0.387 4547. 1413. 0.	2.		53.		67.	1535.
3	S F Q O	4.147 3712. 1430. 0.	-		53.		67.	1550.
4	S F Q O	6.885 1102. 773. 0.	-		53.	0.000 680. 680. 0.	44.	1550.
5	S F Q O	7.279 198. 1113. 0.	-	0.000 45. 45. 0.	50.	0.000 3649. 342. 0.	-	1550.
6	S F Q O	6.181 103. 0. 0.	-	0.000 875. 265. 0.	40.	3.969 3290. 1245. 0.	-	1550.
7	S F Q O	6.304 92. 0. 0.	-	0.732 2913. 1488. 0.	9.	6.423 865. 480. 0.	-	1550.
8	S F Q O	6.415 91. 0. 0.	-	2.959 2613. 1488. 0.	-	6.886 273. 62. 0.	-	1550.
9	S F Q O	6.524 91. 0. 0.	-	4.309 1727. 1488. 0.	-	7.140 132. 62. 0.	-	1550.
	S	= 6.634		4.597		7.224		

Notes:

S = Diversions to storage (10^6 ft^3)
F = Runoff (cfs)
Q = Deliveries to treatment (cfs)
O = Overflow (cfs)
A = Values occurring after the beginning of the storm
B = Throughflows released before the beginning of the storm but arriving at the treatment plant after the beginning of the storm

the maximum delivery to the treatment plant was limited by a combination of total system inputs and individual line capacities. For instance, in period 1 the controlled releases into the interceptor (i.e., releases from reservoirs 20, 25, 26, and 31) totalled 462 cfs. Table IV-2 demonstrates that this is the maximum delivery possible from these reservoirs during time period 1.

As the various level problems are executed the releases shown in Table IV-1 are distributed in greater detail with the total release remaining the same. For example, the interceptor input from section 2 during time period 9 is shown in Table IV-1 to be 1488 cfs. The total 1488 cfs is distributed among four aggregated interceptor input points in the Level 2 problem pertaining to section 2. These are in turn distributed to the eight actual section 2 input points in four Level 3 problems. Level 3 is the final level required for these reservoirs. (See Figure IV-4). Other interceptor inputs would require six levels of disaggregation before they were distributed to the actual input points.

Similarly, the storage utilization from level 1 is allocated to specific reservoirs as the multi-level problems are executed. Table IV-3 shows the final storage in each reservoir and the totals are compared to the final storages in the three sections of level 1. Slight roundoff errors occur because of the passage of *rounded-off* information from one computer program to the next.

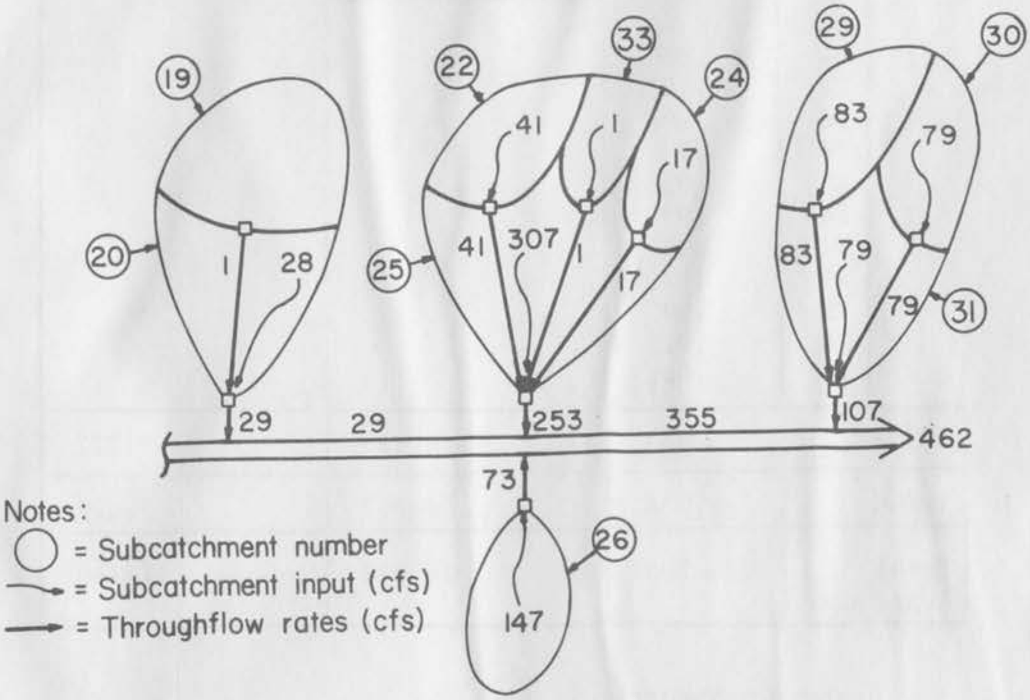
In this example, it is obvious that the control strategy determined would be optimal for the original large-scale problem since no street flooding or overflows occur and delivery to the treatment plant is maximized.

TABLE IV-2

Example 1. Flow Availability for Time Period 1

	(cfs)			
i	Q^i_{MAX}	$F^i(1)$	Flow Available	Maximum Subcatchment Release
19			1(1)	
20	85	28	29	29
22	82	41	41	
23	165	1	1	
24	170	17	17	
25	253	307	366	253
26	73	147	147	73
29	380	83	83	
30	432	79	79	
31	107	79	241	107
				Total = 462

Notes: 1. Dry weather release from previous time period
2. All flowrates are in cfs



SCHEMATIC REPRESENTATION

TABLE IV-3

Final Reservoir Storage versus Aggregated Reservoir Storages of Level 1

Example 1

Section 1 (Res's 1-31)		Section 2 (Res's 32-49)		Section 3 (Res's 50-58)	
i	$S^1(10)$ ($10^6 ft^3$)	i	$S^1(10)$ ($10^6 ft^3$)	i	$S^1(10)$ ($10^6 ft^3$)
1	.650	32	.376	50	.286
2	1.057	33	.142	51	.144
3	.240	34	.147	52	2.211
4	.026	35	.189	53	.329
5	.079	36	.609	54	2.604
6	.097	37	.132	55	.044
7	.161	38	.121	56	1.084
8	.063	39	.003	57	.109
9	.086	40	.173	58	.411
10	.206	41	.480		
11	.557	42	.006		
12	.203	43	.391		
13	.008	44	.082		
14	.007	45	.034		
15	.071	46	.361		
16	.007	47	.110		
17	.050	48	.222		
18	.180	49	1.019		
19	.172				
20	.135				
21	.836				
22	.187				
23	.398				
24	.167				
25	.032				
26	.098				
27	.117				
28	.526				
29	.197				
30	.012				
31	.008				
Totals	6.633		4.597		7.222
Level 1	6.634		4.597		7.224
Total Available	13.390		10.970		11.160

i = Reservoir Number

$S^i(10)$ = Final Storage in Reservoir i

Example 2: Alternative B Storage

In order to illustrate the aggregation technique in a situation requiring overflows, Example 2 is based on Alternative B storage. Reservoirs 2 and 38 do not exist in Alternative B. As these reservoirs are assumed to exist in the formulation of the various l.p.'s, it was necessary to specify their storage capacities to be zero. It is obvious that overflows will be required in this example since the total storage utilized in Example 1 (18.46×10^6 ft^3) is greater than the total system storage capacity of Alternative B (16.85×10^6 ft^3).

Table IV-4 presents the results of the first level optimization. Total throughflows are again maximized and are therefore identical to those of Example 1. Note that at this point it appears that the total system's storage capacity can be utilized since the final storage in each section is equal to that section's total storage capacity.

In this example, each level results in a slightly less desirable solution than that implied by the previous higher level. The exchange is storage utilization for overflow. This is illustrated in Table IV-5 in units of inches of water.

TABLE IV-5

Example 4 - Total System Mass Balance

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Total Runoff(in)	.378	.378	.378	.378	.378	.378
Delivered to Treatment(in)	.169	.169	.169	.169	.169	.169
Total Overflow(in)	.018	.025	.030	.034	.034	.035
Diverted to Storage (in)	.191	.184	.179	.176	.176	.174

TABLE IV-4

Example 2 - Level 1 Mass Balance

k		Section 1 Reservoirs 1-31		Section 2 Reservoirs 32-49		Section 3 Reservoirs 50-58		Total
		A	B	A	B	A	B	
1	S	.000						
	F	782.						
	Q	462.	36		53		67	618.
	O	0.						0.
2	S	.386						
	F	4547.						
	Q	1413.	2		53		67	1535.
	O	0.						0.
3	S	4.147						
	F	3712.						
	Q	1430.	-		53		67	1550.
	O	571.						571.
4	S	6.200				.000		
	F	1102.				680.		
	Q	1102.	-		53	352.	44	1551.
	O	0.			53	0.		0.
5	S	6.200		.000		.394		
	F	198.		45.		3649.		
	Q	264.	-	45.	50	1190.	-	1549.
	O	0.		0.		0.		0.
6	S	6.120	-	.000		3.345		
	F	103.		875.		3290.		
	Q	0.	-	92.	40	1418.	-	1550.
	O	36.		0.		201.		237.
7	S	6.200		940.		5.350		
	F	92.		2917.		865.		
	Q	92.	-	583.	9	865.	-	1549.
	O	0.		0.		0.		0.
8	S	6.200		3.741		5.350		
	F	91.		2613.		273.		
	Q	91.	-	1186.		273.	-	1550.
	O	0.		128.		0.		128.
9	S	6.200		5.300		5.350		
	F	91.		1727.		132.		
	Q	91.	-	1327.	-	132.	-	1550.
	O	0.		401.		0.		401.
S	S	6.200		5.300		5.350		

Notes:

S = Diversions to storage (10^6 ft^3)
F = Runoff (cfs)
Q = Deliveries to treatment (cfs)
O = Overflow (cfs)
A = Values occurring after the beginning of the storm
B = Throughflows released before the beginning of the storm but arriving at the treatment plant after the beginning of the storm

The optimal solution to the actual large-scale problem is known to be bounded by the results of the aggregation technique and the results implied by the first level optimization. In other words, the actual minimum amount of overflow required would not be less than 0.18 inches nor more than 0.35 inches.

E. General Comments on the Aggregation Technique

The foregoing results were obtained on a CDC 6400 computer. Approximately 64000 octal words of memory were required and execution time was about 90 seconds. It is believed that this indicates that the aggregation technique is a feasible method for real time operation. The use of minicomputers would require furthur modification to reduce the dimensionality of the various linear programming problems. It appears that this could be achieved by using an *upper bounding* code rather than the *explicit inverse* code to solve the linear programming problems. It is also believed that this change would result in a savings in execution time requirements. The method can also be used in a feedback mode so that control could be determined several times during a storm based on the most recent system data and storm predictions.

The foregoing examples and other examples not included herein indicate that many solutions obtained by the aggregation technique represent optimal solutions to the original large-scale problem. In others it is only possible to establish upper limits on the degree of suboptimality. However, comparison of various examples indicates that the solutions determined were not highly suboptimal.

CHAPTER V

SUMMARY AND CONCLUSIONS

A. Principal Conclusions

This report describes what might be regarded as an initial approach at control strategy development. Emphasis was placed at the subbasin level and the storm prediction problem was not considered. Only one general subbasin strategy was investigated but several techniques were employed to develop specific control level rule curves for that strategy. The system performance parameters used were number and volume of overflow. A semi-continuous simulation approach was employed using the 66 year San Francisco rainfall record as input.

The results showed that a 25 percent or better reduction in average number and volume of overflows per year can be achieved by a control vs. a no control strategy at the subbasin level. Furthermore, the maximum allowable outflow from the subbasin into the interceptor proved to be an important parameter in determining system performance. As this outflow is increased the performance parameters are substantially reduced. This leads to the conclusion that a good city-wide control strategy is one which makes full use of the storage capacity while at the same time maximizes inflow to the interceptors. These conclusions are subject to the uncertainty imposed by storm prediction capability. This is discussed in the following section.

Water quality parameters were not used in the simulation model. This was not regarded as a serious deficiency for the purposes of this report. However, since the objective of the system is to minimize the pollution of the receiving waters caused by overflows, the final form of the simulation model must include water quality parameters. Which parameters to include and the level of sophistication of the model will depend to some extent on details of water quality regulations which must be met. Since the regulations change with time this may be a difficult decision. However, the state-of-the-art of water quality modeling is rapidly advancing and it is unlikely that quality model generation will be much of a technical problem.

What could be a problem, however, is model calibration. The lack of good quality urban calibration data, both for water quantity and quality, is perhaps the greatest problem today in the modeling area. The flow gages which are presently installed in the San Francisco sewer system can provide useful data. However, it is likely that problems such as those discussed in Chapter II in connection with Flow Gage 125 exist with other gages as well and perhaps hydraulic analysis of each gage site would be worthwhile. Of even greater importance is the need for water quality data. The current lack of in-system quality data is so great that even the most inexpensive data gathering program would be of great benefit as long as the quality of the data is such that it could be confidently used.

Optimization techniques have been shown to be a valuable tool for control strategy development at the subbasin level and are essential at the total system level. The specific techniques employed in this study are not the only ones which could be used. There are a wide variety of large-scale optimization techniques. The aggregation technique described in Chapter IV is not necessarily the best but is a viable approach at this stage of urban systems control research.

The simulation approach to system evaluation has been shown to be a most valuable tool. A system of the size and cost of San Francisco's certainly justifies this evaluation technique rather than considering the results of a few specific design storms. The stochastic nature of the input to the system requires that the output be considered as a stochastic variable as well. Average values and probability distributions of performance parameters based on long term simulation provide a true picture of how the system functions. It is important that the decision makers be aware of the uncertainty associated with any design and the expression of results in terms of uncertainty would and in this awareness. An additional uncertainty variable which was not used in the analysis is the *risk*. Risk can be defined as the probability of exceeding a specific performance parameter value at least once during a given period of time, usually the project life. It is based on the probability of exceeding the particular value in any year (the inverse of the recurrence interval) and given by

$$R = 1 - (1 - P)^N$$

where P = probability of exceeding the value in any year and N = the project life in years. Risk is particularly useful in pointing out the high probability of even storms with relatively large recurrence intervals causing overflows sometime during the life of the project. For example, for a 100 year project life the risk associated with an 11 year recurrence interval event is 1.000, for a 22 year event is 0.990 and for a 100 year event is 0.634. For a 50 year project life these events would have associated risks of 0.992, 0.900 and 0.395 respectively. These high risks give a much better picture of the chances of an overflow event occurring during a given period of time than does the more abstract concept of recurrence interval.

B. Future Studies

Much work needs to be done in the area of control strategy development. Two problems should be given high priority. Although this study has demonstrated that significant technical improvement in system performance can be achieved by system control, the cost effectiveness has not been evaluated. Because the level of sophistication of the control system can be quite variable, it is important that economics be brought into the analysis in order to reduce the feasible range of control system designs. This is not an easy task since water quality standards change with time. Furthermore, dollar costs must be assigned to various pollution levels or a decision concerning maximum allowable pollution levels with associated probabilities must be made. This is not a technical but a political problem, but it has strong design implications.

On a more technical level, the effect of storm prediction uncertainty must be incorporated into the strategy development process. This may have the effect of eliminating some of the more complex strategies from consideration since their potential advantage may be overshadowed by the uncertainty of the input data. There are two basic approaches to evaluating storm prediction uncertainty depending on the equipment available. The use of a telemetered rain gage system will provide only data on what has occurred. Any future projection must be done on the basis of data for the current storm and past history. The addition of weather radar may considerably reduce the short-term prediction uncertianty. However, the question of cost-effectiveness must again be considered.

There are other possible future studies which are discussed in the Phase III report but most of these would be affected by the studies described above.

REFERENCES

[1] California Department of Water Resources, "Summary of Short-Duration Precipitation Frequency in California," August, 1972.

[2] Clark, C. O., "Storage and the Unit Hydrograph," ASCE Transactions, Vol. 110, 1945.

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

[4] Department of Public Works, "San Francisco Master Plan for Wastewater Management - Preliminary Summary Report, San Francisco, September 15, 1971.

[5] Grigg, N. S., Labadie, J. W., Smith, G. L., Hill, D. W. and Bradford, B. H., "Metropolitan Water Intelligence Systems Completion Report - Phase II," Colorado State University, June 1973.

[6] Grigg, N. S., Labadie, J. W., and Wenzel, H. G., "Metropolitan Water Intelligence Systems Completion Report - Phase III," Colorado State University, June 1974.

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

[8] National Oceanic and Atmospheric Administration, National Weather Service, "Precipitation-Frequency Maps for California," prepared by Special Studies Branch, Office of Hydrology, December 1972.

[9] Smith, G. L., Grigg, N. S., Tucker, L. S. and Hill, D. W., "Metropolitan Water Intelligence Systems Completion Report - Phase I," Colorado State University, June 1972.

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

Appendix A

Hydrograph Parameter Identification Model

```
      PROGRAM ROUTE  (INPUT,OUTPUT,TAPE6=OUTPUT)
      DIMENSION STIME(50),TQCALC(50),LAGRAIN(50,60),FRACTN(50),ORD(50)
      DIMENSION PRECIP(50),QOBSVD(50),QCALC(50),C(8),PPECIP(50)
      REAL LAGRAIN
C     A = DRAINAGE AREA IN SQ. MI. , U = IMPERVIOUS AREA/TOTAL AREA
C     XPREC = PRECIPITATION EXCESS IN INCHES, TRAIN = DURATION OF PRECIPITATION
C     EXCESS IN HOURS, DELT = TIME INCREMENT IN HOURS
C     NPREC = NUMBER OF PRECIPITATION INTERVALS, NRUNOFF = NUMBER OF RUNOFF
C     POINTS INCLUDING FIRST AND LAST OBSERVATIONS WHICH ARE USUALLY =0
      DO   171  IZ =  1 ,20
  997 READ 100,(C(I),I=1,8)
  100 FORMAT(8A10)
      PRINT 190,(C(I),I=1,8)
  190 FORMAT("1",8A10)
      READ 101,A,U,          RAINMIN,DELTMIN,NPRECIP,NRUNOFF,KOPT
  101 FORMAT(4F10.4,3I10)
      READ 108,IFIT ,IPLOT,IROUTE
  108 FORMAT(3I10)
      PRINT 107,A,U
  107 FORMAT(" THIS DRAINAGE BASIN HAS AN AREA OF",F10.4," SQ.MI."/" AND
     1 AN URBANIZATION FACTOR OF",F10.4)
      DELT=DELTMIN/60.
      TRAIN=RAINMIN/60.
C     PRECIP(I) = EXCESS PRECIPITATION IN INCHES FOR EACH PRECIPITATION
C     TIME INTERVAL.
      READ 102,(PRECIP(I),I=1,NPRECIP)
      PRINT 102,(PRECIP(I),I=1,NPRECIP)
  102 FORMAT(5F10.2)
C     QOBSVD(I) = THE OBSERVED FLOW IN CFS FOR EACH RUNOFF TIME INTERVAL
      NROP1 = NRUNOFF
      READ 102, (QOBSVD(I), I=1,NROP1)
      DO 444 I=1,NROP1
  444 QOBSVD(I)=.5222*QOBSVD(I)**.9506
      PRINT 102, (QOBSVD(I), I=1,NROP1)
      QTOT = 0.
      PMASS = 0.
      DO 47 I = 1 , NPRECIP
   47 PMASS = PMASS + PRECIP(I)
      DO 48 I = 1 , NROP1
   48 QTOT = QTOT + QOBSVD(I)
      TPREVOL = (5280.**2) * A  *  (PMASS/12.)
      TRUNOF  =  60.  *  DELTMIN  *  QTOT
      RUNOFCF  =  TRUNOF/TPREVOL
      XPRECIP  =  PMASS  *  RUNOFCF
      PRINT 126 ,RAINMIN, PMASS, TPREVOL, TRUNOF,RUNOFCF,XPRECIP
  126 FORMAT("0THIS STORM LASTED ",F6.1," MINUTES"/" A TOTAL OF ",F5.2,"
     1 INCHES OF RAIN FELL.(EQUIVALENT TO A TOTAL VOLUME OF ",F10.1," CU
     2BIC FEET)"/" THE TOTAL RUNOFF WAS ",F10.1," CUBIC FEET"/" THIS RES
     3ULTS IN A RUNOFF COEFFICIENT OF",F6.3,"WITH AN EXCESS PRECIPITATIO
     4N OF",F5.2," INCHES")
      DO 49 I = 1, NPRECIP
   49 PRECIP(I)  =  RUNOFCF  *  PRECIP(I)
C
C
C*****
C     THIS BLOCK FINDS THE MAX. OBSERVED FLOW AND ITS TIME TO PEAK
      QOBSMAX=0.
      TTPOBS=0.
      DO 701 I=2,NROP1
      IF(QOBSVD(I).LE.QOBSMAX)GO TO 701
      QOBSMAX=QOBSVD(I)
      TTPOBS=(I-1)*DELT
  701 CONTINUE
C*****      END OF BLOCK
```

```
C
C
C      PMOMENT = FIRST MOMENT OF THE EXCESS PRECIPITATION
C     .PMASS = AREA UNDER THE PRECIPITATION CURVE
C      RMOMENT = FIRST MOMENT OF THE OBSERVED RUNOFF HYDROGRAPH
C      RMASS = AREA UNDER THE OBSERVED HYDROGRAPH
       PMOMENT=0.
       PMASS=0.
       RMOMENT = 0.
       RMASS = 0.
       NROM1=NRUNOFF-1
C      CALCULATE THE CENTROID OF THE EXCESS PRECIPITATION.
       DO 1 I=1,NPRECIP
       PMOMENT=PMOMENT+PRECIP(I)*      (I*DELT-DELT/2.)
     1 PMASS=PMASS+PRECIP(I)
       PCENTRD=PMOMENT/PMASS
C      CALCULATE THE CENTROID OF THE OBSERVED HYDROGRAPH
       DO 2  I = 1 , NROM1
       RMOMENT=RMOMENT+QOBSVD(I)*DELT*(I*DELT+DELT/2.)+(QOBSVD(I+1)-QOBSV
      1D(I))*DELT/2.*(I*DELT+2./3.*DELT)
     2 RMASS=RMASS+(QOBSVD(I)+QOBSVD(I+1))/2.*DELT
       PCENTRD=RMOMENT/RMASS
       T4=RCENTRD-PCENTRD
C      IROUTE=1=SINGLE LINEAR RESERVOIR
C      IROUTE=2=APPROXIMATE A TRIANGULAR TIME AREA HISTOGRAM FOR CLARK ROUTING
C      IROUTE=3=READ IN TAH FOR CLARK ROUTING
       IF(IROUTE-2)301,302,303
   303 READ 304,IT4
   304 FORMAT(I10)
       READ 305,(FRACTN(I),I=1,IT4)
   305 FORMAT(8F10.4)
       T4=IT4*DELT
       GO TO 306
   302 IT4=IFIX(T4/DELT)
       SUM=0.
       DO 307 J=1,IT4
       XJ=J
       IF((XJ-.5)*DELT.GT.T4/2.)GO TO 308
       ORD(J)=4.*(XJ-.5)*DELT/(T4*T4)
       GO TO 307
   308 ORD(J)=4./T4-(XJ-.5)*DELT*4./(T4*T4)
   307 SUM=SUM+ORD(J)
       DO 309 J=1,IT4
   309 FRACTN(J)=ORD(J)/SUM
   306 PRINT 311,(FRACTN(J),J=1,IT4)
   311 FORMAT(" THE DIMENSIONLESS TIME AREA HISTOGRAM FOR THE CLARK ROUTI
      1NG PROCEDURE IS AS FOLLOWS"/10F8.4)
       JMAX=NPRECIP+IT4-1
       DO 310 J=1,JMAX
       DO 310 I=1,NPRECIP
   310 LAGRAIN(I,J)=0.
       DO 312 I=1,NPRECIP
       JLAST=I+IT4-1
       DO 312 J=I,JLAST
   312 LAGRAIN(I,J)=FRACTN(J-I+1)*PRECIP(I)
       DO 313 J=1,JMAX
       PRECIP(J)=0.
       DO 313 I=1,NPRECIP
   313 PRECIP(J)=PRECIP(J)+LAGRAIN(I,J)
   301 CONTINUE
C      CALCULATE XK1 BY THE REGRESSION EQUATION
       XK1=.887*A**(.49)*(1.+U)**(-1.683)*XPRECIP**(-.24)*TRAIN**(.294)
       XK=T4
C      CALCULATE THE OUTFLOW HYDROGRAPH BASED ON A SINGLE LINEAR RESERVOIR
C      FIRST WITH XK =T4 THEN WITH XK = XK2.
       METHOD=1
       NPREC2=NPRECIP+2
       QCALC(1)=0.
       NPRECP1=NPRECIP+1
       IF(IROUTE-2)7,390,390
```

```
  390 NPRECP1=JMAX+1
      NPREC2=JMAX+2
      GO TO 7
    7 DO 601 I=2,NPRECP1
      QCALC(I)=0.
      IM1=I-1
      DO 601 J=1,IM1
  601 QCALC(I)=QCALC(I)+645.33*A*PRECIP(J)/DELT*(EXP(-(IM1-J)*DELT/XK)
     1-EXP(-(I-J)*DELT/XK))
      DO 602 I=NPREC2,NROP1
  602 QCALC(I)=QCALC(I-1)*EXP(-(DELT/XK))
      IF(IFIT-2)501,502,502
C
C
C*****
C     THIS BLOCK FINDS THE MAX. CALCULATED FLOW AND ITS TIME TO PEAK
  502 TTPCALC=0.
      QCALCMX=0.
      DO 702 I=2,NROP1
      IF(QCALC(I).LE.QCALCMX)GO TO 702
      QCALCMX=QCALC(I)
      TTPCALC=(I-1)*DELT
  702 CONTINUE
C*****      END OF BLOCK
C
C
C     SUMSQ = THE SUM OF THE SQUARES OF THE DIFFERENCES BETWEEN THE CALCULATED
C     AND OBSERVED FLOWS.RUMSQ=THE SQUARE ROOT OF SUMSQ=FIT
C
C
C*****
C     THIS BLOCK DEFINES "FIT" AS THE SQUARE ROOT OF THE FRACTIONAL ERROR OF THE
C     PEAKES SQUARED PLUS THE FRACTIONAL ERROR OF THE TIMES TO PEAK SQUARED
      SUMSQ=((QOBSMAX-QCALCMX)/QOBSMAX)**2+((TTPOBS-TTPCALC)/TTPOBS)**2
      FIT=SQRT(SUMSQ)
      GO TO 555
C*****      END OF BLOCK
C
C
  501 SUMSQ=0.
      DO 4 I = 1 , NROP1
    4 SUMSQ=SUMSQ+(QOBSVD(I)-QCALC(I))**2
      RUMSQ = SQRT(SUMSQ/NRUNOFF)
      FIT=RUMSQ
  555 IF(METHOD-2)5,6,299
    5 IF(IFIT-2)503,504,504
  503 PRINT 505
  505 FORMAT(" FIT=SQRT(SUMMATION FROM I=1,NRUNOFF OF (QOBSVD(I)-QCALC(I
     1))**2)")
      GO TO 506
  504 PRINT 507
  507 FORMAT(" FIT=SQRT(((QOBSMAX-QCALCMX)/QOBSMAX)**2+((TTPOBS-TTPCALC)
     1/TTPOBS)**2)")
  506 PRINT 103,XK,FIT
      RMST4=FIT
  103 FORMAT(//11H WITH K=T4=,F4.2,"HOURS THE INDEX DESCRIBING THE DEGRE
     1E TO WHICH THE CURVES DO NOT FIT="F10.2)
      PRINT 105
  105 FORMAT("1THE FLOWRATES IN CFS ARE AS FOLLOWS"/10X,"TIME",16X,"CALC
     1ULATED",10X,"OBSERVED")
      DO 8   I = 1 , NROP1
      TIME=(I-1)*DELTMIN
      STIME(I) = (I-1)*DELTMIN
    8 PRINT 106,TIME,QCALC(I),QOBSVD(I)
  106 FORMAT(3(10X,F10.1))
      IF (IPLOT - 1 )   151,152,152
  152 CALL MAPA(6,STIME   ,QCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(6,STIME   ,QOBSVD   ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
```

```
      CALL MAPA(1,STIME   ,QCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(2,STIME   ,QOBSVD   ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(2,STIME   ,QCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(4,STIME   ,QCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
  151 CONTINUE
      METHOD=METHOD+1
      XK=XK1
      GO TO 7
    6 PRINT 104,XK,FIT
      RMSK1=FIT
  104 FORMAT(//11H WITH K=K1=,F4.2,"THE INDEX DESCRIBING THE DEGREE TO W
     1HICH THE CURVES DO NOT FIT=",F10.2)
      PRINT 105
      DO 9 I=1,NROP1
      TIME=(I-1)*DELTMIN
      STIME(I) = (I-1)*DELTMIN
    9 PRINT 106,TIME,QCALC(I),QOBSVD(I)
      IF (IPLOT - 1 )  161,162,162
  162 CALL MAPA(6,STIME   ,QCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(6,STIME   ,QOBSVD   ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(1,STIME   ,QCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(2,STIME   ,QOBSVD   ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(2,STIME   ,QCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(4,STIME   ,QCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
  161 CONTINUE
      IF(KOPT.EQ.0)GO TO 999
      IF(RMST4.GT.RMSK1)GO TO 201
      X2=IFIX(XK1*100.+.5)/100.
      X1=IFIX(T4*100.+.5)/100.
      DIRECTN=(XK1-T4)/ABS(XK1-T4)
      RMS=RMSK1
      GO TO 202
  201 X2=IFIX(T4*100.+.5)/100.
      X1=IFIX(XK1*100.+.5)/100.
      DIRECTN=(T4-XK1)/ABS(T4-XK1)
      RMS=RMST4
  202 CONTINUE
      XK=X2
  203 TXK  =  XK
      XK=XK-.10*DIRECTN
      METHOD=METHOD+1
      TFIT  =  FIT
      DO 255 I=1,NROP1
  255 TQCALC(I)=QCALC(I)
      GO TO 7
  299 IF(FIT-RMS)296,296,998
  296 RMS=FIT
      GO TO 203
  998 PRINT 295
  295 FORMAT(//" THE STANDARD ERROR HAS STOPED DECREASING,OPTIMUM CONDIT
     1IONS FOLLOW")
      XK=XK+.1*DIRECTN
      PRINT  298  ,  TXK,  TFIT
  298 FORMAT(//" WITH K ADJUSTED TO K = ",F4.2," THE INDEX DESCRIBING TH
     1E DEGREE TO WHICH THE CURVES DO NOT FIT=",F10.2)
      PRINT 105
      DO 297 I=1,NROP1
      TIME=(I-1)*DELTMIN
      STIME(I) = (I-1)*DELTMIN
  297 PRINT 106,TIME,TQCALC(I),QOBSVD(I)
      IF (IPLOT - 1 )  171,172,172
```

```
  172 CALL MAPA(6,STIME    ,TQCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(6,STIME    ,QOBSVD    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(1,STIME    ,TQCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(2,STIME    ,QOBSVD    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(2,STIME    ,TQCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
      CALL MAPA(4,STIME    ,TQCALC    ,1,NROP1,HL,HH,VL,VH,8HTIME-MIN,8HFL
     1OW-CFS,31HCALCULATED AND OBSERVED VS TIME,1)
  171 CONTINUE
  999 STOP
      END
```

TYPICAL DATA SET

```
 FLOW GAUGE 125  DATE 12/14/71  TIME 4-9 AM
 1.138      .4        120.       15.             2         13          1
        1          0          1
 .0033      .0527     .0094      .0025      .0001
 .0009      .0072     .0003
 0.         .5        21.5       38.0        20.5
 14.        11.       8.2         5.3       4.8
 4.8        1.8        0.
```

Appendix B

Vicente Subbasin Simulation Model

```
C     PROGRAM STAT DEFINES STORMS, OVERFLOW STORMS, AND OVERFLOW
C     VOLUMES FROM PRECIP RECORDS ON MAGNETIC TAPE.
C     A FREQUENCY ANALYSIS IS PERFORMED ALSO.
      PROGRAM STAT(INPUT,OUTPUT,TAPE7,TAPE5=INPUT,TAPE6=OUTPUT)
      COMMON J,VOL( 500),NSTORM(500),NOVFL(500),DEPOF(500)
      DIMENSION YEAR(500),DAY(500), ALPHA(4),    DY(500)
      DIMENSION PRECIP(500),DEP(100),PREC(25),RAIN(500)
      DIMENSION SR(100)
      INTEGER DAY,YEAR,PREC,PRECIP
      SALPHA=0.
      DUM=-1.0
  100 CONTINUE
C     OVERFLOW CURVE COEFFICIENTS ARE READ AND CURVE IS COMPUTED IN
C     STATEMENT 30.
      READ(5,150)C1,C2
  150 FORMAT(2F10.3)
      IF(EOF(5))101,102
  102 CONTINUE
       JJ=3
      CONVER=12/(5280.*5280.*2.)
C     J IS A SUBSCRIPT ON YEAR
      J=1
C     I IS A SUBSCRIPT ON DAY
      I=1
      IEOF=0
      KOVFL=0
      A1=0.
      A2=0.
      DO 30 K1=1,72
   30 DEP(K1)=C1*K1+C2
      WRITE(6,603)C1,C2
  603 FORMAT(*1*,//5X,*NON-OVERFLOW CURVE DEFINED BY     *,F4.2,
     1*(DURATION) + *,F4.2)
      WRITE(6,600)
  600 FORMAT(///,2X,*YEAR*,5X,*NUMBER OF STORMS*,5X,*NUMBER OF OVERFLOWS
     1*,7X,*TOTAL VOL. OF OVERFLOW*,7X,*AVER. DURATION OF *,
     23X,*AVER. TOTAL DUR. OF*/,59X,*MILLION CU. FT.*,2X,*IN.*,8X,
     3*OVERFLOW (HRS.)*,6X,*OVERFLOW STORMS (HRS.)*/)
C     FIRST 24 PRECIP VALUES ARE READ.
      READ(7)DAY(I),YEAR(I),(PREC(K),K=1,24),DY(I)
      DO 201 II=1,24
      IF(PREC(II).LT.6)PREC(II)=0
      PRECIP(II)=PREC(II)
  201 CONTINUE
      YEAR(I)=YEAR(I)/10000
      IYEAR=YEAR(I)+1899
   50 CONTINUE
      IF(JJ.EQ.3)GO TO 51
      IF(IEOF.EQ.1)J=J+1
      L=J-1
      IF(NOVFL(L).EQ.0)GO TO 700
C     COMPUTE THE AVERAGE DURATION OF OVERFLOW FOR EACH YEAR.
      AVOFDUR=          ODUR/FLOAT(NOVFL(L))
C      COMPUTE THE AVERAGE STORM DURATIONS FOR EACH YEAR.
      AVSTDUR=FLOAT(KSTORM)/FLOAT(NOVFL(L))
      A1=A1+AVOFDUR
      A2=A2+AVSTDUR
      KOVFL=KOVFL+1
      GO TO 701
  700 AVOFDUR=0.
      AVSTDUR=0.
  701 CONTINUE
      IF(IEOF.EQ.1) I=I+1
      IYEAR=IYEAR+1
```

```
      WRITE(6,601)IYEAR,NSTORM(L),NOVFL(L),NVIC,VOL(L),DEPOF(L),AVOFDUR,
     1AVSTDUR
  601 FORMAT(3X,I4,10X,I3,20X,I3,5X,I3,5X,
     1                                 F13.1,4X,F7.3,12X,F6.2,12X,F6.2)
      IF(IEOF.EQ.1)GO TO 5
   51 CONTINUE
      JJ=0
      ODUR=0.
      NVIC=0
      KSTORM=0
      NSTORM(J)=0.
      NOVFL(J)=0.
      DEPOF(J)=0.
      VOL(J)=0.
      KMAX=24.
C     READ A SERIES OF 24 PRECIP VALUES UNTIL A NON-SUCCESSIVE DAY OR
C     YEAR IS ENCOUNTERED.
   10 READ(7) DAY(I+1),YEAR(I+1),(PREC(K),K=1,24),DY(I+1)
      YEAR(I+1)=YEAR(I+1)/10000
      IF(YEAR(I+1).EQ.73)IEOF=1
      IF(EOF(7))450,9
  450 IEOF=1
      GO TO 7
    9 CONTINUE
      IF(YEAR(I).EQ.YEAR(I+1)) GO TO 2
      DAY(2)=DAY(I+1)
      YEAR(2)=YEAR(I+1)
      I=1
      GO TO 3
    5 CONTINUE
      IF(KOVFL.GT.0)GO TO 650
      WRITE(6,651)
  651 FORMAT(5X,*NO OVERFLOWS OCCURRED DURING ENTIRE RECORD*)
      GO TO 100
  650 CONTINUE
      B1=A1/KOVFL
      B2=A2/KOVFL
      B3=SALPHA/KOVFL
      WRITE(6,602)B1,B2
  602 FORMAT(//5X,*AVER. DURATION OF OVERFLOW IN HRS.= *,
     2F6.2//,5X,*AVER. TOTAL OVERFLOW STORM DURATION IN HRS. = *,F6.2)
      CALL FREQ
      GO TO 100
    3 JJ=1
      GO TO 7
    2 IF(DAY(I) .EQ. (DAY(I+1)-1)) GO TO 6
      GO TO 7
    6 JMAX=KMAX
      KMAX=KMAX+24
C     STORE SUCCESSIVE SETS OF PRECIPVALUES IN A CONTINUOUS ARRAY OF
C     LENGTH KMAX.
      DO 202 II=1,24
      IF(PREC(II).LT.6)PREC(II)=0
      PRECIP(II+JMAX)=PREC(II)
  202 CONTINUE
      I=I+1
      GO TO 10
C     K=LIMITS  K=1,KMAX
C.    K1= NUMBER OF SUCCESSIVE NON-ZERO PRECIP VALUES=STORM DUR. IN HRS.
    7 K=1
      K1=0
      DO 55 M=1,KMAX
   55 RAIN(M)=0.
      DEPTH=0.
      DIFMAX=0.
      GO TO 13
C     LOOP THRU STAT. 13 FINDS THE FIRST HOUR OF PRECIP ON THE CURRENT
C     STORM.
   18 K=K+1
```

```
      DIFMAX=0.
      IF(IEOF.EQ.1 .AND. K.EQ.KMAX)GO TO 50
      DEPTH=0
      IF(K.GE.KMAX)GO TO 24
      K1=0
13    IF(PRECIP(K).EQ.0 .AND. DEPTH.EQ.0.)GO TO 18
      IF(PRECIP(K).EQ.0)GO TO 800
      DEPTH=DEPTH+FLOAT(PRECIP(K))/100.
      K1=K1+1
      RAIN(K1)=FLOAT(PRECIP(K))/100.
      IF(RAIN(K1).GE.DEP(1))DIFMAX=0.1
C     COMPUTE THE DIFFERENCE BETWEEN THE ACCUMULATED DEPTH AND THE
C     OVERFLOW CURVE FOR EACH HOUR AFTER THE BEGINNING OF THE STORM.
      DIF=DEPTH-DEP(K1)
      IF(DIF.LE.DIFMAX)GO TO 33
      DIFMAX=DIF
33    CONTINUE
      IF(K.GE.KMAX)GO TO 24
      K=K+1
      GO TO 13
C     THE LOOP THRU STAT. 805 DEFINES THE END OF THE STORM BY LOOKING
C     FOR 3 HOURS OF ZERO PRECIP WITHIN THE CURRENT SET OF SUCCESSIVE
C     PRECIP DAYS.
800   IF((K+2).GT.KMAX)GO TO 802
      IF(PRECIP(K+1).EQ.0 .AND. PRECIP(K+2).EQ.0)GO TO 803
      K=K+1
      K1=K1+1
      GO TO 13
803   K=K+2
      GO TO 24
802   IF((K+1).GT.KMAX)GO TO 24
      IF(PRECIP(K+1).EQ.0)GO TO 805
      K=K+1
      K1=K1+1
      GO TO 13
805   K=K+1
24    CONTINUE
C     KEEP A RUNNING SUM OF THE NUMBER OF STORMS IN EACH YEAR.
      IF(DEPTH.GT.0.)NSTORM(J)=NSTORM(J)+1
      IF(K.GE.KMAX.AND.DEPTH.EQ.0.)GO TO 28
      IF(K.GE.KMAX)GO TO 403
      IF(IEOF.EQ.1) GO TO 403
      IF(DEPTH.EQ.0.)GO TO 18
403   CONTINUE
      IF(DIFMAX.GT.0.)GO TO 203
      IF(K.GE.KMAX)GO TO 28
      DO 56 M=1,KMAX
 56   RAIN(M)=0.
      GO TO 18
C     KEEP A RUNNING SUM OF THE FOLLOWING.....
C     1. NOVFL=NUMBER OF OVERFLOWS IN EACH YEAR.
C     2. KMAXDUR=HOURS FROM BEGINNING OF STORM TO TIME WHEN MAX DIFF.
C        BETWEEN STORM MASS CURVE AND OVERFLOW CURVE OCCURS.
C     3. KSTORM=TOTAL STORM DURATION.
C     4. VOL=OVERFLOW VOL. IN CU  FT. FOR EACH YEAR.
C     5. DEPOF=OVERFLOW VOL. IN INCHES FOR EACH YEAR FOR THE 2.0 SQ. MI.
C        VICENTE WATERSHED.
203   CONTINUE
      IPREV=I
      K2=K1
      RMAX=0.
      IDUR=1
      MAXDUR=1
      RATIO=3.
      DO 95 I=1,K1
95    IF(RAIN(I).GT.RMAX)RMAX=RAIN(I)
      DO 96 I=1,K1
       IF(RAIN(I).EQ.0.)GO TO 96
      PREVR=RATIO
      RATIO=RMAX/RAIN(I)
```

```
      IF(PREVR.LE.2.0.AND.RATIO.LE.2.0)GO TO 98
      IDUR=1
      GO TO 96
98    CONTINUE
      IDUR=IDUR+1
      IF(IDUR.LE.MAXDUR)GO TO 96
      MAXDUR=IDUR
      IMAX=I
96    CONTINUE
      IF(MAXDUR.EQ.1)GO TO 103
      EQDEP=0.
      IMIN=IMAX-MAXDUR+1
      DO 99 I=IMIN,IMAX
99     EQDEP=EQDEP+RAIN(I)
      GO TO 104
103   EQDEP=RMAX
104   CONTINUE
      IDUR=MAXDUR
      ALPHA1=-1.166*IDUR**(-1.405)+3.41*EQDEP*IDUR**(-1.06)
      IF(ALPHA1.LT.0.5)ALPHA1=0.5
      IF(ALPHA1.GT.3.0)ALPHA1=3.0
      DO 97 I=1,3
97    ALPHA(I)=ALPHA1
      ALPHA(4)=1.0
C     VOLUME OF OVERFLOW IS COMPUTED FROM SUBROUTINE VICENTE
      DO 106 L=1,K1
106   SR(L)=RAIN(L)
      CALL VICENTE(ALPHA.K1.RAIN,VOFLO,OFDUR,DUM)
      NVIC=NVIC+1
      OFDUR=OFDUR/60.
      IF(VOFLO.EQ.0.)GO TO 90
      SALPHA=SALPHA+ALPHA1
      I=IPREV
      KSTORM=KSTORM + K1/4
      DOF=VOFLO*CONVER
      VOFLO=VOFLO/1000000.
      DEPOF(J)=DEPOF(J)+DOF
      VOL(J)=VOL(J)+ VOFLO
      ODUR=ODUR+OFDUR
       NOVFL(J)=NOVFL(J)+1
90    CONTINUE
      I=IPREV
      IF(IEOF.EQ.1) GO TO 50
 31   IF(K.EQ.KMAX) GO TO 28
      DEPTH=0.
      DIFMAX=0.
      K1=0
      K=K+1
      DO 57 M=1,KMAX
57    RAIN(M)=0.
      GO TO 13
 28   IF(JJ.EQ.1) GO TO 40
      I=I+1
      KMAX=24.
      DO 210 II=1.24
      IF(PREC(II).LT.6)PREC(II)=0.
      PRECIP(II)=PREC(II)
210   CONTINUE
      GO TO 10
 40   J=J+1
      DO 211 II=1.24
      IF(PREC(II).LT.6)PREC(II)=0
      PRECIP(II)=PREC(II)
211   CONTINUE
      I=I+1
      GO TO 50
101   STOP
      END
```

```
      SUBROUTINE FREQ
C     THIS SUBROUTINE PERFORMS A FREQUENCY ANALYSIS OF THE NUMBER AND
C     VOLUME OF OVERFLOWS IN A YEAR.
      COMMON J,VOL( 500),NSTORM(500),NOVFL(500),DEPOF(500)
      DIMENSION NOFPY(300),PEXNO(300),NOFVOL(600)
      INTEGER ZNOVFL,ZVOL,SUMO,SUMS
      JMAX=J-1
C     DELVOL=THE INCREMENT OF OVERFLOW VOL. USED IN THE FREQ. ANALYSIS.
      DELVOL=0.92928
      XX=12000000./(5280.*5280.*2.0)
      I=1
      ZNOVFL=0
      ZVOL=0
      SUMO=0.
      SUMS=0.
      SSNOFPY=0.
      SNOFVOL=0.
      SNOFPY=JMAX
      SUMDEP=0.
      SUMVOL=0.
      MAXNOV=0.
      VOLMAX=0.
    2 SUMS=SUMS+NSTORM(I)
      SUMO=SUMO+NOVFL(I)
      SUMVOL=SUMVOL+VOL(I)
      IF(NOVFL(I).GT.MAXNOV) MAXNOV=NOVFL(I)
      IF(VOL(I).GT.VOLMAX) VOLMAX=VOL(I)
      I=I+1
      IF(I.GT.JMAX) GO TO 5
      GO TO 2
C     COMPUTE VARIOUS OVERALL STATISTICAL PARAMETERS AS DESCRIBED IN
C     FORMAT STATEMENTS 201 AND 200.
    5 ANOFPY=SUMO/FLOAT(JMAX)
      AOFVPE=SUMVOL/SUMO
      AOFVPY=SUMVOL/JMAX
      VPEI=AOFVPE*XX
      VPYI=AOFVPY*XX
      POFFAS=SUMO/FLOAT(SUMS)
      WRITE(6,350)
  350 FORMAT(*1*)
      WRITE(6,201)JMAX,SUMS,SUMO
  201 FORMAT(5X,*ANALYSIS BASED ON *,I3,* YEARS OF RECORD*//,5X,
     1*TOTAL NUMBER OF STORMS= *I5//,5X,*TOTAL NUMBER OF OVERFLOWS= *I5/
     2)
      WRITE(6,200)ANOFPY,AOFVPE,VPEI,AOFVPY,VPYI,POFFAS
  200 FORMAT(5X,*AVER. NUMBER OF OVERFLOWS/YR.= *F7.2//5X,*AVER. VOL. OF
     1 OVERFLOW/OVERFLOW EVENT= *F6.2,* MILLION CU. FT.=*
     4                                              F6.3,* IN.*//5X, *AVER.
     2 VOL. OF OVERFLOW/YR.= *F7.2,* MILLION CU. FT.=*F6.3,* IN.*//5X,*P
     3ROBABILITY OF OVERFLOW FROM ANY STORM= *F6.4/////)
      DO 20 L=1,MAXNOV
      NOFPY(L)=0.
   20 CONTINUE
      J=1
      WRITE(6,351)
  351 FORMAT(5X,*NUMBER OF OVERFLOWS*,5X,*PROBABILITY OF EXCEEDING*,5X,
     2*PROBABILITY OF EXCEEDING AT LEAST*/,11X,*PER YEAR*,14X,*DURING AN
     3Y YEAR*,10X,*ONCE DURING 100 YEAR PERIOD*/)
    9 CONTINUE
      IF(NOVFL(J).GT.0)GO TO 300
      ZNOVFL=ZNOVFL+1
      GO TO 310
  300 CONTINUE
      NOFPY(NOVFL(J))=NOFPY(NOVFL(J))+1
  310  CONTINUE
      J=J+1
      IF(J.GT.JMAX) GO TO 10
      GO TO 9
   10 I=1
      L=0
      PZO=1.-ZNOVFL/SNOFPY
```

```
      RISK=1.-(1.-PZO)**100
      SSNOFPY=ZNOVFL
      WRITE(6,352)L,PZO,RISK
352   FORMAT(13X,I3,22X,F5.3,26X,F5.3)
11     CONTINUE
C     COMPUTE THE PROBABILITY OF EXCEEDING N OVERFLOWS IN ANY YEAR AND
C     THE ASSOCIATED RISK BASED ON A 100 YEAR PERIOD.
      SSNOFPY=SSNOFPY+NOFPY(I)
      PEXNO(I)=1.-(SSNOFPY/SNOFPY)
      RISK=1.-(1.-PEXNO(I))**100
      WRITE(6,352)I,PEXNO(I),RISK
      I=I+1
      IF(I.GT.MAXNOV) GO TO 14
      GO TO 11
  14  KMAX=(VOLMAX/DELVOL)+1
      DO 30 N=1,KMAX
      NOFVOL(N)=0.
  30  CONTINUE
      WRITE(6,355)
355   FORMAT(/////7X,*VOLUME OF OVERFLOW/YR.*6X,*PROBABILITY OF EXCEEDIN
     1G*,5X,*PROBABILITY OF EXCEEDING AT LEAST*/2X,*MILLION CU. FT.*,5X,
     2*IN.*,13X,*DURING ANY YEAR*,11X,*ONCE DURING 100 YEAR PERIOD*/)
      J=1
C     THIS LOOP DETERMINES THE NUMBER OF YEARS IN WHICH THE OVERFLOW
C     VOLUME FALLS WHTHIN A PARTICULAR RANGE.
 41   CONTINUE
      IVOL=VOL(J)/DELVOL
      IF(VOL(J).GT.0.)GO TO 400
      ZVOL=ZVOL+1.
      GO TO 410
 400  CONTINUE
      NOFVOL(IVOL+1)=NOFVOL(IVOL+1)+1
410   CONTINUE
      J=J+1
      IF(J.GT.JMAX) GO TO 50
      GO TO 41
  50  NTOT=JMAX
      SNOFVOL=ZVOL
      PZV=1.-SNOFVOL/NTOT
      RISK=1.-(1.-PZV)**100
      WRITE(6,356)L,L,PZV,RISK
356   FORMAT(2X,I10,8X,I3,19X,F6.3,26X,F5.3)
      I=1
51    CONTINUE
C     COMPUTE THE PROBABILITY OF EXCEEDING A PARTICULAR OVERFLOW
C     VOLUME IN ANY YEAR AND THE ASSOCIATED RISK.
      SNOFVOL=SNOFVOL+NOFVOL(I)
      PEXIDEL=1. -(SNOFVOL/NTOT)
      RISK=1.-(1.-PEXIDEL)**100
      VOF=I*DELVOL
      DOF=VOF*XX
      WRITE(6,357)VOF,DOF,PEXIDEL,RISK
357   FORMAT(2X,F10.1,8X,F6.3,16X,F6.3,26X,F5.3)
       I=I+1
      IF(I.GT.KMAX)RETURN
      GO TO 51
      END

      SUBROUTINE VICENTE(ALPHA,NR,R,TTOVF,OFDUR,DUM)
      DIMENSION T(500),QUP(500),QDN(500),QBASIN(500),Q36(500),Q3T09(500)
     1,AREA(7),K(7),C(7),BASE(7),QCAP(4),VMAX(4),S(10),D(10),N(10),
     2DELX(10),TVOVF(4),LSTOVF(4),NDOVF(4),ALPHA(4),R(500),RR(100),
     3ZERO(4)
      COMMON/VIC/DELTBAS,DELTREA,JRETAIN,JS,JD,JR
      EQUIVALENCE(Q36(1),Q3T09(1))
      REAL K,N
      DO 150 I=1,500
      QUP(I)=0.
      QDN(I)=0.
      QBASIN(I)=0.
      Q36(I)=0.
      Q3T09(I)=0.
```

```
150   CONTINUE
      IF(DUM.GT.0.0)GO TO 333
      READ(5,100)DELTRAS,DELTREA,JRETAIN
  100 FORMAT(2F10.0,I10)
      READ(5,101)(AREA(I),I=1,7)
      READ(5,101)(K(I),I=1,7)
      READ(5,101)(C(I),I=1,7)
      READ(5,101)(BASE(I),I=1,7)
      READ(5,101)(QCAP(I),I=1,4)
      READ(5,101)(QZERO(I),I=1,4)
      READ(5,101)(VMAX(I),I=1,4)
  101 FORMAT(8F10.2)
      READ(5,101)(S(I),I=1,10)
      READ(5,101)(D(I),I=1,10)
      READ(5,101)(N(I),I=1,10)
      READ(5,101)(DELX(I),I=1,10)
  333 CONTINUE
      NK=0
      DO 250 I=1,NR
250   RR(I)=R(I)/4.
      DO 200 I=1,NR
      DO 200 J=1,4
      NK=J+(I-1)*4
200   R(NK)=RR(I)
      NR=4*NR
      DUM=5.0
C     SUB-BASIN 6
      JS=1
      CALL BASIN(AREA(JS),K(JS),BASE(JS),C(JS),NR,R,QBASIN,NUP)
C     RETENTION BASIN  12-3
      JD=1
      CALL RETNTN(QBASIN,NUP,QUP,QCAP(JD),ALPHA(JD),VMAX(JD),TVOVF(JD),
     1LSTOVF(JD),NDOVF(JD),QZERO(JD))
C     REACH 5
      JR=1
      CALL REACH(QUP,NUP,QDN,NDN,S(JR),D(JR),N(JR),DELX(JR))
C     SUB-BASIN 3
      JS=2
      CALL BASIN(AREA(JS),K(JS),BASE(JS),C(JS),NR,R,QBASIN,NUP)
      IF(NUP.GE.NDN)NQ=NUP
      IF(NUP.LT.NDN)NQ=NDN
      DO 1 I=1,NQ
    1 QUP(I)=QBASIN(I)+QDN(I)
      NQP1=NQ+1
      DO 9 I=NQP1,500
    9 QUP(I)=QBASIN(1)+QDN(1)
C     REACH 4
      JR=2
      CALL REACH(QUP, NQ,Q36,N36,S(JR),D(JR),N(JR),DELX(JR))
C     SUB-BASIN 8
      JS=3
      CALL BASIN(AREA(JS),K(JS),BASE(JS),C(JS),NR,R,QBASIN,NUP)
C     RETENTION BASIN 12-4
      JD=2
      CALL RETNTN(QBASIN,NUP,QUP,QCAP(JD),ALPHA(JD),VMAX(JD),TVOVF(JD),
     1LSTOVF(JD),NDOVF(JD),QZERO(JD))
C     REACH 11
      JR=3
      CALL REACH(QUP, NQ,QDN,NDN,S(JR),D(JR),N(JR),DELX(JR))
C     SUB-BASIN 5
      JS=4
      CALL BASIN(AREA(JS),K(JS),BASE(JS),C(JS),NR,R,QBASIN,NUP)
      IF(NUP.GE.NDN)NQ=NUP
      IF(NUP.LT.NDN)NQ=NDN
      DO 2 I=1,NQ
    2 QUP(I)=QBASIN(I)+QDN(I)
      NQP1=NQ+1
      DO 19 I=NQP1,500
  19  QUP(I)=QBASIN(1)+QDN(1)
C     REACH 10
      JR=4
      CALL REACH(QUP, NQ,QDN,NDN,S(JR),D(JR),N(JR),DELX(JR))
```

```
      DO 3 I=1,500
    3 QUP(I)=QDN(I)
      NUP=NDN
C     REACH 7
      JR=5
      CALL REACH(QUP,NUP,QDN,NDN,S(JR),D(JR),N(JR),DELX(JR))
C     SUB-BASIN 4
      JS=5
      CALL BASIN(AREA(JS),K(JS),BASE(JS),C(JS),NR,R,QBASIN,NUP)
      IF(NUP.GE.NDN)NQ=NUP
      IF(NUP.LT.NDN)NQ=NDN
      DO 4 I=1,NQ
    4 QUP(I)=QBASIN(I)+QDN(I)
      NQP1=NQ+1
      DO 29 I=NQP1,500
   29 QUP(I)=QBASIN(1)+QDN(1)
C     REACH 13
      JR=6
      CALL REACH(QUP, NQ,QDN,NDN,S(JR),D(JR),N(JR),DELX(JR))
      IF(NDN.GE.N36)NUP=NDN
      IF(NDN.LT.N36)NUP=N36
      DO 5 I=1,NUP
    5 QUP(I)=QDN(I)+Q36(I)
      NUPP1=NUP+1
      DO 39 I=NUPP1,500
   39 QUP(I)=QDN(1)+Q36(1)
C     REACH 3
      JR=7
      CALL REACH(QUP,NUP,Q3TO9,N3TO9,S(JR),D(JR),N(JR),DELX(JR))
C     SUB-BASIN 9
      JS=6
      CALL BASIN(AREA(JS),K(JS),BASE(JS),C(JS),NR,R,QBASIN,NUP)
C     RETENTION BASIN 12-5
      JD=3
      CALL RETNTN(QBASIN,NUP,QUP,QCAP(JD),ALPHA(JD),VMAX(JD),TVOVF(JD),
     1LSTOVF(JD),NDOVF(JD),QZERO(JD))
C     REACH 9
      JR=8
      CALL REACH(QUP,NUP,QDN,NDN,S(JR),D(JR),N(JR),DELX(JR))
C     SUB-BASIN 7
      JS=7
      CALL BASIN(AREA(JS),K(JS),BASE(JS),C(JS),NR,R,QBASIN,NUP)
      IF(NUP.GE.NDN)NQ=NUP
      IF(NUP.LT.NDN)NQ=NDN
      DO 6 I=1,NQ
    6 QUP(I)=QBASIN(I)+QDN(I)
      NQP1=NQ+1
      DO 49 I=NQP1,500
   49 QUP(I)=QBASIN(1)+QDN(1)
C     REACH 8
      JR=9
      CALL REACH(QUP,NUP,QDN,NDN,S(JR),D(JR),N(JR),DELX(JR))
      DO 7 I=1,500
    7 QUP(I)=QDN(I)
      NUP=NDN
C     REACH 6
      JR=10
      CALL REACH(QUP,NUP,QDN,NDN,S(JR),D(JR),N(JR),DELX(JR))
      IF(NDN.GE.N3TO9)NQ=NDN
      IF(NDN.LT.N3TO9)NQ=N3TO9
      DO 8 I=1,NQ
    8 QDN(I)=QDN(I)+Q3TO9(I)
      NQP1=NQ+1
      DO 59 I=NQP1,500
   59 QDN(I)=QDN(1)+Q3TO9(1)
C     RETENTION BASIN 12-2
      JD=4
      CALL RETNTN(QDN,    NQ,QUP,QCAP(JD),ALPHA(JD),VMAX(JD),TVOVF(JD),
     1LSTOVF(JD),NDOVF(JD),QZERO(JD))
  999 CONTINUE
      NOVF=0
```

```
      LTOVF=3000
      TTOVF=0.0
      DO 222  JJ=1,4
      TTOVF=TVOVF(JJ)+TTOVF
      IF(LSTOVF(JJ).LT.LTOVF)LTOVF=LSTOVF(JJ)
      IF(NDOVF(JJ).GT.NOVF)NOVF=NDOVF(JJ)
  222 CONTINUE
      OFDUR=(NOVF -LTOVF+1)*DELTREA
      IF(OFDUR.LT.0.)OFDUR=0.
      RETURN
      END

      SUBROUTINE RETNTN(QIN,NQIN,QOUT,QCAP,ALPHA,VMAX,TVOVF,LSTOVF,
     1NDOVF,QZERO)
      DIMENSION QIN(500),QOUT(500),V(501),T(500)
      COMMON/VIC/DELTBAS,DELTREA,JRETAIN,JS,JD,JR
C     QMAX IS THE MAXIMUM FLOW RATE OUT OF THE RETENSION BASIN (CFS)
C     VMAX IS THE MAXIMUM RETENSION CAPACITY (CFS)
C     VSTART IS THE QUANTITY OF WATER (CF) INITIALLY IN THE RETENSION BASI
      VOLOF=0.
      IF(JRETAIN.EQ.0)GO TO 16
      IF(JD.GT.1)GO TO 50
      DO 49 I=1,500
      QOUT(I)=0.
      V(I)=0.
      T(I)=0.
 49   CONTINUE
 50   CONTINUE
      QMAX=ALPHA*QZERO
      VSTART=0.0
      V(1)=VSTART
      DO 6 I=1,500
      IF(QIN(I)-QMAX)7,7,8
    7 IF(V(I))9,9,10
    9 V(I+1)=0.
      QOUT(I)=QIN(I)
      IF(I-NQIN)6,6,11
   10 IF(V(I)-(QMAX-QIN(I))*60.*DELTREA)12,12,13
   12 QOUT(I)=QIN(I)+V(I)/(60.*DELTREA)
      V(I+1)=0.
      GO TO 6
   13 V(I+1)=V(I)-(QMAX-QIN(I))*60.*DELTREA
      QOUT(I)=QMAX
      GO TO 6
    8 IF(V(I).EQ.VMAX)GO TO 1
      IF(V(I)+(QIN(I)-QMAX)*DELTREA*60.-VMAX)14,15,15
   14 V(I+1)=V(I)+(QIN(I)-QMAX)*60.*DELTREA
      QOUT(I)=QMAX
      GO TO 6
   15 QOUT(I)=QIN(I)-(VMAX-V(I))/(60.*DELTREA)
      V(I+1)=VMAX
      GO TO 6
    1 V(I+1)=VMAX
      QOUT(I)=QIN(I)
    6 CONTINUE
 11   CONTINUE
      TVOVF=0.
      NDOVF=0
      LSTOVF=3000
      DO 802 I=1,NQIN
      IF(V(I+1).EQ.VMAX)GO TO 801
      GO TO 800
  801 RATEOF=QOUT(I)-QMAX
      VOLOF=VOLOF+RATEOF*60.*DELTREA
      TVOVF=VOLOF
      IF(I.LT.LSTOVF)LSTOVF=I
      IF(I.GT.NDOVF)NDOVF=I
      GO TO 802
800    CONTINUE
 802  CONTINUE
      RETURN
```

```
 16 DO 17 I=1,NQIN
 17 QOUT(I)=QIN(I)
    RETURN
    END
    *

    SUBROUTINE BASIN(A,K,BASE,C,NR,R,Q,NQ)
    DIMENSION      Q(500),R(500),T(500),QF(500),RB(500)
    COMMON/VIC/DELTBAS,DELTREA,JRETAIN,JS,JD,JR
    REAL K
    IF(JS.GT.1)GO TO 11
    DO 50 I=1,500
    Q(I)=0.
    T(I)=0.
    QF(I)=0.
    RB(I)=0.
50   CONTINUE
 11 DO 4 I=1,NR
4   RB(I)=R(I)*C
    DFLT=DELTBAS/60.
    Q(1)=0.
    T(1)=0.
    I=1
  2 I=I+1
    IM1=I-1
    Q(I)=0.
    DO 1 J=1,IM1
  1 Q(I)=Q(I)+645.33*A *RB(J)/DFLT*(EXP(-(I-J-1)*DELT/K)-EXP(-(I-J)*
   1DELT/K))
    T(I)=IM1*DELTBAS
    IF(IM1.LT.NR) GO TO 2
    QEND=Q(I)
  3 I=I+1
    IM1=I-1
    Q(I)=Q(I-1)*EXP(-DELT/K)
    T(I)=DELTBAS*IM1
    IF(Q(I).GT..01*QEND) GO TO 3
    NQ=I
    NRP1=NR+1.
    F=DELTBAS/DELTREA
    NQ=(NQ-1)*F+1.5
    DO 7 J=1,NQ
    FI=(J-1)/F+1.
    II=FI
    IF((II+1).GT.NQ)GO TO 7
    QF(J)=Q(II)+(FI-II)*(Q(II+1)-Q(II))
7   CONTINUE
    DO 8 J=1,NQ
  8 Q(J)=QF(J)
    T(1)=0.
    Q(1)=Q(1)+BASE
    DO 6 I=2,500
    T(I)=T(I-1)+DELTREA
  6 Q(I)=Q(I)+BASE
    RETURN
    END


    SUBROUTINE REACH(QJ,NQ,QJP1,NQJP1,S,D,N,DELX)
    DIMENSION QJ(500),QJP1(500),T(500)
    REAL N,K,KBYDELT
    COMMON/VIC/DELTBAS,DELTREA,JRETAIN,JS,JD,JR
    IF(JR.GT.1)GO TO 50
    DO 49 I=1,500
    QJP1(I)=0.
    T(I)=0.
49  CONTINUE
50  CONTINUE
    R=D/2.
    DELT=DELTREA
    T(1)=0.
    I=1
```

```
      QUMAX=0.
      QDMAX=0.
      QJP1(1)=QJ(1)
      Q=3.1416*D*D/8.*1.486/N*(D/4.)**.6667*SQRT(S)
      CALL KALFA(Q,S,R,N,DELX,K,ALPHA)
      X=(1.-ALPHA)/2.
      C1=(1.+2.*X)/(3.-2.*X)
      C2=(1.-2.*X)/(3.-2.*X)
      C3=(1.-2.*X)/(3.-2.*X)
      K=K/60.
      KBYDELT=K/DELT
    6 T(I+1)=I*DELT
      IF(QJ(I).GT.QUMAX)QUMAX=QJ(I)
      IF(QJP1(I).GT.QDMAX)QDMAX=QJP1(I)
    3 IF(T(I+1)-K.GT.0.) GO TO 1
      QJB=QJ(1)
      QJP1B=QJP1(1)
      GO TO 2
    1 FJ0=I+1-KBYDELT
      J0=FJ0
      F=(FJ0-J0)
      QJB=QJ(J0)+F*(QJ(J0+1)-QJ(J0))
      IF(KBYDELT.GT.1.0)GO TO 13
      FJ0=I+1-KBYDELT
      J0=FJ0
      J0M1=J0-1
      QJP1B=QJP1(J0M1)+(FJ0-J0M1)*(QJP1(J0)-QJP1(J0M1))
      GO TO 2
13    CONTINUE
      QJP1B=QJP1(J0)+F*(QJP1(J0+1)-QJP1(J0))
    2 CONTINUE
      IF(I.LT.NQ) GO TO 5
      QJ(I+1)=QJ(1)
    5 QJP1(I+1)=C1*QJB+C2*QJ(I+1)+C3*QJP1B
    9 I=I+1
      IF(I.GE.NQ)QJ(I+1)=QJ(1)
      IF(I.LT.NQ+10) GO TO 6
      NQJP1=I
      SUM1=0.
      SUM2=0.
      DO 55 I=2,NQJP1
      SUM1=SUM1+.5*(QJ(I)+QJ(I-1))
   55 SUM2=SUM2+.5*(QJP1(I)+QJP1(I-1))
      VOL1=SUM1*DELT*60.
      VOL2=SUM2*DELT*60.
 12   IP1=I+1
      DO 10 J=IP1,500
   10 QJP1(J)=QJP1(1)
      RETURN
      END


      SUBROUTINE KALFA(Q,S,R,N,DELTX,K,ALPHA)
      REAL N,K
C     INITIALIZE T (THE CENTRAL ANGLE THETA) AT 3.14
      T=3.14
      I=0
C     CHECK TO SEE IF Q IS GREATER THAN THE FULL FLOW CAPACITY.
C     IF IT IS LET T=3.5 AND GO TO 13 AFTER PRINTING Q FULL AND Q ENTERED
      QM=1.49/N*(R/2.)**.667*3.1416*R*R*SQRT(S)
      IF(Q-QM.LT.0.)GO TO 1
      T=3.5
      GO TO 13
C     WITH T CALCULATE AREA (A) AND HYDRAULIC RADIUS (RH)
    1 A=((R**2)/2.)*(T-SIN(T))
    2 RH=(R*(T-SIN(T))/(2.*T))
C     CALCULATE FLOW (QI) FROM MANNING EQUATION
    3 QI=(1.49/N)*A*(RH**(2./3.))*(S**.5)
C     COMPARE OBSERVED AND CALCULATED
    5 IF(ABS(Q-QI)-.00005) 8,6,6
```

```
C     IF NEEDED, ADJUST T (THETA) TO GET BETTER CALCULATED FLOW
C     CALLING FT THE DIFFERENCE BETWEEN Q AND QI WE
C     FIND FT=0 BY NEWTONS METHOD.
C     DFT IS THE FIRST DERIVATIVE OF FT (WITH RESPECT TO T)
    6 DFT=-(((.4693*R**(8./3.)*S**.5)/N)*
     1(((5./3.)*T**(-2./3.)*((T-SIN(T))**(2./3.))
     2*(1.-COS(T)))-(((2./3.)*T**(-5./3.))*
     3((T-SIN(T))**(5./3.)))))
      FT=Q-QI
C     APPLYING NEWTONS METHOD
      T=T-(FT/DFT)
      I=I+1
      IF(I.EQ.20) GO TO 99
      GO TO 1
C     TEST T FOR EXTREMES
    8 IF(T.LE..1) T=.1
      IF(T.GE.3.5) T=3.5
C     CALCULATE K
C     FIRST CALCULATE DS/DQ, THEN K=DELTX*(DS/DQ)
   13 DSDQ=((3.*N)/(2.*R*(R**(2./3.))*(S**.5)))*
     1(1-COS(T))/((((T-SIN(T))/(2.*T))**(2./3.))*
     2(((5./2.)*(1.-COS(T)))-((T-SIN(T))/T)))
      K=DELTX*DSDQ
      C=1./DSDQ
C     CALCULATE ALPHA
      ALPHA=(K*Q)/(2*S*R*(DELTX**2.)*SIN(T/2.))
      IF(ALPHA.GT.1.)ALPHA=1.
      GO TO 100
   99 WRITE(6,12)
   12 FORMAT(* ITERATIONS OVER 20 THEREFORE STOP*)
  100 RETURN
      END
```

APPENDIX C

SUBCATCHMENT DATA

Reservoir No. (i)	[1] SFMP No.	S^i_{max} [2] Alternate C (10^6 ft^3)	S^i_{max} Alternate B [10^6 ft^3]	[3] Q^i_{max} (cfs)	Drainage Area (acres)	Reservoir Routing Constant, K (hrs)	Runoff Coefficient C	Dry Weather Flow (cfs)	Travel Time (min)
1	16-6	.66	.32	530	456	.1239	.66	4.6	50.0
2	5	1.09	".00"	240	748	.1586	.66	7.5	48.3
3	4	.24	.12	260	168	.0752	.66	1.7	45.9
4	3	.16	.10	295	112	.0614	.66	1.1	42.0
5	8	.30	.14	226	204	.0828	.66	2.0	44.2
6	2	.13	.10	370*	88	.0544	.66	.9	40.8
7	1	.22	.22	18	90	.0550	.66	0.9	43.6
8	7	.18	.10	25	124	.0646	.66	1.2	39.4
9	14-1	.14	.14	63	60	.0449	.66	.6	33.1
10	2	.79	.38	119	541	.1349	.66	5.4	32.7
11	13-11	.57	.27	155	387	.1141	.66	3.9	33.6
12	10	.23	.11	190	153	.0717	.66	1.5	27.7
13	9	.25	.12	140	165	.0745	.66	1.6	27.3
14	8	.21	.10	250	145	.0698	.66	1.5	28.5
15	7	.15	.10	95	101	.0583	.66	1.0	29.1
16	6	.23	.10	185	154	.0720	.66	1.5	24.9
17	5	1.45	.70	419	1012	.1845	.66	10.1	22.1
18	4	.18	.10	110	126	.0651	.66	1.3	30.7
19	3	.18	.10	200	122	.0641	.66	1.2	25.1
20	2	.27	.13	85	186	.0791	.66	1.9	19.4
21	1	1.13	.54	151	770	.1609	.66	7.7	24.7
22	12-3	.19	.10	82	129	.0659	.66	1.3	15.9
23	5	.40	.19	165	276	.0964	.66	2.8	29.4
24	4	.32	.15	170	222	.0864	.66	2.2	22.2
25	2	.95	.46	253	655	.1484	.66	6.6	10.4
26	1	.54	.25	73	370	.1116	.66	3.7	13.2
27	11-1	.26	.12	35	175	.0767	.66	1.8	24.1
28	4	1.10	.53	151	761	.1600	.66	7.6	28.8
29	2	.27	.13	380	182	.0782	.66	1.8	13.2
30	5	.36	.17	432	246	.0910	.66	2.5	15.1
31	1	.24	.11	107	165	.0745	.66	1.6	13.2
32	21-4	.44	.21	210	299	.1003	.66	3.0	156.0
33	3	.16	.10	33	"107"	.0600	.66	1.1	155.6
34	2	.16	.10	78	109	.0606	.66	1.1	152.4
35	1	.77	.37	207	529	.1334	.66	5.3	148.3

APPENDIX C

Reservoir No. (i)	[1] SFMP No.	S^{i}_{max} [2] Alternate C $(10^6\ ft^3)$	S^{i}_{max} Alternate B $[10^6\ ft^3)$	[3] Q^{i}_{max} (cfs)	Drainage Area (acres)	Reservoir Routing Constant, K (hrs)	Runoff Coefficient C	Dry Weather Flow (cfs)	Travel Time (min)
36	28-1	.76	.36	102	517	.1313	.66	5.2	156.8
37	24-3	.14	.10	84	102	.0586	.66	1.0	158.1
38	2	.13	".00"	70	90	.0550	.66	.9	154.7
39	1	.59	.28	220	405	.1167	.66	4.0	145.6
40	37-8	.40	.19	54	271	.0955	.66	2.7	125.8
41	7	.48	.23	155	330	.1054	.66	3.3	121.8
42	9	.67	.32	340	453	.1234	.66	4.5	127.3
43	5	1.33	.63	183	923	.1762	.66	9.2	106.8
44	3	.45	.21	60	303	.1010	.66	3.0	99.0
45	4	.16	.10	22	111	.0611	.66	1.1	101.0
46	30-1	.39	.18	52	262	.0939	.66	2.6	132.0
47	37-6	.11	.11	200	80	.0519	.66	.8	129.7
48	37-2	.23	.10	180	161	.0736	.66	1.6	130.1
49	37-1	3.60	1.71	587	2463	.2878	.66	24.6	121.9
50	44-3	.68	.32	93	469	.1256	.66	4.7	81.5
51	2	.32	.15	44	220	.0860	.66	2.2	85.7
52	5	3.34	1.59	"1393"	2289	.2775	.66	22.9	71.0
53	40-1	.34	.16	46	235	.0889	.66	2.4	98.5
54	44-1	3.31	1.57	532	2328	.2798	.66	23.3	90.2
55	48-1	.15	.15	12	62	.0457	.66	.6	91.7
56	54-1	1.13	.54	153	774	.1614	.66	7.7	91.6
57	52-2	.79	.36	260	540	.1348	.66	5.4	82.5
58	52-1	1.10	.51	136	749	.1587	.66	7.5	92.6

Notes:

1. Corresponding numbering San Francisco Master Plan
2. S^{i}_{max} = The storage capacity of reservoir i
3. Q^{i}_{max} = The flow capacity of the line leading from reservoir i to an interceptor or another reservoir.

APPENDIX D

Statistical Analysis of San Francisco Storm Data

As discussed in Chapter III, the semi-continuous simulation approach requires that storm events be defined from the continuous precipitation record. The definition that was adopted was that a 3-hour period of zero precipitation signified the end of a storm. In addition, all hourly precipitation values less than or equal to 0.05 inches were ignored in order to eliminate many small storms from consideration which would not cause overflows.

A statistical analysis was performed of the average depth and number of storms as a function of duration for the 66 year San Francisco rainfall record using the above definition. Two cases were examined: all hourly precipitation values less than or equal to 0.05 in. ignored and all values used. The average depth as a function of storm duration is shown in Figure D-1. The data is approximately linear with the difference in slopes of the lines approximately equal to 0.05 in./hr. This difference is expected since this average rate of rainfall is being neglected in the former case.

The cumulative probability curves for duration for the two cases are shown in Figure D-2. It is interesting to note that for the same exceedence probability the average storm depth depth obtained from using the corresponding durations in Figure D-1 is approximately the same for the two cases except at low durations.

This data is presented for reference purposes since it may be of interest in future work.

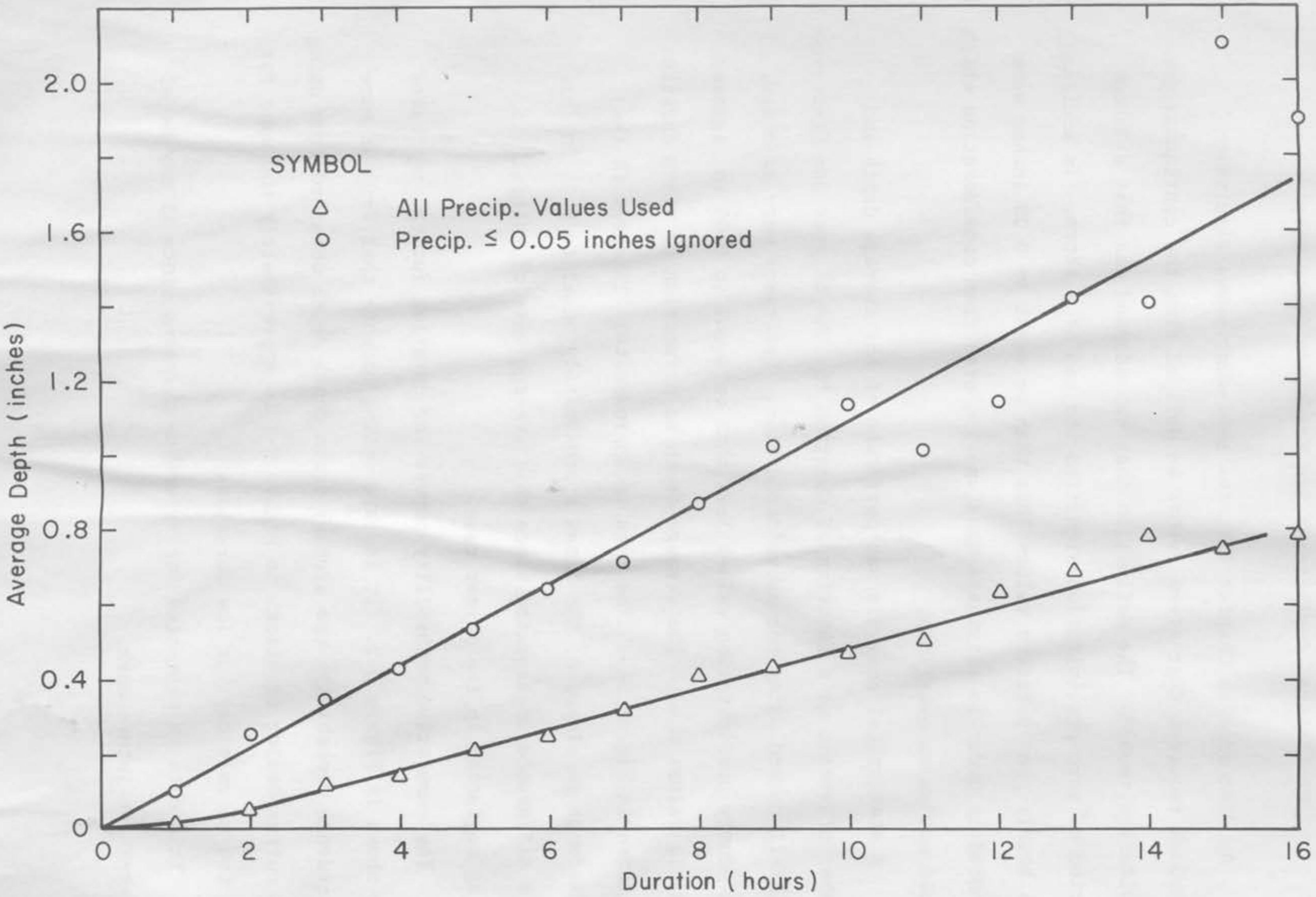


FIGURE D-1

AVERAGE STORM DEPTH VS DURATION

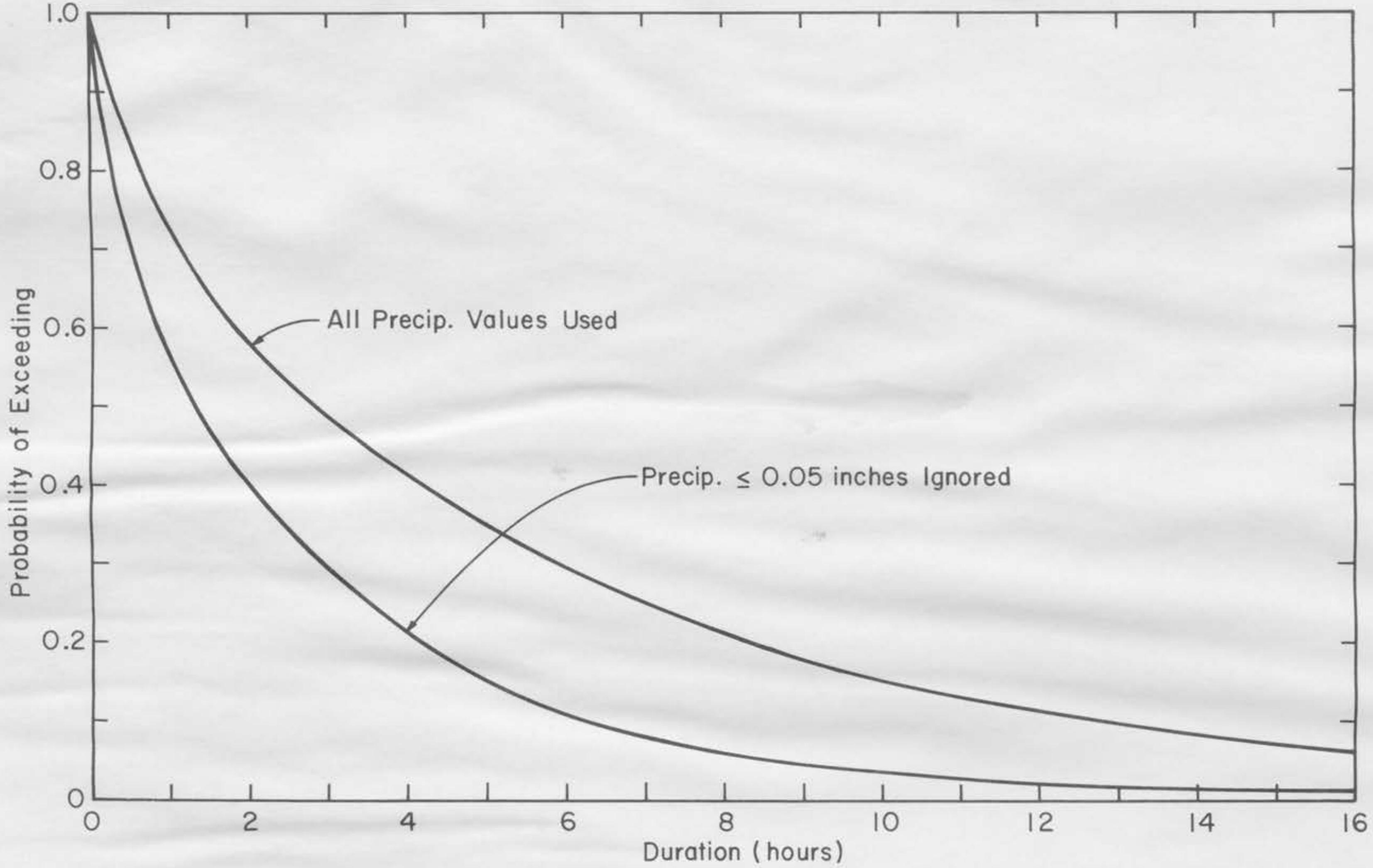


FIGURE D-2

PROBABILITY DISTRIBUTION OF STORM DURATIONS

APPENDIX E

Rainfall Data

During the earlier phases of the study considerable rainfall data were gathered. Although not all were used in the analysis, some effort was spent in the gathering and the data may be of subsequent value, therefore it is summarized here.

Rainfall depth-duration-frequency analyses were obtained from three sources [1,4,8]. The three analyses were not in perfrct agreement. A detailed study to find out the reason for this was not carried out but the same basic data may not have been used and the analysis procedures probably were not the same. Typical intensity-duration-frequency curves are shown in Figure E-1.

In addition, 5 minute precipitation data for excessive precipitation storms from 1896 to 1973 were obtained from the National Climatic Center, Federal Building, Asheville, North Carolina. These data were obtained with the tipping bucket raingage located on the roof of the Federal Office Building in San Francisco. They are presented in the following table. The 5 minute values are in hundredths of an inch and where these values were not available, hourly totals in inches are given.

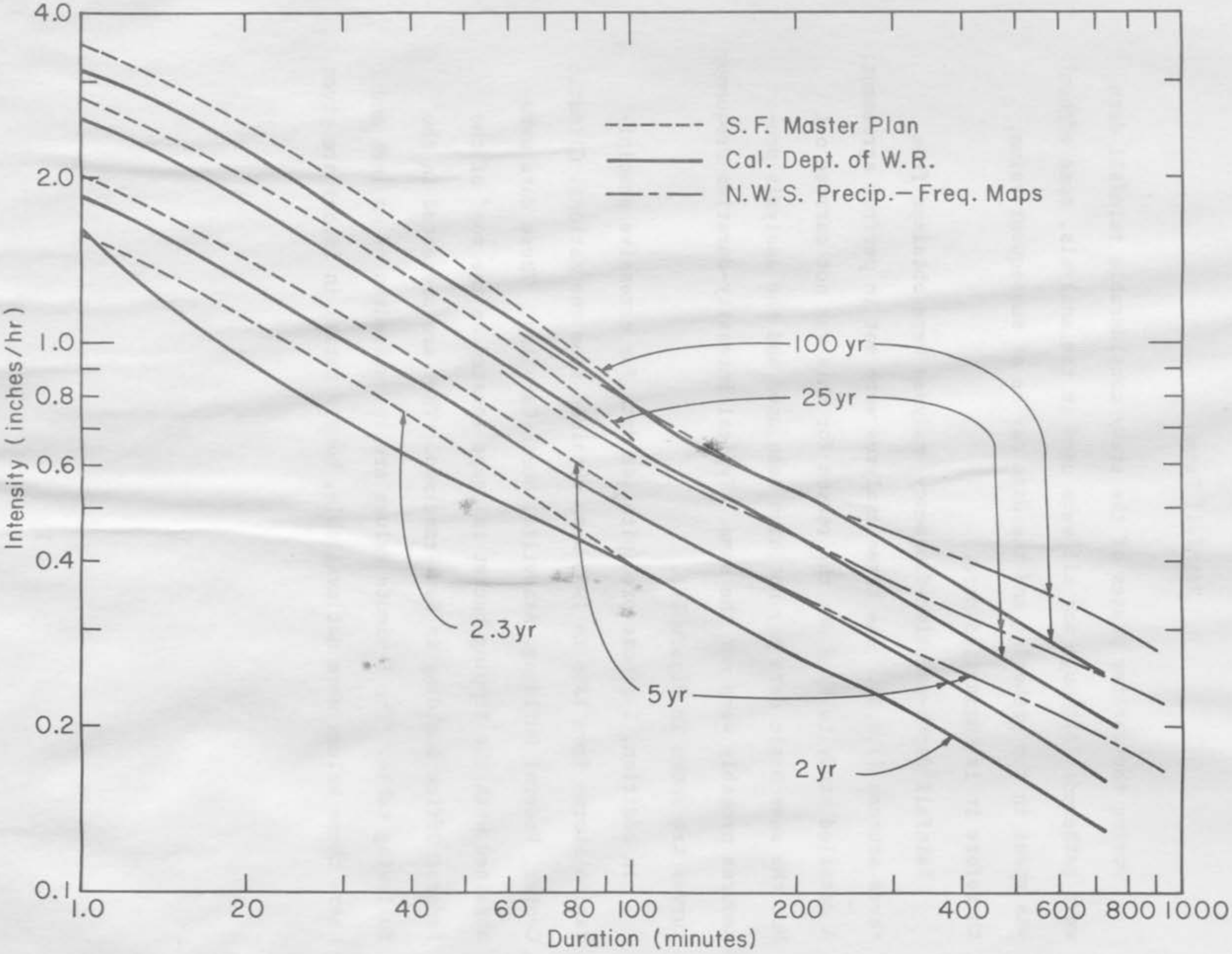


FIGURE E-1

INTENSITY-DURATION-FREQUENCY CURVES FOR SAN FRANCISCO

TABLE E-1

EXCESSIVE PRECIPITATION DATA

Date: 1/20/1894 Total Precipitation: 2.33"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5																								
10																								
15																								
20																								
25																								
30																								
35																								
40																								
45																								
50																								
55																								
60																								
HOUR TOTAL	.10	.10	.25	.20	.35	.20	.05	.05	.10	.20	.20	.13	.05	.05	.05		.05	.05	.05	.05		.05		

Date: 11/23/1896 Total Precipitation: 2.27"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5																								
10																								
15																								
20																								
25																								
30																								
35																								
40																								
45																								
50																								
55																								
60																								
HOUR TOTAL							.03	.06	.05	.05	.08	.15	.20	.44	.44	.17	.06	.04	.07	.16	.27			

Date: 11/24/1896 Total Precipitation: 0.73"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5																								
10																								
15																								
20																								
25																								
30																								
35																								
40																								
45																								
50																								
55																								
60																								
HOUR TOTAL	.01	.05	.12	.03	.02	.01	.03	.24	.07	.06	.07	.02												

Date: 2/11/04 Total Precipitation: 0.75"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5																		2	2	1	1			
10																		2		2				1
15																		2	3	1	1			1
20																		2					1	2
25																		1	1	1			1	1
30																	1	2	1				1	1
35																	1	2	1	1				2
40																		3	1	1			1	1
45																	2	2	1	2			1	1
50																	1	1		1			1	2
55																	1	1	1	1		1	1	2
60																	1			1			1	
HOUR TOTAL																								

Date: 2/12/04 Total Precipitation: 2.01"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	2	1	1		1	2	1	1	1	1		1	1		2		1							
10	2	1	1		2	1	2		2	1	1	1	1	1	1									
15	2	1			1	1	3	1	2	1	1	1	1	1	2	1								
20	2	1	2	1	1	3	1	1	2	1	1	1	1	1	2									
25	2	1	1	1	1	2	2	1		1	1	1	1		2	1								
30	1	1	1		1	1	2	1	1		1		1	1	2									
35	2	1		1	1	1	1	3	1	1		1	1	1		1	1							
40	1	1	2	1	2	1	3	1	1		1	1	1	1			1							
45	1	1	1	1	1	1	1	1	1	1	1	1		1	1		1							
50	2	1		1	2	3	1	1	1	1	1	1		1	1									
55	2	1	1	1	1	1		1	1	1	1	1		1	1		1							
60	1	2	1	1	1	1	1	1		1	1	1	1	1	1									
HOUR TOTAL																								

Date: 9/23/04   Total Precipitation: 3.58"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5				1	4	3	8			1	2									1	1	1		
10			1	3	3	5	4			1										2	1	8		6
15		1		4	6	4			1	5									1			4		1
20		1		11	4	4	1		1	2	1								2			7		
25		1		13	5				1	3									2					
30	2	1		7	2		2		1	5									1		1			2
35		1	1	8	2		2			3	1									2		1		8
40	1		1	18	2	1	1			5										2		1		1
45			2	16	2	1	3		1	9														3
50		1	3	5	2		9		2	5										1				
55			4	3	1	3	15			4									1	1		1	1	2
60	1		2	2		14			1	1									1	2		1	3	1
HOUR TOTAL																								

Date: 3/5/12 Total Precipitation: 2.07"

Minutes	Hour 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5						1	5	2									1							
10							20	1						3			1	2		1	1			
15					1		20	1						2		2	1	4		1				
20					3		24							1		4		2	1					
25					3		10							1			2		1	1				
30					3	1		1						1		1	1							
35					2		2	1							1	1			1					
40					1	1	3	1							1	1	1	1			1			
45					2	2	5								1	1	2			1				
50					1	2	7	1					1			2	1			1				
55						4	7								1	2	1		1			1		
60						4	2						1			1	1		1					
HOUR TOTAL																								

Date: 12/3/15 Total Precipitation: 3.28"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	2	2	2		3			1	1	1	3	1	1		2									
10	2	3	2		6	3	2		5	3	2	1	3	1										
15	2	2	2	5	3	2		1	2	4	4	1	2		1	1								
20	1		3	6	2				5	3	8	2	1	1					6					
25	2		2	3	2	2				4	7	2	3						5					
30	1	1	1	1	6	3	1		2	5	3	2	3			1			3					
35		1		1	3	3	1		3	5	1	1		1		1			1					
40	1	1	5	2	1	3		2	5	2	1	2	1	2					2					
45		1	3	5	1	1	1	4	2	2	1	1	2	1	2	1								
50			1	3	2	1	1	2	5	3	2	1	1	1	1									
55			1	3	2	1		1	1	2		2		1	1									
60	1		1	4	2			2	1	4		2	1		1									
HOUR TOTAL																								

Date: 1/2/16 Total Precipitation: 2.01"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5													1	3	4									
10													2	3	2						1			
15													3	4	3		1				1			
20													2	3	3									
25														2	1									
30													1	4	2									
35													2	2	2									
40													1	6	3	1					1			
45													4	3	1	1					1			
50													3	2	2									
55								1					2	3	1									
60													1	2	1									
HOUR TOTAL												.06										.32	.40	.30

Date: 1/3/16 Total Precipitation: 1.66"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	2										1											8		
10																						1		
15																								
20																								
25																								
30	1																							
35	2												2											
40															1									
45	2												1											
50	1																							
55	1																				1			
60																								

HOUR TOTAL .25 .25 .25 .15 .12 .10 .10 .10 .10

Date: 2/10/19 Total Precipitation: 2.86"

Minutes	Hour 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5				2	1	1		2	2	1	4	1	2	1	1	2	2							
10				1	1			1	1	2	2	1		1	1	2	1						1	1
15			1	1	1			1	1	1	2	1	1	1	1	2	1	1				1		1
20		1		1	1				1	3	3	2	1	1	1	2	1							2
25				1	5			1	1	3	6	1	1	1	1	2	1					1		1
30				3	1				2	3	7	1	1	1	2	2						1		1
35			1	2		1		1	2	2	2	2		2	1	2		1					1	
40			1	1			1	3	1	9	2	2		2	1	2					1			1
45			2	2			2	1	2	2	1	3	1	1	1	1	1				4			1
50			2	2				1	1	6	1	5		1	1	1		1			4		2	1
55			2		1		1	2	1	12	1	4	1	1	1	1				1	1		1	1
60			2	1			1		2	9	1	1	1		4	1					1		2	
HOUR TOTAL																								

Date: 1/17/21 Total Precipitation: 2.40"

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
												Hour												
5	2	2		3	5						1													
10	2	5		2	2						1	1												
15	1	4		2	3							3		2										
20		3		5	2							1		2										
25		1		7	8					1	1	1		1										
30		3		2	1					1		1		1										1
35	1	1		3									1	1										
40	2	2		2	1					1		1		1										1
45	2	1		2	2					1	1			1										
50	1			1	6					1													1	1
55	5	1		1	8					1														
60	3	3								1			1											1
HOUR TOTAL			.22			.09	.09	.28	.23															

Date: 12/21/24 Total Precipitation: 2.85"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	1	1	1	1			1		1	1	1		2	7	2	1	1		2					
10	1	1	1	1	1	1		1		2	1		4	6	1	1		1				1		
15	1	1	1		1		2			2		1	5	7	3	2			1					
20	1	1	1	1	1	1	1	1	1	1		1	4	5	1	1	1							
25			1	1					1	1	1		3	5	1	2	1							
30	1	1	1	1	1	3	1	1	1	1	1		3	6	2	2	1		1					
35	1	1			2	2	1		2	1		1	5	3	1	1								
40	1		1		1				1	1	1	3	5	3	2		1							
45	1	1	1	1	1	1	1		2	1		3	5	3	2	1	1	1						
50	1	1	1	1	1			1	2	1	1	3	7	4	1	1		1	1					
55		1	1	1	1	1			2	1	1	4	5	2	1	1	1			1				
60	1	1	1	2		1	1	1	2	1		1	8	2	1			1						
HOUR TOTAL																								

Date: 2/2/26 Total Precipitation: 1.18"

Minutes	Hour 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5												1			4	19				1			1	
10										1	1				2	5					1			
15										2	1	1			5	1				1				
20										1	1	1			2									
25												1		1	3									
30												1			3					1				
35										1		1		1	3	1								
40												2		1	3									
45										1	1	1		2	4	1								
50											1	1			3									
55									1	1	1			3	2									
60									1					4	10									
HOUR TOTAL													.04											

Date: 1/12/30 Total Precipitation: 0.75"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5			1																					
10																								
15			1	1																				
20																								
25				1																				
30			1	1																				
35				1																				
40																								
45																								
50				1																				
55			1	1																				
60																								
HOUR TOTAL					.62	.01					.01													

Date: 4/13/30 Total Precipitation: 1.14"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5								2	3														2	
10								4	1													2	2	
15									1													1	1	
20																						1	2	
25																							2	3
30									1													1	2	
35																						1		3
40								4	1													2	1	1
45								4	1													2	2	1
50								5														5	2	
55							3	4													1	17	1	
60								3														15	2	
HOUR TOTAL																								

Date: 12/27/31 Total Precipitation: 1.93"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	2		1	2	1	1	2	2	1					1										
10	1	1		2	1	2	3	1	1	1			1											
15	1	1	1	1	2	2	1	2	2															
20	1	1		3		1	1	4	1							1								
25			2	1	1	2	2	3	1			2			3									
30	1		3	2	1	2	1	2	1			1		1	4									
35		1	4	2	1	1	1	3	1			1			1	1								
40	1	1	4	2	1	1	1	3		1			2			1								
45			4	1	1	2	2	3	1			1	2											
50		1	2	2	2	3	3	3	1			1				1								
55	1	1	3	2	1	2	1	4	1		1					1								
60	1	1	4	1	1	2	1	3			1													
HOUR TOTAL																								

Date: 2/11/36 Total Precipitation: 2.04"

	Hour																						
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5			1			4	1		1	1	4								1			1		
10	1	2		1		1	1	1	6	3	12													
15		1	1	1		1	2		9	1	2											1		
20						1			3	1	2													
25	1	3	2	1		1			1													1		
30		9	1			1	1		1		1										1			
35		5	2			3		1				3									2	1		
40		2	2			4			1		1	1												
45		3	2			1	1				1	2												
50	1	3				3			1	1		2						9				1		
55		1	1		1	3			7	1	1	1						2						
60		1	1	1					17		1													
HOUR TOTAL													.02											

Date: 2/26/40   Total Precipitation: 1.57"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5									1		1				1		1		2		1	1	1	1
10														1			1					1	1	1
15												1			1		1	1		1	1	1	1	1
20										1			1		1	1	2		1	1	3	1	1	1
25													1	1	2	3	2	1		2	1	1	1	2
30											1		1		1		2	1		1	6	3	1	1
35										1			1	1	2	1	3			2	2	5		1
40										1						1		1			1	1	1	1
45										1	1		1	1	1		1		1	1	1	1	1	1
50								1				1		2			1	1	2			1	2	2
55								2			1			2		1	1	1	2	1	2	1	2	2
60													1	2		2	1	1	1	2	1		1	
HOUR TOTAL																								

Date: 2/28/40 Total Precipitation: 1.18"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5		1																			1		1	
10		1	1		2																			
15		1			3															1				
20	1	1	1		2																1		1	1
25	1		1		4																1		1	1
30		1			17																		1	
35	1	1	1		16																			1
40	1		1	1	4														1					
45	1				3														1					
50		1			5														1	1				1
55	1		1	1	5														1	3	1			
60	1	1		1	1														1	1	1			
HOUR TOTAL																								

Date: 3/29/40 Total Precipitation: 1.79"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5					3	1										1	1	2	2	2	1	1	1	1
10					1											1	1	1	5	1		1	1	1
15		1			1											1	1	1	2		1	3		
20																3	2	1	3	2	1	3	1	1
25				1												3	2	3	5	1	1		1	
30																2	3	1	1	1		2	2	2
35																4	1	2	1	2	1	5	1	2
40																2	1		1	3		1	1	1
45																2	5	2	1	1	1	1	1	
50																2	6	2				1	1	
55				1												5	3	2		2	1	1	1	3
60				2											2	1	2	2	1	2	2	1	1	1
HOUR TOTAL																								

Date: 3/30/40 Total Precipitation: 2.18"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	1		4	1	1	3	1	1	2			2	1	1				1	1					
10			4	3	1	3	1	1	3	1		1	1		2			1						
15	2		1	2	2	2	1	2	2			3				3	1							
20	1		1	1	2	1	2	2	1	1		1		1		2		1						
25			1	2	4	2	2	1	2	1	1	1			1	1	1	1						
30	1	1		1	2	1	3		1	1					2	1								
35			2	1	2		1	1	1	2	1	1				1		1						
40			3		2		1	1	1	2	5		2			1								
45	1	3	2	1	4	1	1	1	2	2	5		1		1									
50			2	1	1			2	1	1	1		1			1		1						
55		4	2	2	1	1	1	1	1	1	2		1											
60		4	1	1	1		1	2	1		1		1	1										
HOUR TOTAL																								

Date: 1/21/41   Total Precipitation: 0.98"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5					1		1	1		5														
10					1		1			7														
15					1		1			7														
20							1			16														
25							1			20														
30							1			10														
35						2	1	1		7														
40									1															
45							1																	
50							1														8			
55				1																				
60				1																	1			
HOUR TOTAL																								

Date: 4/4/41 Total Precipitation: 1.89"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5		1		1	2	1			4															
10		2	1	1	2			1	8	3	1	1												
15	1	1	1	1	1	1	1	1	17	2	1	2												
20	1	1	1	1	1				21	1		1	1											
25		1	1	1	1	1		1	8	1	1													
30		1	1	1	1		1	1	5	2	1												2	
35	1	2	2	1	1	1		1	3	2	1												3	
40		1	1	1	1				1	1	1													
45	1	1	2	1	1	1	1		2	2	1													
50		2	1	1		1	1		1	2	1													
55	1	1	2	1				1	1	2														
60		1	3	1		1		2	1	2	1													
HOUR TOTAL																								

Date: 3/27/47   Total Precipitation: 1.52"

												Hour												
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5														1								1	2	22
10									1													1	1	19
15					1														1			1		6
20							1			1											1	1	1	9
25																1			1		1	1	3	1
30					1			1													1		5	
35													1						1				1	
40																					1	1		1
45																					2	2	11	
50						1														1	2	1	9	
55													1							2	2		6	
60									1											1	1		10	4
HOUR TOTAL												.01												

Date: 10/24/50 Total Precipitation: 2.14"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5													5	5	5	5								
10													5	5	5	5								
15													5	5	5	5								
20													5	5	5	5								
25													5	5	5	5								
30													5	5	5	5								
35													5	5	5	4								
40													5	5	5									
45													5	5	5									
50													5	5	5									
55													5	5	5									
60													5	5										
HOUR TOTAL																								

Date: 10/25/50 Total Precipitation: 1.94"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5											5				1	1	1	1			1			
10											5			1		1					9	1		
15											5				1	2		1			1	1		3
20											5		1	1		14		5				2		
25											1			1	1	26		2			3			
30													1	1	1	12		1			2			
35													1	1		3		1			4			
40										3			1	2	2	2			1	1	2	2		
45										4			1	2	2				1		2			
50										5				2	2	1			2	1	1			
55										5			1	1	3	1				1	1			
60										5			1		1	1								
HOUR TOTAL																								

Date: 10/25/50 Total Precipitation: 1.94"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5											5				1	1	1	1			1			
10											5			1		1					9	1		
15											5				1	2		1			1	1		3
20											5		1	1		14		5				2		
25											1			1	1	26		2			3			
30													1	1	1	12		1			2			
35													1	1		3		1			4			
40										3			1	2	2	2			1	1	2	2		
45										4			1	2	2				1		2			
50										5				2	2	1			2	1	1			
55										5			1	1	3	1				1	1			
60										5			1		1	1								
HOUR TOTAL																								

Date: 10/26/50   Total Precipitation: 0.99"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5																	1							
10															1									
15		1								4														
20										5			5			16	1							
25		1								5			5			16	1					1		
30										5			4			1								
35		1								5			2			1								
40													4			1								
45													5			2								
50																								
55																1							1	
60	1															1							1	
HOUR TOTAL																								

Date: 10/27/74 Total Precipitation: 3.24"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5										1		5	5	5	5									
10	1									5		5	5	5	5									
15	1		3							5		5	5	5	5									
20			1							5		5	5	5	5									
25										5		5	5	5	5	4								
30										5	2	5	5	5	5	5								
35										5	5	5	5	5	5	5								
40							1			5	2	5	5	5	3	5								
45										5	2	5	5	5		5								
50								4			5	5	5	5		3								
55							1				5	5	5	5										
60											5	5	5	5										
HOUR TOTAL																								

Date: 11/29/51   Total Precipitation:0 .64"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5															5	5								
10															4	5								
15																5								
20																5								
25																5								
30																5								
35																5								
40														2		5								
45														5		1								
50														3										
55																								
60														4										
HOUR TOTAL																								

Date: 11/30/51 Total Precipitation: 0.43"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5						2	1		6															
10									5			1												
15						1	1		2									1						
20							1		2								1							1
25								1	2															
30									1								1							
35							1		1															
40																								
45						5																		
50						1			1			2												
55									1															
60																		1						
HOUR TOTAL																								

Date: 12/1/51   Total Precipitation: 2.54"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	1				1	2	3	1																
10					1	1	3	3	1				2											
15					2	2	4	4				4	5	6						1				
20					1	2	12	4	1			3	5	3										
25					1	2	17	5				4	5											
30	1			1	2	2	9	4				4	5		2									
35					2		3				1	5	5		5									
40	1				1	1	1	1			4	5	5		5									
45					1	2	1	1				3	5		5									
50			1	1	2	9	1					1			3									
55	1			1	1	12	2	1																
60				2	2	2	2	1			4													
HOUR TOTAL																								

Date: 1/13/52   Total Precipitation: 0.86"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5													5	5										
10													5	5										
15													5	3										
20													5	5		1								
25													5				3							
30													5		1				2					
35													5				1							
40													5						1					
45													5											
50													5											
55													5											
60													5											
HOUR TOTAL																								

Date: 1/14/52 Total Precipitation: 2.06"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5			2	1			22		2													1		
10			1			1	3		1		2		1							1		2		
15						1	2				5											1		1
20					1		3	3	5		1									1			1	
25					1	1	1	2	5											1		1		
30							2	1	7		2		3								1	1		
35						1	1	1		1	5		5								2	1		
40						1	1				5	9	1						2		6	1		1
45		1			1	1	1		1			3									10			
50		1	1			1	1					2	1								2			
55			1			18	1	2				3	1						2		1	1	1	
60		1			1	4	1	1				1									1			
HOUR TOTAL																								

Date: 1/15/52 Total Precipitation: 1.47"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5											2		5											
10					1		1				5		5										2	
15											5	1	5											
20	1										5	1	5											
25							1				7	1	5	2										
30											1		5	5										
35							2					1	5	3						1				
40							2			2			5		1	1								
45							2					4	5											
50							8	1			3	5	5											1
55			1				3				1	5				1								2
60							1					5												
HOUR TOTAL																								

Date: 4/2/58 Total Precipitation: 2.04"

Minutes	Hour 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5							1	1			2	2	8	1		1				1	3			
10								1		1	1	1	16		1					1	3			
15			1			1			1	1	1		7	1						2	1			
20									1	1	1	1	10	1						2				
25			1			1			1	1	1	1	11		1					3	1			1
30												1	7	2				1		2				
35									1	1	1	1	8	1						2				1
40						1	2	1		1	1	1	11		1					1				1
45							1				2	1	9							1				1
50							1	1	1		2	1	4	2	1					1				
55					1	1	1	1			1	1	2	1					1	1				1
60									1	1	1	3	2	1						2				1
HOUR TOTAL																								

Date: 5/23/58 Total Precipitation: 0.63"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5																			5	1				
10																			19					
15														1					4					
20																			2	3				
25																				1				
30																								
35												1		1										
40																								
45																			1					
50																			14					
55																			8					
60																			2					
HOUR TOTAL																								

Date: 10/12/62 Total Precipitation: 1.92"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5		3		1					1									1	2	2	1	1	1	
10		3	1															1	2	3	1		1	1
15		2		1	1													1	1	3	2	1	1	
20		1	1	1		1												1	1	2	1	1	2	1
25	1	1	1					1										1	2	2	1	1		
30		2				1												3	2	1		3	1	1
35		1	1															17	3	2	1	2		
40																		8	2	2		2		1
45	1	1	1														4	12	2	1		1		1
50		1	2		1												3	8	1	1	1	1	1	
55	1			1			1										1	6	2	1	1	1	1	
60	2																2	4	1		2	1	1	
HOUR TOTAL																								

Date: 10/13/62 Total Precipitation: 1.80"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5			1		3		1	1	1		1	2	1	1	1	2					1			
10	1	1			1	1	1	1		1		1			2	1								
15	1		1	1	1	2		1		1	1	1	1	2	2	2								
20		1	1		1	1	2			1					1					1				
25		1	2	1	1		3	1		1		1	1	1	1	1				7				
30	1		1	1	1	1	2	1	1		1			1	1					6	1	1		
35	1	1	1	1		1	2	2		1			1	1	1			1		1				
40	2	1		1	1	1	2			2	1	1		1	1					1	1			
45	1		1	2	1		1	1	1	1				3	1	1			1					
50	1	1	1	1	2	1				1	1		1	2	1					1				
55	1	2		1	1	1	1	1			1	1			2	1								
60	1	1	1	1	1		1		1	1	1			2	1									
HOUR TOTAL																								

Date: 12/17/67 Total Precipitation: 0.70"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5							1															1		
10																					1	2		
15																						1		
20				3	1															1		20		
25				1							1											11		
30																					1			
35					1															1	1	1		
40					3	2	1														1			
45							1														1			
50						2	1	1													1			
55					1							1							1		1			
60					1																1	1		
HOUR TOTAL																								

Date: 1/10/68 Total Precipitation: 1.15"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5		1	2	2	2	1	1																	
10			1	1	2	3																		
15		1	2	2	1	2																		
20			2	1	2	1	3																	
25			1	1	3	2																		
30		1	1	2	2	1																		
35	1	1		2	2	1																		
40			1	2	26		1																	
45		1	1	3	3																			
50	1	1		2	5																			
55		1	1	2	3																			
60	1	1	1	3	1																			
HOUR TOTAL																								

Date: 3/12/71 Total Precipitation: 1.35"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5					1						1	1	1	1		1								
10								1				2	2	1	1									
15						1					1	1	8	1	1	1								
20												3	23	1	1	1								
25												1	21	1										
30										1	1	1	7	1		2								
35							1					3	2	1	1	3								
40			1	1	1						1	1	1	1	1									
45			1								1	1	1	1	1									
50			1				1			1	1	1	1	1										
55								1		1	1	1	1	1	1									
60			1								1	1		1										
HOUR TOTAL																								

Date: 3/30/73 Total Precipitation: 0.40"

Hour

Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5								1	1					1	1		1							
10									1	1					1									
15													1		1	1	1	1						
20											1													
25						1		1	1							1	1							
30							1						1											
35															1	1	1	1						
40																								
45							1	1	1						1		1							
50																1								
55						1		1				1			2		1		1					
60											1				1									
HOUR TOTAL																								

Date: 10/11/73 Total Precipitation: 1.74"

	Hour																							
Minutes	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5			2	2	1	15	1						1											
10			2	3	1	7	2						1	1	1									
15			2	2	1	3				1			1	1										
20			2	2	2	4	1																	
25		1	1	2	1	3						1	1											
30				2	1	2							1	1										
35		1	2	2	1	5					1													
40		2	1	1	6	3							1											
45		3	2	2	7	1						1	1											
50		1	1	2	8	5						1		1										
55		2	2		3	5							1											
60		3	2		7	5				1		1	1											
HOUR TOTAL																								

5622495