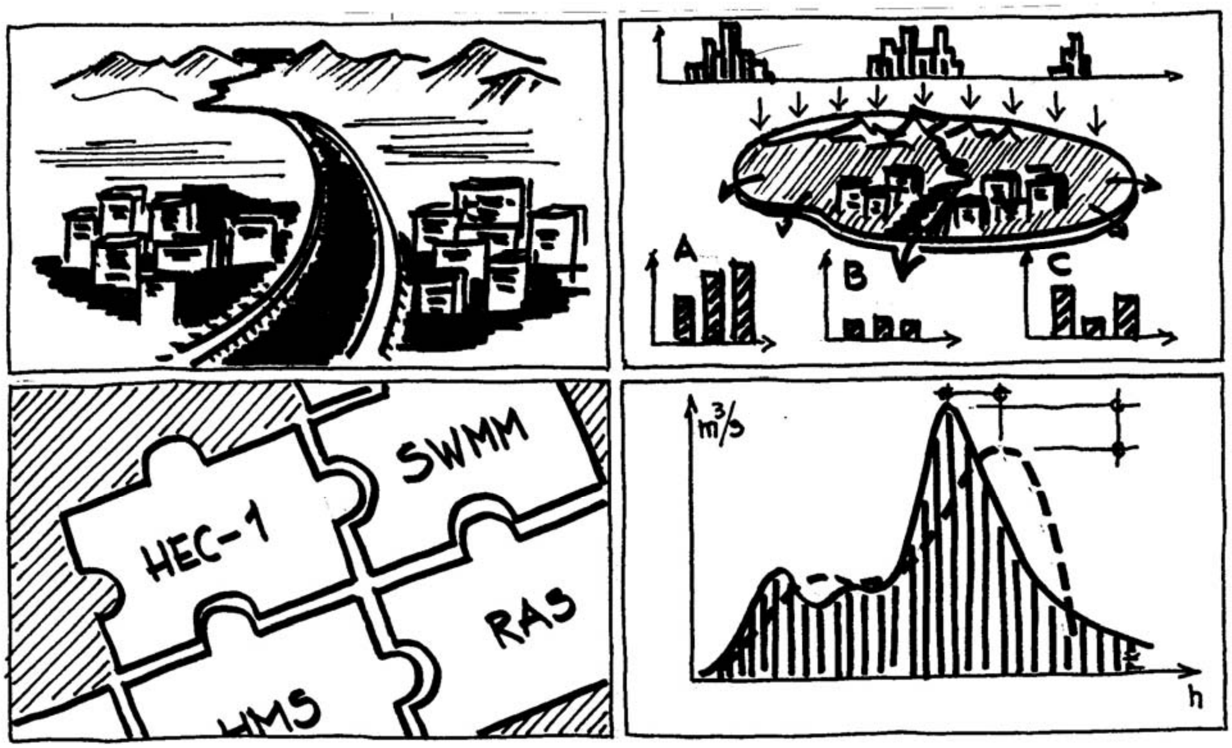
US Army Corps
of Engineers
Hydrologic Engineering Center

Proceedings of a Workshop on

Urban Hydrology & Hydraulics



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14. ABSTRACT

A workshop on Urban Hydrology and Hydraulics was held on 13 - 15 September 1994 at Davis, California. The purpose of the workshop was to examine the persistent issues in urban hydrology and see if the issues are now surmountable because of advances in technology, increased understanding of the fundamental processes, or more pragmatically, because circumventive procedures have been developed. The other purpose is to quantify the real and current problems being faced by Corps staff in the field of urban hydrology, and propose new methods or tools to solve the problems.

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13 - 15 September 1994

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King County
Sacramento County
University of California, Davis
University of Guelph
US Geologic Survey
Denver Urban Drainage and Flood Control District
Bechtel Corp.
Virginia Tech
University of Colorado at Boulder
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FOREWORD

The Hydrologic Engineering Center last examined urban hydrology in detail in the Seventies. A workshop was held, problems and solutions identified, and courses delivered which conveyed the latest methods to the Corps practitioner. This past effort was successful in that several improvements were made to existing models and modeling expertise was disseminated from researchers to the users. Other issues related to urban hydrology were identified for which significant progress was not made. Insufficient data for model calibration is the best example of these.

The current research effort logically began with an examination of past efforts by HEC and the urban hydrology community at large. Interestingly, some of the issues discussed during the HEC sponsored workshop in 1971 survived the intervening twenty years to reappear, almost word for word, in the workshop described in these proceedings. Therefore, one goal of this workshop was to examine these persistent issues to see if they are now surmountable because of advances in technology, increased understanding of the fundamental processes, or more pragmatically, because circumventive procedures have been developed. Several themes of the papers herein are directed towards this goal. The second fundamental task of the workshop was to quantify the real and current problems being faced by Corps staff, some of which evolved since the last workshop, and propose new methods or tools to solve them. Several of the papers do just this. Therefore, it is hoped that these proceedings are not just a repetitive discussion of the material covered in the 1971 proceedings, but are instead a natural secession to them.

ACKNOWLEDGEMENTS

Credit for the successful completion of the workshop, and these proceedings, lies with the workshop participants. Though backgrounds and opinions varied quite a bit, a friendly and productive rapport existed throughout the workshop. Therefore, the names of those to be acknowledged for their efforts in this endeavor are listed in the last section of these proceedings, which is the list of participants. Peter Kaufmann deserves additional mention for the fine illustrations found on the cover and at the session headings. The coordination of the workshop was a joint effort by Troy Nicolini, Arlen Feldman, and Peter Kaufmann.

EXECUTIVE SUMMARY

INTRODUCTION

The Hydrologic Engineering Center hosted a workshop on urban hydrology and hydraulics on 13 - 15 September 1994, as a component of the ongoing research work unit (#32875) titled Urban Hydrology Methods/Models. The workshop consisted of four sessions: Corps perspective, hydrology, hydraulics, and modeling methods. Twenty-six papers were presented. The participants included representatives from the Corps and the USGS, private consulting industry, academia, and state and local agencies. The major objective of the workshop was to bring together software developers and users to discuss use of existing models, as well as the needs for future models.

These proceedings are intended as a means of sharing the experiences of the participants in dealing with a variety of urban hydrology and hydraulics modeling issues. Ideally, lessons learned, or standard procedures could be extracted and converted into guidance for future urban work. Although there is a portion of urban hydrology for which this can occur, the varied nature of urban studies is such that creative, never-before-done solutions will always be needed. Therefore, the summaries of the proceedings that follow attempt to draw together the quantifiable notions from the papers, while the papers themselves serve as a library of creative solutions to the unique elements in each study.

On several issues, there were exactly opposite opinions between the authors. Many of these instances of disagreement stem from the different environments in which the participants work. This was to be expected since the workshop roster spanned a broad spectrum of backgrounds. Although these summaries describe some of these disagreements, no attempt is made to choose the correct side of an issue. Instead, consensuses - when they exist - are noted.

THE STATE OF URBAN HYDROLOGY IN THE CORPS

The workshop illustrated that there are two predominant ways that the Corps becomes involved in urban hydrology and hydraulics. First, through interior flood hydrology studies, and second through assistance to states projects. These two common contexts show up a preponderance of certain types of urban hydrology and hydraulics modeling problems. For example, since interior flooding is low in the flood plain, it frequently involves very mild slopes. Resultant issues include instability of routing computations, and accounting of the volume of ponded water. Hammond, Webb, Dotson, and Yamamoto illustrate some of these issues in their case studies. Also for interior conditions, if the river and local flow are coincident and the urban drainage is complex, the timing of runoff is important. For these reasons, hydrologic methods must be used which perform well from volume accounting and timing respects, in addition to estimating a peak flow. In the case of assistance to states projects, the full range of urban studies is encountered. One item specific to these projects, illustrated in Rodman's paper, is the lead role the Corps can take in a regional approach to doing urban (and general) hydrology.

Corps offices are applying the urban hydrology features and capabilities of HEC models such as HEC-1 and IFH. The kinematic wave method with curve number loss rate computation is often used. Additionally, EPA models such as HSPF and SWMM are being used for portions of studies, with ad hoc communication of results between HEC and non-HEC models.

Several Corps Districts are quite far along in doing urban hydrology and hydraulics. Chicago, Huntington, and Los Angeles have been involved in some major urban studies, as demonstrated by the papers. Others, such as Charleston are in the midst of major studies. Interestingly, some of the difficulties being encountered by Charleston, for example, caused nods of empathy from other districts. This verified the need for dissemination of knowledge between districts. Of course, this dissemination is ongoing in several informal ways. The workshop was intended as a formal mechanism for such exchange, and the proceedings are intended to further broadcast it to others.

ISSUES AND RECOMMENDATIONS SURFACED

The paragraphs below highlight issues that were mentioned by several participants, with particular attention paid to concurrences of opinion and visa versa. Many other equally important issues can be found in the individual papers and session summaries. The material is broken down into two categories: that which is related to using existing tools and technology, and that which is related to the development of new methods and tools.

Guidance on the Use of Existing Technology

1. Several papers stressed the importance of not modeling in too much detail. The difficulty lies in determining how much is too much. Unfortunately, the only sure way of knowing is after budgets are exceeded trying to calibrate and make sense out of an inappropriate model. Webb's paper documents such a case and also illustrates why there is a temptation to model at a detailed level. In urban interior hydrology, the need frequently arises to characterize the transition from overland to channel to closed conduit flow. This is especially difficult with larger flows which surcharge the pipes, pond and travel unexpected overland flow paths. An instinctive approach in this case is to model in great detail because there is no apparent way to average the impact of the system on the flow. However, Webb's experience, as well as that of some of the county representatives, indicates that too much detail can result in a less tractable model, unpredictable results, and fail to give the expected increment in accuracy. A corollary to not using too much detail is to make sure the accuracy which demands detailed analysis is needed in the first place. In fact, the process of deciding on the accuracy needed and the process of deciding on level of detail are highly dependent on each other.

2. Taking a top down approach and starting with simple methods are explicitly recommended in Kaufmann's paper to ensure efficiency of the study. In papers by Webb and Dotson, the virtues of top down are illustrated. Webb describes how early in his study, excessive effort was spent trying calibrate a SWMM model for the closed conduit portion of the flow hydrograph. Later it was recognized that the closed conduit portion of the hydrograph was a small portion of the total flow for larger events. He subsequently realigned the priority of the closed conduit analysis and got the project back on schedule. Webb summarizes that an earlier overview evaluation of the system could have avoided the excessive effort in the first place. In a similar, but smaller study, Dotson shows how establishment of the coincident nature between the interior areas and the receiving river allowed simpler hypothetical storm analysis of the interior flow. The message is that stepping back and looking at the whole system <u>first</u> saves time during the detailed analysis. These are just two examples, others reverberated the same idea. It appears that designing an urban study involves a lot of choosing the level of detail of analysis for the various components and regions of the study. This is an area for which it is particularly hard to give specific advice. The best that can be offered is to use simple methods first to gain an appreciation for the important elements of a study; then, proceed to complex methods only after the need for their use is verified.

3. Doan and Kibler demonstrate that storms and loss-rates combinations used for larger watersheds may not work well, or at least have unpredictable results, for small basins. This is because of their short response times. With short response times, the runoff from a basin is dominated by the initial loss function used and the rainfall duration. DeVries also points out that the common practice of using a very small rainfall time step for small basins can inadvertently over-predict rainfall peak intensity when several small basins are modeled together.

4. Not surprisingly, there was disagreement about the use of continuous simulation modeling. The advocates of continuous simulations included James, Hartley, and Lumb. There was little compromise on this point; the strongest advocates of continuous simulation would not concede to a single scenario where design event modeling was appropriate, and vice a versa. Several Corps attendees, including Webb, Fogarty, Hammond, held that the Corps uses continuous simulation for portions of a study for which it is needed. For example, Fogarty's case study involved using continuous simulation to analyze the interactions in a system of tunnels, channels, and reservoirs.

5. During the application of kinematic wave method to urban situations, the issue comes up of the maximum length for overland flow. McCuen offered reasoning for setting limits based on physical characteristics, slope and roughness, instead of hard and fast rules. Effectively, flat, rough surfaces support overland sheet flow for a shorter distance than steep, smooth surfaces. A similar issue is faced when using the travel time method of TR-55 to determine time of concentration. Hall offers limits suitable for his area which agree qualitatively with McCuen's findings. McCuen points out, as does Hall, that the overland flow portion of flow path can contribute the single most amount to the total travel time. Therefore, its accurate characterization is important.

6. The assumption that rainfall frequency equals flow frequency was recognized by several as deserving scrutiny, but especially questioned by James and DeVries.

7. Hammond and Webb used EXTRAN for their closed conduit hydraulic modeling while Fogarty and Powell used UNET. Their papers have the details of how each option had its pluses and minuses. In general, EXTRAN has built in capability for such features as surcharged pipes, tidal boundary conditions, which have been refined over the years so that they work in a predictable, mathematically stable fashion. The difficulty lies in learning to use the model, and in obtaining technical support for it (the EPA is no longer providing general support although it is available from private sources). Another potential problem is that EXTRAN is not currently directly connected with Corps supported models such as HEC-1 and HEC-IFH. UNET, on the other hand, has the look and feel of other Corps models, is connected via DSS to other Corps models, and is supported by HEC. Unfortunately, UNET hasn't been around for a long time or been applied in a wide variety of urban situations. Therefore, it doesn't have all the built in features that EXTRAN has.

8. For the unit hydrograph method, the relationship between lag and time of concentration was mentioned by Hall and DeVries. SCS gives a general relationship of lag=.6TC. For small urban basins, DeVries mentions lag = .8TC while Hall used unity. Looking at historical data is the only way to be sure of a relationship for a particular region. However, lacking the data, the SCS value is normally used as a default. It would be valuable to have a different default value for small urban basins.

9. CN loss rates and kinematic wave rainfall runoff transformation were predominant in the Corps studies presented, while in general the county representatives advocated not using the kinematic wave method. The county perspective seems natural for their work where reproducibility is paramount. The kinematic wave method needs more guidance so that different analysts can come to similar answers.

10. During discussions at the end of the workshop, it was pointed out that there is a lack of information regarding urban models. This makes it difficult to choose a model for a particular application. Important information includes model capability, availability, source of support, hardware requirements, and dissemination of guidance and error corrections. As Wang pointed out, sharing of information between users is very valuable. Several participants, including Huber, Heaney, Schmidt, and Kibler addressed this issue with mention of recent papers which profile available models, and with descriptions of their agencies protocols for model distribution, update, support, and maintenance. A common thread was the use of the internet for exchange of ideas, and bug correction news. A profile of available urban models is given in chapter six of the ASCE's "Design and Construction of Urban Stormwater Management Systems" (1992). Also useful is a paper by Donigian and Huber (1991).

11. The issue came up regarding the development and use of regional equations and standard models or practices. Hall and Urbonas support the idea and have worked to develop

standards that can be used by all entities within their area. This consistency of approach is held up as a virtue in itself because it's conducive to taking a regional approach to urban planning. Rodman described attending scores of meetings in an attempt to rectify the problems from neighboring public agencies working together, and he concludes that standardized models for all to use are a solution. Webber and Harvey, also call for consistent regulations and design standards for dealing with their urban studies. Hartley's King County takes a somewhat different approach by developing much of the base model data - such as infiltration for particular soils - upon which a site-specific model is built. It appears that both Corps and non-Corps participants doing work within the urban agency environment advocate developing and adopting regional equations and standardized models and methods. On the other hand, James essentially said that the use of standardized, equations-based methods was taking a step back because it may limit appropriate thinking about the problem.

12. Kaufmann, Heaney, James, Hartley, and others advocated an integrated approach to doing urban hydrology which involves looking at the entire system before choosing methods and design criteria. As an ideal, in Switzerland, all urban hydrologic studies are required to first look at the entire system, assess the important points, and to lay out possible solutions - before any detailed analysis takes place. Similar thinking takes place in King County, where Hartley calls the first phase of a study "triage" during which the sick portions of a study area are identified and critiqued as to their savability. James goes further to say that the entire system in which hydrology takes place should be examined. Therefore, flood control studies should consider the effect that making the flood plain safe will have on human population, and whether the ecosystem as a whole can sustain that increase. Heaney focussed on using an integrated view of urban hydrology as a fundamental platform for future model development. The current practice of water quality models being developed by water quality agencies, and flood models being developed by flood control agencies does not recognize the inherent interaction amongst urban hydrologic processes.

13. During the presentation of a comparison of calibrated unit hydrograph and un-calibrated kinematic wave results, several participants pointed out that methods should be compared on an equal basis. This highlights the disparity in the quality of models developed with and without calibration data. The need for better ungaged parameter estimation techniques has been acknowledged for a while. This is of particular importance for urban modeling since the majority of work occurs without the benefit of gaged data.

14. Improved accuracy comes from an increase in input quality, not from more complex methods with the same old data. Urbonas argues this point from the standpoint of moving to distributed models when the data is barely adequate to do lumped modeling. McCuen calls for using as few parameters as possible so that sources of variation can be kept under control.

15. Both Kibler and Huber stated that one of the more important measures of a model's worth is consistency, defined by similar results from different modelers using the same information. Huber goes on to conclude that consistency is achieved through guidance, and that

the key to successful guidance is for the subject method to be as physically based as possible. He also states that in urban studies, physically based methods are more achievable because characteristics such as pipe dimensions and percent impervious are more easily estimated. Urbonas also desires consistency and believes that it is essential to harmony amongst the many different interests found in urban studies. However, his answer to consistency is simplicity in method reflected in a minimum number of input parameters.

16. Several participants pointed to volume accounting, water ponding, and backwater effects as important considerations in urban settings and illustrate how they addressed them. Yamamoto identifies backwater effects of urban structures as the largest cause of uncertainty. Both Webb and Hammond faced the problem of modeling the storage within a flooded interior area. In their studies as well as in Fogarty's, surcharge from pipes was particularly hard to quantify. The difficulty lies in the fact that some surcharged flow may travel overland before reentering the closed conduit system at a different point. Schmidt described modeling this phenomenon and also gave an approach for designing detention facilities using volume criteria instead of peak criteria.

17. Parameter sensitivity in the application of hydrology and hydraulics models was brought up by a couple participants. A few participants specifically asked that guidance be developed for existing models, and identified this as a much more important task than developing new models with more parameters for which sensitivity is not known. Of particular importance is the sensitivity of parameters for methods which have been developed to address larger, non-urban flooding (and for which the user community has more expertise), but are now being applied in small urban basins. Both McCuen and Hall offer some guidance in that they identify the overland flow portion of travel time computation as the more driving component, i.e. the travel time is more sensitive to this parameter. Kibler and Doan present a similar finding that the rainfall-runoff response is highly sensitive to the rainfall duration and temporal distribution.

18. Webb did sensitivity analysis on representative basins in his study and found that lumping several small subbasins gave similar results as when they were separate, with much less work. DeVries supported this idea in general, and in particular from the perspective of rainfall concerns. Their message is to not automatically subdivide in pursuit of accuracy because it is time consuming and may not give better results. This does not imply that subdividing is always a bad idea, but that for any study there is an appropriate level of subdivision which balances the desired accuracy against the effort required to achieve it.

<u>New methods and tools needed</u>

1. A few workshop participants made a case for an integrated modelling package capable of all tasks at all levels of detail/effort. Webb explicitly made this one of his suggestions, others implied it by the nature of their wish lists. For example, a suggestion was made for a single package capable of doing reconnaissance level analysis all the way through very complex

analysis so the user doesn't have to change models for different phases of a project. This would also allow the user to do work with available data/expertise, with the option of upgrading the model later without starting over. Barkau's modification of UNET to handle closed conduit flow is a step in this direction. As an interim (and perhaps final), solution it is likely that connections between available models will be stressed. There was unanimous support for this amongst participants. In fact Webb's, Hammond's, Hartley's, and Fogarty's case studies involved connecting several models via HEC-DSS as the central database. Each of these connections was custom, or ad-hoc. A coordinated effort was recognized as the logical next step. This approach is in agreement with Fogarty's idea that the urban hydrologist must have a good "tool box" of varied hydrologic and hydraulic capabilities. The notion of an integrated model, as well as the notion of several models connected via DSS are both in support of Heaney's principle of integrated urban hydrology.

2. Urban studies frequently involve large amounts of spatial data which is obtained, managed, and supplied to models in a variety of ways. This was illustrated in several case studies presented. A natural aid in the use of spatial data is the coupling of GIS with hydrology and hydraulic models. Efforts in this area are ongoing. Huber discusses work to develop an ARC/INFO shell for the SWMM model, and HEC and others are involved in similar GIS interfaces for their models.

3. Several participants urged HEC to reflect a more positive bias towards environmental issues in simulation models. This seems in order since as Choate points out, Corps studies are increasingly addressing water quality and related issues. Although HEC is not the principal agency responsible for environmental modeling, it was acknowledged that modeling development decisions should be made so as not to preclude water quality analysis. The notion of integrated urban hydrology - as put forth by Heaney, James, and Kaufmann in various forms - leads to the conclusion that HEC models should at least have easy-to-use connections to water quality models. Otherwise, the model user is being biased against looking at the whole system by the very nature of the tool.

4. Finally, an important issue discussed by Heaney and others is the development of new methods. Sources of funding for model development were discussed, as well as mechanisms for model support and maintenance. There was no answer to the funding problem. In fact, it was pointed out that the Corps' enviable means of providing model support may be changing. Also discussed by Huber and others was the philosophy of developing new generations of models. In particular, the virtues of building on the old - shorter development time and proven routines - were compared to the virtues of starting fresh - new paradigm advantages and easier to maintain final product.

CONCLUSIONS AND FUTURE WORK

As mentioned in previous paragraphs, one purpose of the workshop was to disseminate existing technologies and practices. The workshop proceedings are the first step. Subsequent guidance may come in various forms with the goal of answering questions which were brought out during the workshop, but not necessarily resolved. For example, choosing storm durations needs to be addressed, as pointed out by Doan, Kibler, and DeVries. Future guidance work will also address issues not brought up at the workshop, but that have surfaced through other means such as the technical assistance HEC provides to districts. For example. the application of the kinematic wave method to urban systems seems to be causing problems for some districts, yet it wasn't mentioned a great deal in the workshop.

A second goal was to determine what modeling needs are not being met. A few needs were identified during the workshop which are described above. HEC will proceed on those which are clearly needed such as the connection between model using DSS (Huber is also working on this). Others will need further investigation before a course of action is planned.

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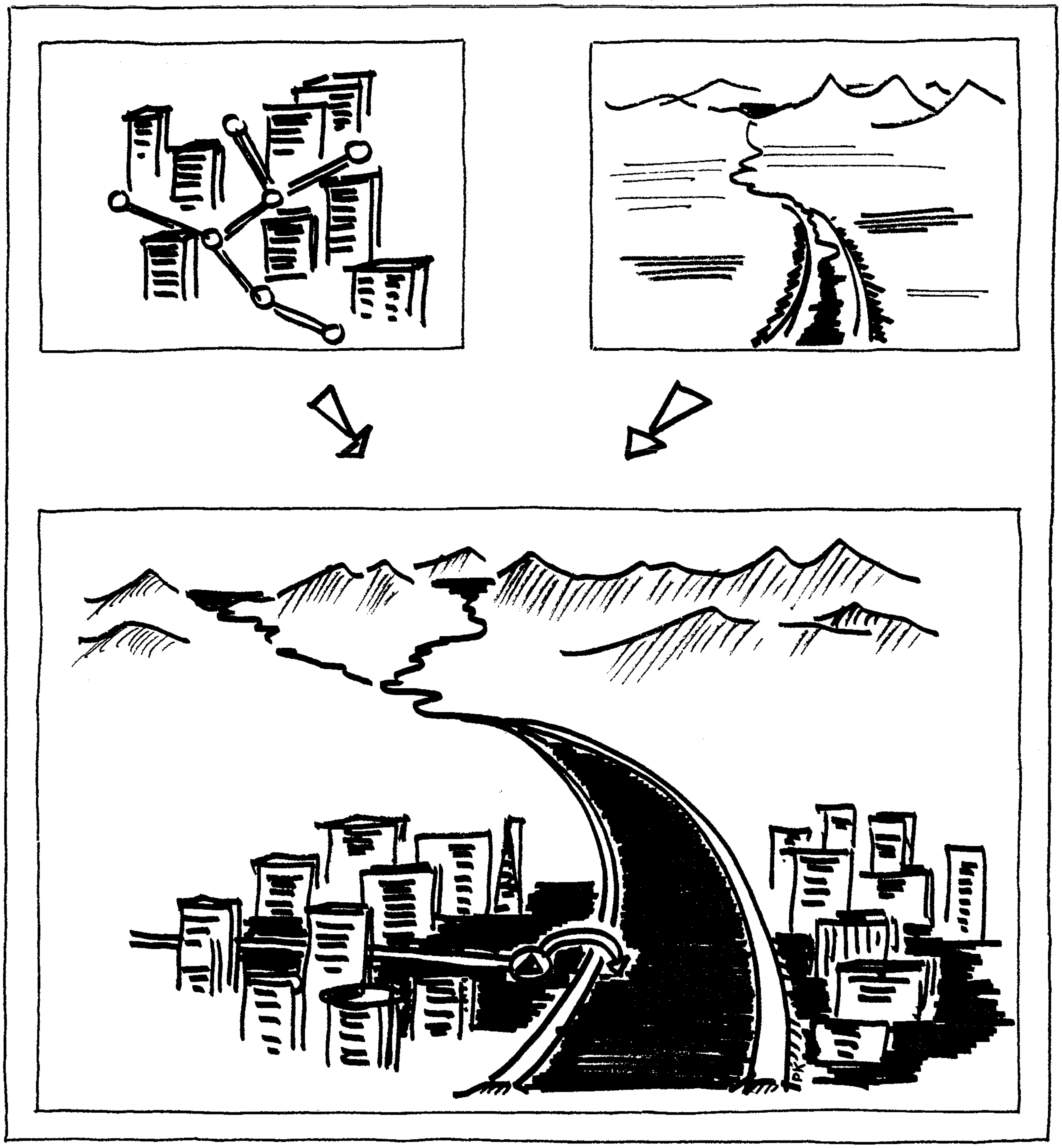
Donigian, A.S., and Huber, W.C. (1991). "Modeling of non-point source water quality in urban and non-urban areas." EPA/600/3-91/039 (NTIS PB92-109115) EPA, Athens, GA.

Session 1:

Current Corps' Practices



SUMMARY OF SESSION 1: **Current Corps' Practices**

Overview

This session was intended to set the stage for the rest of the workshop by tabling the types of problems being faced by Corps districts, as well as the solution methods being applied to solve these problems. The session was started off with a paper by Arlen Feldman and introductory comments by Ming Tseng in which he thanked the participants for bringing their experience to the workshop. A panel discussion was the main activity of the session.

Paper Presentations

Paper 1. Arlen Feldman, Chief, Research Division, Hydrologic Engineering Center, presented a paper titled "HEC Models for Urban Hydrologic Analysis." After reviewing the role HEC plays in the Corps and HEC's past activities in urban hydrology, Arlen gave an overview of the modeling capabilities within several HEC models for urban analysis. Particular attention was paid to the methods of the rainfall-runoff model, HEC-1, which are suited for analyses typically found in urban studies. These include various loss rate methods, as well as rainfall transformation using the unit hydrograph method or using a variable distance step, finite difference solution of the kinematic wave equations. Other models described were the interior flood hydrology model (HEC-IFH), and STORM which is used for sizing of storage and treatment facilities for storm water runoff. The communication between models was shown to be facilitated by the use of HEC-DSS as a central data base management system. Arlen finished by describing HEC's current efforts to develop new generation software which take advantage of recent computer advances.

Panel: Recent Corps Urban Studies

The panel session included five Corps engineers who described their efforts on recent studies.

A. Mike Choate, Chief, Hydrology Section, Jacksonville, discussed his activity in urban hydrology and hydraulics. Most of their technical participation in urban studies occurs in Puerto Rico and the U.S. Virgin Islands. The hydrology in those areas typically involves steep catchments which empty onto flat urbanized alluvial floodplains. TR-55 methods are frequently used, as well as the Interior Flood Hydrology model (HEC-IFH). Mike listed a few areas of concern related to the level of detail at each study phase, overestimation of flows resulting from routing, and environmental requirements.

B. Steve Yamamoto, Pacific Ocean Division was not able to attend the workshop, but he submitted a paper titled "Kawainui Marsh Flood Control Project, Case Study in Urban Hydrologic Modeling." The paper describes an interior flood hydrology study with interesting hydraulic conditions in the interior as well as exterior areas.

C. Joe Evelyn, Chief, Hydraulics and Hydrology Branch, Los Angeles District, described several studies in the Southwest. He used the example studies to illustrate the types of products demanded of hydrologic and hydraulic analysis. He then described the improvements to procedures or tools needed for urban hydrology studies.

D. Bill Doan, Omaha District, described a recent effort which compared several models for their suitability to perform urban hydrology. The models compared were HEC-1, SWWM, CUHP, and KINNET. The impact on results was shown for different implementations of kinematic wave for runoff computation, as well as different routing methods, loss rates, and temporal storm distributions. Rainfall distribution, in conjunction with the loss rate function were found to have more impact on results than the runoff computation method.

F. Paul Rodman, Fort Worth District, described recent studies. His emphasis was on how the modeling efforts were carried out to aid in the decision making process through effective communication of decision impacts. He concluded that a consistent, standard set of models is indispensable from this standpoint.

G. Joe Weber, Hydraulics and Flood Plain Management Services Section, Seattle District described two different case studies. In both cases, the technical issues of the studies were made more difficult to resolve because of varying regulations and standards across respective jurisdictional boundaries. Based on his experience in these studies, Joe recommends standardization of models, as well as regulations.

During discussion, Heaney pointed out that a recurring theme in the case studies was that an integrated approach was needed, both from a modeling and interagency perspective.

HEC Models for Urban Hydrologic Analysis*

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ABSTRACT

The Hydrologic Engineering Center, HEC, has several numerical models for simulation of hydrologic and hydraulic processes in urban areas. This paper will focus on new developments and applications procedures for the surface water hydrology models. The primary surface water hydrology model is the HEC-1 Flood Hydrograph Package. It can simulate the precipitation-runoff process in a wide variety of basins, from small urban areas to large river basins. It also has many features which facilitate its application to urban areas. The next generation of HEC-1, termed the NexGen Hydrologic Modeling System, HMS, is currently under development. A new model to analyze flooding in interior areas (e.g. on the land side of a levee) was just released. An older model (STORM) for urban storm water and combined sewer storage and treatment is still used in the profession but not actively supported by HEC. These models (primarily HEC-1) will be discussed in relation to urban hydrologic design. Future directions of the Corps new "Urban Hydrology Methods/Models" research work unit will also be discussed.

* Presented at the 73rd Annual Meeting of the Transportation Research Board, National Research Council, January 9-13, 1994, Washington, D.C.

HEC Models for Urban Hydrologic Analysis

INTRODUCTION

HEC Hydrologic Background

As the Corps' national center for hydrologic engineering and analytical planning methods, the Hydrologic Engineering Center, HEC, addresses the needs of the Corps field offices. HEC has developed and supported a full range of simulation models for understanding how water resources systems function[1]. It is assumed that experienced professionals can deduce the appropriate solution to a problem given the insight provided by selective execution of the simulation models. This deduction process has historically been and will continue to be the dominant methodology for planning and operational decisions in the water resources community.

This paper primarily addresses the urban-hydrology software of HEC. Not included in the paper is information about HEC's river hydraulics, flood damage, flood frequency, reservoir system, and water quality models. In recent years, many improvements have been made to: the steady state water surface profile model; the sediment scour and deposition model; and the unsteady flow model. In the area of flood damage analysis, a flood damage analysis package provides a comprehensive set of tools to evaluate flood damage reduction, and a new program accounts for project benefits during flood event operations. HEC also maintains a set of non-point source, river, and reservoir water quality simulation models.

Urban Hydrology Background

The U.S. Army Corps of Engineers flood control responsibility has applied to many geographic settings. The flood damages to be prevented are primarily in urban areas. HEC developed several computer programs and methods to analyze and compute urban flood damage. The principal program is the Flood Damage Analysis Package[2]. The subject of this paper is not the damage computation, but the hydrologic simulation software (e.g. HEC-1) developed for urban areas. The Corps has made flood damage reduction investigations of just about every type of urban area, from sparsely developed areas to major metropolitan areas. For these studies, traditional hydrologic models have been applied to the urban areas; oftentimes, new runoff parameters were added to the models to simulate the particular watershed characteristics of urban areas. Many of the computer simulation models are simply adapted to the particular infiltration, runoff, and channel characteristics of urban areas.

In the Corps Urban Studies Program of the 1970's, new models were developed to meet the specific needs of those studies. In other studies, e.g. the Expanded Floodplain Information Studies, major changes in use of geographic data were made; this was the start of geographic information system, GIS, usage in hydrologic modeling. In all cases, HEC strives to develop physically-based simulation models which are easily applied in ungaged areas. This is especially true in the urban situation. The following sections of this paper discuss the urban hydrology software development and application activities of the HEC.

HEC-1 FLOOD HYDROGRAPH PACKAGE

Background

The HEC-1 Flood Hydrograph Package[3] computer program was originally developed in 1967 by Leo R. Beard and other members of the HEC staff. The first version of the HEC-1 program package was published in October 1968. It was expanded and revised and published again in 1969 and 1970. To simplify input requirements and to make the program output more meaningful and readable, the 1970 version underwent a major revision in 1973. In the mid 1970's, increasing emphasis was being placed on urban storm-water runoff. A special version of HEC-1 was developed which incorporated the kinematic wave runoff techniques that were being used in several urban runoff models. Special versions of HEC-1 were also developed for other purposes. In 1981, the computational capabilities of the dam-break, project optimization, and kinematic wave special versions were combined into a single package. In late 1984 a microcomputer version (PC version) was developed. A menu capability was added to facilitate user interaction with the model; an interactive input developer, a data editor, and output display features were also added.

Current Version

The latest version, Version 4.0 (September 1990), represents improvements and expansions to the hydrologic simulation capabilities together with interfaces to the HEC Data Storage System, HEC-DSS (see later section of this paper). The DSS connection allows HEC-1 to interact with the input and output of many HEC and other models. (DSS will be an important capability for marrying several models needed for complex urban runoff situations as discussed later.) New hydrologic capabilities in HEC-1 include Green and Ampt infiltration, Muskingum-Cunge flood routing, prespecified reservoir releases, and improved numerical solution of kinematic wave equations. The Muskingum-Cunge routing may also be used for the collector and main channels in a kinematic wave land-surface runoff calculation. This new version also automatically performs numerical analysis stability checks for the kinematic wave and Muskingum-Cunge routings. The numerical stability check was added because many users did not check the validity of the time and distance steps used in the model.

In September 1991, an alternate version of HEC-1 (version 4.0.1E) was released for use on extended memory PC's. A hydrograph-array size of 2,000 time intervals is now available in this version. The increased array size reduces limitations encountered when simulating long storms using short time intervals. For example, simulation of a long (96-hour) storm in an urban area at 15-minute intervals requires 384 ordinates just for the storm; more time intervals would be required to simulate the full runoff hydrograph and route it through the channel system. The large-array version also allows greater flexibility in checking for numerical stability of simulation processes (e.g., kinematic runoff and routing computations). The large-array version uses an extended or virtual memory operating system available on 386 and 486 machines.

Urban Hydrology Features

Watershed Runoff

HEC-1 computes runoff using one of several loss methods (e.g. SCS Curve Number or Green and Ampt) together with a either a unit hydrograph or kinematic wave. Kinematic wave was added specifically to address the issues of urban hydrology. It also provided a better physical basis for application in ungaged areas. Before adding the kinematic wave runoff capability to HEC-1, the overall structure of urban runoff computations was analyzed. Urbanization impacts on both infiltration and runoff characteristics were considered. Also, the procedure for applying the model to large urban areas was reviewed. It was noted that runoff is usually computed from two types of surfaces: pervious and impervious. Within a subbasin (the smallest land surface area for which precipitation-runoff calculations will be made), urban drainage systems were found to have a regular structure of overland flow leading to collector channels of increasing size. For example, runoff from a property goes into a gutter, then into various sizes of storm drains until it reaches the main channel (storm sewer).

These characteristics of urban runoff were taken into account in designing the kinematic wave urban capability in HEC-1[4]. The result is a series of runoff elements, Fig. 1, which are linked together. These elements are linked together into a "typical" collector subsystem within the subbasin, Fig. 2. The rationale is that urban developments often have fairly regular storm drain systems which are tributary to a main channel. As shown in Fig. 2, there are two overland flow elements (pervious, the longer and impervious, shorter) which flow into the first collector channel. These overland flow elements allow specification of different infiltration and runoff characteristics, usually one representing pervious surfaces and the other representing impervious surfaces. If the "typical collector system" capability is not appropriate, the simulation may be accomplished on a detailed lot-by-lot and block-by-block basis. That detailed simulation is performed by specifying each runoff plane and channel element as a separate subbasin and routing reach.

The capability to represent impervious and pervious areas could be accomplished by artificially separating the subbasin into two more subbasins. The artificial subbasins would be characteristic of pervious and impervious runoff. This would have the same effect for the overland flow segments, but not for the channels. Both types of overland runoff typically flow into the same runoff collector channel (maybe a gutter). Thus, when they are treated together, there is a larger volume of flow in the channel than when each is done separately, and the nonlinear flow characteristics could not be reproduced by doing two separate smaller flow components.

This collector system capability allows for use of either kinematic wave or Muskingum-Cunge channel routing. Two main deficiencies in kinematic wave routing (both channel and overland) have been noted in our experience: lack of attenuation and numerical instability. The Muskingum-Cunge method has been found to apply to a much wider range of flow conditions, and be as good as the full unsteady flow solution much of the time[5]. Muskingum-Cunge routing still has numerical stability problems but they are not as limiting as for kinematic wave.

Flow Diversions

Urban storm runoff often encounters blocked or insufficiently sized channels and/or inlets to channels. Typically runoff from the land surface flows down gutters and ponds at storm drain

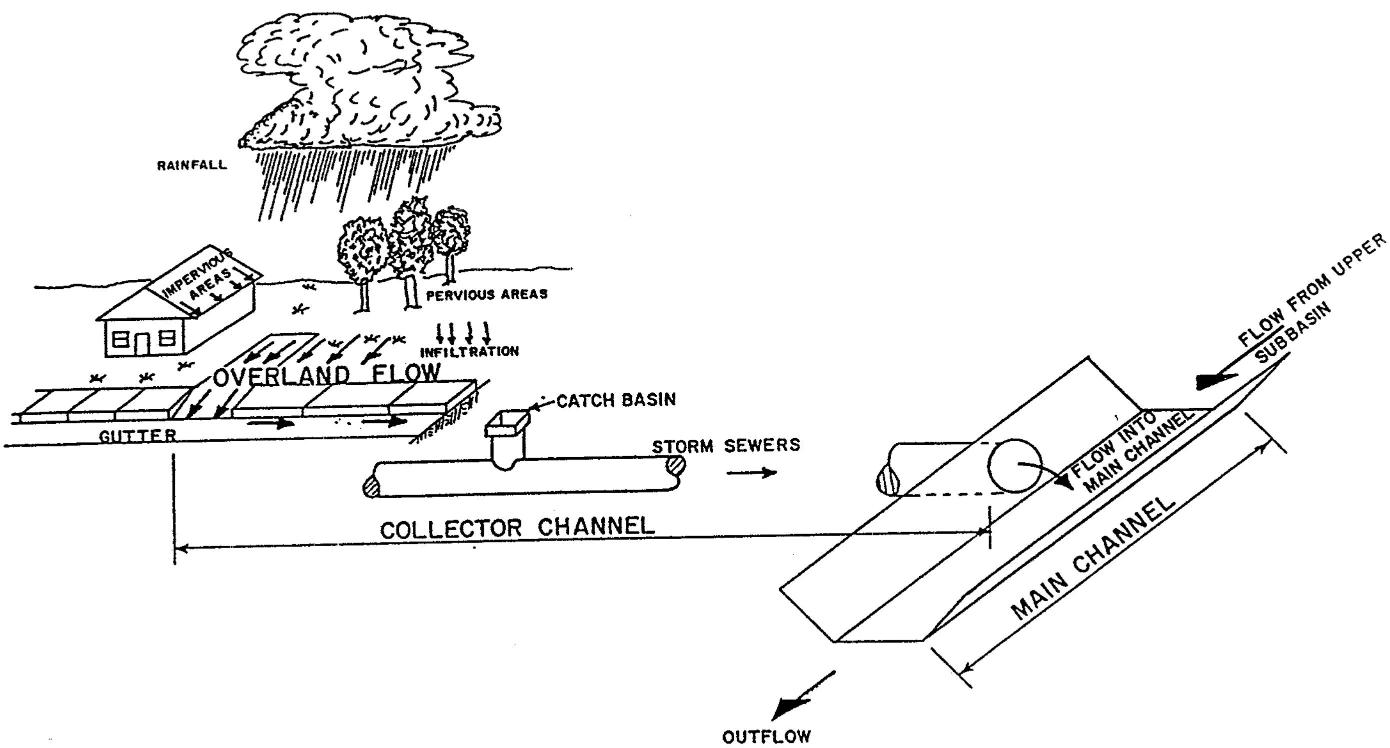

Figure 1. Elements Used in Urban-Runoff Calculations

inlets because the inlet is undersized or the storm drain is already full. Where does the water go? Depending on the terrain, the water may pond at the inlet or flow along the surface streets (channels). To accommodate this situation, HEC-1 has a diversion capability which allows the user to specify the inlet capacity and divert the remaining flow. The amount of the diversion is a function of the incoming flow. The diverted flow is saved and can be retrieved in any subsequent computation; it can be routed through a reservoir to simulate ponding or routed through a separate channel system (e.g., down the streets or in an adjacent channel). Because this flow separation mechanism is so unique to the particular terrain and storm drain characteristics, the diversion and disposition of the diverted flow was not made an automated process in HEC-1.

Flow Constrictions

Another problem in urban areas is insufficiently sized culverts at road crossings. These constrictions turn the open flow channel into a reservoir. The level-pool reservoir routing feature of HEC-1 can handle these conditions very effectively. If the culvert is submerged most of the time, then the orifice outlet capability of the dam routing may be used. If the flow exceeds the roadway, then the road may be represented as a nonlevel top-of-dam. For cases where open channel flow conditions are prevalent in the culvert, then a separately-determined rating curve for the culvert and top of the roadway must be computed and input to the dam routing.

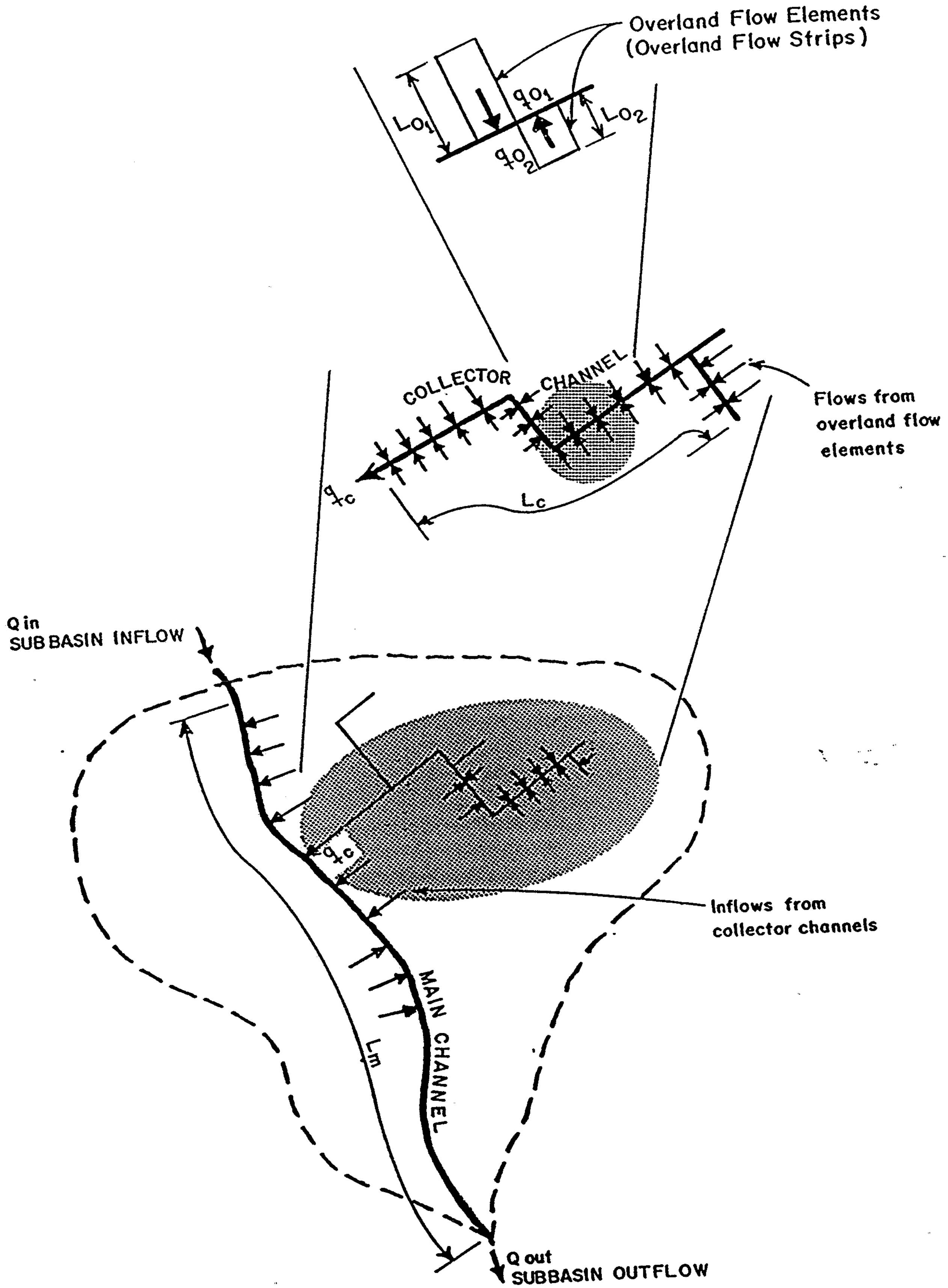

Figure 2. Subbasin Collector System

Pumping

In relatively flat urban areas, pumps are often installed to lift the flow to reduce excessive depth construction costs and to dispose of the water into a river or flood control channel. A pumping capability was added to HEC-1 to lift flood water over levees in interior flooding situations. (A more thorough capability, HEC-IFH, for simulating interior flood situations is presented in a subsequent section of this paper.) Pumping is accomplished as part of a reservoir routing. As the reservoir water surface elevation increases, pumps are turned on at different elevations. There are separate on and off elevations for each of five pumps in HEC-1. The pumped water is handled in the same manner as the flow diversions, i.e. it can be retrieved in any subsequent computation. The water that is not pumped either stays in the reservoir (sump or bay) or flows out a gravity outlet.

Modifying Flow-Frequency Curves

One of the key questions in analyzing a flood damage reduction project is: what is the modified flow-frequency curve with the proposed project in place? The frequency curve together with a flow-damage relationship allows one to compute expected annual damage reduction to analyze the economic efficiency of the proposed project. The multi-ratio and multi-plan capability in HEC-1 performs this modified frequency curve determination. It requires that a flow frequency curve be provided for existing conditions. Then HEC-1 simulates a series of different-sized storms (ratios) for both the existing and alternative future basin conditions to compute the modified frequency curve. The typical application in an urban area is to compute the modified frequency curve for a change in land use (alternative 'plan'). Then, compute the modified frequency curve for a change in land use and the addition of a detention storage reservoir (another 'plan'). This is easily accomplished with the HEC-1 urban runoff and reservoir routing simulation in a multi-ratio, multi-plan format.

Optimizing the Size of Flood Damage Reduction Projects

The size of a flood damage reduction project can be optimized for economic efficiency[6]. This computation combines two major features of HEC-1: expected annual damage computation and the automated parameter estimation algorithm. With the addition of cost-versus-size relationships for the projects in question, HEC-1 searches different project sizes to find which one minimizes the sum of costs and damage. This is equivalent to maximum-net-benefit measure of economic efficiency. Thus, a detention reservoir may be automatically sized for maximum economic efficiency.

HEC-IFH, INTERIOR FLOOD HYDROLOGY

Some flood damage reduction projects, such as levees and flood walls, usually involve special problems associated with isolated interior urban areas. Storm runoff patterns are altered and remedial measures are often required to prevent increased or residual flooding in the interior area due to blockage of the natural flow paths. Hydrologic analyses are needed to characterize the interior area flood hazard and to evaluate the performance of the potential flood damage reduction measures and plans. The HEC-IFH program[7] was conceived to meet this need.

HEC-IFH is a comprehensive, interactive program that is operational on extended memory PC's. It is particularly powerful for performing long, historical-period simulations to derive annual- or partial-series interior elevation-frequency relationships for various configurations of interior features such as gravity outlets, pumps, and diversions. It makes extensive use of a menu-driven user interface, statistical and graphical data representations, and data summaries. An engineer may use either a continuous simulation or hypothetical event approach depending on the type of study.

Continuous simulation analysis (also called a period-of-record analysis) uses continuous historical precipitation to derive streamflow records, see Fig. 3. HEC-IFH is designed to accommodate complete continuous simulations for at least 50 years of hourly records. However, these are not the absolute limits of the program's capabilities. For example, total periods of up to 100 years and time increments as small as 5 minutes may be used, although significant increases in data storage requirements and computation time will result.

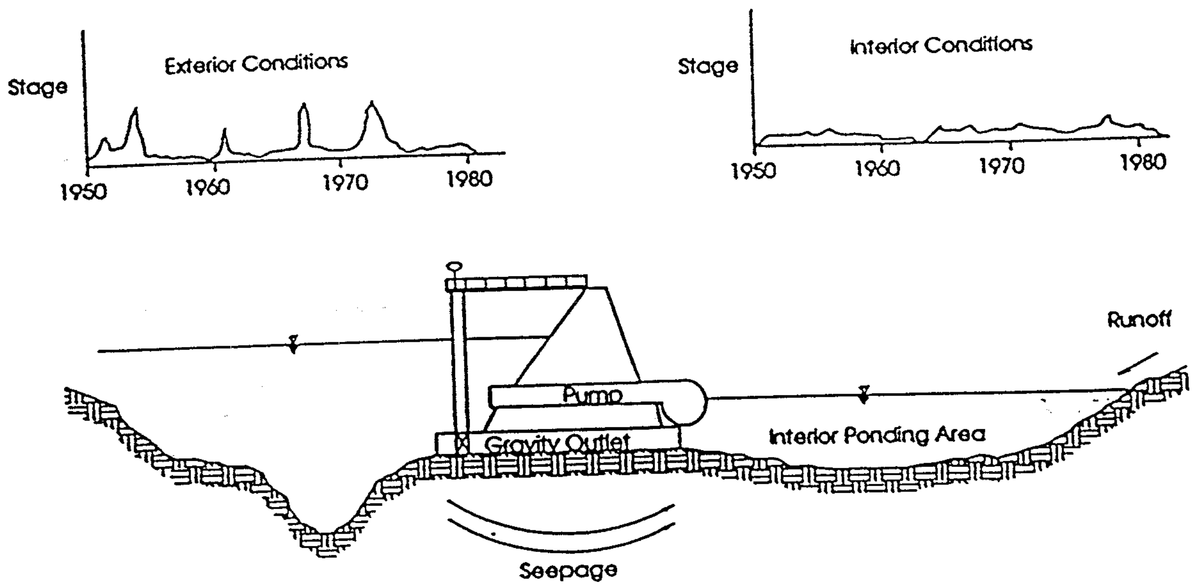

Figure 3. Interior-Flood-Hydrology Continuous Simulation

Hypothetical-event analysis is generally applicable when interior and exterior flood events are dependent. The analysis can be conducted so that the same series of synthetic storm events occur over both the interior and exterior areas. This analysis method can also be applied using a constant exterior stage, or for any "blocked" or "unblocked" gravity outlet condition.

STORM

The original version of the STORM program[8] was completed in January 1973 by Water Resources Engineers, Inc., WRE, of Walnut Creek, California, while under contract with HEC. STORM analyzes the quantity and quality of runoff from urban or nonurban watersheds. The purpose of the analysis is to aid in the sizing of storage and treatment facilities to control the quality and quality of storm water runoff and land surface erosion. The model considers the interaction of seven storm water elements: rainfall/snowmelt; runoff; dry weather flow; pollutant accumulation and washoff; land surface erosion; treatment rates; and detention reservoir storage. The program is designed for period-of-record analysis using continuous hourly precipitation data.

The quantity of storm water runoff has traditionally been estimated by using a design storm approach. The design storm was often developed from frequency-duration-intensity curves based on rainfall records. This approach neglects the time interval between storms and the capacity of the system to control some types of storms better than others. Infrequent, high intensity storms may be completely contained within treatment plant storage so that no untreated storm water overflows to receiving waters. Alternately, a series of closely spaced storms of moderate intensity may tax the system to the point that excess water must be released untreated. It seems reasonable, therefore, to assume that precipitation cannot be considered without the system, and design storm cannot be defined by itself, but must be defined in the light of the characteristics of storm water facilities. The approach used in this program recognizes not only the properties of storm duration and intensity, but also storm spacing and the storage capacity of the runoff system.

Runoff quantity is computed from hourly precipitation (and air temperature for snow) by one of three methods: the coefficient method; the U.S Soil Conservation Service Curve Number technique; or a combination of the two. Any sized basin may be used; for small basins, the calculation is simply an hourly volume accounting. Runoff quality is computed from the washoff of pollutants that accumulate on the land surface and from dry weather sanitary flow. The amount of pollutants washed into the storm drains and eventually to the treatment facilities for receiving waters is related to several factors including the intensity of rainfall, rate of runoff, the accumulation of pollutants on the watershed and the frequency and efficiency of street sweeping operations.

The resulting runoff is routed to the treatment-storage facilities where runoff less than or equal to the treatment rate is treated and released. Runoff exceeding the capacity of the treatment plant is stored for treatment at a later time. If storage is exceeded, the untreated excess is wasted through overflow directly into the receiving waters The magnitude and frequency of these overflows are often important in a storm water study. STORM provides statistical information on washoff, as well as overflows. The quantity, quality, and number of overflows are functions of hydrologic characteristics, land use, treatment rate, and storage capacity.

When the Corps urban studies program ended in the late 1970's, HEC discontinued development of STORM. It has remained in its original form since then, but some private engineering organizations have converted it to the PC environment. STORM was used extensively by consultants doing waste water management studies for EPA. Currently, there is much renewed interest in STORM for use in EPA's National Pollutant Discharge Elimination System effort.

HEC-DSS, DATA STORAGE SYSTEM

Background and Purpose

HEC-DSS[9] was the outgrowth of a need that emerged in the mid 1970's. During that time most studies were performed in a step-wise fashion, passing data from one analysis program to another in a manual mode. While this was functional, it was not very productive. Programs that used the same type of data, or were sequentially related, did not use a common data format. Also this required that each program has its own set of graphics routines, or other such functions, to aid in the program's use.

HEC-DSS was developed to manage data storage and retrieval needs for water resource studies. The system enables efficient storage and retrieval of hydrologic and meteorologic time-series data. The HEC-DSS consists of a library of subroutines that can be readily used with virtually any applications programs to enable retrieval and storage of information. At present approximately 20 applications programs have been adapted to interface with DSS.

Approximately 17 DSS utility programs have been developed. A number of these programs are for data entry from such files as the U.S. Geological Survey's WATSTORE data base or from the National Weather Services's precipitation data files. Other utility programs include a powerful graphics program, a general editor, and a program for performing mathematical transformations. Macros, selection screens, and other user interface features combine with DSS products to provide a set of tools whose application is limited only by the ingenuity of the user. HEC-DSS is depicted in Fig. 4.

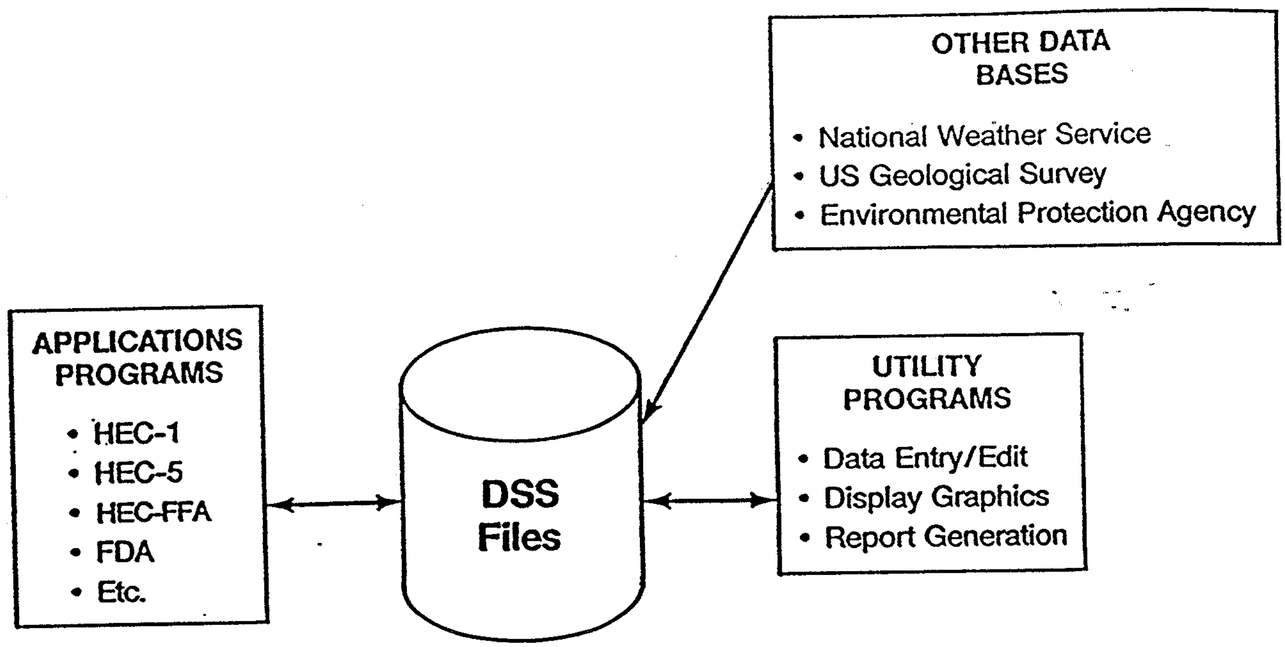

Figure 4. HEC-DSS Data Storage System

Using DSS to Link Several Models

HEC-DSS has played an integral link in several urban modeling studies where more than one model was necessary to solve the problem. In West Columbus, Ohio, the Corps Huntington District used three models to analyze urban flooding: HEC-1, SWMM[10], and HEC-IFH. The land surface runoff was computed with HEC-1 and routed to storm drain inlets. Flow into the inlet was computed and flow in excess of the inlet capacity was stored or routed through another part of the surface system as warranted by the terrain. The extended transport, EXTRAN, module of SWMM was used to collect the surface inlet flows and route them through the storm sewer system. The surface and subsurface systems drain naturally to several low areas which are now blocked by a levee protecting West Columbus from the main Scioto River. Thus, an interior flooding problem is created at those areas and the runoff must be pumped over the levee. The HEC-IFH model was used to solve the interior flooding problem. HEC-DSS was used to connect the output of one model to the input of the next model. The result was that a complex urban flood problem was disaggregated by detailed simulations of three specialized models whose results were managed by HEC-DSS.

NEXT GENERATION HYDROLOGIC MODELING SYSTEM

In 1990, HEC embarked on a project to develop the next generation, termed NexGen, of its simulation models. The objectives of the new modeling capabilities were to provide the user with better means to visualize and understand the process being simulated, and to build more engineering expertise into the models themselves. The capabilities of modern workstations and PC's using the Windows-NT and UNIX operating systems offer a new level of processing power that could meet these next-generation software needs. Four technical areas are being addressed in the current NexGen effort: river hydraulics, watershed runoff, reservoir system, and flood damage analysis. The new models will have most of the capabilities of the existing HEC models in those areas plus new algorithms where appropriate. The watershed runoff project is called the Hydrologic Modeling System, HEC-HMS.

The intent of this next generation of models is to put the users inside the model and give them the tools to easily work with the data, simulation processes, and results. The user will enter data into a data base that is constructed in a logical engineering-analysis format, not a format for some computer input device. Output will also be stored in the data base for analysis. A graphical user interface will let the user view the data, computations, and results for maximum understanding and analysis of the data and the physical processes.

The ultimate goal is to have smarter models that automatically evaluate numerical stability (time and distance steps) and physical constraints of the process being simulated. The user will be advised of process-simulation problems, and alternative methods and analyses will be recommended where possible. Thus, more engineering expertise will be built into the models to enhance their application and interaction with the user.

New Software Structure

The HEC-HMS model is being developed using "object oriented" technology. Object oriented technology provides a natural way to express a problem, decompose its complexity into understandable entities, and implement program code to solve the problem. Previously most models have been developed by looking at problems from a procedural viewpoint. The procedures that operate on the data were identified, defined, and executed using data supplied when the procedure was invoked. Either the object or the procedural approach can be used to solve a problem. The object perspective coupled with an object oriented computer language can offer some interesting advantages over the procedural approach.

In the HEC-HMS model the hydrologic element objects are the main building blocks. A watershed may be comprised of any number of Subbasins, Reaches, Junctions, or other components. Each hydrologic component object is linked to its associated neighbors to form a dendritic network. Fig. 5 shows an actual network for an area above the Allegheny Reservoir in

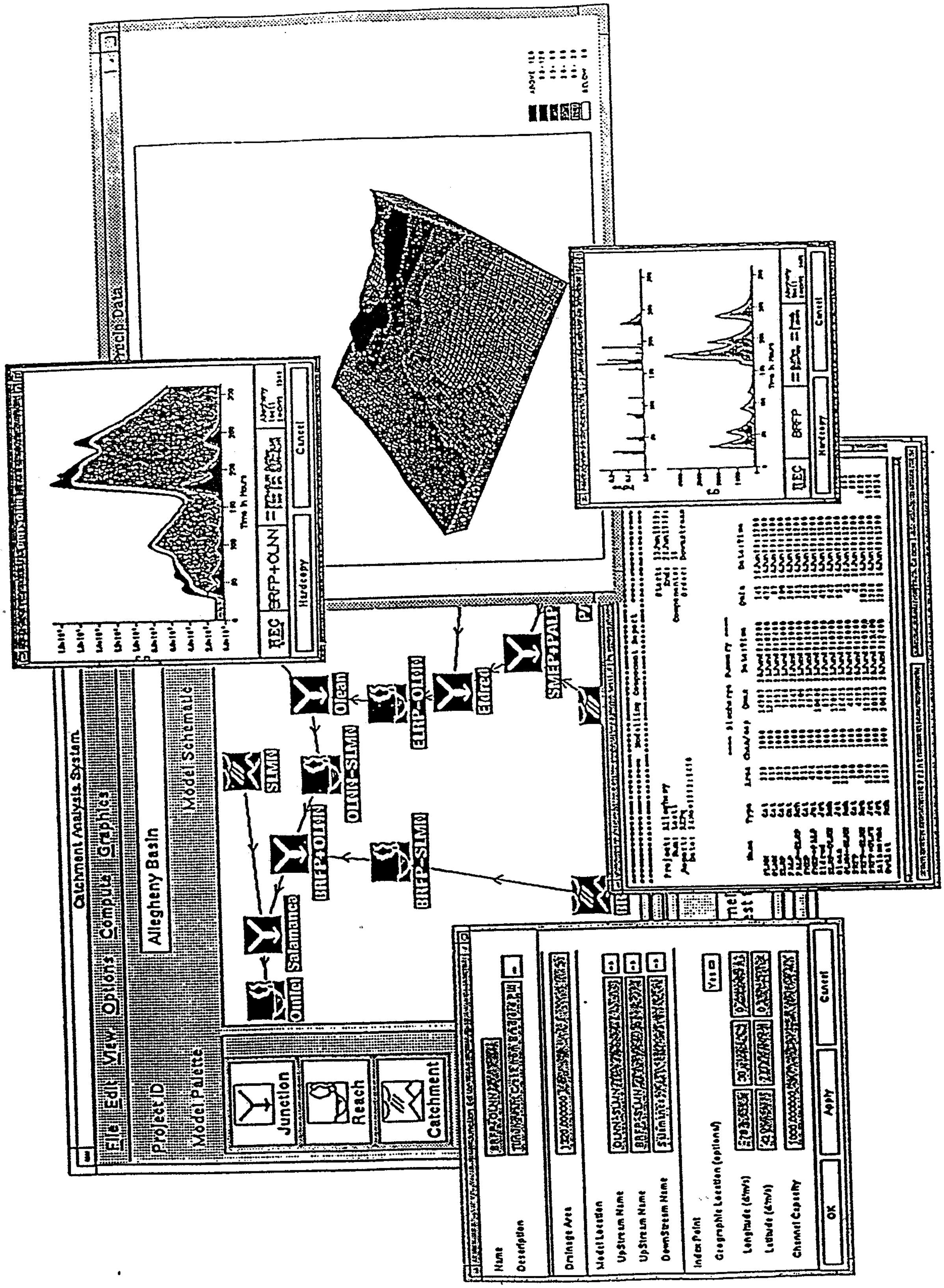

Figure 5. NexGen Hydrologic Modeling System

Pennsylvania. Once the model is configured, some interesting capabilities are possible by using the behavior defined in the component objects.

To compute flows in the system, the outlet is found, and it is requested to compute its flow. The outlet component requests the upstream neighbor for its flow. The compute request works itself up through the object network until components such as Subbasins compute flows from precipitation. As the component flows work their way back downstream, each Reach encountered performs a routing operation, and each Junction combines its inflows. The final result of outflow is then available at the outlet. This illustrates the concept of an object oriented model and the interrelationships that can be defined between objects.

New Algorithms

The initial goal of the project is to field a working model that is useful for accomplishing work similar to that currently done with HEC-1. As such, the model will initially contain the most frequently used algorithms in HEC-1. Later releases will incorporate newer engineering algorithms. The new model framework discussed in the previous section greatly facilitates the expansion of the model to include new technologies. Such new algorithms under consideration include the following.

- Both gaged and spatially distributed precipitation.
- A continuous soil moisture accounting procedure to permit long-period analysis, and improved low-flow simulation and flow forecasting.
- Improved direct runoff response may be possible by use of a transform that accepts a non-uniform excess distribution, where the spatial distribution of precipitation and excess is available.
- Improved baseflow and total runoff simulation.
- Automatic calibration of loss rates to reproduce given flow-frequency curves and given volume-duration frequency curves.

New User Interface

The user interface is the portion of the model that the user "sees" and "touches". In existing models the interface is a text input file, an execution command line, and a text output file. The HEC-HMS makes dramatic changes to the user interface by operating in a window environment with a graphical user interface, GUI. With the GUI the user has the ability to edit, execute, and view model data and results. The watershed configuration is depicted in the main window in schematic form. The schematic in Fig. 5 shows the name and graphic icon for each runoff subbasin, routing reach, and combining junction, along with the linkages that make up the model. The schematic itself may be altered on the screen to add, delete, or change subbasins, reaches, or junctions. Because of the object oriented framework, a newly reconfigured model is able to continue to perform all of its runoff functions without other user actions. The internal object representation used to perform model functions is always consistent with the visual presentation in the GUI schematic.

The GUI is currently the only access to model functionality. While not yet designed and implemented, it has been recognized from the outset that for large project requirements the HEC-HMS model must be able to be driven from other processes, as well as by the interactive GUI user. With the ability to accept commands from other processes, it will be possible to use the HEC-HMS as one component of a larger model encompassing reservoir, river hydraulic, water quality, and flood damage evaluation models. This will eventually make it possible to investigate a broader range of water resource problems producing an integrated solution across multiple modeling tools.

GIS HYDROLOGY

Some of HEC's earliest work in GIS hydrology involved development of a systematic methodology for automating the data preparation process. The raster-based organization chosen by HEC was called a grid cell data bank. Techniques for use of satellite data, for conversion of polygon data to grid format, and for use of commercially available software to manipulate and convert the data were developed. Parameters for HEC-1 and other hydrologic models were computed by a program called HYDPAR[11] which accessed the grid cell data. In 1975, the grid cell data bank approach was formalized in the HEC Spatial Analysis Methodology, HEC-SAM. Remotely sensed land use and other hydrologic characteristics were also incorporated in the SAM methodology. Later, HEC explored the use of triangular irregular network elements, TINs, for representation of watershed characteristics. A program linking HEC-1 with the TIN was developed in the late 1970's. Because of various hardware, software, and study-management problems associated with the GIS approach, HEC has been less active in the evolution of GIS technology for the past decade.

Recent HEC efforts have included a review of GIS applications in hydrologic modeling[12], and research into a method for combining the spatial GIS data with lineal hydrologic networks. A hybrid grid-network procedure for adapting these existing GIS capabilities for hydrologic modeling is being investigated. Spatially distributed processes are represented on a grid and one-dimensional flow and transport occurs through an associated network. There is a duality between a grid and a network in that once the direction of flow on each grid cell is defined to a single neighboring grid cell, an implied flow network is created. These ideas are being further investigated in HEC's NexGen and remote sensing/GIS projects.

CURRENT R & D ACTIVITIES

In FY 1994, HEC will begin a new urban hydrology and hydraulics R & D work unit. Many more of the Corps flood control investigations are now being conducted in urban areas. Several simulation models exist in the profession to perform these analyses, but each has its particular purpose. Each of the models has limitations too. The intent of this research is to: review the needs of Corps field offices; understand the limitations of existing models; develop guidance for the use of existing models for different types of investigations; and develop new or modified models as necessary to meet the needs. Table 1 summarizes the present status and perceived needs for urban hydrologic modeling capabilities at HEC.

One of the areas of concern is the transition from surface runoff to storm sewer flow. Inadequate inlet capacity and surcharging storm sewers make the simulation difficult. The SWMM EXTRAN module addresses this problem, but has limitations. One consideration will be to use HEC's new unsteady flow routing program, UNET[13], to perform the storm sewer routing. UNET has the capability to simulate flow in looped networks under free-surface and pressure-flow conditions. It presently interacts with HEC-DSS for input and output. UNET will have to be tested in such urban applications in conjunction with HEC-1. A dynamic link between the two models may be desirable.

TABLE 1 HEC Urban Hydrology Capabilities and Needs

Existing	Needed
Hypothetical frequency-based storms:	
Nested intensities balanced in time	Other hypothetical storm distributions
Infiltration:	
Land use-based	Replacement for SCS Curve Number
Lump sum computation	Distribute in time while water is on plane
No recovery during dry periods	Soil moisture accounting
Impervious area	OK
Drainage system:	
Typical collector system or detailed simulation	OK
Two land-surface-runoff planes	OK
No inlet control or check on storm drain capacity	Check for inlet control and storm drain capacity
Collector channel routing	Test diffusion routing in collectors
Channel routing:	
Several hydrologic methods	Additional checks for stability of computation
Kinematic wave and Muskingum-Cunge for typical urban channels	Check for channel capacity and limit flow.
Muskingum-Cunge widely applicable	Test diffusion routing for improvements
No storm sewer hydraulics except via data storage system	More direct connection of HEC-1 to unsteady flow model UNET or SWMM EXTRAN
Diversions:	
Single monotonic function	More flexible function with changes based on time and stage
Pumping stations:	
Fixed computation time interval causes oscillations between "on" and "off" times	Dynamic time interval
Detention storage:	
Level pool reservoir routing	OK
Culvert outflow submerged	Include free surface and pressure flow
Water quality:	
Only STORM computes land surface runoff quality	Add water quality routing or connect through data storage system
Project analysis:	
Fixed channel sizes	Automatic storm drain sizing
Modified flow-frequency curves	Automatic calibration to frequency curve
Flood damage calculation	OK
Multi-flood and multi-project simulation	OK

CONCLUSIONS

Several existing and emerging software packages for urban hydrologic modeling and analysis have been presented. More detailed information on any of the capabilities can be obtained from HEC. The purpose of HEC software is to help solve hydrologic analysis problems faced by Corps field offices. HEC follows a very applications-oriented approach to software development and problem solving. The development of new urban hydrology software will follow this same approach. One of the first tasks in the new R & D work unit will be to host a seminar to bring Corps users in contact with leaders in the urban hydrology profession. It will serve to both apprise the profession of the Corps needs and review the latest capabilities of the profession. With that information in hand, new and modified methods, models, and guidance will be developed. The result will be a set of physically based models with applications guidance for solution of urban hydrology problems in gaged and ungaged areas.

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Jacksonville District Experience with Urban Hydrology

Mike Choate[1]

1. The Jacksonville District's experience with urban hydrology has been predominately in Puerto Rico and the U.S. Virgin Islands. Urban flood damage is the primary justification for flood control projects in the islands. Agriculture, at one time, was a major economic influence; however, it now is secondary to urban losses. Flood damages are caused by a major river, originating in the high central mountains, passing quickly through the foothills and overflowing its banks in the alluvial floodplain inundating the urban area. Hydrographs are very steep and typically have low volumes.

2. The urban area is modeled under two different scenarios. First as an existing condition where the urban area is a contributing watershed to the main stream. The second is after the flood protection project, which is usually a levee, turns the urban area into an interior drainage study. During the interior drainage study, the storm drainage system and the routing of the overland flow become very important. Comprehensive drainage plans of the urban area are not usually available and drawings of existing facilities are sometimes difficult to obtain.

2. Urban drainage systems are first encountered with our request for surveys. Locations and diameters of channel inlets are requested along with channel surveys. Our experience has been that depressions and isolated areas that lack outfalls most often have storm drainage or pumping stations. Streets and gutters provide the conveyance for most of the storm runoff. Flow paths of both subsurface and surface flow are difficult to determine from maps. A site visit is almost always needed, but not always feasible.

3. SCS Urban Hydrology TR-55 is used for estimating curve numbers and lag times. Protected urban areas, since they are usually protected from the main stream, are treated as interior drainage problems and the Corps Interior Flood Hydrology Package (HEC-IFH) is used.

4. Recently completed Jacksonville District studies which included urban analysis:
   a) Rio Puerto Nuevo, San Juan, PR - 39 sq km, densely urban with five dendritic steams.
   b) Savan Gut, St. Thomas, VI - 2.3 sq km, steep watershed with high velocity outflow channel.
   c) Rio La Plata, Dorado, PR - 583 sq km, five urban areas protected.
   d) Rio Grande de Arecibo, Arecibo, PR - 487 sq km, town is subject to flooding from two sources.

Chief, Hydrologic Investigation Section, Jacksonville District, US Army Corps of Engineers

5. Areas of concern.

a) Level of detail at each study phase. We now do detailed H&H during preparation of the Detailed Design Memorandum. There is never time or money during Feasibility.
b) We always over estimate our design discharges. Our experience during calibration is that flows should be routed behind every culvert crossing and roadway crest.
c) Increasing environmental regulation is requiring that we develop methods to retain the first one-half inch of runoff and detain any increase in discharge.

6. Urban modeling is traditionally done by the municipalities for urban drainage planning. The Jacksonville District encounters these existing models and drainage plans during flood control studies and work-for-others projects. Our experience has been that the detail of these urban models (usually EXTRAN) is very good and calibration to local high waters marks is satisfactory. The survey information and system description contained in the model provides a very good data base for our hydrologic study.

7. The Jacksonville District has also utilized urban models (again EXTRAN) to model areas that no other existing model could handle. In ares of high intensity farming, with each farm leveed and diked and pumping into common canals, each farm is allowed to surcharge up to its existing levee height until overflow occurs. At this point, linking the nodes allows exchange of flow volume and equalization of storage. Routings using this method have produced realistic flood stages and good estimates of ponding durations which are needed for crop damages.

KAWAINUI MARSH FLOOD CONTROL PROJECT CASE STUDY IN URBAN HYDROLOGIC MODELING PACIFIC OCEAN DIVISION

by

Steven H. Yamamoto, P.E.[1]

Abstract

The combination of record rainfall intensity and volume during the New Year's Eve storm of December 31, 1987 - January 1, 1988 caused numerous damages to localized southeastern areas on the island of Oahu. Coconut Grove, located in the town of Kailua on the eastern windward side of Oahu, was flooded to depths exceeding 4-feet after waters from Kawainui Marsh overtopped its 9 1/2 -foot high earthen levee, which was constructed by the Corps of Engineers in 1966. Although the primary flood waters inundating Coconut Grove were from the Kawainui Marsh drainage basin, a simplified interior hydrologic analysis was conducted to resolve project formulation issues. This paper discusses the methodology and results of the interior hydrologic analysis.

Introduction

The New Year's Eve storm of December 31, 1987 - January 1, 1988 caused numerous damages to localized southeastern areas on the island of Oahu. Rainfall amounts exceeding 22 inches in 24 hours and exceeding 15 inches in 6 hours were recorded near the heaviest rainfall areas along the windward side and crest of the Koolau Mountains in East Oahu (Figure 1). These amounts were greater than the probable rainfall amounts for a return period of 100 years for several gaged locations. This storm caused estimated damages of $34.6 million to residences, businesses, agriculture and public property. Damages to residences and personal property within Coconut Grove were estimated at $10 million (DLNR, 1988).

A thorough hydrologic investigation of the 10.62 square mile Kawainui Marsh drainage basin (Figure 2) was conducted in support of flood control improvements to the existing levee. Rainfall-runoff hydrographs were developed from design storm rainfall hyetographs, and from Snyder's unit hydrograph and loss rate parameters, which were determined from the reconstitution of several observed runoff events within the basin (USACOE, 1992). Runoff hydrographs into the marsh were then routed using the RMA-2V finite element model to hydraulically simulate the marsh conditions, and to determine the levee-overtopping hydrographs. These overtopping hydrographs were then used to conduct an interior drainage analysis of Coconut Grove.

Study Area

The interior area, as defined for this study, is a predominately residential community located in Kailua, Oahu and has a total land surface area of approximately 820 acres. The area is bordered by Kailua Bay, Oneawa Channel, Kawainui Marsh levee and Kuulei Road (Figure 3). A natural ridge divides the study area into two natural drainage areas. Closest to the levee, the southwestern drainage area, consists of 550 acres (0.85 square mile) and drains runoff into Kaelepulu Stream. The northeastern area drains toward

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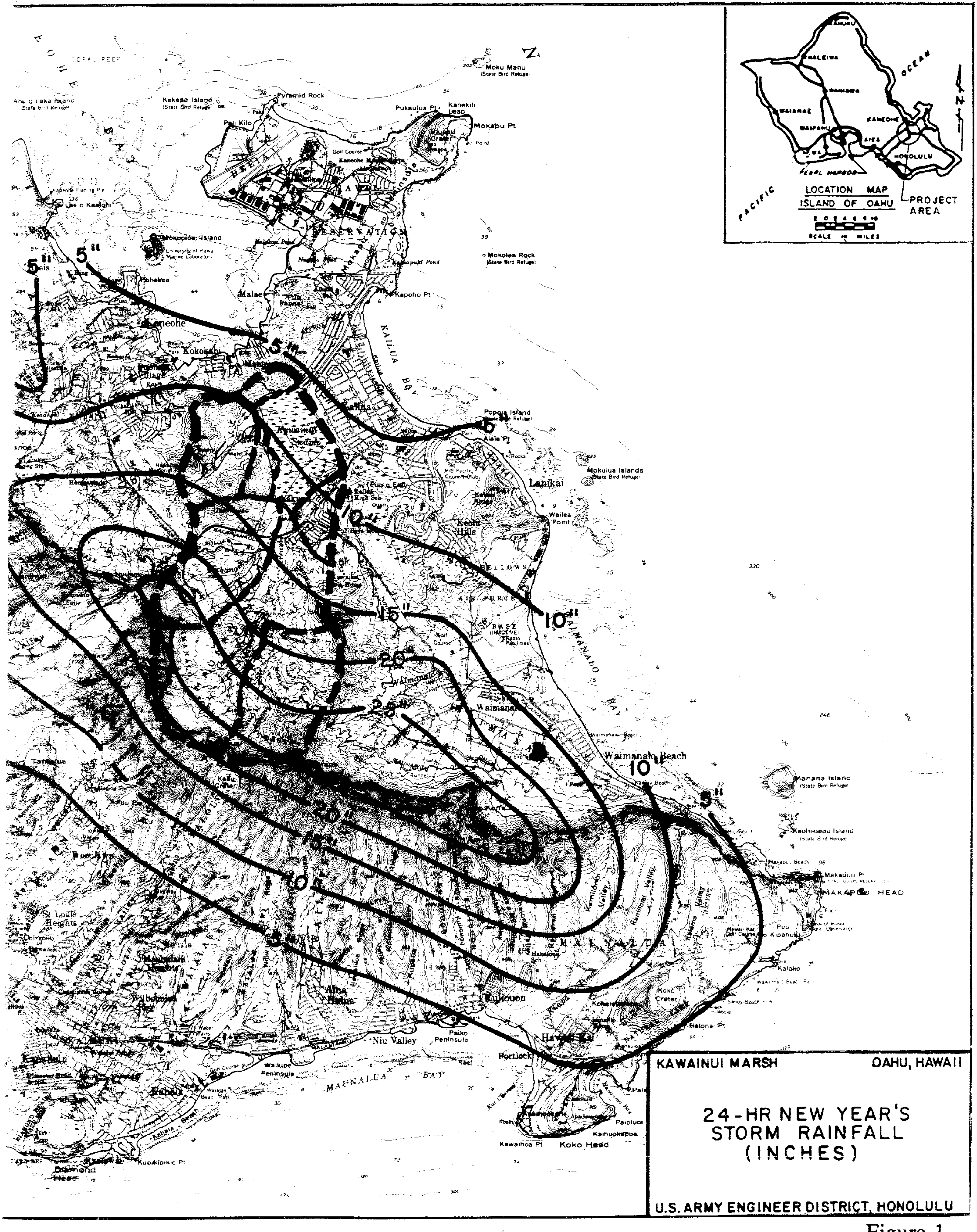

KAWAINUI MARSH
OAHU, HAWAII
24-HR NEW YEAR'S STORM RAINFALL (INCHES)
U.S. ARMY ENGINEER DISTRICT, HONOLULU
LOCATION MAP
ISLAND OF OAHU
PROJECT AREA
PACIFIC
OCEAN
HALEIWA
WAHIAWA
WAIANAE
WAIPAHU
KANEOHE
AIEA
HONOLULU
PEARL HARBOR
Moku Manu
Pyramid Rock
Kekepa Island
Ahu o Laka Island
Pukaulua Pt
Kahekili Leap
Mokapu Pt
Moku o Loe Island
Mokolea Rock
Kapoho Pt
Kaneohe
Kokokahi
KAILUA BAY
Kailua
Popoia Island
Alala Pt
Mokulua Islands
Lanikai
Wailea Point
Keolu Hills
BELLOWS
AIR FORCE
Waimanalo
Waimanalo Beach
Manana Island
Kaohikaipu Island
Makapuu Pt
MAKAPUU HEAD
Koko Crater
Halona Pt
Hawaii Kai
Kuliouou
Portlock
Niu Valley
Paiko Peninsula
Aina Haina
Wailupe Peninsula
MAUNALUA BAY
Kahala
St Louis Heights
Wilhelmina Rise
Kawaihoa Pt
Koko Head
Kaihuokapua
Paiolul
Diamond Head
Kupikipikio Pt
5"
10"
15"
20"
25"

Figure 1

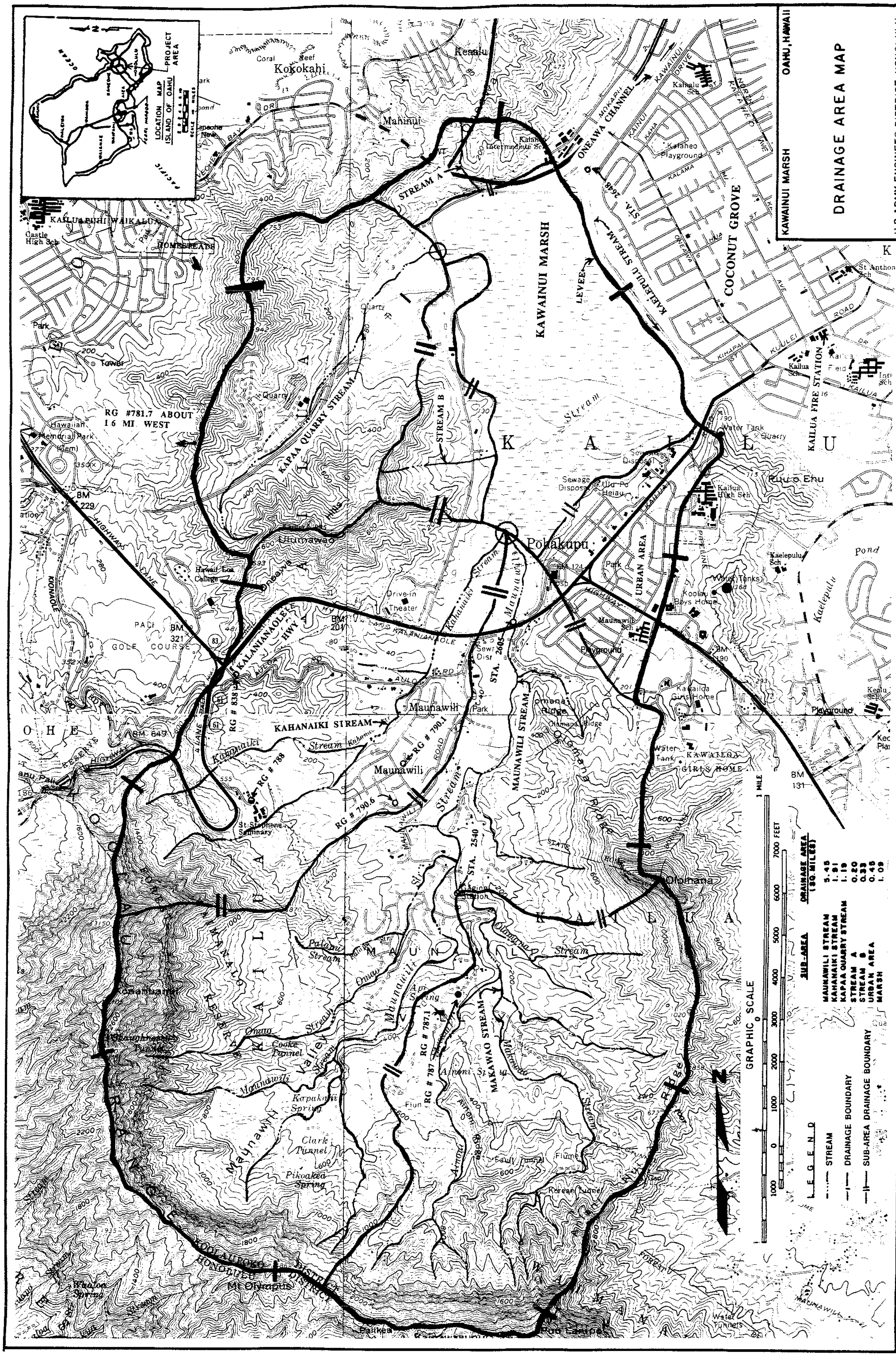

KAWAINUI MARSH
OAHU, HAWAII
DRAINAGE AREA MAP
U.S. ARMY ENGINEER DISTRICT, HONOLULU
LOCATION MAP
ISLAND OF OAHU
PROJECT AREA
KAWAINUI MARSH
COCONUT GROVE
STREAM A
STREAM B
KAPAA QUARRY STREAM
KAHANAIKI STREAM
MAUNAWILI STREAM
MAKAWAO STREAM
KAELEPULU STREAM
ONEAWA CHANNEL
LEVEE
URBAN AREA
KAILUA FIRE STATION
RG #781.7 ABOUT 1.6 MI. WEST
RG # 838
RG # 788
RG # 790.1
RG # 790.6
RG # 787.1
RG # 787
STA. 2005
STA. 2540
KALANIANAOLE HWY
Pohakupu
Maunawili
Olomana
Mt Olympus
Kokokahi
Mahinui
Kailua High Sch
Castle High Sch
St Stephens Seminary
Pali Golf Course
KAWAILOA GIRLS HOME
GRAPHIC SCALE
1 MILE
7000 FEET
LEGEND
STREAM
DRAINAGE BOUNDARY
SUB-AREA DRAINAGE BOUNDARY
SUB-AREA
DRAINAGE AREA (SQ. MILES)
MAUNAWILI STREAM 5.45
KAHANAIKI STREAM 1.91
KAPAA QUARRY STREAM 1.19
STREAM A 0.20
STREAM B 0.33
URBAN AREA 0.45
MARSH 1.09

Figure 2

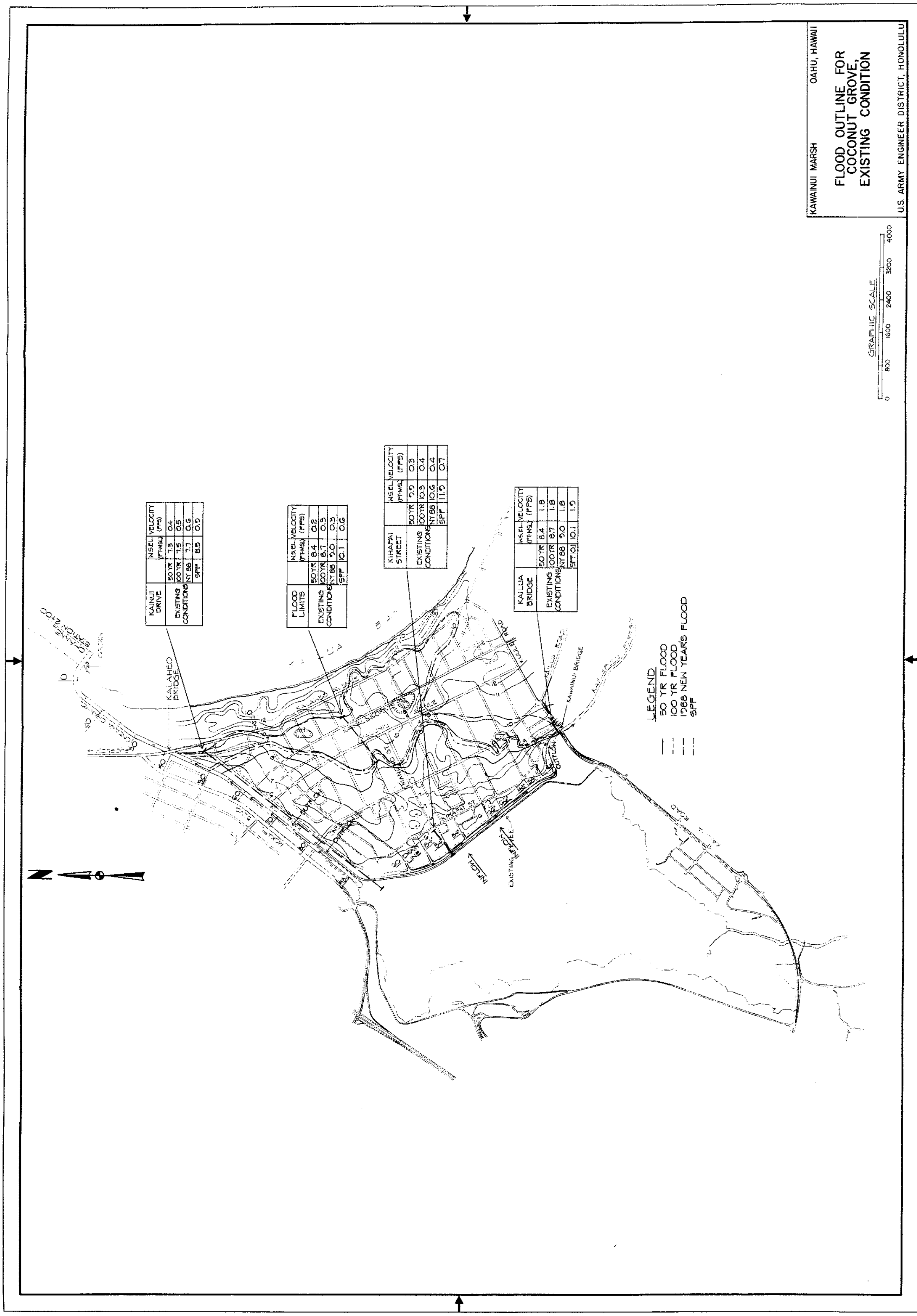

KAWAINUI MARSH
OAHU, HAWAII
FLOOD OUTLINE FOR COCONUT GROVE, EXISTING CONDITION
U.S. ARMY ENGINEER DISTRICT, HONOLULU
GRAPHIC SCALE
0 800 1600 2400 3200 4000
KAINUI DRIVE
EXISTING CONDITIONS
W.S.EL. (FT-MSL) VELOCITY (FPS)
50 YR 7.3 0.4
100 YR 7.5 0.5
NY 88 7.7 0.6
SPF 8.5 0.9
FLOOD LIMITS
EXISTING CONDITIONS
W.S.EL. (FT-MSL) VELOCITY (FPS)
50YR 8.4 0.2
100YR 8.7 0.3
NY 88 9.0 0.3
SPF 10.1 0.6
KIHAPAI STREET
EXISTING CONDITIONS
W.S.EL. (FT-MSL) VELOCITY (FPS)
50YR 9.9 0.3
100YR 10.3 0.4
NY 88 10.6 0.4
SPF 11.9 0.7
KAILUA BRIDGE
EXISTING CONDITIONS
W.S.EL. (FT-MSL) VELOCITY (FPS)
50YR 8.4 1.8
100YR 8.7 1.8
NY 88 9.0 1.8
SPF 10.1 1.9
KALAHEO BRIDGE
KAWAINUI BRIDGE
KUULEI ROAD
LEGEND
50 YR FLOOD
100 YR FLOOD
1988 NEW YEARS FLOOD
SPF
EXISTING LEVEE
INLET

Figure 3

Kailua Bay. Only the southwestern drainage area, which comprises the Coconut Grove tract, was considered for hydrologic analysis.

The existing City and County of Honolulu storm drain system, which was installed in 1973, is located at the southwestern end of Coconut Grove and discharges runoff into Kaelepulu Stream. The system, which consists of nine drain lines and servicing an area of 271 acres, was designed to discharge the 10-year storm with one-foot of freeboard. In low-lying areas where the elevation difference between the existing ground water are less than one foot, the pipes were sized to discharge the 50-year storm with no freeboard. The total design discharge for the storm drains, into Kaelepulu Stream, is 705 cfs (Towill, 1969).

Hydrologic Analysis of Interior Ponding

The hydrologic study procedure used in evaluating the interior drainage area followed the guidance and criteria of EM 1110-2-1413, "Engineering and Design, Hydrologic Analysis of Interior Areas," dated January 15, 1987. The four objectives of the analysis were to:

(1) define the existing interior area,
(2) develop interior inflow hydrographs for the existing system,
(3) perform storage routing using the Modified Puls method for various storm events, and
(4) determine the ponding limits and elevations for the with- and without-project conditions and verify the modeled results.

The hydrologic study of the interior drainage area was formulated on the assumption that:

(1) future land development of the interior area will be negligible and that the hydrologic characteristics of the study area will remain essentially unchanged, and

(2) complete coincidence occurs between the exterior event (high stage conditions which would produce levee overtopping) with the interior event (ponding due to interior runoff).

The first assumption was made after an evaluation of the City and County of Honolulu Kailua District Base Map which indicates that the study area has undergone maximum residential development and that improvements to existing drainage systems are not anticipated. The second assumption was concluded from analyzing the physical and meteorologic processes of both interior and exterior conditions, and deducing reasonable correlation for both interior and exterior events.

Interior Inflows

Rainfall estimates for the 10-, 50- and 100-year frequencies and Standard Project Storm were determined using design storm isohyetal maps. Rainfall depth-duration curves (Figure 4) were plotted and a unit time interval of 10 minutes was used to describe the rainfall inflow hydrographs into Coconut Grove. The time interval of 10 minutes was considered the most practical for storm computations to adequately define the computed hydrograph.

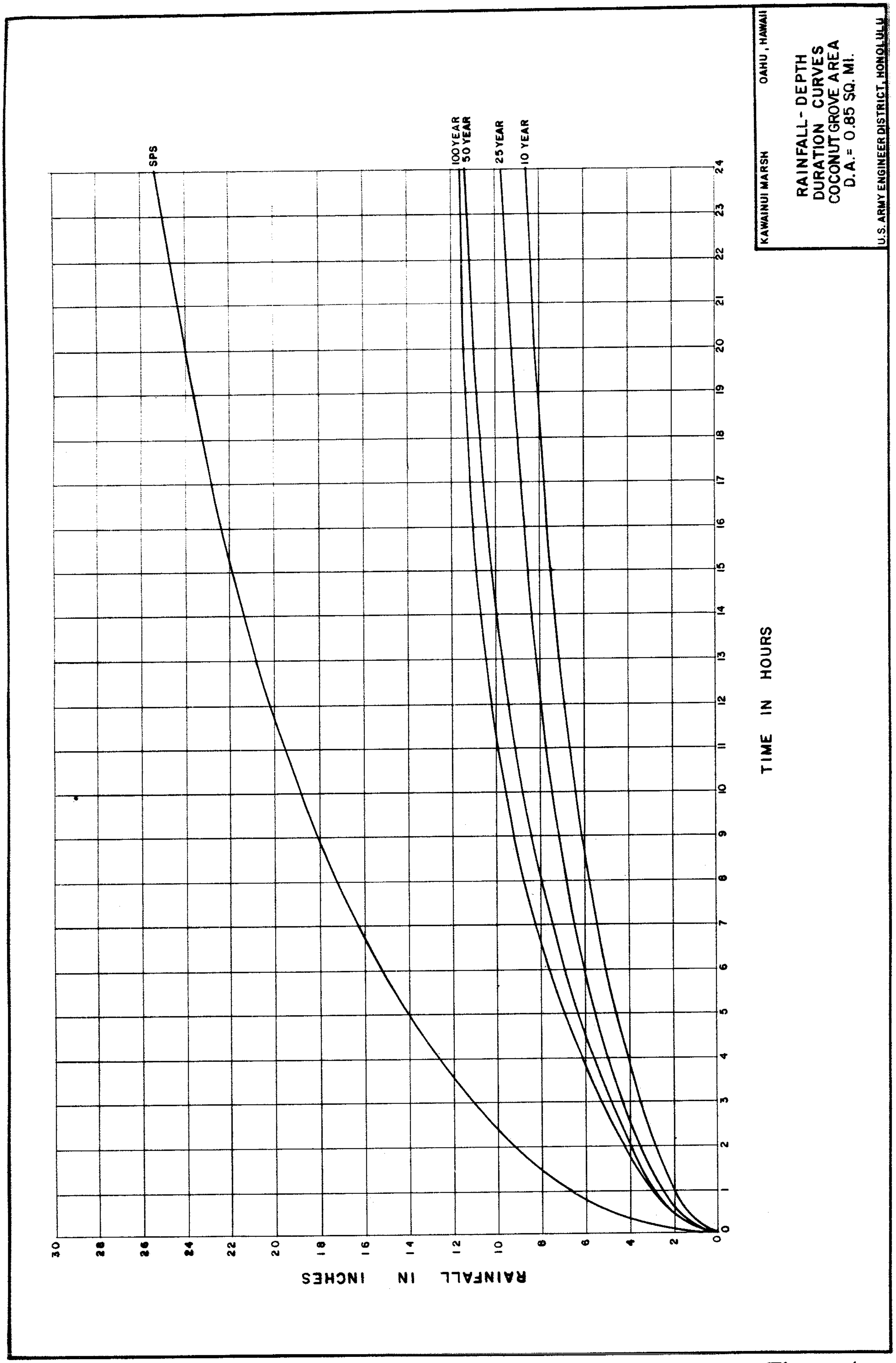

KAWAINUI MARSH
OAHU, HAWAII
RAINFALL - DEPTH
DURATION CURVES
COCONUT GROVE AREA
D.A.= 0.85 SQ. MI.
U.S. ARMY ENGINEER DISTRICT, HONOLULU
SPS
100 YEAR
50 YEAR
25 YEAR
10 YEAR
TIME IN HOURS
RAINFALL IN INCHES

Figure 4

Infiltration and depression losses were considered negligible in determining interior runoff because of the following reasons:

(1) Initial loss estimates for the 100-year frequency storm using the SCS loss method yielded an average loss of 0.11 inches per hour, which would have a minimal effect on the total rainfall volume; and

(2) During periods of heavy rainfall, ponding occurs in low-lying areas along Kihapai Street due to the rapid rise of the shallow water table (USGS, 1971). Infiltration in this area is retarded because of the high soil saturation and the capillary rise of soil moisture.

The volume of water overtopping the existing Kawainui Marsh levee was determined by the Corps of Engineers Waterways Experiment Station (WES) using the RMA-2V model. Discharges for the 25-, 50- and 100-year flood frequencies as well as the 1988 New Year's flood and the Standard Project Flood, were presented as levee-overtopping inflow hydrographs into Coconut Grove and used for storage routing.

The total inflow hydrograph into Coconut Grove, for the existing without-project improvements, consists of combining the interior drainage rainfall hydrograph with the levee-overtopping hydrograph. This combination simulates the worst case scenario by assuming 100 percent coincidence for both interior and exterior events. Only runoff from interior rainfall was analyzed in the storage routing program for the with-project conditions, which assumes no inflow over the levee.

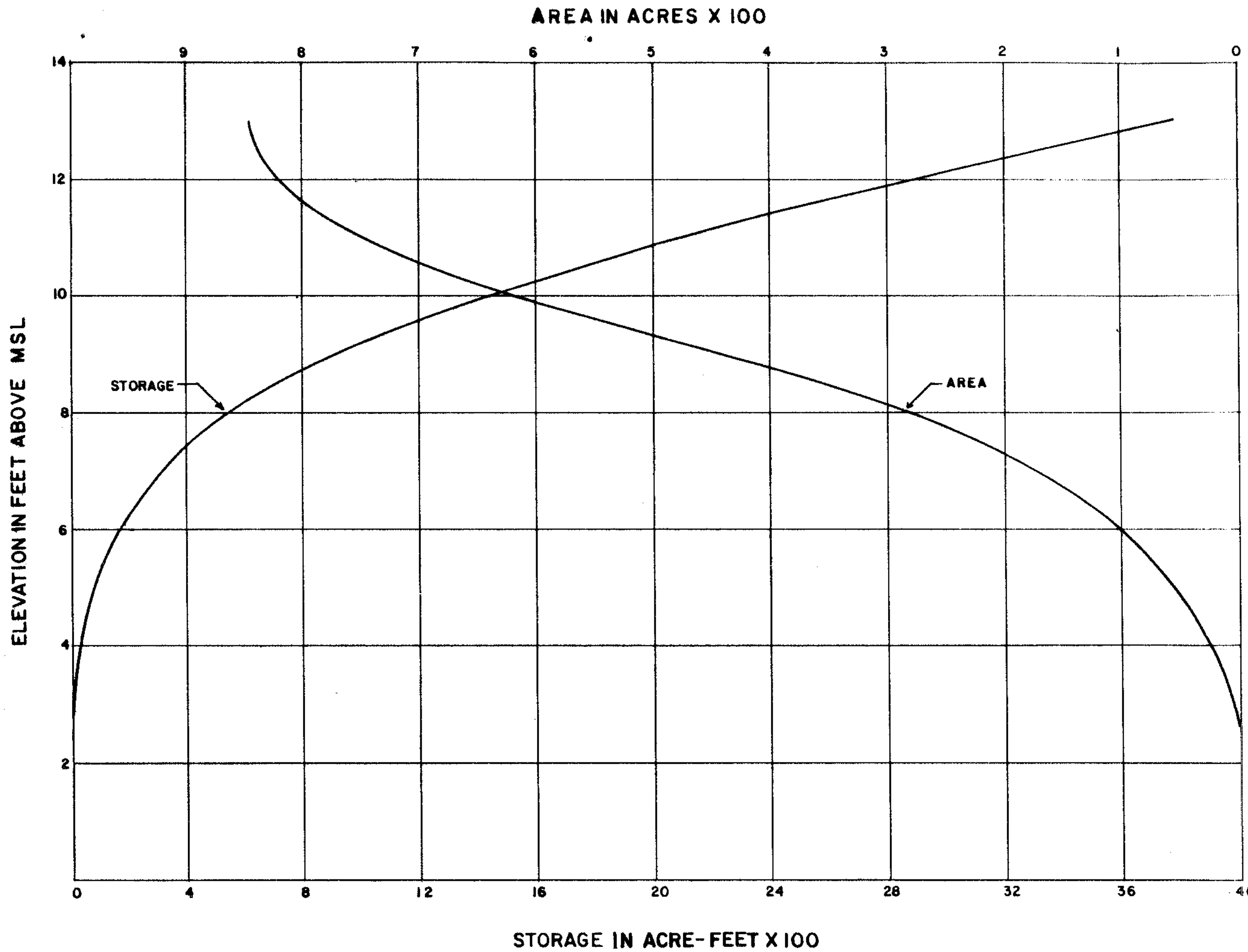
Interior Storage Routing

Storage routing was accomplished by the Modified Puls method, which calculates storage, elevation, and outflow. The information required to perform the storage routing are the inflow hydrographs, area-storage curve, and discharge rating curves.

The area-storage curve (Figure 5) was derived by computing increments of volume from intervals of 1-foot elevation using the average-end-area method.

Runoff from Coconut Grove is conveyed by two primary modes: via the existing storm drains which discharges into Kaelepulu Stream and, by overflowing the low-lying right bank into Oneawa Channel (between Oneawa Street and Kainalu Drive) during high flood conditions. Discharge rating curves for both were developed using the water surface profile program, HEC-2. The rating curve for Kaelepulu Stream was developed at a cross section fifty feet upstream of Kawainui bridge using channel and overbank roughness of 0.045, and contraction and expansion coefficients of 0.3 and 0.5, respectively. The starting water surface elevation of 2.2 feet mean sea level (msl) was used which assumed the highest tidal conditions (USCGS, 1956).

The rating curve for discharge into Oneawa Channel was developed at an interior section, 2,000 feet southeast from Oneawa Channel. Manning's roughness of 0.20 and contraction and expansion coefficients of 0.3 and 0.5, respectively, were used to model the residential conditions. Cross sections for the right bank of Oneawa Channel were obtained from as-built drawings of the Kawainui Flood Control Project dated October 1966. Water surface profiles were computed using critical depth as the starting elevation along the Oneawa Channel right bank. Critical depth was used because of the rapid change in grade along the right bank.



REFERENCE: CONTOUR MAP (SCALE 1"= 400') FROM R. M. TOWILL REPORT "HYDROLOGIC STUDY FOR COCONUT GROVE IMPROVEMENT DISTRICT, KAILUA, OAHU, HAWAII" MARCH 1969

KAWAINUI MARSH	OAHU, HAWAII
AREA-STORAGE CURVES FOR COCONUT GROVE	
U.S. ARMY ENGINEER DISTRICT, HONOLULU	

Figure 5

Verification of Flood Model

The New Year's flood of Coconut Grove was used to verify the accuracy of the computed results. High-water marks in Coconut Grove were obtained from the American Red Cross Damage Assessment Reports conducted on January 2, 1988. Together with randomly surveyed residential first floor elevations conducted by the Corps of Engineers, flood heights along Kihapai Street ranged in elevations from 10.0 to 11.0 feet above msl, compared to the storage routing model result of 10.6 feet.

During the New Year's flood, it was observed that:

(1) runoff from Coconut Grove flowed into Oneawa Channel over its right bank, and

(2) the USGS stage gage #2648 located at Oneawa Channel (Station 90+00) reported a maximum tailwater elevation of 2.5 feet msl from high water marks.

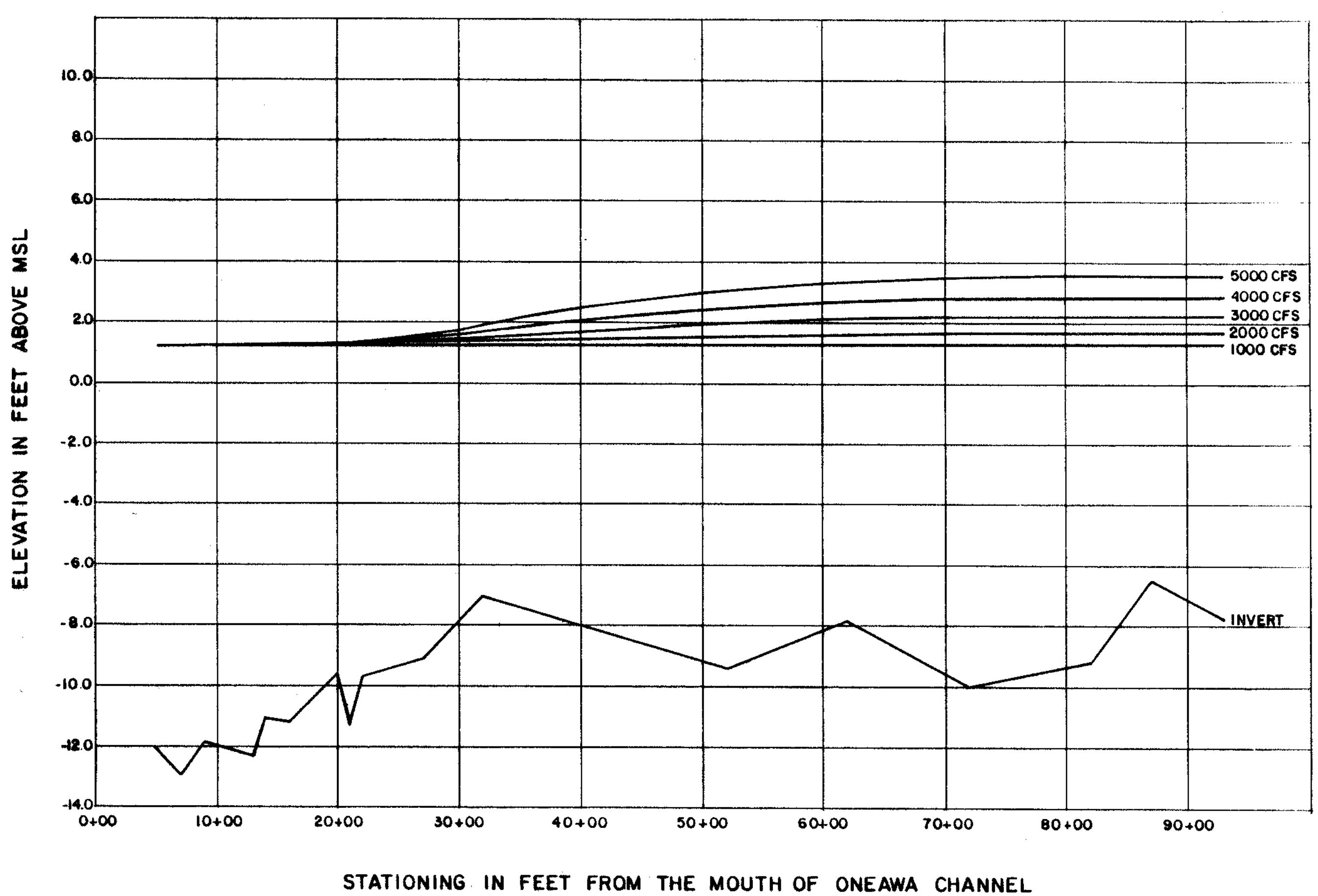
To verify the computed outflow of 3370 cfs from the storage routing model, the New Year's flood profiles of Oneawa Channel (Figure 6) were calculated using HEC-2. The highest predicted tide during the New Year's flood of 1.2 feet msl was used as the starting water elevation. Channel roughness of 0.025 and contraction and expansion coefficients of 0.1 and 0.3, respectively, were utilized throughout the Oneawa Channel reaches. From the water surface profiles, a maximum discharge of 3500 cfs was calculated which could still produce a tailwater elevation of 2.5 feet msl at the USGS gage.

The flood limits for the existing without project conditions of Coconut Grove are shown on Figure 3. During the New Year's flood, the maximum observed limit of inundation followed the 9-foot contour (DLNR, 1988). However, reported flood heights along Kihapai Street, near the vicinity of the levee, were predominately much higher and reached elevations as high as 11 feet msl. To account for this backwater effect, a reduction factor of 15 percent was applied to the modeled 50-, 100-, and SPF water surface elevations to determine the extent of flooding. This reduction factor was based on the modeled 1988 New Year's flood elevation of 10.6 feet and the 9-foot contour flood limit. The flood elevations and outflow discharges using the Modified Puls storage routing method for both existing and with project conditions are listed below:

COCONUT GROVE
FLOOD ELEVATIONS AND DISCHARGES

	Existing Without Project			Existing With Project	
Flood Event	Peak Elevations (ft msl)	Flood Limit Elevation (ft msl)	Total* Outflow (cfs)	Flood Limit Elevation (ft msl)	Total Outflow (cfs)
SPF	11.9	10.1	10,600	7.3	1,100
Jan 1, 1988	10.6	9.0	5,270	---	---
100-yr	10.3	8.7	4,370	5.8	710
50-yr	9.9	8.4	3,480	5.7	690

* Total outflow via Oneawa Channel and Kaelepulu Stream



KAWAINUI MARSH — OAHU, HAWAII

WATER SURFACE PROFILES
ONEAWA CHANNEL

U.S. ARMY ENGINEER DISTRICT, HONOLULU

Figure 6

Conclusion

The overall correlation between the results from the simplified hydrologic interior analysis model and the actual flood elevations in Coconut Grove during the New Year's flood are reasonably good. Storage routing using the Modified Puls method yielded acceptable flood elevations for areas adjacent to the levee, although actual flood elevations during the New Year's event were considerably less along the outer inundation limits. One explanation for this is believed to be the models inability to consider the back water effects caused by the presence of buildings and other man-made structures. Certainly, future studies may require the use of 2-dimensional models, such as RMA-2V, to better define flooding within highly urbanized interior areas.

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Urban Hydrology Problems of the Southwest

by Joseph B. Evelyn[1]

The rapid population growth in the Southwest during the post World War II era has resulted in large scale urbanization in southwest cities of southern coastal California (Los Angeles to San Diego); Phoenix and Tucson, Arizona; and Las Vegas, Nevada. Although a relatively dry region in terms of average annual precipitation, southwest water resource problems include flood control as well as water supply. Urban hydrology problems in the southwest encompass the conventional issues of quantifying runoff rates from hypothetical storm events and determination of discharge frequency relationships. Difficulties arise because urbanizing watersheds are in a state of physical change so that hydrologic data is non-homogeneous and therefore must be adjusted or analyzed in smaller quasi-homogeneous time periods. The objective of this paper is to identify urban hydrology research and development needs based on Los Angeles District Corps of Engineers experience in planning, designing, and operating flood control projects in the Southwest.

Urban hydrology products needed in the plan formulation and evaluation process, project design phase, and for project operation include: discharge frequency curves under present and future basin conditions, both with and without project (channelization and/or detention storage); specific return period flood hydrographs; design floods such as probable maximum flood; flood overflow areas, depths, and velocities; probability distributions for uncertainty of discharge frequency and stage-discharge curves; and real-time flood forecasting.

Urban hydrology studies in the Southwest would benefit from improved procedures or computational tools in the following areas.

A. Capability to optimize (automatically) rainfall-runoff parameters and channel routing characteristics in the same reconstitution run of a rainfall-runoff model such as HEC-1.

B. Rainfall-runoff models with the capability to use variable time distribution/intensity pattern over a watershed.

C. Guidance on the best ways to treat non-homogeneous streamgage records in discharge frequency analysis.

D. Improved (more accuracy with a reasonable increase in computational effort) methods for overflow area determinations in urban areas.

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E. Hydrology and hydraulic computational tools that can take advantage of the increasing availability of Geographic Information System (GIS) data.

F. Increase the amount of hydrologic data available for calibration/verification of hydrologic models used in urban areas.

G. Improved procedures for development of runoff volume frequency relationships in areas with little data.

H. Improved procedures for determination of discharge frequency relationships downstream of detention storage in regions where the dominate flood producing storms are local storms of limited areal coverage.

I. Methodology that enables use of Monte Carlo Analysis to address the local storm centering issue described in subparagraph H above.

J. Better flood forecasting tools to enable real-time analysis (rapid input/output of data, appropriate handling of missing input data, automated runoff forecast procedure).

K. Additional design guidance to enable more environmentally oriented approaches to flood control in urban areas, e.g., use of flexible porous channel revetment that is compatible with vegetation.

L. More esthetic yet functional approaches to flood control on alluvial fans.

Within the Corps the research and development needs with respect to urban hydrology have changed with time as the hydrologic output requirements have changed. In the pre-1970's the primary area of emphasis was development of a design flood (Standard Project Flood) because it was deemed the appropriate level of protection for urban areas. In the 1970's more rigorous discharge frequency analysis for economic analysis, and for FEMA studies was required. In the 1980's Corps guidance shifted to National Economic Development (NED) analysis, and away from a design flood basis for establishing level of protection in urban areas. In the 1990's the Corps is approaching flood problem definition, analysis of alternatives, and project design using risk-based analysis in which the uncertainty inherent in all hydrologic determinations is quantified explicitly. Development of procedures and computational tools to determine probability distributions for the error of hydrologic and hydraulic estimates is currently of most importance.

COMPARISONS OF RAINFALL/RUNOFF MODELS FOR TWO WATERSHEDS IN COLORADO

William P. Doan[1]

INTRODUCTION

The purpose of this paper is to summarize recent experiences of applying and comparing rainfall/runoff models for two actual watersheds in Colorado. These simulations allowed for an unique opportunity to evaluate the similarities and differences between several commonly used rainfall/runoff models. This paper was prepared for presentation at the Hydrologic Engineering Center's Urban Hydrology and Hydraulics Workshop, Davis, Ca., 13-15 September, 1994.

RURAL WATERSHED

The Omaha District has been requested to help the Colorado Water Conservation Board prepare a Flood Hydrology Manual for the state of Colorado. The purpose of this manual is to provide guidance and consistent methodology in analyzing flood hydrology within the state. As part of the Manual, example applications of utilizing several commonly used rainfall/runoff models to determine design discharges for an example watershed were performed. The example watershed selected was a relatively "hydrologically simple" 5.7 square mile basin in northeastern Colorado, Darby Creek near Buchanan. The drainage basin was modelled with HEC-1, Storm Water Management Model (SWMM), Colorado Urban Hydrograph Procedure (CUHP), and Kinematic Wave Rainfall-

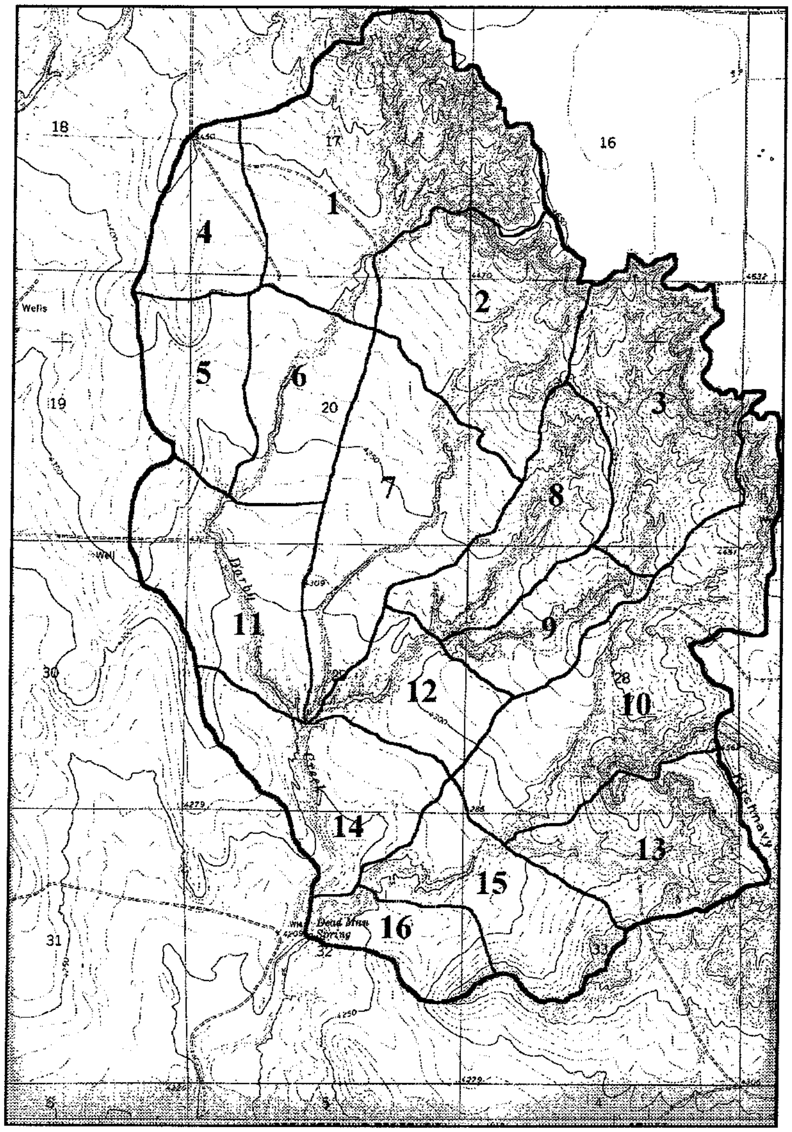


Figure 1

[1]Hydraulic Engineer, Hydrology and Meteorology Section, Omaha District, U.S. Army Corps of Engineers

Runoff Stream Network Model (KINNET). The basin was divided into the sixteen subbasins shown in Figure 1. The average size of the subbasins was approximately 0.3 square miles. All hydrologic parameters; channel slopes, channel dimensions, n-values, percent imperviousness, infiltration rates, etc. were estimated and remain consistent in all models.

Results

The outflow hydrographs from the basin for a 100-year rainfall event is shown in Figure 2 for the models tested.

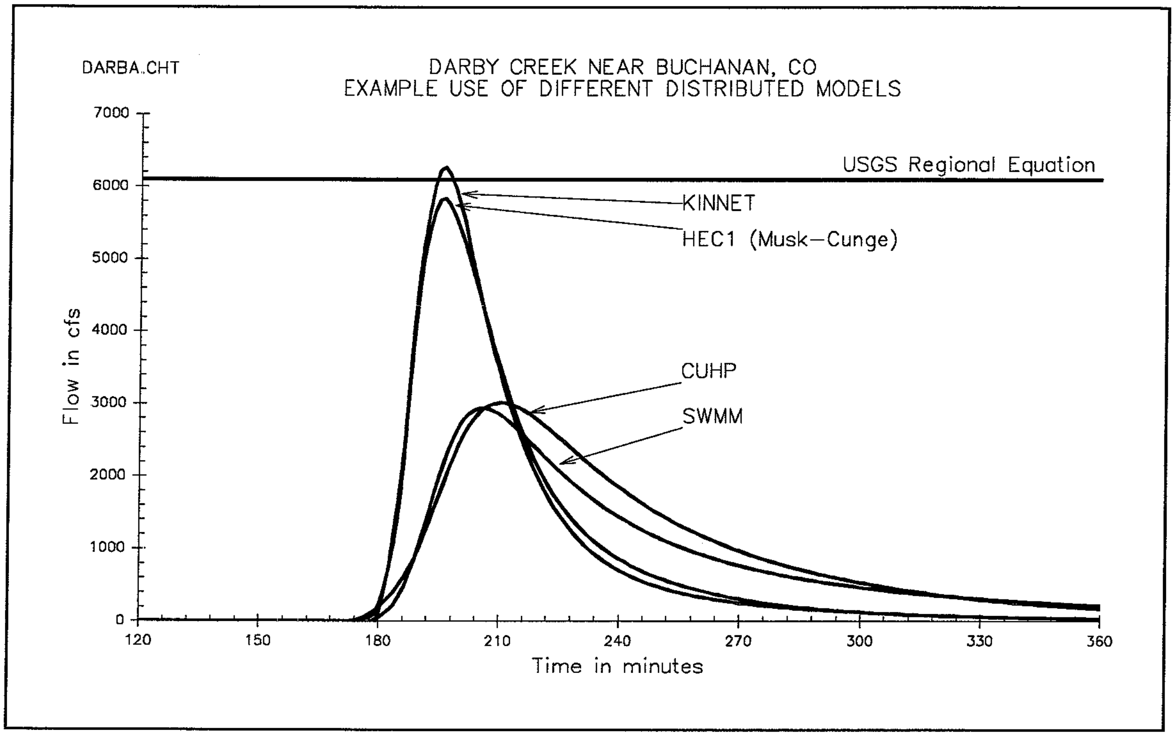


Figure 2

For comparison purposes, the estimated 100-year peak flow rate based on USGS regional equations (TM-1 - Manual for Estimating Flood Characteristics of Natural-Flow Streams in Colorado) for the basin was calculated as 6100 cfs.

Analysis of Results SWMM's resulting hydrograph appears to have slightly less volume than the other models' results. This may be due to the infiltration algorithm in SWMM that applies infiltration to the depth of flow rather than precipitation. By applying infiltration to the depth of flow spread over the entire subbasin results in greater infiltration losses since the duration of overland flow is longer than the duration of precipitation. The SWMM and CUHP models gave lower flow rates than the other models. This may be because both models utilize non-linear reservoir routing method as the channel routing option. The basin is very steep, with slopes ranging from 130 feet per mile to 80 feet per mile which generate rapidly rising hydrographs. Routing these hydrographs through relatively long channels, using a horizontal water surface or level-pool routing technique, may overestimate the physical attenuation effects. To test the effects of the spatial grid size, each of the sixteen basins were divided up into: two smaller subbasins for a total of 32, five smaller subbasins for a total of 80 subbasins, and 10 smaller subbasins for a total of 160. The results are shown in Figure 3.

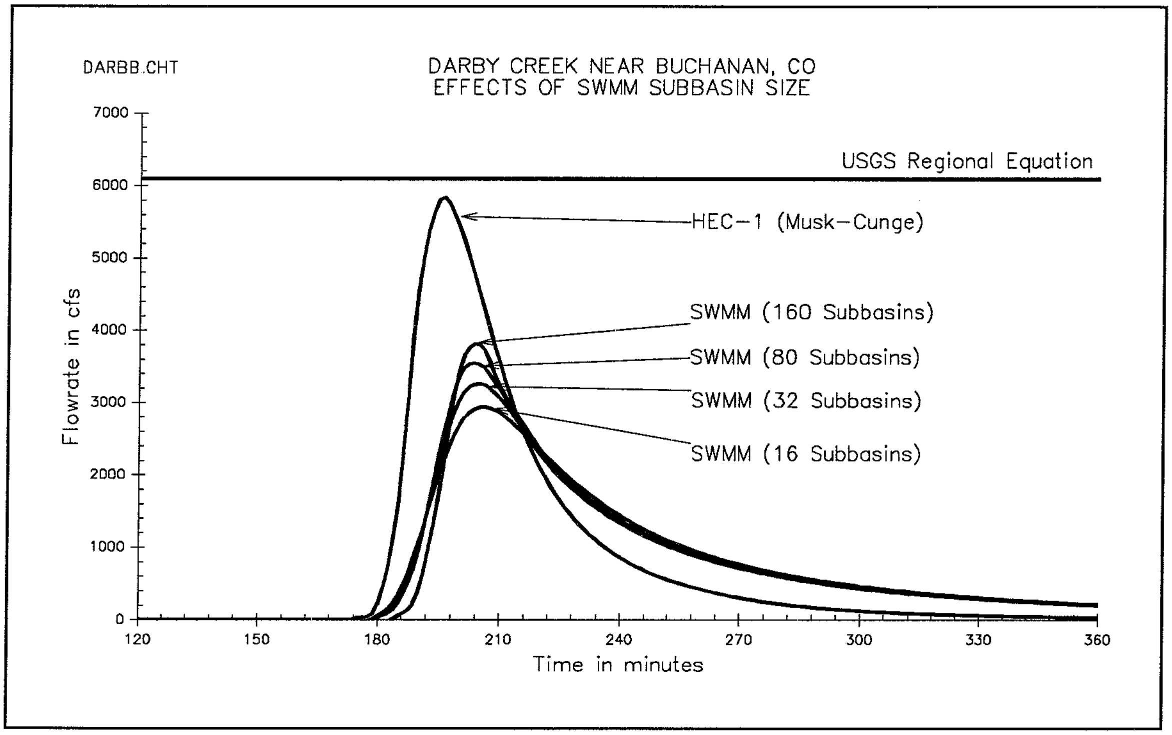


Figure 3

The results indicate that the finer the subbasins are delineated, the closer the results approach the kinematic or Muskingum-Cunge solution. Although it was not performed, a similar analysis could be done in further discretizing the *overland flow planes* into finer spatial grid sizes. It is assumed the results would match the kinematic solution even closer.

URBAN BASIN

The Omaha District has helped the Army Command of the Rocky Mountain Arsenal (RMA) to improve the hydrologic adequacy of one of their dams along the Irondale Gulch Drainage Basin. Rocky Mountain Arsenal is located just northeast of Denver, Colorado. The Irondale Dale Gulch covers a total area of 11.5 square miles upstream of the Arsenal and 18 square miles at the RMA northwest boundary as shown in Figure 4. The watershed is undergoing rapid urbanization in the basin upstream of the Arsenal. Continual development of residential and industrial sites upstream of the Arsenal increases both the volume and peak flow rates entering the Arsenal. As a consequence of this upstream development, the dams on the Arsenal have become increasingly hydrologically inadequate.

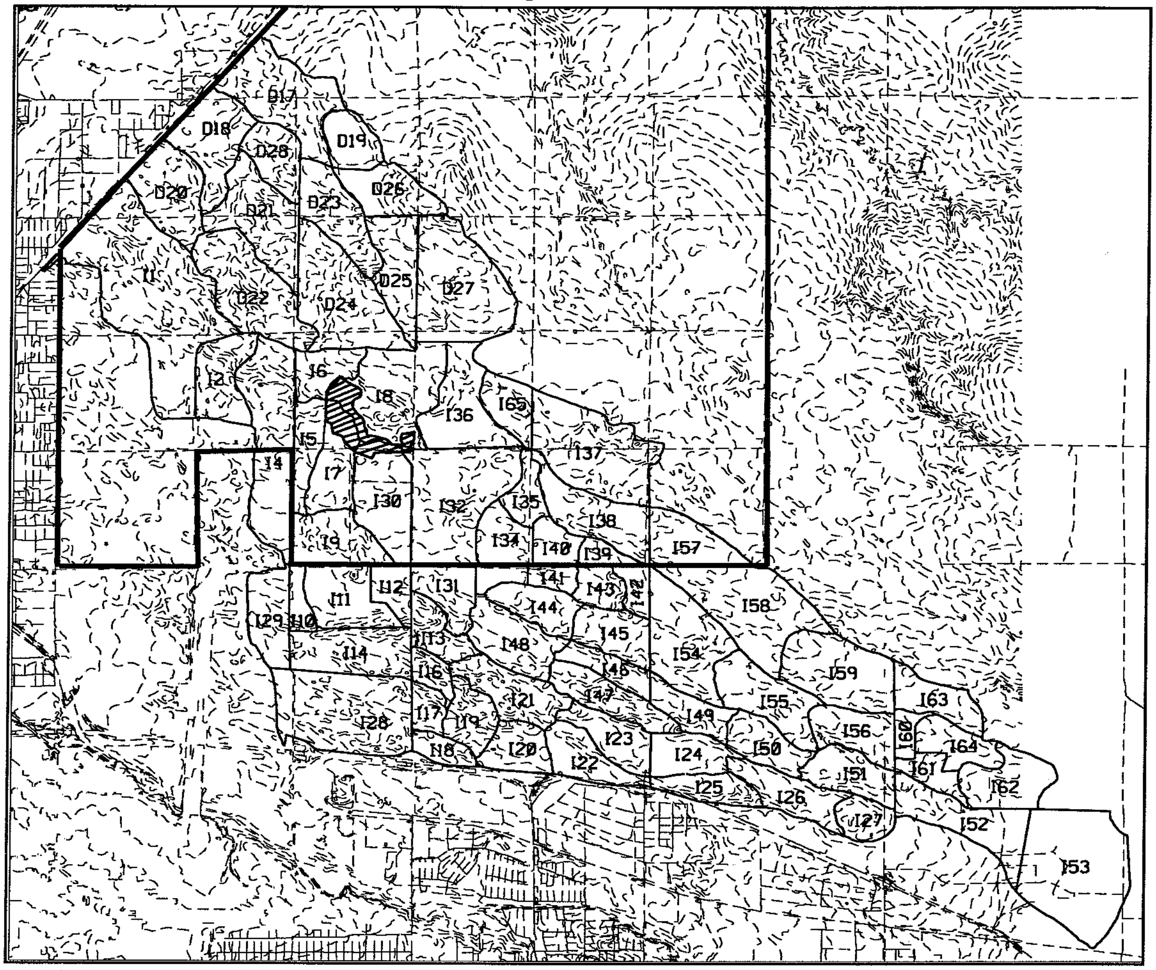


Figure 4

Model Development

As part of a Master Planning Analysis completed in 1990 by the local flood control district, the Irondale Gulch drainage basin had been modelled with the Colorado Urban Hydrograph Procedure (CUHP) and the Storm Water Management Model (SWMM). The CUHP method incorporates unit hydrograph technique utilizing calibrated unit hydrograph coefficients to generate hydrographs for subbasins. An earlier version of SWMM is used to perform channel and reservoir routings. To take advantage of improved channel routing techniques and the dam failure simulation capabilities in HEC-1, the basic hydrologic data in the CUHP/SWMM model was transferred to a HEC-1 model. The basic hydrologic data were: drainage basin delineations, subbasin connectivities, channel slopes, channel dimensions, n-values, etc.

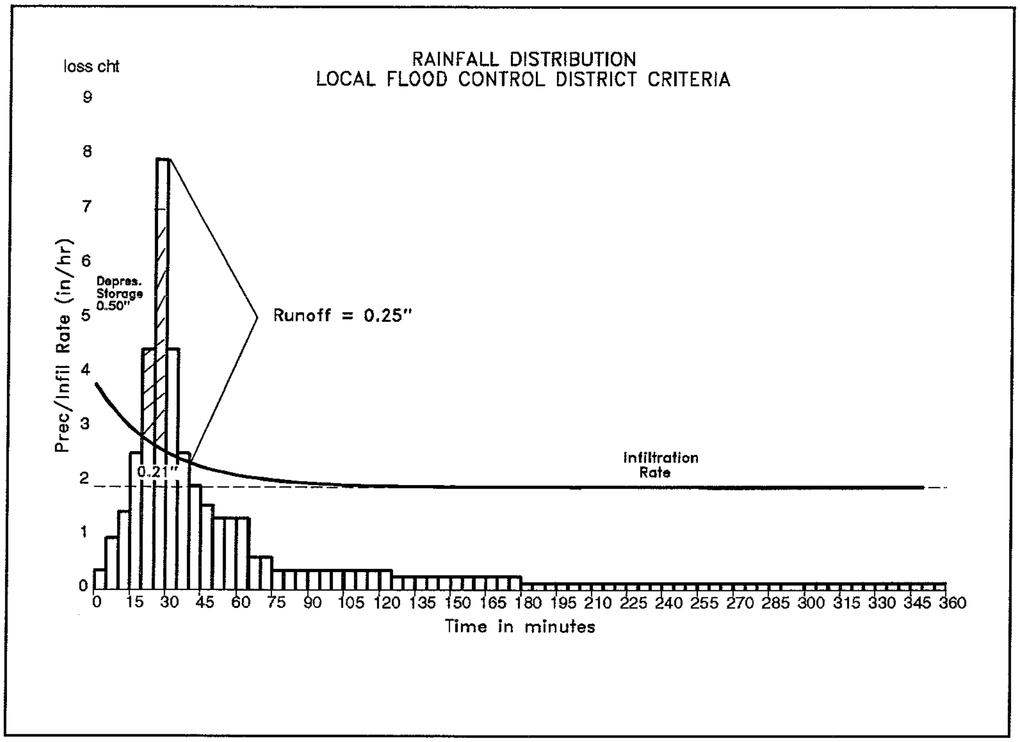
Results

Comparison between the models results at the inflow point for the reservoir being investigated showed the HEC-1 model results to have approximately three times larger peak flows than the results from CUHP/SWMM.

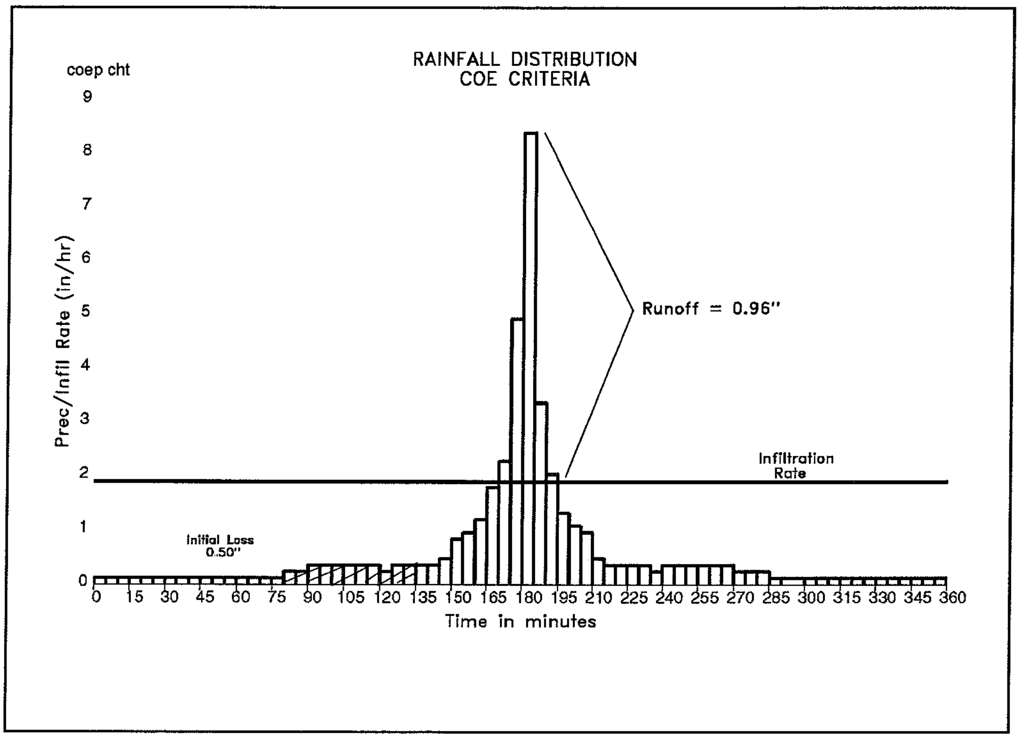
Analysis of Results

The large differences in peak discharges were determined to be the result of five processes in the rainfall/runoff models: rainfall distribution, infiltration methods, depression storage losses, different methods of overland flow runoff generation, and different methods for channel routing.

1. Rainfall Distribution- Both models used the same accumulated rainfall amount for the six-hour design storm. The rainfall distribution, though, varied between the two methods. The method used in the HEC-1 model was a balanced storm with the peak intensity occurring at the center of the storm as shown in Figure 5. The CUHP method, based on typical storms in the Denver area, placed the most intense portion of the storm within the first thirty minutes of the start of the storm also shown in Figure 5. While rainfall distributions with peaks occurring near the beginning of the storm may be the distribution for commonly occurring storms in the area, it was felt that rainfall distributions of extreme events are not as well known, and as such, should be arranged to reflect the most conservative possible distribution.



loss cht
RAINFALL DISTRIBUTION
LOCAL FLOOD CONTROL DISTRICT CRITERIA
Prec/Infil Rate (in/hr)
Depres. Storage 0.50"
Runoff = 0.25"
0.21"
Infiltration Rate
0 15 30 45 60 75 90 105 120 135 150 165 180 195 210 225 240 255 270 285 300 315 330 345 360
Time in minutes



coep cht
RAINFALL DISTRIBUTION
COE CRITERIA
Prec/Infil Rate (in/hr)
Runoff = 0.96"
Infiltration Rate
Initial Loss 0.50"
0 15 30 45 60 75 90 105 120 135 150 165 180 195 210 225 240 255 270 285 300 315 330 345 360
Time in minutes

Figure 5

2. Infiltration Process- The CUHP method used an exponential decaying infiltration rate, while the HEC-1 model used an initial and uniform loss, with the uniform rate being set equal to the minimum rate used in CUHP. The uniform loss rate was used in HEC-1 because the exponential loss rate assumes there is a soil moisture deficit conditions prior to the start of the storm. Without knowing the antecedent soil moisture conditions, it would be conservative to assume that there would be rainfall prior to the six-hour design storm that would satisfy any soil moisture deficit conditions. The numeric consequences of using a rainfall distribution with the peak intensities near the beginning of the storm combined with using an exponentially decaying infiltration rate are shown in Figure 5. Figure 5 shows there is approximately 0.21 inches of potential runoff out of a possible 0.96 inches of runoff that is lost to satisfying the a soil moisture deficit which may or may not be there during the actual storm event.

3. Depression Storage- The two methods treat storage depression losses differently. In HEC-1, using the initial and uniform loss option, the depression storage loss is simulated with the initial loss, which is satisfied by the initial rainfall prior to infiltrating occurring. In CUHP, though, the depression storage does not begin to be satisfied until the precipitation rate exceeds the infiltration rate, or when ponding begins. Both methods are shown in Figure 5.

The difference between methods is significant in determining the volume of runoff. The total potential runoff is 0.96 inches (volume of rainfall that exceeds infiltration). CUHP subtracts the 0.50 inches of depression storage directly from this amount, leaving 0.46 inches of runoff. The 0.50 inches of depression storage in HEC-1 is lost at the beginning of the storm, so the effect on runoff is minimal.

The total in runoff volume for pervious areas due to rainfall distribution, infiltration methods, and depression storage is 0.96 inches for HEC-1 and 0.25 inches for CUHP/SWMM, a difference of 0.71 inches or nearly 300 percent in this case.

4. Overland Flow Generation- CUHP generates subbasin or overland flow hydrographs using the unit hydrograph method with calibrated unit hydrograph coefficients. The method used in HEC-1 was kinematic wave. A direct comparison of the two methods was completed for a headwater basin. To negate the effect of difference in runoff volumes for pervious areas, a primarily urban subbasin was compared. The generated subbasins hydrographs are shown in Figure 6 with the CUHP hydrograph time advanced to offset the differences in rainfall distributions.

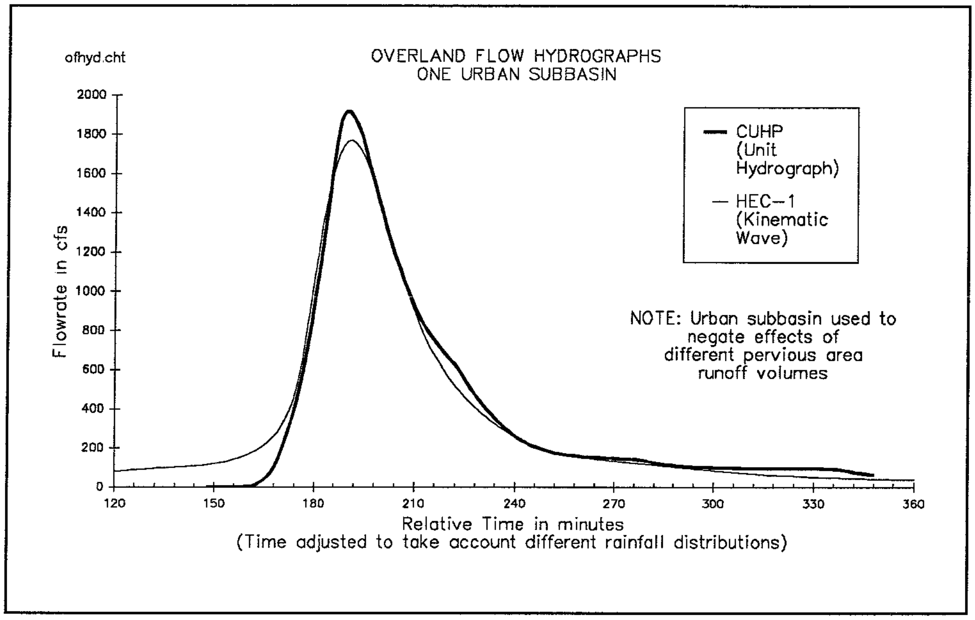


Figure 6

As can be seen the similar hydrographs, it does not appear if overland flow was a significant factor in the difference in runoff at the reservoir.

5. Channel Routing- The last significant difference between the two models was the channel routing methods. The Muskingum-Cunge routing option was used in HEC-1, while the CUHP/SWMM model apparently used the non-linear reservoir routing method. The effect of different channel routing options on floodwave attenuation throughout a routing reach was tested by routing the same inflow hydrograph though a test channel (length=6600', bottom width=100', slope=0.002) for the following options:

Model	Routing Method
1. CUHP/SWMM	non-linear reservoir
2. HEC-1	Muskingum-Cunge
3. HEC-1	Reservoir routing (using normal depth rating curve outflow at d/s end and elevation capacity relationship of channel)
4. UNET	Full dynamic wave

The results for the channel routings are shown in Figure 7.

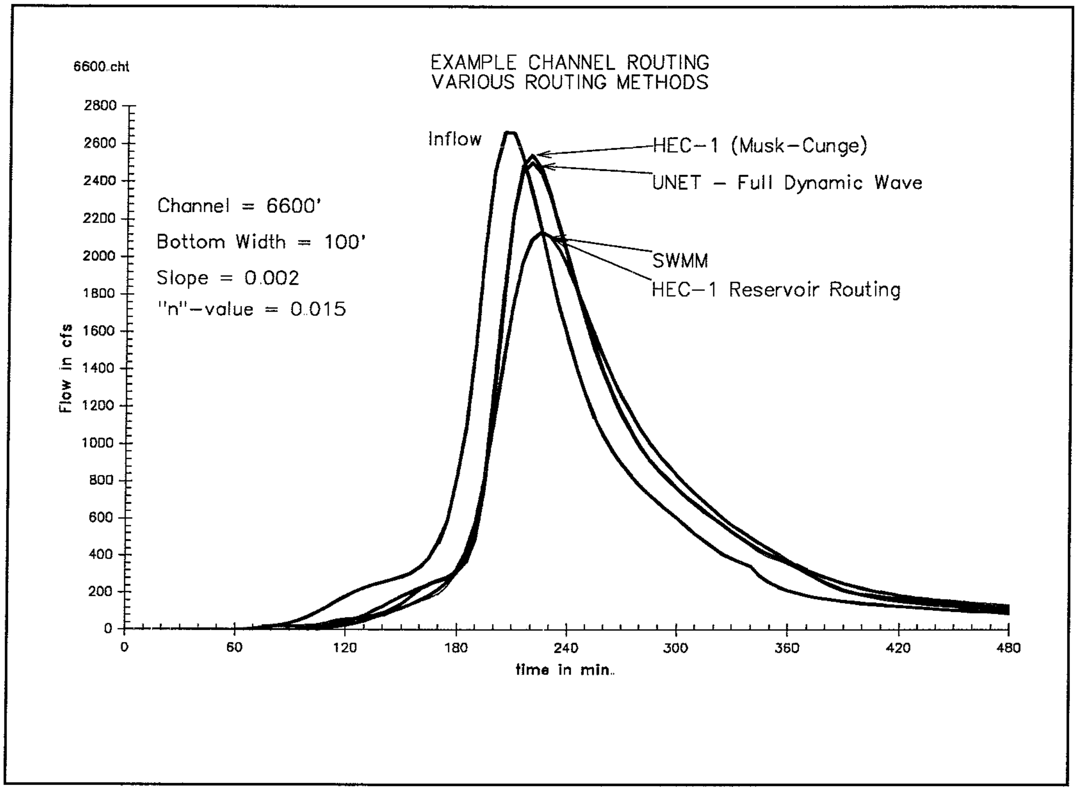


Figure 7

The results of the routings indicate:

1.) The CUHP/SWMM and HEC-1 reservoir routing give nearly identical results, confirming that CUHP/SWMM uses a reservoir routing routine.

2.) The Muskingum-Cunge and UNET full dynamic wave simulations gave virtually identical results. This indicates that at least for this channel, the Muskingum-Cunge method most closely simulates the physical processes in flood routing.

SUMMARY

The intent of this paper is to summarize recent experiences of the Omaha District U.S. Army Corps of Engineers in applying rainfall/runoff models to two watersheds in Colorado. It is not intended to provide recommendations for various models and methods. Regardless of which models and methods are used for basin modelling, the most important factors are the calibration, verification, and understanding the physical processes involved in the model simulation of the hydrologic features of the basin.

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Hydrology and Hydraulic Models as Shapers of Floodplain Development Policy: Dallas-Fort Worth Metroplex

by Paul K. Rodman[1]

1. Introduction.

A. Purpose. This paper presents a summary of the use of hydrology and hydraulic (H&H) models in shaping floodplain development policies of the Corps of Engineers regulatory program and of City ordinances for the Upper Trinity River and Tributaries in the Dallas-Fort Worth Metroplex.

B. Key Issues. Potential cumulative impacts of numerous unrelated development projects being proposed along the Trinity River and its tributaries in the Dallas, Denton, and Tarrant Counties in the mid 1980's caused significant concern. Hydrology (HEC-1) and Hydraulic (HEC-2) models were used to show the impacts of various development scenarios and policies. In 1988 the Corps published the Regional Environmental Impact Statement (REIS) for the Trinity River and Tributaries and issued a Record of Decision with hydrology and hydraulics criteria for Section 404 decisions for floodplain development in the Metroplex. (Enclosure 1). Significant portions of the floodplain are not controlled under Section 404. Indicated impacts of alternative development scenarios were so severe that by 1990 the nine cities and three counties involved had all endorsed a resolution in support of using criteria similar to those of the Corps Record of Decision in evaluating floodplain development. Those criteria were incorporated into the Corridor Development Certificate (CDC) process and implementing ordinances were adopted by all cities and some counties in late 1993 and early 1994. Updated H&H models from the Upper Trinity Feasibility Study will be used in implementing City Ordinances and the Record of Decision.

2. Physical Setting and Available Data.

A. Description of Project Characteristics. The area hydrologically modeled in this study consists of the entire drainage area of the Trinity River upstream of the point where Five Mile Creek flows into the Trinity River near the intersection of the Trinity River and Interstate Highway 20 (about 10 miles southeast of downtown Dallas).

The total drainage area at that point is approximately 6,275 square miles. Included in this area is the Fort Worth-Dallas Metroplex. The total drainage areas of the Trinity River at the Elm Fork-West Fork confluence and at the Dallas Gage are 6,061 and 6,106 square miles, respectively. The terrain elevation

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varies from 1200 feet NGVD at the headwaters of the West Fork of the Trinity River approximately 35 miles south-southwest of Wichita Falls, Texas, to 380 feet NGVD at the confluence of Five Mile Creek and the Trinity River. Figure 1 is a general watershed map for the study area.

Of the five Corps of Engineers flood control lakes in the study area, Lakes Benbrook, Lewisville, and Grapevine were impounded in the early 1950's. The two remaining Corps lakes, Lakes Joe Pool and Ray Roberts, were impounded in January 1987 and June 1987, respectively. Additional Corps of Engineers major flood control projects in the study area include the Dallas Floodway and the Fort worth Floodway.

The two largest non-Federal lakes in the study are Lake Bridgeport and Eagle Mountain Lake. Lake Bridgeport is located on the upper West Fork near the city of Bridgeport in Wise County. Eagle Mountain Lake is located in Tarrant County on the West Fork above the much smaller Lake Worth owned by the city of Fort Worth. Eagle Mountain Lake has two sets of gates and an emergency spillway. Since it has no dedicated flood control storage, large and long-duration releases are required during floods. Lake Amon Carter, located in Montague County, is a small lake on Big Sandy Creek north of Lake Bridgeport. Lake Weatherford is a small lake on the upper Clear Fork, located in Parker County. Lake Arlington is a small lake on Village Creek in Tarrant County within the city limits of Arlington. Mountain Creek Lake is a power plant cooling lake on Mountain Creek in Dallas County near the city of Grand Prairie.

The Trinity River watershed is located in a region of temperate mean climatological conditions, experiencing occasional extremes of temperature and rainfall of relatively short duration. The National Oceanic and Atmospheric Administration Station at Fort Worth, Texas shows an average annual rainfall of 32.3 inches during a recent ten year period (1976-1985). The extreme annual rainfall values since 1887 are a maximum of 53.54 inches occurring in 1991, and a minimum of 17.91 inches occurring in 1921. The mean relative humidity is 65 percent and the average temperature is 65.8 degrees.

Generally the major storms experienced in the study area are produced by heavy rainfall from the frontal-type storms which occur in the spring and summer months, but major flooding can also be produced by intense rainfall associated with localized thunderstorms. These thunderstorms may occur at any time during the year but are more prevalent in spring and summer months. Precipitation from hurricane moisture can be very intense and occur over a large area. Hurricane related storms generally occur from July to October.

B. Description of available pertinent data. Some previous Corps of Engineers hydrology and hydraulics studies of the Trinity River Basin above the Dallas gage include, the "Definite Project Report on the Dallas Floodway" (1952), the "Comprehensive Survey Report of the Trinity River and Tributaries" (1962), the "Trinity River Project Memorandum No. 2" (1978), and the "Reconnaissance Report for the Upper Trinity River Basin" (1990).

In addition, hydrology and hydraulics modelling has been performed by the Fort Worth District for many portions of the upper Trinity Basin and the Dallas-Fort Worth Metroplex. Urban studies conducted by the U.S. Geological Survey have been utilized in developing curves indicating impact of urbanization on time to peak (Nelson, 1970) (Rodman, 1977). Urban curves for the upper Trinity River, Dallas-Fort Worth Metroplex, are included as figures 2 and 3. A daily period of record system model is available for the Trinity River Basin for the period 1940 to 1992 for evaluating larger flood control storage projects.

3. Study Approach.

The March 1990 Reconnaissance Report for the Upper Trinity River Basin, "Common Vision", utilized H&H models with outdated topography in the cross sections for the hydraulic model. As part of the cost-shared Feasibility Study, detailed topography (two feet vertical, one inch equals 200 feet horizontal) was flown and developed for the mainstem Trinity floodplain in the Metroplex for approximately three million dollars in 1991. GIS land use provided by the North Central Texas Council of Governments was used as the basis for urbanization and imperviousness for each subarea of the hydrology model. GIS soils data was used to estimate percent sand and clay for use with urban curves and to estimate loss rates. In order to support the Corps and City regulation programs, the decision was made to continue using HEC-1 and HEC-2 since engineering consultants for cities and developers were already familiar with them. The HEC-2 model was used to develop new storage discharge data for routing. New calibration runs were made for the HEC-1 model. Historical flood hydrographs at streamgages were simulated with the model for June 1989, April - May 1990, and December 1991 events. In the HEC-1 model, loss rate values, unit hydrograph "time-to-peak", and Cp values were adjusted in order to generate computed hydrographs which plotted as closely as possible to the observed hydrographs at stream gages and lakes throughout the Trinity River basin. Also, the Muskingum X, K, and number of routing steps in Muskingum and modified Puls routing methods were adjusted. The results of the recent hydrograph reproductions were compared with those of October 1974, March 1977, October-November 1981, and May 1982 events obtained in the Upper Trinity Reconnaissance Study in deciding on model parameters for the large, rural upper watershed subareas. The HEC-1 model was further calibrated by adjusting parameters such as loss rates and time-to-peak within reasonable limits in order to match as closely as possible the expected probability peak values of eight different frequency floods based on analyses of historic peaks at several streamgages. The gage record considered starts in 1953 (since most major lakes were in place by 1952) and continues to 1991.

The Standard Project Storm was assumed to have a total rainfall amount equal to 50 percent of the Probable Maximum Storm rainfall. The Probable Maximum Storm rainfall was determined in accordance with the method described in Hydrometeorological Report No. 51, dated June 1978, Subject: "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," and Hydrometeorological Report No. 52, dated August 1982, Subject: "Application of Probable Maximum Precipitation Estimates - United States East of the 105th Meridian." The computer program used to develop the Standard Project Storm was the HMR52 Program developed by the Corps of Engineers' Hydrologic Engineering Center. The duration of the Standard Project Storm was 72 hours. Four separate elliptical storm centers were used to obtain critical discharges on the West Fork, Clear Fork, Elm Fork, and mainstem Trinity River.

The modified Puls routing method was used to route through the urban and urbanizing reaches downstream of Lake Worth, Benbrook, Grapevine, and Lewisville Lakes. Storage-discharge data were based on HEC-2 backwater analyses using the latest 2-foot topography. Non-conveying portions of cross sections were considered in estimating storage. The number of routing steps for the modified Puls method was determined by dividing the storage (acre-feet) for a particular discharge (cfs) by the discharge and multiplying by 12.1 (units conversion factor) to get travel time in hours. The travel time was divided by time interval used in the HEC-1 model to get the number of steps.

Permitted mainstem floodplain projects were included in the HEC-2 model for Baseline (year 2000) conditions. Land use changes for Baseline and Future (year 2050) were projected by the North Central Texas Council of Governments. These projections were used with the GIS data base to estimate urbanization and imperviousness for Baseline and Future conditions. Expected probability discharge-frequency relationships were developed using the HEC-1 model with Baseline condition storages. It was assumed that the cities would adopt the Corridor Development Certificate (CDC) process into their ordinances and that future development would meet the CDC H&H criteria. Thus, since conveyances should not be significantly reduced for the one percent (100-year) flood and SPF and since maximum valley storage reduction should not significantly exceed five percent, the Baseline HEC-2 model was assumed to be a reasonable approximation for hydraulic evaluation of Future conditions.

Adoption of the CDC process into ordinances by the nine cities involved in this study in late 1993 and early 1994 made the assumption that future development would meet the CDC criteria more plausible. However, the CDC process has a regulatory zone (one percent event floodplain) and a review zone (Standard Project Flood floodplain). Projects located in the review zone generally must meet the CDC H&H criteria. Projects located in the review zone can be exempted by a city from the CDC process and most cities are choosing to grant that exemption. Cities have postulated that development in the zone above the one percent floodplain will not fill excessively, since that is not needed for protection. Thus, cities do not anticipate significant valley storage loss in the review zone, even though most are going to allow development there.

During the last 10 years I have attended several hundred meetings on the Upper Trinity River basin. These have included study team meetings within the Corps, City and County Staff Task Force, Steering Committee (elected political representatives), Wise and Jack County Watershed Committees and public meetings, special committee meetings for drafting the CDC process, special meetings with the State of Texas and FEMA, H&H modelling subcommittee meetings, and meetings with contractors for developers and Corps regulatory personnel. In 1993, I met with individual city staffs and with City managers to answer their questions on the CDC process. Public information meetings were held at various locations in the Metroplex. Significant effort has been expended in evaluating alternative development scenarios and presenting those results to city staffs and politicians so that they can see reasons and common interests in limiting impacts of floodplain development. Appeals were made to the goodwill and cooperativeness of the participants. Some cities sacrificed more than others in limiting their development. However, the results of the H&H models convinced them that this was still in their best interest and that no city was given an unfair competitive advantage in seeking additional development in the floodplain.

Several newspaper articles in mid 1993 revived various cities' interest in passing CDC ordinances. Dallas was especially impressed by an article on the potential failure of the Dallas Floodway with the occurrence of the Standard Project Flood (the original design flood). After the 1990 flood, high water marks indicated that roughness within the Dallas Floodway was higher than originally estimated. Downstream from the Dallas Floodway, trees had not been removed for many years and several landfills, some unpermitted, were found to exist in the floodplain. Computed conveyance was reduced to the point that the Standard Project Flood was estimated to overtop the levee protecting downtown Dallas and a major industrial area. This was a radical change from the 1990 Reconnaissance study where the existing conditions SPF water surface was computed a couple of feet below the top of the levee.

4. <u>Study Results</u>.

Preliminary results for the H&H models for the Feasibility Study for the Upper Trinity River are now available. Generally, expected probability discharges have increased 15 to 20 percent and computed water surfaces for frequency specific events have increased as much as two feet over those previously available. These models are being used for analysis of flood control alternatives for both the Upper Trinity and the Dallas Floodway Extension Study. The Corps Southwest Division has done a preliminary H&H review for us. Since these new H&H models will eventually be used in the CDC process and must be approved by the cities' political representatives, we have sought review and comment by the cities and their consultants. Some of the reviews have been very detailed. We are currently preparing responses to comments on the H&H models. It is anticipated that some changes will be made to the models in response to the comments.

The H&H models of the Upper Trinity River have been very instrumental in shaping floodplain development policy in the Dallas-Fort Worth Metroplex in the last 10 years. Floodplain development projects being designed today are very different from those that were designed 10 years ago. Conveyance and storage criteria require the use of part of the tract for those purposes. For a project deep in the floodplain, more than half of the land may be required for H&H mitigation. The H&H mitigation is usually complementary to mitigation requirements for wetlands and habitat under Section 404 and Section 10. For public projects such as roads, sewage treatment plants, and flood control projects protecting existing life and property, some variance from the criteria is allowed. However, even these type of projects must show a good faith attempt to meet the H&H criteria. Floodplain road design has been made much more complex. Public entities frequently desire to raise roads above the one percent flood water surface. Generally, H&H criteria must be met for roads so that conveyance and valley storage are not significantly impacted. We are currently awaiting the designs for two major roads crossing the West Fork Trinity River between Dallas and Fort Worth. In both cases, we have asked the designers to meet the H&H criteria of the Record of Decision. We are currently reviewing a major floodplain development project involving approximately 2000 acres along the West Fork Trinity River, under both Section 404 and the CDC process. Conveyance and storage mitigation will be done so that the H&H criteria of the Record of Decision are met. Negative H&H impacts of current and future development will be minimized as projects are designed to meet the criteria of the Record of Decision and CDC.

5. <u>Conclusions</u>.

The hydrology and hydraulic modelling used for these studies is neither particularly advanced nor exceptionally complex. The significant accomplishment of this set of studies is the communication of the impact of policy decisions to the decision makers and the subsequent effect on policy.

Floodplain management policies have been the subject of considerable national discussion and debate since the upper Mississippi River floods of 1993. The experience of the Upper Trinity has at least some application to the Mississippi and to floodplain management in general. Our experience indicates that, in a watershed such as the upper Mississippi, one master set of hydrology and hydraulics models is essential to planning and analyzing impacts of existing and future projects. Examination of alternative development scenarios with the H&H models can be very helpful in obtaining public and political support for implementing floodplain development criteria.

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RECORD OF DECISION
REGIONAL ENVIRONMENTAL IMPACT STATEMENT
TRINITY RIVER AND TRIBUTARIES

I. Introduction

Since its early history, the U.S. Army Corps of Engineers has played an important role in the development of the nation's water resources. Originally, this involved construction of harbor fortifications and coastal defenses. Later duties included the improvement of waterways to provide avenues of commerce and reduce flood hazards. An important part of its mission today is the protection of the nation's waterways through the administration of the Regulatory Program. The Corps is directed by Congress under Section 10 of the Rivers and Harbors Act of 1899 (33 USC 403) to regulate all work or structures in or affecting the course, condition, or capacity of navigable waters of the United States. Section 9 (33 USC 401) directs the Corps to regulate the construction of any dam or dike across a navigable water of the United States. The intent of these laws is to protect the navigable capacity of waters important to interstate commerce.

Additionally, the Corps is directed by Congress under Section 404 of the Clean Water Act (33 USC 1344) to regulate the discharge of dredged and fill material into all waters of the United States, including adjacent wetlands. The intent of this law is to protect the nation's waters from the indiscriminate discharge of material capable of causing pollution, and to restore and maintain their chemical, physical, and biological integrity. Because the District Engineer's decision to issue or deny a permit under these laws is a significant Federal Action, various other statutes, principally Public Law 91-190 (the National Environmental Policy Act, or NEPA) come into play. Among other things, NEPA requires the consideration of the direct, indirect, and cumulative impacts of an action (40 CFR 1508.25(C)).

Late in 1984 and early in 1985, it became apparent that numerous unrelated development projects were being proposed along the Trinity River and its tributaries in Dallas, Denton, and Tarrant Counties, Texas. Most involved modification of the river channel and/or flood plain in some form or another, and most required a Corps of Engineers permit as a result. Because, individually or cumulatively, these projects were felt to have the potential to compromise the existing protection afforded to flood plain residents, because of perceived impacts to wetlands and other natural resources, and because of competing public demands for other uses of the river channel and flood plain, the District Engineer determined that it was necessary to develop a regional perspective in order to properly evaluate the impacts of individual permit decisions in accordance with the spirit and intent of NEPA and other applicable laws.

The Draft Regional Environmental Impact Statement (EIS), published in May 1986, analyzed a number of scenarios which were specifically designed to identify possible, significant cumulative impacts associated with different permitting strategies for the Trinity River flood plain. In addition to developing a baseline condition, it examined three groups of conditions based on a) maximizing environmental quality, b) ultimate implementation of the

Federal Emergency Management Agency's (FEMA) minimum criteria for the flood insurance program, and c) maximizing economic development.

The results of the Draft Regional EIS indicated strongly that there are potential cumulative impacts associated with individual flood plain development projects which are both measurable and significant. Additionally, the Draft Regional EIS indicated that the permitting approach adopted by the Corps of Engineers had the potential to have significantly different impacts on a number of regional parameters, especially flood hazards. Even though the analyses were not complete, and the public comment on the Draft Regional EIS indicated that there was much work to follow, the implications to the ongoing Regulatory Program could not be overlooked. In response to this, the Corps formulated a set of interim criteria to be in effect until the Record of Decision was rendered.

Many of the comments received on the Draft Regional EIS indicated that the slate of alternatives analyzed did not represent a realistic approach to regulatory strategies. In many cases, the predicted results were publicly unacceptable. Two important examples include the overtopping of the Dallas Floodway levees under two of the scenarios, and a substantial downstream shift in the Dissolved Oxygen "sag" resulting in noncompliance with State Water Quality Standards in the reach below the Trinidad gage. After careful analysis of the public and agency input, several new scenarios were formulated for analysis in the Final Regional EIS.

In addition to updating the baseline, three scenarios, representing the same three broad categories that had been previously addressed, were developed. Many people suggested that the Maximum Development scenarios analyzed in the Draft Regional EIS were too extreme, either because they conflicted with an ongoing project, or because levees were physically impractical in some portions of the flood plain. In response to this criticism, we agreed to replace them with a "Composite Future" scenario. Each city was tasked to provide the North Central Texas Council of Governments (NCTCOG) a delineation of the "most likely" limits of maximum encroachment within their jurisdiction. NCTCOG compiled each city's individual prediction and presented the resultant set of maps to local staffs and local elected officials before providing them to the Corps for analysis.

The Modified Floodway scenario of the Final Regional EIS replaced the floodway-based scenarios of the Draft Regional EIS as a representative compromise between maximum (realistic) development and maximum (realistic) environmental quality. In this scenario, the Corps defined the geographic limits of a drainageway incorporating the FEMA concept with significant technical variations. For the third scenario, the Corps revised and represented a Maximum Environmental Quality scenario, hydraulically identical to the revised baseline because it incorporated no additional flood plain projects except water quality, recreation, and wildlife enhancements. Of the scenarios, or alternatives, examined in the Final Regional EIS, this is the environmentally preferred alternative.

The extensive coordination and public involvement characteristic of the Regional EIS process continued during the comment period on the Final Regional EIS, which extended from its release on October 22, 1987, through January 31, 1988. During this period, I held a public meeting at Lamar High School at which eleven people submitted statements. My staff attended in excess of twenty meetings with local government staffs, public agencies, and citizen groups. In addition, sixty-six written comments on the Final Regional EIS were received.

II. Discussion of Issues and Factors

Most of the formal public comment and discussion with local governments centered on three general issues: the appropriate level of flood protection (100-year vs. SPF), the level of accuracy of the hydraulic and hydrologic analyses displayed in the Regional EIS, and the issue of equity as it pertains to governmental regulation. "Benefits" and "costs" of an action, whether it be a proposed project or a proposed regulation, do not always occur to the same group of people, let alone in the same order of magnitude. The definition of the "public interest" which is at the heart of the Regional EIS calls for an assessment of the tradeoffs inherent between public demands for enhanced environmental quality in the river corridor and for its use for needed public facilities, and economic development and the rights of private landowners.

A major concensus achieved through the review of the Final Regional EIS is that additional regional increases in flood hazards for either the 100-year or Standard Project Flood are undesirable, and that the thrust of flood plain management, in the short term, should be to stabilize the flood hazard at existing levels through regulation. Future efforts on the part of both the Corps and local organizations may be required to reduce flood hazard over the long term.

The Regional EIS is probably the most comprehensive such study done in the United States. It has highlighted the need for planning for the region and cooperation among the governmental entities along the Trinity River corridor to achieve quality development. The document was developed for the sole purpose of establishing a permitting strategy for the Trinity River and its tributaries. It does not contain a technical baseline that will remain current over time and is not to be used as a design document. Design decisions requiring water surface predictions based on critical storm centerings, and which are sensitive to valley storage computations, must be based on detailed site-specific engineering analyses. Other site-specific public or private flood control management decisions should likewise be based on current technical analyses. Further, flood insurance data must be obtained from the FEMA and not from the Regional EIS.

Neither the Regional EIS nor this Record of Decision encroaches upon the responsibility of design engineers or the authority of local governments. The Regional EIS, its public review, and this Record of Decision serve only to establish and document the "best overall public interest" as it applies to the Trinity River and its tributaries. It remains the responsibility of design engineers to perform competent work in accordance with professional design practices. Permit applicants who proposed flood plain modifications and/or site-specific flood control structures will need to satisfy review agencies as to the reasonableness of design assumptions.

Throughout the development of this Record of Decision, the Corps has worked closely with the NCTCOG to insure consistency with their COMMON VISION program. The criteria listed below for the West Fork, Elm Fork, and Main Stem are consistent with the Statement of Principles for Common Permit Criteria submitted by the Steering Committee of local government officials. Because of the massiveness of this undertaking and the importance of its impact on future growth, the comments from the cities and other governmental entities have been carefully considered.

III. Decision

Based on my consideration of the data developed and presented in both the Draft and Final Regional EIS's and my careful consideration of all public input, I have determined that, for the purposes of the Regional EIS study area, my Regulatory Program will be henceforth based on the following criteria. The baseline to be used in analyzing permit applications will be the most current hydraulic and hydrologic model of the specific site in question. The burden of proof of compliance with these criteria rests with the permit applicant. Variance from the criteria would be made only if public interest factors not accounted for in the Regional EIS overwhelmingly indicate that the "best overall public interest" is served by allowing such variance.

A. Hydraulic Impacts--Projects within the SPF Flood Plain of the Elm Fork, West Fork, and Main Stem. The following maximum allowable hydraulic impacts will be satisfied, using reasonable judgment based on the degree of accuracy of the evaluation, and using cross sections and land elevations which are representative of the reaches under consideration:

1. No rise in the 100-year or SPF elevation for the proposed condition will be allowed.

2. The maximum allowable loss in storage capacity for 100-year and SPF discharges will be 0% and 5% respectively. (on Site and Generally)

3. Alterations of the flood plain may not create or increase an erosive water velocity on-or off-site.

4. The flood plain may be altered only to the extent permitted by equal conveyance reduction on both sides of the channel.

B. Hydraulic Impacts--Tributary Projects. For tributaries with drainage areas less then 10 square miles, valley storage reductions of up to 15% and 20% for the 100-year and Standard Project Floods, respectively, will be allowed. For tributaries with intermediately-sized drainage areas (10 square miles to 100 square miles), the maximum valley storage reduction allowed will fall between 0% and 15% for the 100-year flood and 5% and 20% for the Standard Project Flood. Increases in water surface elevations for the 100-year flood will be limited to approximately zero feet. Increases in water surface elevations for the Standard Project Flood will be limited to those which do not cause significant additional flooding or damage to others. Projects involving tributary streams with drainage areas in excess of 100 square miles will be required to meet the same criteria as main stem projects (see "A" above).

C. Cumulative Impacts. The upstream, adjacent, and downstream effects of the applicant's proposal will be considered. The proposal will be reviewed on the assumption that adjacent projects will be allowed to have an equitable chance to be built, such that the cumulative impacts of both will not exceed the common criteria.

D. Design Level of Flood Protection. The engineering analysis will include the effects of the applicant's proposal on the 100-year and Standard

Project Floods and should demonstrate meeting FEMA, Texas Water Commission, and local criteria, as well as Corps, for both flood events.

1. For levees protecting urban development, the minimum design criterion for the top of levee is the SPF plus 4.0, unless a relief system can be designed which will prevent catastrophic failure of the levee system.
2. For fills, the minimum design criterion is the 100-year elevation, see above, plus one foot.

E. Borrow Areas. The excavation of "borrow" areas to elevations lower than the bottom elevation of the stream is generally hydrologically undesirable. The volume of such excavations, above the elevation to which the area can be kept drained, can be considered in hydrologic storage computations.

F. Preservation of Adjacent Project Storage. The applicant will be required to respect the valley storage provided by adjacent projects by ensuring that their hydraulic connection to the river is maintained. If the project blocks the hydraulic connection of the adjacent project, then the applicant will be required to provide additional valley storage to offset the loss caused by the blockage of the hydraulic connection.

G. Special Aquatic Sites. Value-for-value replacement of special aquatic sites (i.e. wetlands, pool and riffle complexes, mud flats, etc.) impacted by non-water dependent proposals will be required.

These criteria will be used by the Corps for the express purpose of evaluating new permit applications received subsequent to the effective data. They will not be used to reevaluate any flood plain project already constructed or permitted. They apply to permit applications from public agencies as well as private sector applications. In addition to the criteria discussed above, the following guidelines will be used by my staff in evaluating permit applications:

A. Runoff. Site drainage systems should minimize potential erosion and sedimentation problems both on site and in receiving water bodies.

B. Habitat Mitigation. A standardized, habitat-based evaluation method should be used to evaluate the impacts of the applicant's proposal to fish and wildlife resources. Guidelines for the quality and quantity of mitigation are as follows:

1. Category 2 resources--habitat of high value which is scarce, or is becoming scarce in the ecoregion--no net loss of habitat value. Category 2 resources in the study area include vegetated shallows, riffle and pool complexes, and riparian forests, as well as wetlands (see above for mitigation of wetlands). A buffer strip of natural vegetation 100' feet wide on each side of the channel for main stem projects, and 50' feet for tributaries, should be maintained.
2. Category 3 resources--habitat of medium-to-high value that is relatively abundant in the ecoregion--no net loss of habitat value while minimizing the loss of the habitat type. (This means to reduce the loss of the habitat and compensate the remainder of loss of habitat value by creation or improvement of other Category 2 or 3 resources.) Category 3 resources in the study area include deep water, native rangeland, upland forests, and upland

shrubland.

3. Category 4 resources--habitat of low-to-medium value--mitigation should be to minimize the loss of habitat value, which can be accomplished by avoidance or improving other habitat types. Category 4 resources in the study area include cropland and improved pasture.

C. Cultural Resources. Cultural resources, including prehistoric and historic sites, will be identified and evaluated according to National Register of Historic Places Criteria. Identification procedures may involve literature review, pedestrian survey, and excavation to identify buried cultural materials. Sites which are eligible for inclusion in the National Register of Historic Places will be treated by measures which range from avoidance, to preservation in place, to mitigation through excavation.

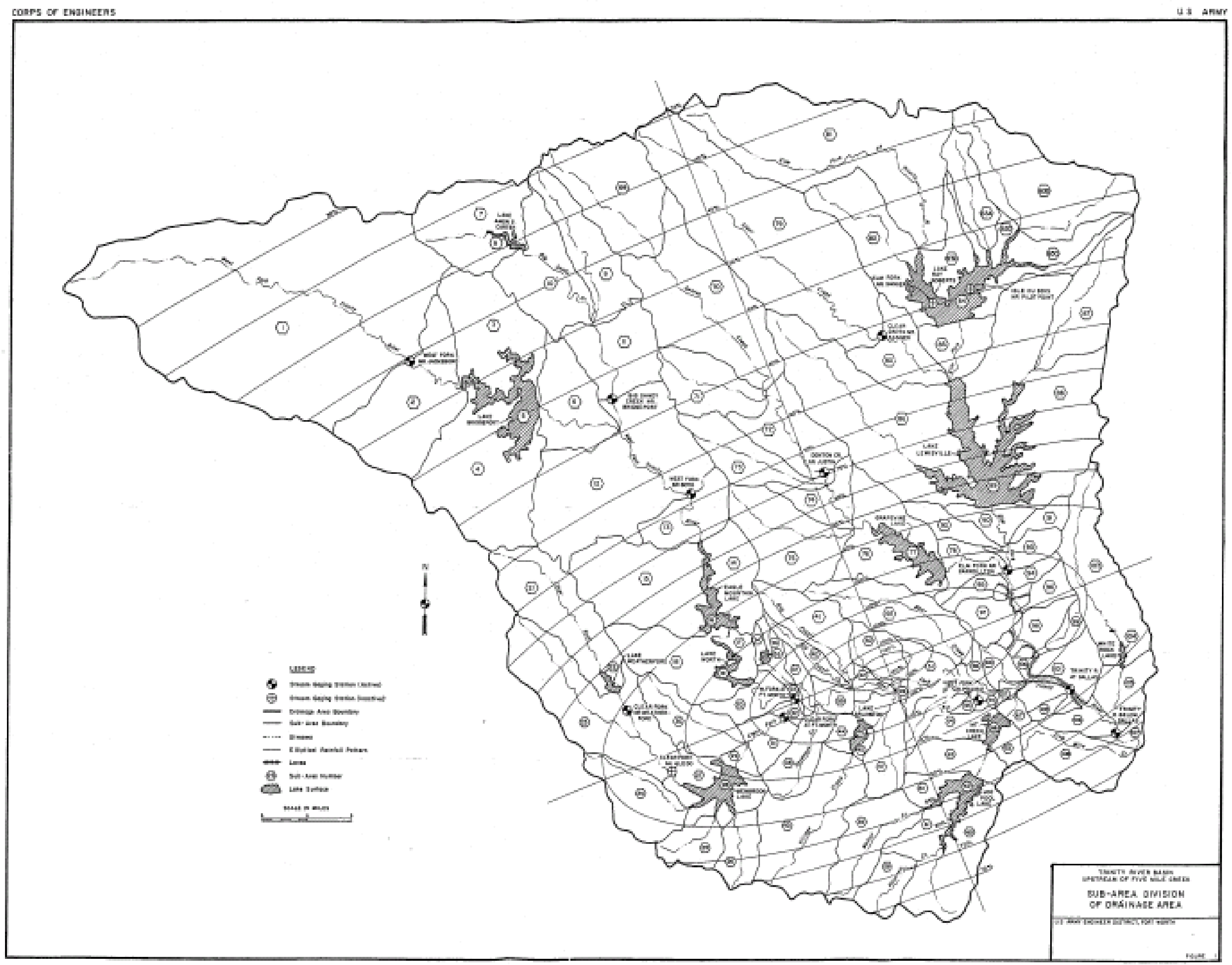
D. Other Regional Needs and Plans. Consideration will be given when evaluating permit applications of the proposal's impact on regional facilities which have been identified as important through the Regional EIS process. These include, but are not limited to, a linear hike/bike system linking large flood plain parks throughout the Metroplex, the Trinity Tollway, Master Thoroughfare Plans and sites for regional stormwater detention basins. (Specific locations and plans for these facilities will continue to evolve through coordination with NCTCOG and local governments.) Applicants will be urged to design projects which do not preclude future implementation of these regional assets.

It is my conclusion that the criteria and guidelines set forth above represent the best available definition of the "overall public interest," taking into account the rights of individual landowners and the direct, indirect, and cumulative impacts of individual actions under my purview. Further, I conclude that these policies represent all the practical means known to me to avoid or minimize environmental harm within that framework. This document will therefore provide the specific framework within which we will operate the Fort Worth District's Regulatory Program within the Regional EIS study area.

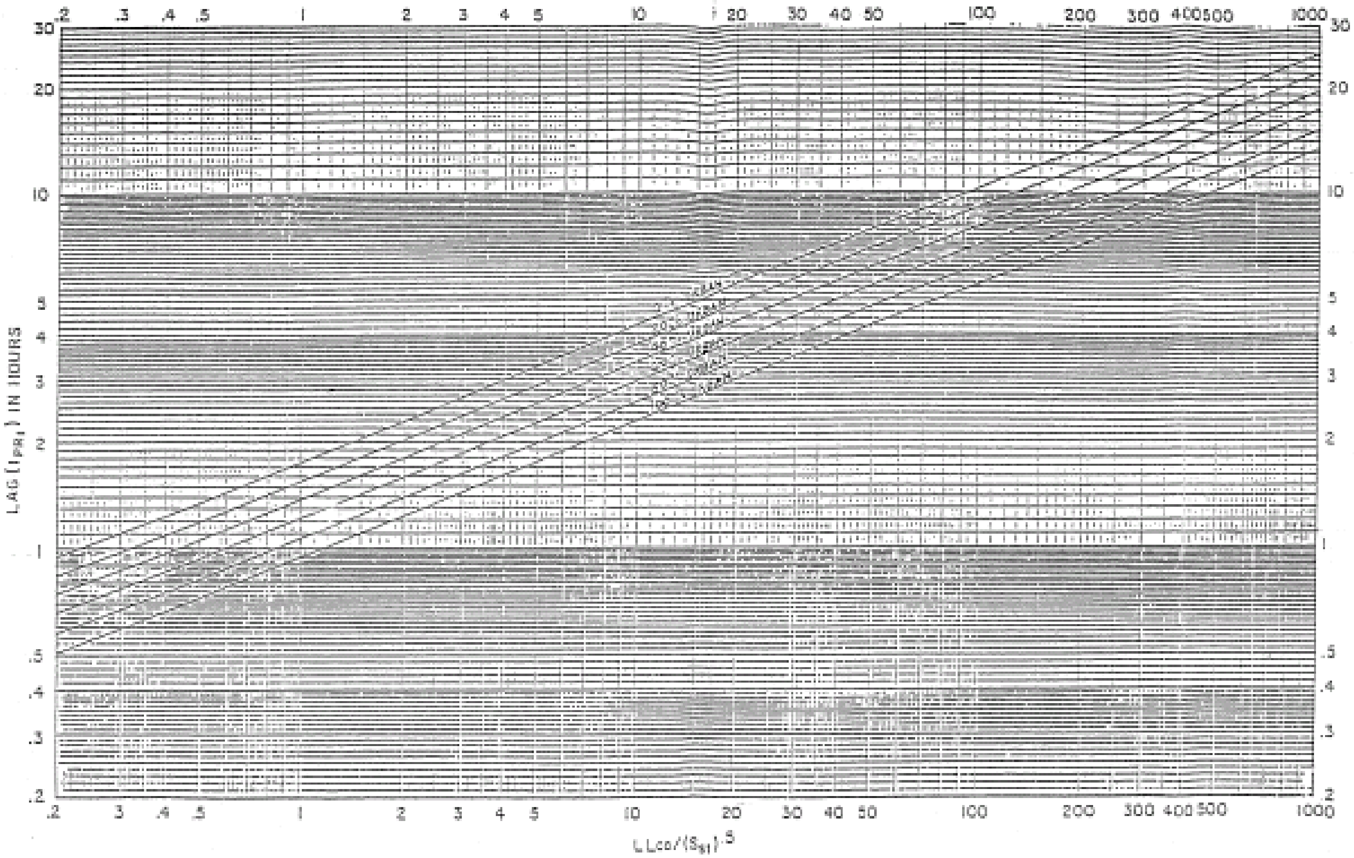
John E. Schaufelberger

JOHN E. SCHAUFELBERGER
Colonel, Corps of Engineers
District Engineer

Date: APRIL 29, 1988


CORPS OF ENGINEERS
U.S. ARMY
TRINITY RIVER BASIN
UPSTREAM OF FIVE MILE CREEK
SUB-AREA DIVISION
OF DRAINAGE AREA
U.S. ARMY ENGINEER DISTRICT, FORT WORTH
FIGURE 1
LEGEND
Stream Gaging Station (Active)
Drainage Area Boundary
Sub-Area Boundary
Streams
Elliptical Rainfall Pattern
Levee
Sub-Area Number
Lake Surface
SCALE IN MILES

CORPS OF ENGINEERS | U.S. ARMY



TRINITY RIVER BASIN UPSTREAM
OF FIVE MILE CREEK

EAST-WEST CROSS TIMBERS
URBANIZATION CURVE

U.S. ARMY ENGINEER DISTRICT, FT. WORTH

FIGURE 2

CORPS OF ENGINEERS

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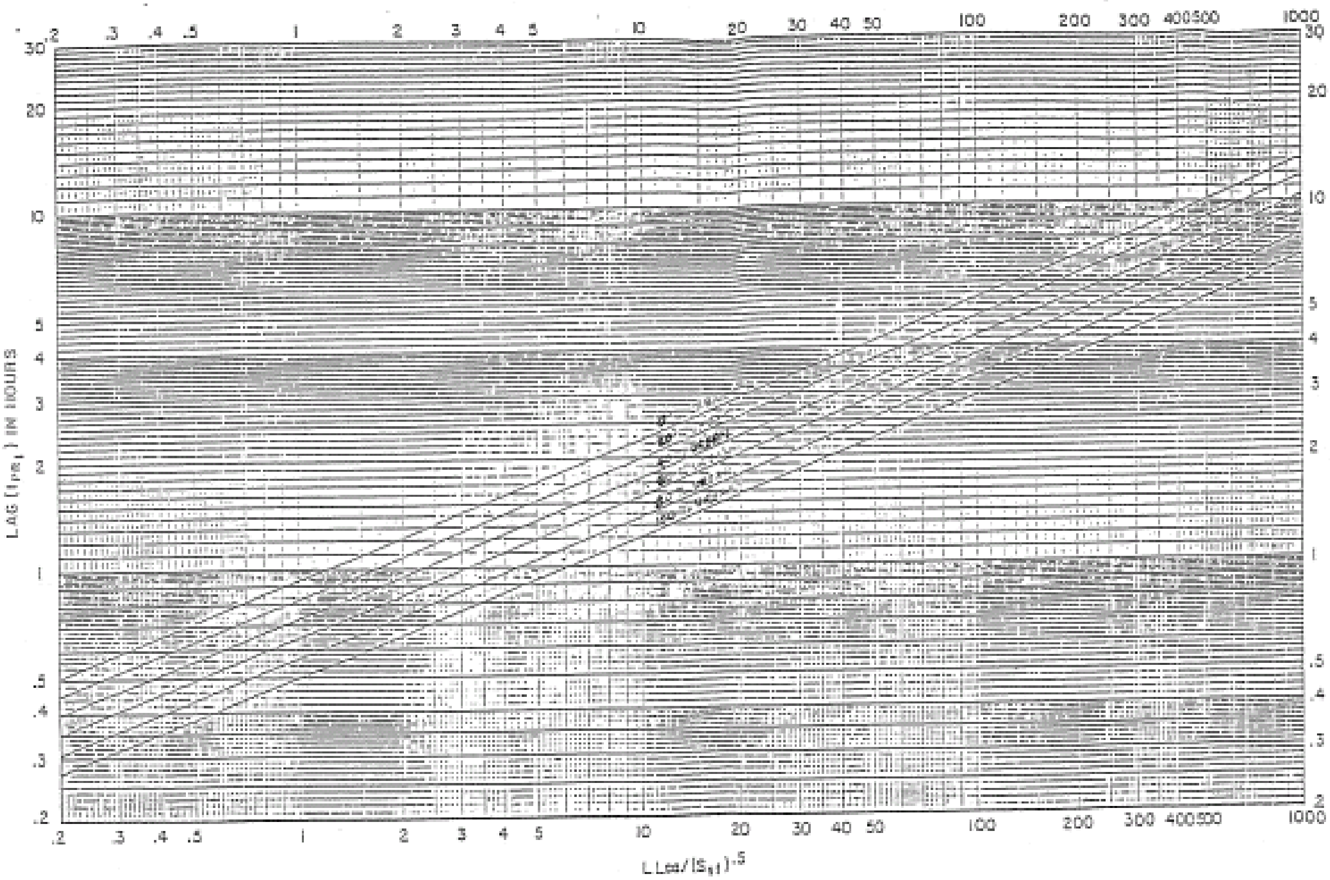

TRINITY RIVER BASIN UPSTREAM OF FIVE MILE CREEK

BLACKLAND PRAIRIE URBANIZATION CURVE

U.S. ARMY ENGINEER DISTRICT, FT. WORTH

FIGURE 3

DRAFT

CASE STUDIES
URBANIZATION PROBLEMS AND DESIGNS IN HYDROLOGIC ENGINEERING

by

Dan Harvey, P.E.[1]

INTRODUCTION

This paper discusses two quite different commercial developments under the 404 permit process, the Northwest Auburn Industrial Park and the Auburn Racetrack (figure 1), both of which involved evaluation of flood impacts on Mill Creek and the effectiveness of mitigation design features to eliminate these impacts. Mill Creek is located south of Seattle within the city limits of Auburn in King county. Mill Creek has a drainage area of about 22 sq. mi. and is a tributary to the Green River which is regulated by Howard Hanson (HAH) Dam, a Corps of Engineers project. Both of these commercial sites are situated near the lower end of Mill Creek on the upstream boundary of a major natural storage reserve for flood waters from the Green River. The Mill Creek watershed as well as others in the region are experiencing significant growth pressures and associated flood problems. As a result, the public is becoming very much aware of the potential flood consequences of poorly designed developments and becoming more vocal in demanding that these developments not cause impacts to their properties. In some cases even relatively minor flood stage increases have stymied or killed projects even in sparsely populated areas. Each of these developments cause impacts to various hydrologic regimes including infiltration, evapotranspiration, detention/wetland storage, timing of runoff, groundwater levels, etc. Interactions of these regimes are extremely complex and ultimately affect flood flows not only on a short term but also on a seasonal and long term basis, e.g., base flow during the summer. An additional major problem in many instances, particularly where projects are smaller, is how to determine the overall flood impacts resulting from development without expending huge sums of money and time, and how to develop reliable data demonstrating that these mitigation measures will be effective. Political pressures to get the project built often mean that insufficient time is available to collect adequate data and develop hydrologic models. This frequently results in analyses that are done on a site-specific basis using single event flood hydrology and simpler analytic techniques, which was the case for the developments discussed in this paper. These analyses lack sufficient detailed investigations of the longer term impacts to the entire watershed, especially the cumulative flood effects. Furthermore, governmental entities often have varying permitting authorities and differing regulations and design requirements to limit flooding. In many cases these regulations are conflicting, as when

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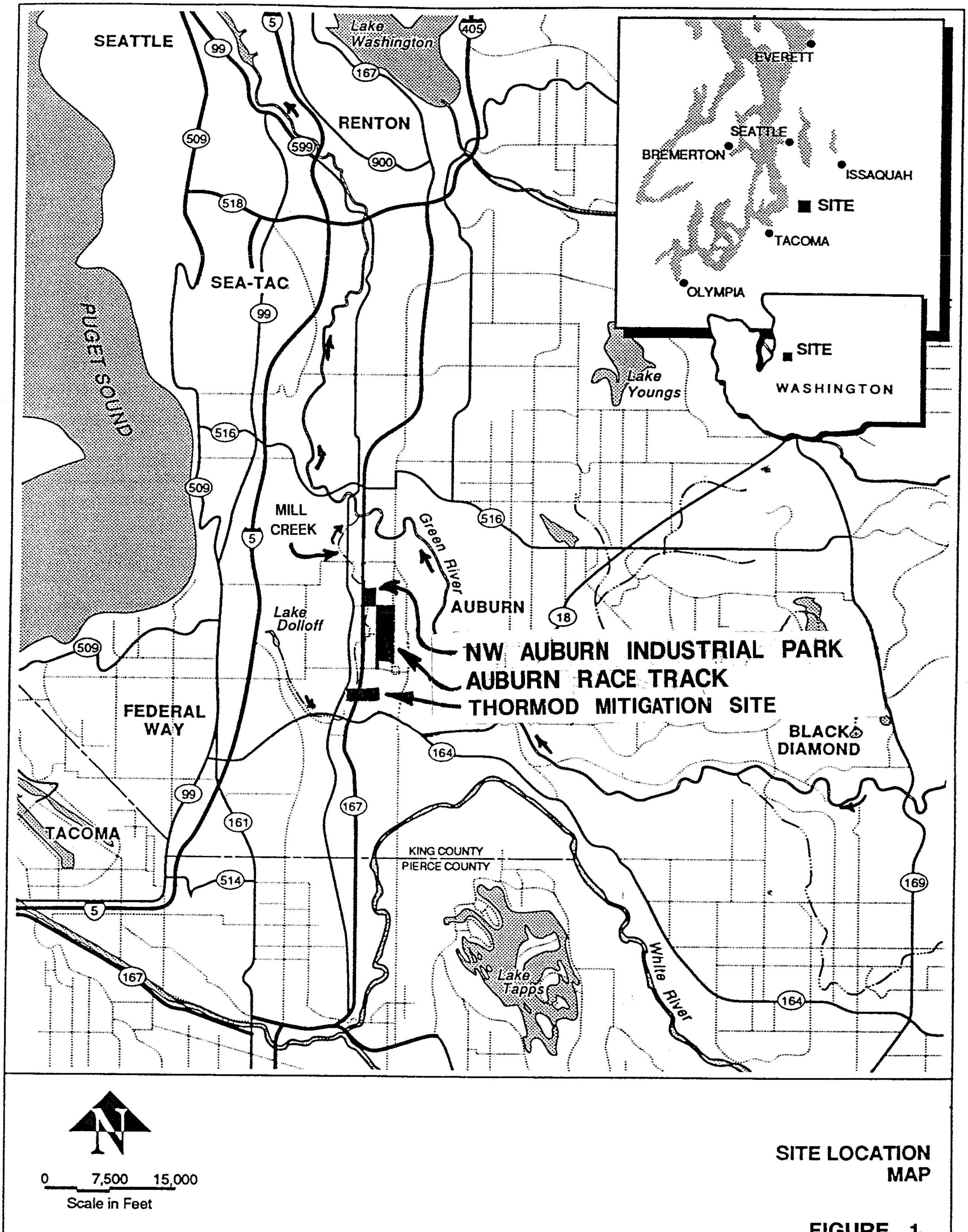

SITE LOCATION MAP

FIGURE 1

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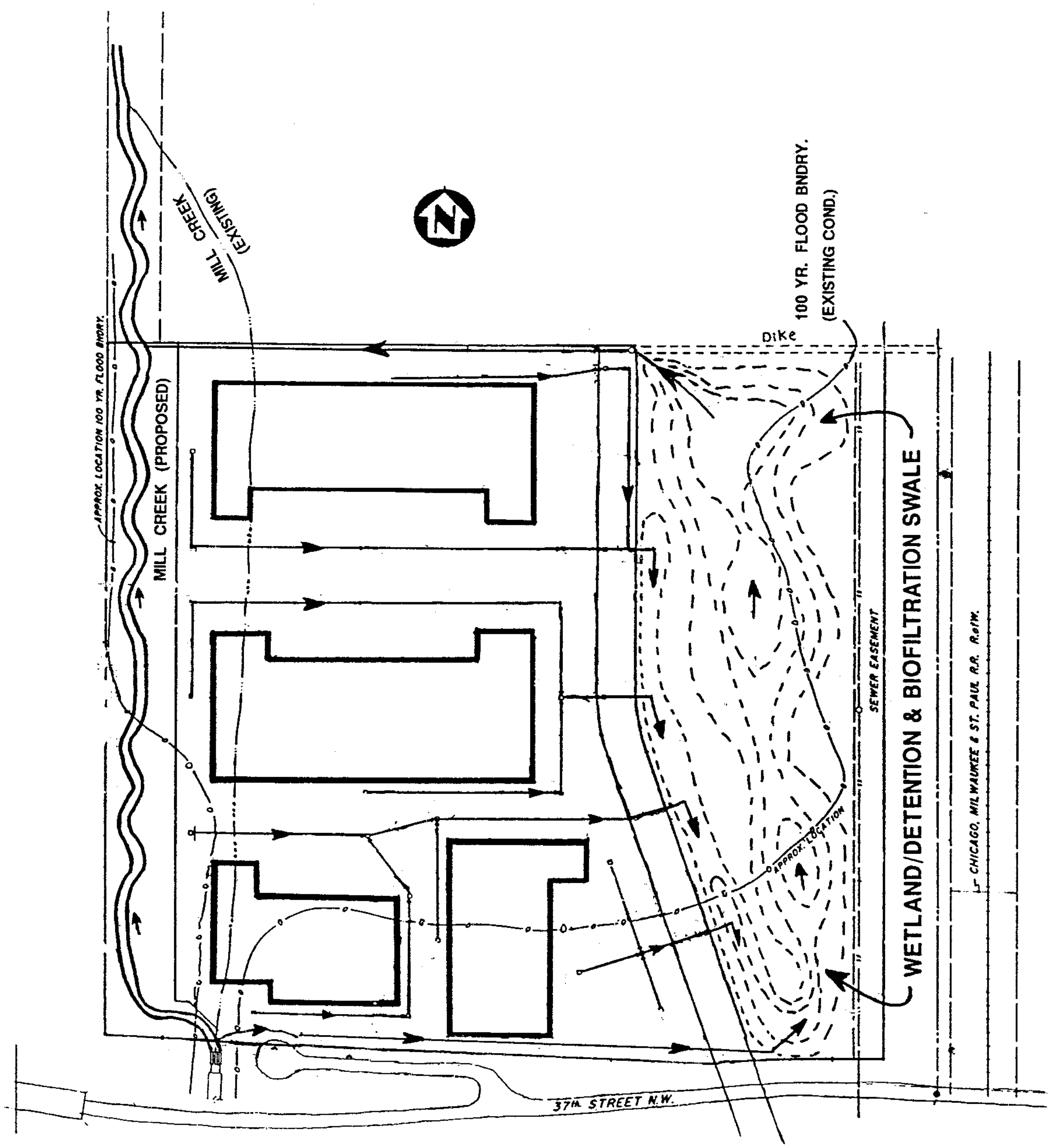
one governmental body may have a zero rise flood stage policy while the other may not. This was also the situation in both the case studies presented herein; the city of Auburn has local jurisdictional authority for issuing building permits but does not have as stringent regulations as King County, including a zero rise policy, even though King County would be impacted by Auburn's decisions.

King County Surface Water Management (KCSWM), particularly Mr. Andy LeVesque, was the primary contributor to the review efforts for these permitting actions. KCSWM provided extremely valuable insight to Mill Creek flood problems based on prior studies performed for this system.

NW AUBURN INDUSTRIAL PARK

Proposed Project Features: The NW Auburn Industrial Park development is typical of many commercial projects being built in watersheds in the region today. The NW Auburn Industrial Park is located on the right bank of Mill Creek (figure 2). The site encompasses 31 acres of undeveloped farm land which would be transformed by extensive placement of fill to provide 14.8 acres of office/warehouse space and associated parking, i.e, impervious area. About 7.44 acres of this fill would be placed in 11.26 acres of wetlands and existing stream channel. Approximately 22 acres of land would be inundated by the 100-year flood based on FEMA 1989 flood insurance maps. Mitigation for these impacts would consist of creating 12.37 acres of combined off-stream wetlands and flood storage detention, and enhancement of 0.54 acre of the relocated stream channel for wetlands and fisheries improvements. The off-stream wetland/detention area would be connected to Mill Creek by an outlet channel with drop board structure at the downstream (northern) edge of the property. Increased onsite runoff from the 25-year flood requiring water quality treatment, i.e., roofs, parking areas, etc., would be detained in an underground pipe and tank network in accordance with city of Auburn design criteria. The detained flow would be released into bio-filtration swales and oil/water separators located in the wetland/detention area before release to Mill Creek. Controlled storage within the wetland/detention area would further attenuate the 25-year onsite flow to pre-project levels.

Developer's Initial Flood Analysis: Evaluation of the hydrology for all floods was performed using the rational method to determine surface runoff and the Yrjanainen and Warren (Y&W) method for storage determination. Runoff from the site is entirely from rainfall-runoff. HEC-2 was used to compute water surface profiles on Mill Creek and flows were taken from the FEMA flood insurance study. According to the developer's initial computations, the project would displace about 18.66 acre-feet of existing flood storage for the 100-year event. Storage detention computations were performed for the wetlands/detention area to ensure that the 25-year post development onsite flood runoff was diminished to existing condition levels. It was also estimated that the project would increase onsite runoff for the 100-year, 24-hour flood from 4.65 cfs to 9.45 cfs, and increase the Mill Creek 100-year flow of 320 cfs by about 7.5 cfs. No onsite post development computations were made for the 100-year flood to evaluate flow reduction



SITE PLAN - NW AUBURN INDUSTRIAL PARK

FIGURE 2

designs or to estimate post project floodplain storage to compare against pre-existing conditions. The developer reported that the increased runoff would not present a significant flood problem since channel changes resulted in a net reduction in the water surface profile and floodplain limits within, and upstream of, the project reach; and downstream impacts would not be significant because increased runoff (7.5 cfs) only raised the downstream water surface elevation by 0.07 foot. No determination of the discharge increase on Mill Creek resulting from the lost floodplain storage was made.

Corps Flood Evaluation: The developer's flood computations were site specific in accordance with city of Auburn design requirements but did not evaluate flood impacts upstream and downstream from changes in onsite runoff and Mill Creek channel changes. They also did not demonstrate that mitigation features would result in runoff conditions that would produce negligible localized or cumulative downstream impacts in accordance with 404 permit requirements and Executive Order 11988 (Floodplain Management) for the 100-year event. In reviewing this permit, the Corps applied relatively strict flood reduction requirements due to the lack of engineering data to show basin impacts.

Review by COE (and KCSWM) of storage loss and compensation figures were made by planimetering contours since detailed data was not attainable from the applicant or other sources. Several problems were detected: adequate onsite storage was not available to attenuate the increased onsite peak runoff from the 100-year event; reduction in water surface elevations on Mill Creek for the 100-year flood, due to widening of the relocated Mill Creek channel and onsite fill material, had produced a pronounced loss in onsite and upstream flood storage of about 11 acre-feet and 40 acre-feet (roughly estimated by KCSWM), respectively. It was suggested to the developer that storage losses could be eliminated by several means including raising the water surface profile on Mill Creek to pre-project levels to eliminate upstream storage losses, and additional detention storage to reduce onsite runoff. These measures would reduce the potential for significant downstream cumulative impacts. Raising the water surface profile could be accomplished by either constricting the Mill Creek culvert on the upstream side of the property and/or reconstructing, i.e., constricting, the Mill Creek channel. The developer acted to correct these problems by adding onsite storage and reconfiguring the Mill Creek channel to raise the water surface profile.

A subsequent check of this design by COE and KCSWM, however, revealed that post project onsite flood storage was still deficient due to use of a sloping water level in the wetland/detention area projected from Mill Creek. Since, according to the developer's design drawings, this area is only backwatered from Mill Creek for floods greater than 25-years through the north (downstream) outlet channel, as discussed earlier, the water level in the wetland/detention area should be a level-pool value, i.e., no appreciable gradient, equal to the elevation of Mill Creek at the confluence with the detention pond outlet. The developer was informed that lack of adequate compensatory onsite flood storage would increase flows downstream. The cumulative effects of this increase would most likely be significant if similar projects were allowed to be built to the same standards.

An additional problem in flood storage determination was also detected due to inclusion of storage below groundwater levels in the relocated Mill Creek channel.

Recommendations: The following recommendations were made by the Corps and accepted by the developer in order to reach compliance with regulatory requirements:

1) To mitigate for the 100-year onsite flood storage loss several design changes were suggested. Construct an overflow channel and control structure on the south (upstream) side of the property to convey high flows greater than 25-years from Mill Creek to the wetland/detention area. This would hydraulically link Mill Creek with the wetland/detention area providing a continuous flowing system. Constrict the flow at the north (downstream) end of the detention area with a culvert to pond the 100-year flood to the level of Mill Creek at the upstream confluence. Prepare performance curves for the inlet and outlet culverts in the wetland/detention area and perform storage routings to confirm that the system functions as designed.

2) Build the project to the contours on the grading plan to ensure that flood storage is provided in accordance with the HEC-2 cross-sections and computations.

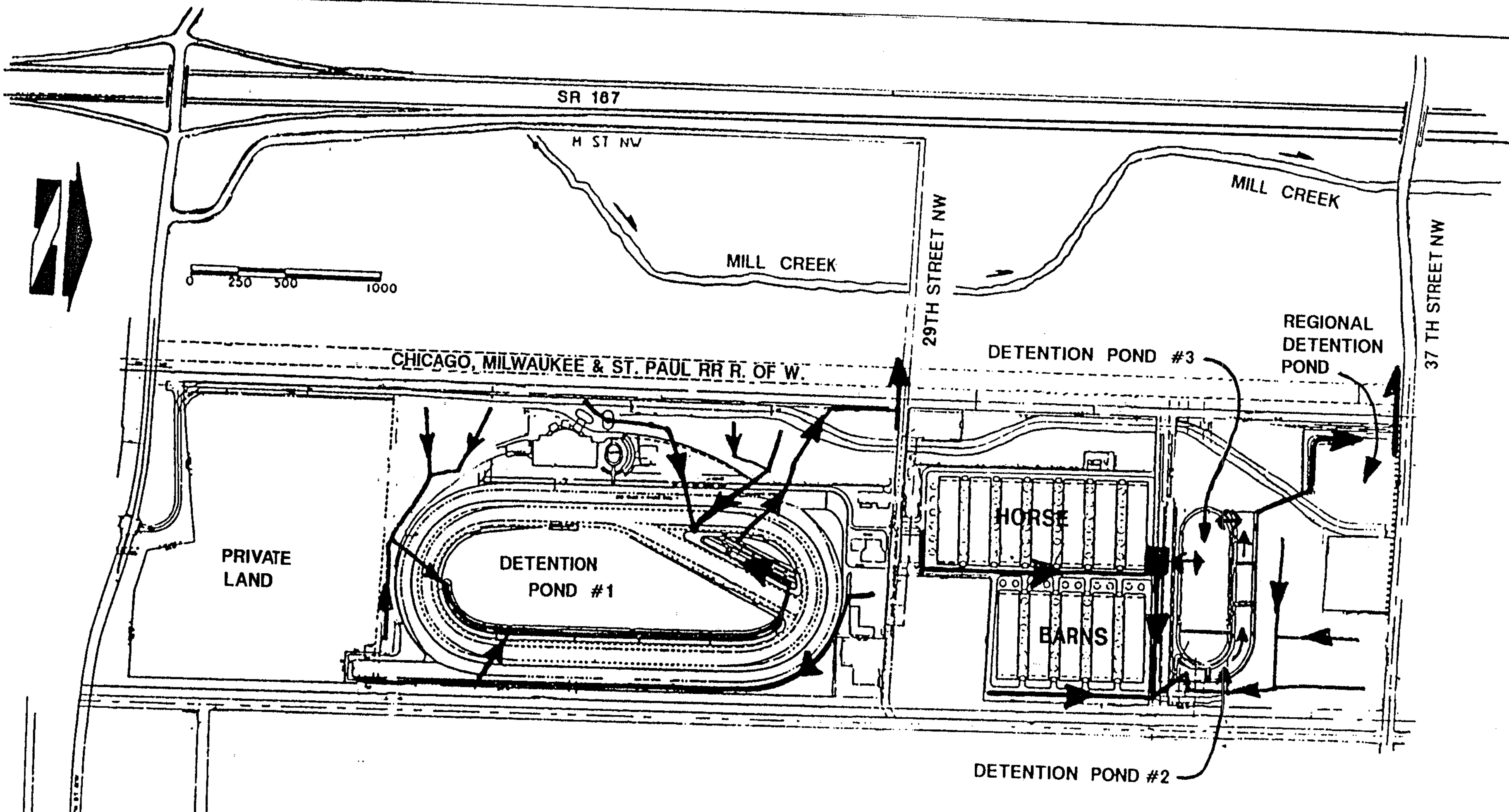
3). Redesign the Mill Creek channel to raise water surface elevations to the existing condition level in agreement with FEMA flood insurance study results.

Status: The developer accomplished the analysis and design changes recommended in paragraph (1) above prior to this permit being granted. Final onsite existing and developed condition 100-year runoff values to the detention pond for storage routing analysis were computed to be 5.88 cfs and 11.35 cfs, respectively, with wetland/detention area outflow essentially equal to the existing condition runoff of 5.88 cfs.

The aforementioned recommendations ultimately became conditions of the 404 permit.

AUBURN RACETRACK

Proposed Project Features: This project is still in the review stages. A draft EIS was distributed for public review in July 1994. The Auburn Racetrack is located about 600 feet from the right bank of Mill Creek (figure 3). The site encompasses about 168 acres of undeveloped farm land which would be transformed by extensive placement of fill to provide horse barns, parking, a thoroughbred horse track and related structures. Much of the land south of 29th Street had been previously filled to a depth of about 2 to 3 feet but would require an additional 3 feet of structural fill on average. The area north of 29th Street would be filled to a depth of 3 to 5 feet. Nearly 91.5 acres of the site would become impervious. About 17.4 acres of new fill would be placed in 20.7 acres of onsite wetlands and depression storage. Only about 1.5 acres of the site are situated in the 100-year floodplain of Mill Creek based on FEMA 1989 flood insurance maps.



SITE PLAN - AUBURN THOROUGHBRED RACE TRACK

FIGURE 3

Principal mitigation for wetland and other flood related impacts on the racetrack property would consist primarily of: (1) offsite enhancement of the 56.5-acre Thormod site (located about 0.75 mile upstream) by expansion of existing wetlands and raprian habitat, and by providing additional flood storage; (2) managing 3.5 acres of existing onsite wetlands in a 5-acre regional detention facility for the city of Auburn (located in the northwest corner of the racetrack property); and (3) providing three onsite vegetated flood storage detention ponds to control increased surface runoff with grass lined bio-filtration swales and oil/water separators for water quality treatment. This paper will primarily discuss the more complicated flood control features related to the racetrack property rather than that of the offsite Thormod property.

The three racetrack detention ponds encompass a large segment of the site. Detention pond No. 1, drainage area 89.7 acres, is located within the racetrack on the south side of 29th Street; detention ponds No. 2 and 3, drainage areas 67.4 acres and 10.5 acres, respectively, are located on the north side of 29th Street. Detention ponds No. 1 and 2 would drain to Mill Creek through storm drain systems at 29th and 37th Streets, respectively. Because of hydraulic capacity problems with the 37th Street system, detention pond No. 2 would first drain to the regional detention site before entering the 37th Street system. Detention system No. 3 would provide long term storage and treatment of runoff from the horse barn area and would only release to Mill Creek via an interconnection with detention pond No. 2 during the off-season period from November through January. During the racing season, February through October, detention pond No. 3 would store the runoff and use it during the summer to augment irrigation for the racetrack infield. Emergency releases from detention pond No. 3 would be to the METRO sewage treatment plant. All three detention ponds would be sized to provide water quality treatment for floods up to the 25-year event

.

Because of known downstream flood problems along Mill Creek, very restrictive release rates from the detention ponds were set by the city of Auburn. Multi-orifice control structures would release stored flows to the bio-filtration swales and storm drain systems at 50 percent of the 2-year, 100 percent of the 10-, 25-, and 100-year pre-project flow rates from the low, middle, and high orifices, respectively. Maximum detention storage for flood control would be sized to control the 100-year, 7-day event to pre-project levels.

Developer's Initial Flood Analysis: Hydrology for all floods evaluated for the racetrack was developed using city of Auburn criteria. Since runoff from the racetrack site is entirely from rainfall-runoff, the developer selected the Santa Barbara unit hydrograph (SBUH) method to determine this runoff, which was also a Washington State Department of Fisheries permitting criteria. The Y&W method was used for storage detention determination for these floods with a 1.2 safety factor applied based on city of Auburn requirement due to size of the project and known downstream flood problems on Mill Creek. Level pool routing was used on each detention pond to compute system outflows. The King County model, Backwater Program, was used to compute pipe flow and hydraulic grade lines for the pond routings. Results of the analysis showed that the ponds would function as proposed, i.e., none of the detention ponds would overflow for the 100-year event.

Because of the limited space available to construct detention pond No. 2, flood storage was determined based on controlling the 100-year, 24-hr event and not the 100-year, 7-day event. To offset this, detention pond No. 1 storage was increased and release rates reduced from that of the other ponds to provide an average 100-year 7-day flood reduction capability for the combined No. 1 and 2 systems, i.e., the sum of outflows from these two ponds would be equal to the 100-year 7-day flow. Computed winter peak runoff and release values for each detention pond for pertinent flood events are listed below:

	Detention Ponds		
	No. 1	No. 2	No. 3
		100-Year, 24-hr Event	
existing inflow	11.3 cfs	8.7 cfs	1.8 cfs
developed inflow	64.6 cfs	46.1 cfs	6.3 cfs
developed outflow	6.9 cfs	11.0 cfs	1.8 cfs
detained storage	19.8 ac-ft	9.2 ac-ft	1.2 ac-ft
		100-Year, 7-Day Event	
existing inflow	12.6 cfs	9.6 cfs	1.9 cfs
developed inflow	37.2 cfs	27.4 cfs	3.9 cfs
developed outflow	9.2 cfs	13.0 cfs	1.94 cfs
detained storage	22.7 ac-ft	10.9 ac-ft	1.5 ac-ft

The regional detention area would have additional flood storage capacity of about 13 acre-feet. The combined flood storage for the three detention ponds of approximately 35 acre-feet is greater, according to the developer's report, than that required using design criteria by the city of Auburn (9.5 acre-feet), or the WSDF (between 19.5 and 23.0 acre-feet).

Groundwater data was used in the detention storage calculations to assess ineffective/dead storage elevations. Groundwater field measurements were made at various times during the period September 1992 through April 1993. Generally, groundwater depths below the ground surface were observed to be about 4 to 8.7 feet in early fall, 1.2 to 5.3 feet in late fall, and 0 to 3.3 feet in the spring of 1993. These depths may not represent typical conditons, however, since precipitation for the year was below normal.

Estimated lost floodplain storage on Mill Creek at the racetrack was about 0.2 acre-foot for the 100-year event, and lost depression storage was estimated at 3.8 acre-feet. Flood storage evaluation at the Thormod mitigation site was limited to insuring that sufficient storage volume was provided to more than offset lost flood storage on Mill Creek and onsite depression storage caused by the racetrack. No detailed routings to evaluate floodplain storage changes at the Thormod site that would affect Mill Creek were performed.

HEC-2 was used by the developer to compute water surface profiles on Mill Creek with flows taken from the FEMA flood insurance study (1989). The purpose of the analysis was to evaluate potential impacts on Mill Creek and the racetrack site relative to changes in peak flows, water surface elevations, and velocities due to the racetrack project. No adjustments were made in the Mill Creek flows for what was assumed to be small changes in runoff from the racetrack and Thormod sites. The only change in water surface profiles occurred at the Thormod site due to channel modifications proposed for this reach. Here, the water surface elevation was lower by about 1.1 feet at the upstream end and higher by about 0.14 foot at the downstream end of the site (KCSWM).

Corps Flood Evaluation: In reviewing this permit for hydrologic concerns, the Corps used strict requirements for flood reduction due to the lack of an overall basin analysis. As with the Auburn Industrial Park, the developer's flood computations were site specific in accordance with city of Auburn design requirements but did not demonstrate that flood flow impacts upstream and downstream for the 100-year event would not result from development or mitigation measures in accord with 404 and EO 11988 requirements. This was especially the case for the Thormod site where lower backwater profiles could significantly reduce upstream floodplain storage availability and result in increased runoff downstream.

The developer's initial onsite runoff calculations using the SBUH method were determined to misrepresent runoff. A comparison of the SBUH against hydrographs produced by the detailed Hydrologic Simulation Program-FORTRAN model (HSPF) for a 50-year event on a representative 10 sq.mi. basin (figure 4) indicates that the SBUH produces higher, more concentrated runoff which is not characteristic of runoff from sites similar to the racetrack. HSPF hydrographs would be much flatter with a longer recession time. Overall runoff volumes (excluding baseflow) for the SBUH and HSPF hydrographs appeared to be similar but the difference in magnitude and volumes around the peaks could affect the detention storage/discharge routing balance and produce higher runoff to Mill Creek than expected particularly if the outlet structures are sized too large.

KCSWM was very concerned that increased flow rates and volumes on the recession side of the racetrack runoff hydrographs from detention pond releases would affect the Green River natural storage detention area at the mouth of Mill Creek and ultimately add to Green River flood levels and problems. The Green River is generally at high stages for long durations during flood events, about 9 days for a 100-year event, due to flood regulation of HAH dam. Therefore, increased onsite runoff on the recession side

HYDROGRAPH PLOT

(50 YR = 24 HR STORM)

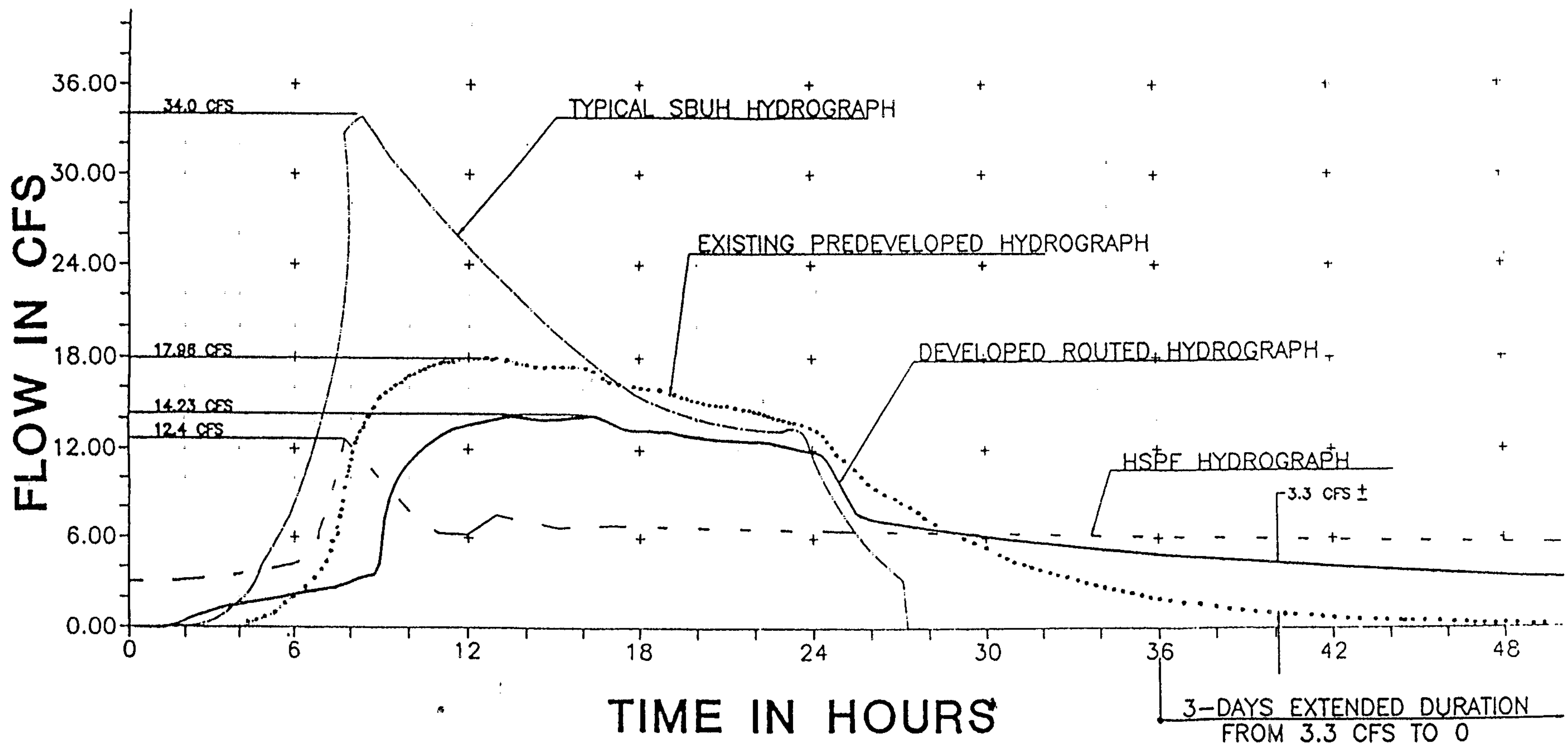


FIGURE 4

of the Mill Creek hydrographs may coincide with high Green River flows and cause stage increases on the Green River particularily if there are cumulative affects from other future developments. These increases, if significant, may have to be compensated for by either reducing the HAH outflow, which would reduce the overall protection level, by reducing the interior pumping along the Green River, thereby increasing interior flooding, or some other means. No routing or flow timing data were available to evaluate these effects.

A problem was also noted with the amount of storage in the detention ponds for flood control. It appeared that groundwater levels and dead storage were set too low based on field observations by Corps personnel. Winter, groundwater levels where the land had not been previously filled was near at the ground surface at many locations.

No cumulative impacts analysis was performed for the Auburn racetrack site to investigate the impacts from proposed new development in the basin at other sites with similar flood control design/mitigation features.

Recommendations: The following recommendations have been or may be provided to the developer during the course of this permitting process to ensure that urban hydrology has been adequately investigated:

1) Evaluate groundwater table levels and groundwater (seepage) flow through the ponds in the winter.

2) Reevaluate flood storage in the detention ponds to consider new groundwater data and variations in SBUH and HSPF hydrographs to ensure proper functioning of ponds. Special attention should be paid to detention pond No. 3 which may not have sufficient storage to control the 100-year event with a higher groundwater level. It was suggested to the developer that, since King County and city of Auburn had just completed cooperative development of an HSPF model for Mill Creek, it might be used to help address onsite runoff and impacts to the Mill Creek system.

3) Review flood storage requirements for February through April (or later) especially for detention pond No. 3 which would begin to fill in February from retained horse barn runoff.

4) Perform flood routings on Mill Creek to evaluate floodplain storage changes, flow timing and flow rate differences, and downstream impacts on the Green River natural flood detention area. Determine mitigation measures needed.

5) Perform a continuous water surface profile analysis with HEC-2 and include onsite runoff changes, including the Thormod site, to provide a continuous impact analysis of floodplain changes.

6) Evaluate cumulative impacts of any changes in flow and flood conditions on Mill Creek and Green River and determine mitigation measures.

7) Develop a monitoring program and performance standards that will ensure detention ponds function as designed. Provide alternative operating or contingency plans and design changes that would be implemented to assure proper functioning of the detention systems if changes to the storage/release features are needed. It was suggested by KCSWM that the monitoring program might be patterned after that developed by the Department of Ecology.

Status: Some of the recommendations listed above for determining Mill Creek impacts have been addressed by the developer; these include:

1) The developer acted to adjust the SBUH hydrographs to more closely represent an HSPF type hydrograph by applying adjustment factors (figure 4). Storage routings were rerun and designs reevaluated. The adjustments were only partially effective, therefore, the previous comment with regard to use of the SBUH method stands. (King County is currently contracting to have an HSPF analysis performed for the racetrack site.)

2) The HEC-2 model was revised to extend from the mouth of Mill Creek to the upstream end of the Thormod site, a distance of about 6 miles. Flow information on Mill Creek for the 2-year through 100-year, 24-hr events was updated from the 1989 FEMA study based on results of a continuous rainfall-runoff simulation study using the HSPF model performed for the city of Auburn and King County in October 1993. Generally, flows on Mill Creek for developed conditions show a slight increase in the peak due to an extended duration of peak flows from the racetrack detention ponds. Results of the HEC-2 analysis showed that for the developed condition flow increases produced only minor changes in the water surface elevations and velocities on Mill Creek. At the Thormod site, however, more significant changes occurred due to channel modifications. Here, although flows did not changed, velocities decreased from 2.25 fps to 0.9 fps at a mid-point location and increased from 4.83 fps to 5.49 fps at the upstream end; elevations for the same points were 0.14 ft. higher and 0.31 ft lower, respectively. The developer reported that partial mitigation for flood storage loss upstream of the Thormod site as a result of these changes would be made by increasing the Thormod site excavation.

3) A cumulative impacts analysis was performed for proposed future projects in the Mill Creek basin by increasing flow rates at various designated points along Mill Creek. Results of this analysis generally showed an increase in peak flow rates with a relatively minimal increases in velocities and water surface elevations.

4) A preliminary reevaluation of the groundwater table during the winter indicated that higher than anticipated water levels during this period would not affect detention pond No. 1 but could impact to some extent flood storage in detention ponds No. 1 and 2. The use of impervious pond liners where groundwater is of concern will be considered to alleviate this problem.

5) King County is currently developing an unsteady flow model for the confluence area of Mill Creek and the Green River that they might use to analyze impacts of increased Mill Creek recession flows on the Green River.

SUMMARY

The preceding two case studies for the Northwest Auburn Industrial Park and the Auburn Racetrack, demonstrate some of the complexities involved in urban flood analyses and in designing effective mitigation features. These studies point to the need for better procedures, guidelines, models, and consistent regulations and design standards to perform these investigations including evaluation of cumulative downstream impacts. New models should be flexible enough to allow input of non-standard designs similar to capability of Object Oriented Programming, "OOP", type models. These tools are needed not only for governmental agencies involved in regulating actions but also by private engineering firms so that reanalysis is minimized and contentiousness diminished as projects proceed through the review and permitting process.

An encouraging step in the direction of standardizing and streamlining the permitting process is being pursued by the Washington Department of Ecology where landowners and project proponents can apply for various needed permits simultaneously. Only one permit would be required. The consolidated form would cover activities regulated under state and local shoreline management programs, the state short-term water quality standards, and local floodplain management ordinances. The form will also include information for the Corps section 10 and Section 404 permits together with hydraulic project proposals issued by the state fishery and wildlife departments. If a six month pilot test is successful, statewide implementation will begin in 1995.

Also, King and other counties appear to be recommending continuous simulation modelling in some cases to evaluate runoff and development impacts. The principal model recommended is HSPF. King County design standards now require 100-year flood evaluation for any new construction. This appears to be becoming the regional design standard.

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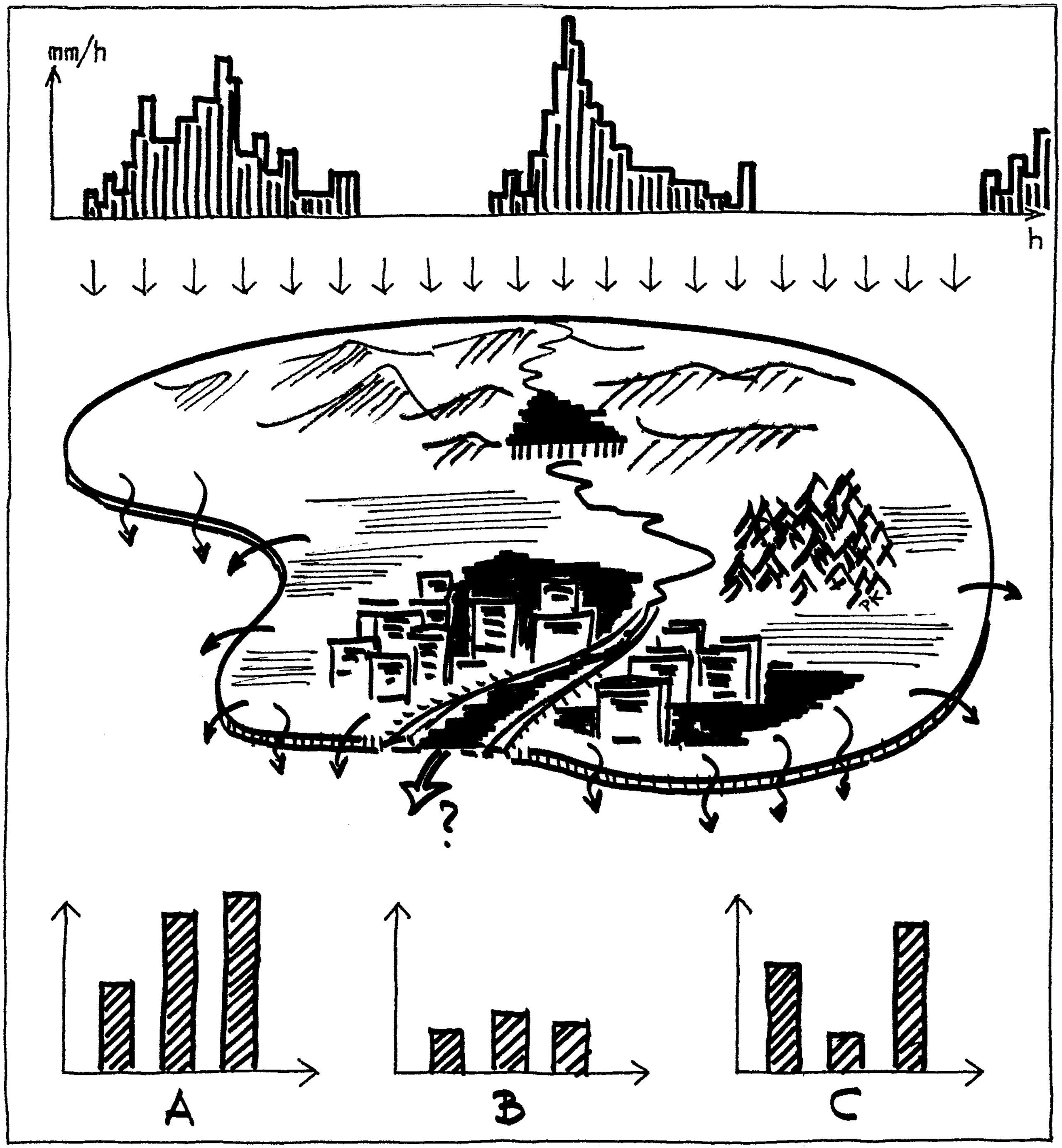
Session 2:

Hydrology



SUMMARY OF SESSION 2: **Hydrology**

Overview

This session included six papers on issues related to rainfall-runoff modeling. Discussion also occurred on issues related to modeling approaches. The papers address topics from rainfall to overland flow, and from the rational method to distributed, continuous simulation.

Paper Presentations

Paper 2. Richard McCuen, Professor, Department of Civil Engineering, University of Maryland, presented a paper titled "Implications of Variations in Small Watershed Modeling to Engineering Design." His emphasis was on the sources of variation in hydrologic modeling. His goal was to bring out the major sources of such variation to aid the modeler in understanding the inputs into a model, as well as their impact on the results. Sources discussed were: within-model, storm-to-storm, watershed-to-watershed, and location-to-location variation. These were discussed in the context of several model inputs, highlighting the peak rate factor of the SCS method, concluding that its estimation should be given more attention. Time of concentration is also discussed. He finds that: 1) the overland sheet flow component of a travel time derived TC is an important component, and 2) that the hard and fast rule of an upper limit for assuming overland sheet flow is not good to use. He, instead, proposed using criteria which account for physical characteristics. He quoted past efforts to develop criteria, assessed some of them statistically, and recommended a couple of options. The criteria which he suggests implies that the limit is dependent on slope and roughness; steep, smooth surfaces can support sheet flow for a longer distance than rough, flat surfaces. He concluded that pursuing more complex models should receive less emphasis when so much is yet to be understood about the inputs to existing models.

Paper 3. David Hartley, Hydrologist, King County Surface Water Management Division, presented a paper titled "Role of Hydrologic Modeling within an Interdisciplinary Approach to Urban Watershed Planning and Management - Experiences from King County." He described the conditions under which hydrology is carried out in King County. In particular, expanding urbanization has occurred such that aquatic resources need to be protected, and flood control implemented. Because of the wide range of impacts to rivers that King County manages, continuous simulation is used. The need to do analysis stems from issues such as the need to consider the entire salmonoid life cycle in relation to stream flow. HSPF is used by King County for several reasons, including the fact that it is used by most counties and cities in the area. David gave an overview of using HSPF, including an idea of the philosophy needed when calibrating. He presented this overview by stepping through a typical study in King county, illustrating the types of information required and why continuous simulation is needed. An important portion of the information needed for HSPF is related to the fundamental hydrologic response units, or PERLNDS. Regional default PERLNDS parameters have been developed by the USGS. This is being used by King County to develop a database of unit area discharges for use by land developers.

Paper 4. Pete Hall, Associate Engineer, Sacramento County Department of Public Works, presented a paper titled "Development of Hydrologic Standards in Sacramento County Using HEC-1." Pete described his efforts to include HEC-1 in his modeling approach without discarding all the past work and success of their rational method and regional equations. A preprocessor was developed for HEC-1 to facilitate the model development process for entities working within Sacramento County. Some of its features include: 1) lag computable from travel time method or Snyder's equations, 2) short and long storm capabilities, and 3) a lag/frequency adjustment factor to account for the delay imparted at the inlet to closed conduit systems during larger events. The results from this new approach worked well for larger events, but didn't agree with those from the old method for smaller events. Since the old method has proved accurate over time, the new method was further scrutinized. Pete's team carried out an investigation at this point in which they calibrated their HEC-1 method with existing flow gage data, as well as with data from temporarily placed flow gages. For the rage of basin areas looked at, consistent results were obtainable with a single set of "n" values (tabulated in the paper) for use in Snyder's equation. Adjustments were also decided upon for cases where travel time is used to derive lag. The relationship between lag and TC was changed from lag=.7TC to lag=TC, and the upper limit for the overland portion of the travel time was modified to be dependent on the slope and roughness. With these adjustments, a range of basins can be modeled using different methods with agreement between them.

Paper 5. Joe DeVries, Department of Civil and Environmental Engineering, University of California, Davis, presented a paper titled "Role of Design Storms in Urban Hydrology." Joe described some of the issues that should be in the forefront when choosing a design storm for urban analysis. One item is the assumption that the percent chance exceedance of the rainfall equals that of the flow. He pointed out that this is less true for smaller storms because of the variance in antecedent conditions, i.e. twenty five year rain can cause either a five year flow or a forty year flow. For larger events, though, the antecedent conditions are less varied (consistently wet) so that a 100 year rain causes a 100 year flow. Another issue is the result of the common practice today of having large complex watershed models containing very small subbasins. The small subbasins require small time intervals and rainfall definition. However, when aggregated into a large basin model, the short duration intensity of the rainfall is inappropriate. Therefore, Joe recommends that as the size of watershed assemblages gets larger, hyetograph increments should be made progressively larger. Finally, Joe mentioned that more complex models may increase uncertainty of model predictions. As an illustration of the difficult task of developing a design storm for an urban area, efforts for San Joaquin County were described. Of note is the existence of strong correlation between mean annual precipitation and average maximum 24-hour rainfall. This fact was used to develop intensity/duration/frequency relations for which the independent variable is the mean annual precipitation.

Paper 6. Bill James, Professor of Environmental and Water Resources Engineering, University of Guelph, Ontario, Canada, presented a paper titled "On Reasons why Traditional Single-Valued, Single-Event Hydrology (Typical Design Storm Methodology) has become Simple-Minded, Dishonest and Unethical." Bill ultimately argued that continuous simulations is superior, based on a technical evaluation of the uses for which it is ideal, as well as its ability to accomplish these uses. However, he first established why the extra questions that it answers should be addressed at all, and perhaps more importantly, that ignoring them is unethical - not just bad modeling. With this stated, Bill went on to describe in detail why continuous simulation works where event modeling does not. He began with listing the crucial issues for aquatic health: They are: 1) Regulation of flows, 2) sediment and sediment-attached pollutants, and 3) thermal enrichment over blacktop. In none of these cases, Bill argued, can event modeling address the key issues for the simple reason they are dependent upon more that a single event. During discussions that followed, several participants debated the point that design event modeling is never appropriate.

Paper 7. Allen Lumb, Chief, Hydrologic Analysis Support Section, U.S. Geological Survey, presented a paper titled "U.S. Geological Survey Urban Studies, Methods and Data Considerations." The paper addressed both regional regression and watershed modeling. A recent effort of the USGS is a compilation of current regional regression equations into a computer program called the National Flood Frequency Program (NFF). It can be used directly for selected urban areas. For other areas, additional equations and information, such as a development factor, are required. The method also includes capability to generate a full hydrograph. In the watershed modeling arena, Allen described the USGS role in three models, DRSM, HSPF, and RRM. He supports continuous simulation as a major criteria for modeling in urban areas, and also believes that distributed models are more capable of reflecting land use changes. He summarized efforts to improve modeling through model control programs, incorporation of central databases, and the use of GIS. He finished with a description of GENSCN, a shell to facilitate decision making process which facilitates evaluation of scenarios and presentation of results.

IMPLICATIONS OF VARIATIONS IN SMALL WATERSHED MODELING TO ENGINEERING DESIGN

Richard H. McCuen[1]

INTRODUCTION

Urban and suburban development usually proceeds at a piecemeal rate. Over the decades, political jurisdictions have given little thought to the cumulative effects of development, except in planned communities. Zoning laws are changed regularly, often to meet the economic needs of the jurisdiction. This pattern, or lack of one, has had hydrological consequences.

Where land use change has not been coordinated, drainage design is controlled by conditions at the site of development. While stormwater regulations are being improved, on-site drainage control was, in the past, the only consideration. Since much of the urban and suburban development has been taken on by independent developers, development has traditionally involved small parcels of land. Thus, suburban drainage control has been intended for small watersheds.

This pattern of development has been partially responsible for the evolution of hydrologic models intended for use in the design and evaluation of small urban watersheds. Since design requirements for urban/suburban development have been intended to control local flooding, the focus of design models has been towards the small watershed. Thus, models such as the Rational method and the SCS TR-55 Graphical method (SCS, 1986) have been widely used. Only when we recognized that the spatial hydrologic interactions were important did spatially segmented models such as HEC-1 and TR-20 receive wider usage for small watersheds. These models enable the assessment of the hydrologic effects of development in one small drainage area on other downstream areas.

The populations of urban areas have increased dramatically over the past several decades; these increases have been paralleled by an even greater increase in the potential damages expected with urban flooding. This has created a need for a wider array of hydrologic models. Small watershed and spatially distributed models are necessary for problems such as on-site design, assessment of downstream impacts, the comparison of watershed responses for both before-development and after-development conditions, the solution of legal cases, retrofitting of existing drainage facilities to control the effects of increased development, the assessment of water quality and aquatic habitat, and the assessment of the hydrologic effects on channel morphology. Such an array of problems requires an array of hydrologic models.

Design storm models, which are used for most urban design, are viewed by some as being woefully inadequate because of their simplicity and the assumption that the exceedence probability of the runoff equals that of the rainfall. Does the simplicity of the model

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limit its accuracy for design? Is the assumption about the exceedence probability realistic, and if not, what error does this introduce into design and flood drainage computations? Both knowledge of the potential accuracy of urban hydrologic models and an understanding of the assumptions that underlie the models are needed. These are necessary to assess their appropriateness for solving design problems that are typically encountered in engineering practice. These two elements of small watershed modeling will be approached herein.

SOURCES OF VARIATION

Designs are almost always made on watersheds where measured hydrologic data do not exist, i.e., on ungaged watersheds. Thus, design rainfall data and watershed characteristics are used as input to a model that yields one or more design discharges as output. Given the lack of measured data at the site of the design, the designer should be interested in the expected accuracy of the model-predicted design discharges. Estimates of the expected accuracy can be made by analyzing data from watersheds where measured data are available. The degree to which the accuracy criteria are applicable to sites where the model is used for design depends on the similarity between the design watershed and the calibration watershed.

One common design practice is to use a regionalized unit hydrograph model as the basis for obtaining peak discharge estimates for design. The HEC-1 and TR-20 programs are commonly used regionalized models that include unit hydrographs. The unit hydrographs are assumed to apply over large regions, if not most of the country. Tests of these models, as well as other models, have been made. Because of the widespread use of unit hydrograph models, the unit hydrograph approach will be used to assess the two modeling problems identified above.

Four sources of variation can be identified in the assessment of the accuracy of a regional model. The separation of variation described here is just one possible paradigm and consists of the following four elements:

1. Within-model variation,
2. Storm-to-storm variation,
3. Watershed-to-watershed variation, and
4. Location-to-location variation.

Each of these will be assessed.

Unit hydrograph models are composed of several elements, such as a baseflow model, a loss function, an antecedent moisture element that may or may not be part of the loss function, and a function that describes the unit hydrograph. The specific functional form selected for each of these elements can significantly affect the accuracy; these sources contribute to variation that is referred to here as the <u>within-model variation</u>.

When analyzing a unit hydrograph model, the storm events used to calibrate the model components can significantly effect the prediction accuracy of the model. For example, if the model is calibrated on small volume storms, it may not provide accurate predictions for large volume storms. Also, if it is calibrated for storms that occurred when the watershed was saturated, then the model may provide poor prediction accuracy for storms that occur during periods of low rainfall. This storm-to-storm variation is an important element of prediction accuracy.

Adjacent watersheds can have quite different hydrologic responses to a storm event, even when their land uses and watershed characteristics are similar. Variations in soil characteristics, antecedent conditions, watershed shape, and rainfall patterns will contribute to different hydrologic responses and, thus, unit hydrographs from different storm events. This introduces variation in predictions made with the unit hydrographs. This watershed-to-watershed variation can be significant.

Variation is expected between watersheds in different locations, with the magnitude of the variation depending in part on the differences in hydrometeorological characteristics of the two locations. Whether or not one unit hydrograph model is appropriate for both locations will depend on the variation introduced by these locational differences. For example, coastal-zone unit hydrographs are known to be different from unit hydrographs developed for mountainous areas. This location-to-location variation should be considered when selecting a hydrologic model for design.

PEAK RATE FACTOR UNIT HYDROGRAPHS

Unit hydrograph models require fitting of one or more coefficients. The SCS unit hydrograph uses a peak rate factor (K) to compute the peak discharge:

$$q_p = \frac{KAQ}{t_p} \tag{1}$$

where q_p is the peak discharge (cfs) of the unit hydrograph; A is the drainage area (sq. mi.); Q is the volume of direct runoff (inches), which is 1 inch for a unit hydrograph; and t_p is the time to peak (hours). The time to peak is usually computed from the time of concentration (t_c) by:

$$t_p = \frac{2}{3}\, t_c \tag{2}$$

Thus, for a given watershed with drainage area A, the unit hydrograph requires two inputs, the t_c and the peak rate factor.

The peak rate factor is usually believed to be a function of watershed slope, with large values in mountainous areas and low values in coastal areas. SCS uses a value of 484 for most of U.S., with a lower value of 284 used in coastal areas in Maryland (Welle, Woodward, and Moody, 1989). Ordinates of the SCS UH are obtained from a dimensionless model, with the time to peak and the peak rate factor used to quantify the unit hydrograph.

The gamma distribution is often used as a unit hydrograph because it takes the shape commonly evident in unit hydrographs. Its shape is essentially identical to that of the SCS dimensionless UH. The gamma distribution is a function of shape (c) and scale (b) parameters:

$$q = \frac{t^{c-1}\, e^{-t/b}}{b^c\, g(c)} \tag{3}$$

in which t is time and g(c) is the gamma function with argument c. The time to peak of the UH is related to the parameters by:

$$t_p = b\,(c - 1) \tag{4}$$

The peak rate factor and the shape parameter are related by:

$$c = 1.006 + 1.104 * 10^{-3} * K + 1.267 * 10^{-5} * K^2 + 1.646 * 10^{-9} * K^3 \tag{5}$$

Thus, for a given K and t_c, a gamma distribution unit hydrograph can be computed using Eqs. 2, 5, 4, and then 3. Table 1 gives values of c and K from Eq. 5.

EFFECT OF WITHIN-MODEL VARIATION

It is known that different storms on the same watershed provide different unit hydrographs. It is also true that different models and fitting criteria will produce different unit hydrographs. Rainfall and runoff data from Guthrie, OK, and LaCrosse, WI, were input to a unit hydrograph model that has the option of selecting one of two loss function methods, specifically the curve number and the Green-Ampt equation, one of two parameter options (two- and three-parameters), and one of three weighting criteria for errors at the peak discharge. This yields twelve different unit hydrographs for each storm event. Each of the twelve represents a different model, although there are obvious intercorrelations among the twelve.

The model was used to analyze ten storm events on the Ravine A watershed at Guthrie, OK. For each storm a peak rate factor was computed for each of the twelve model options. The results are given in Table 2. The computed peak rate factors varied from 102 to 529, which is a considerable amount of variation when one considers that SCS uses 284 for coastal watersheds and 484 for general usage. The mean value of the 120 peak rate factors is 302.

An examination of the data suggests that the loss function is a significant source of the within-model variation. The means and standard deviations for the six peak rate factors from each storm are computed for the curve number and Green-Ampt loss functions (see Table 2), with the absolute difference of the means also shown. While some storm-event differences are not hydrologically meaningful, others are quite large and would produce significantly different unit hydrographs. Five of the ten differences in peak rate factors exceed 50.

STORM-TO-STORM VARIATION OF THE PEAK RATE FACTOR

The peak rate factor for a storm-event unit hydrograph can be obtained by transforming Eq. 1:

TABLE 1. **Peak Rate Factors (K) for Selected Values of the Gamma Distribution Shape Parameter (c)**

c	K	c	K	c	K
1.0	0	2.6	309	5.0	504
1.6	175	3.0	349	7.0	622
2.0	237	4.0	434	10.0	765

TABLE 2. **Variation of Peak Rate Factors (K) for Twelve Model Options on Ten Storm Events (Ravine A, Guthrie, OK)**

Model[1]	Storm Event										
Options	1	2	3	4	5	6	7	8	9	10	Mean
C21	215	162	281	508	419	459	264	268	374	275	
C23	207	166	244	518	427	459	261	273	360	273	
C25	197	164	248	529	431	462	263	271	360	272	
C31	205	162	172	441	406	456	276	296	359	273	
C33	198	162	172	431	419	456	276	302	359	275	
C35	194	161	173	455	429	456	277	302	359	273	
G21	161	194	117	361	323	347	270	412	370	222	
G23	206	190	113	380	351	350	269	413	375	223	
G25	199	189	110	410	349	354	267	420	375	226	
G31	186	194	106	449	338	368	278	423	360	236	
G33	190	193	105	454	338	373	277	423	360	237	
G35	186	193	102	482	342	377	278	430	360	238	
Mean	195	178	162	452	381	410	271	353	364	252	302
St. dev	14	15	64	52	44	51	6	71	7	23	
Range	54	33	179	168	108	115	17	162	16	53	
CN mean[2]	203	163	215	480	422	458	270	285	362	274	313
CN st. dev	8	2	48	43	9	2	8	16	6	1	
GA mean[2]	188	192	109	423	340	362	273	420	367	230	290
GA st dev	15	2	6	47	10	13	5	7	8	7	
Difference[3]	15	29	106	57	82	96	3	135	5	44	23

Notes:

(1) For the model options:
C denotes curve number loss function
G denotes Green-Ampt loss function
2 or 3 indicates two-parameter and three-parameter models
1, 3, or 5 indicates a weighting for goodness of fit

(2) CN mean and CN st. dev are for the first six model options
GA mean and GA st. dev are for the second six model options

(3) The difference is the absolute value of the difference between CN mean and GA mean.

$$K = \frac{q_p\, t_p}{AQ} \tag{6}$$

where q_p and t_p are obtained from the unit hydrograph. Where more than one storm event is used to derive a unit hydrograph for a watershed, a K can be computed for each storm event; these can then be used to compute a mean K for the watershed.

Using data from 41 storm events on eight watersheds in two states, peak rate factors were computed for each storm (see Table 3). In most cases, the values of K showed considerable variation from storm to storm on any one watershed. The standard deviations of the computed K values for a watershed ranged from 44 to 162. This reflects the fact that different storms on the same watershed can produce drastically different unit hydrographs. Figure 1 shows the four unit hydrographs derived for the storms on Plot J, Guthrie, OK. The variation between the unit hydrographs is approximately the average found for the other seven watersheds of Table 3. For this source of variation, the K values of Table 3 and the four unit hydrographs of Figure 1 suggest that the storms selected for an analysis can be responsible for significant variation in the resulting unit hydrographs and thus in the accuracy of estimated peak discharges.

WATERSHED-TO-WATERSHED VARIATION OF THE PEAK RATE FACTOR

The data of Table 2 were used to characterize the within-storm variation of the peak rate factor. The best available estimate of K for each watershed is given in Table 4. Variation between these eight peak rate factors are indicative of watershed-to-watershed variation. While the watersheds vary in size, shape, slope, and land use, the differences are not sufficient to justify using the same peak rate factor for a regional unit hydrograph. For the four Wisconsin watersheds, the weighted mean peak rate factor is 503, with a minimum of 401 and a maximum of 617. For the four Oklahoma watersheds the weighted mean is 343, with a minimum of 305 and a maximum of 461. Both of these variations are significant because the mean would produce quite different unit hydrographs than that produced by either of the extremes.

To study the watershed-to-watershed variation the weighted average peak rate factors are given in Table 4 with the average watershed slope. Slope was chosen since past efforts have suggested that the peak rate factor increases with increases in slope. The data of Table 4 suggest a positive correlation although the computed correlation of 0.483 is not statistically significant at the 5% level of significance. This probably reflects the small sample size rather than a lack of association between K and either slope or watershed storage.

WATERSHED AND REGIONAL VARIATION OF THE PEAK RATE FACTOR

The four watersheds in each of the two regions were very close in proximity to each other. Thus, they are of little value in studying the location-to-location variation of the peak rate factor.

TABLE 3. Storm-to-Storm Variation of the Peak Rate Factor

State	Watershed	Event	CN	K	Mean K	Std. Dev. of K
Wisconsin	UC (2.34)	1	82	382	458	162
		2	95	602		
		3	96	582		
		4	83	266		
	UC(2.25)	1	97	642	617	44
		2	92	643		
		3	87	567		
	UP	1	80	391	401	85
		2	76	322		
		3	88	491		
	C	1	98	527	550	82
		2	93	641		
		3	88	482		
Oklahoma	Ravine A	1	82	186	305	98
		2	90	194		
		3	85	244		
		4	92	455		
		5	81	338		
		6	83	459		
		7	92	278		
		8	86	296		
		9	96	359		
		10	95	236		
	Terrace 2B	1	89	476	326	111
		2	76	359		
		3	93	332		
		4	85	432		
		5	91	160		
		6	92	125		
		7	95	387		
		8	99	288		
		9	99	386		
		10	95	311		
	Plot 13	1	94	550	461	105
		2	96	460		
		3	98	313		
		4	99	520		
	Plot J	1	65	235	363	115
		2	73	511		
		3	93	376		
		4	91	328		

TABLE 4. Peak Rate Factor (K) and Watershed Slope (S)

S (%)	K	Watershed	State
2.8	326	Terrace 2B	OK
4.4	363	Plot J	OK
5.0	305	Ravine A	OK
5.1	461	Plot 13	OK
15.0	458	UC (2.34 ac)	WI
15.5	617	UC (2.25 ac)	WI
17.0	550	Controlled	WI
26.4	401	UP	WI

TABLE 5. Statistics for Peak Rate Factors Based on an Average Curve Number

County	Region	Average K	Maximum K	Minimum K
Garrett	1	560	1241	198
Alleghany	1	475	837	166
Washington	1	344	501	201
Frederick	2	449	643	361
Carroll	2	398	518	275
Baltimore	2	349	498	235
Harford	2	695	791	551
Cecil	2	528	770	201
N. New Castle	2	531	755	281
Howard	2	339	436	263
Montgomery	2	499	773	275
Prince George	2	370	623	164
Anne Arundel	2	150	228	77
Calvert	3	111	134	89
Charles	3	223	303	115
St. Mary	3	143	259	56
Dorchester	3	77	91	64
Wicomico	3	103	107	100
Somerset	3	137	137	137
Worchester	3	110	110	110
Kent, MD	3	202	202	202
Queen Anne	3	118	141	95
Caroline	3	147	211	76
Talbot	3	189	238	139
S. New Castle	3	206	354	102
Kent, DE	3	116	135	69
Sussex	3	69	178	19
Region 1		502	1241	166
Region 2		445	791	77
Region 3		122	354	19
Overall		364	1241	19

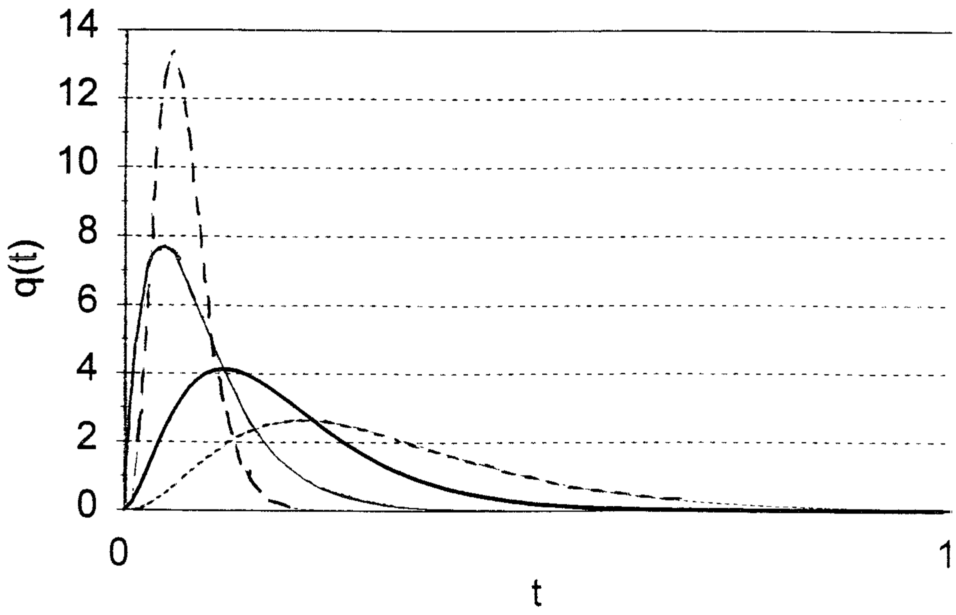


FIGURE 1. Unit Hydrographs for Four Storm Events: Plot J, Guthrie, OK

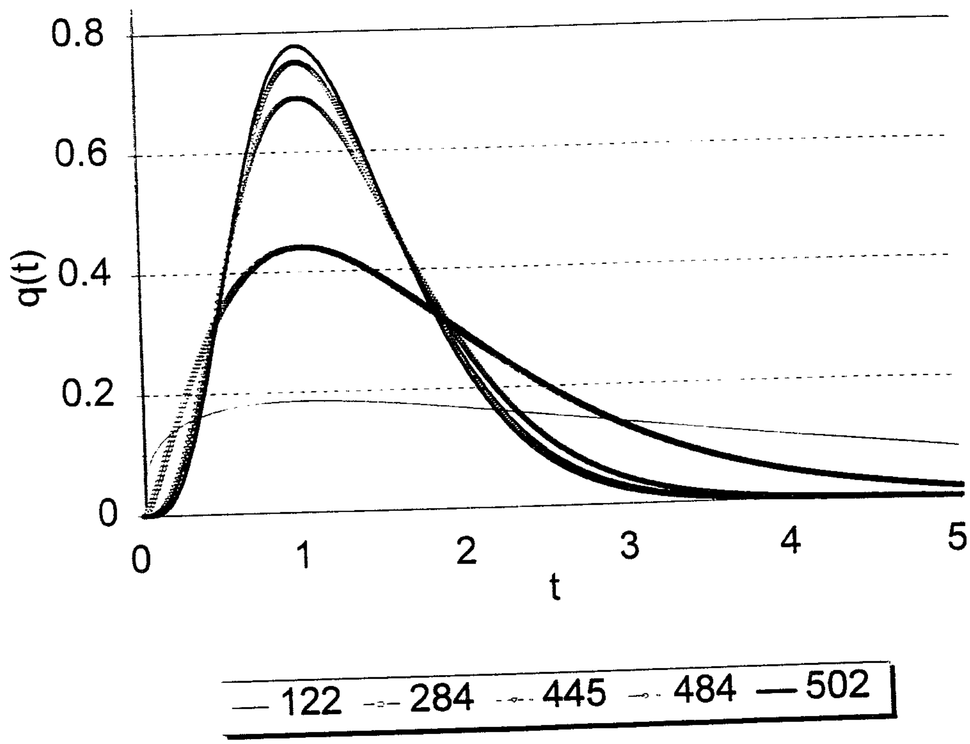


FIGURE 3. Unit Hydrographs for Peak Rate Factors of 122, 284, 445, 484, and 502

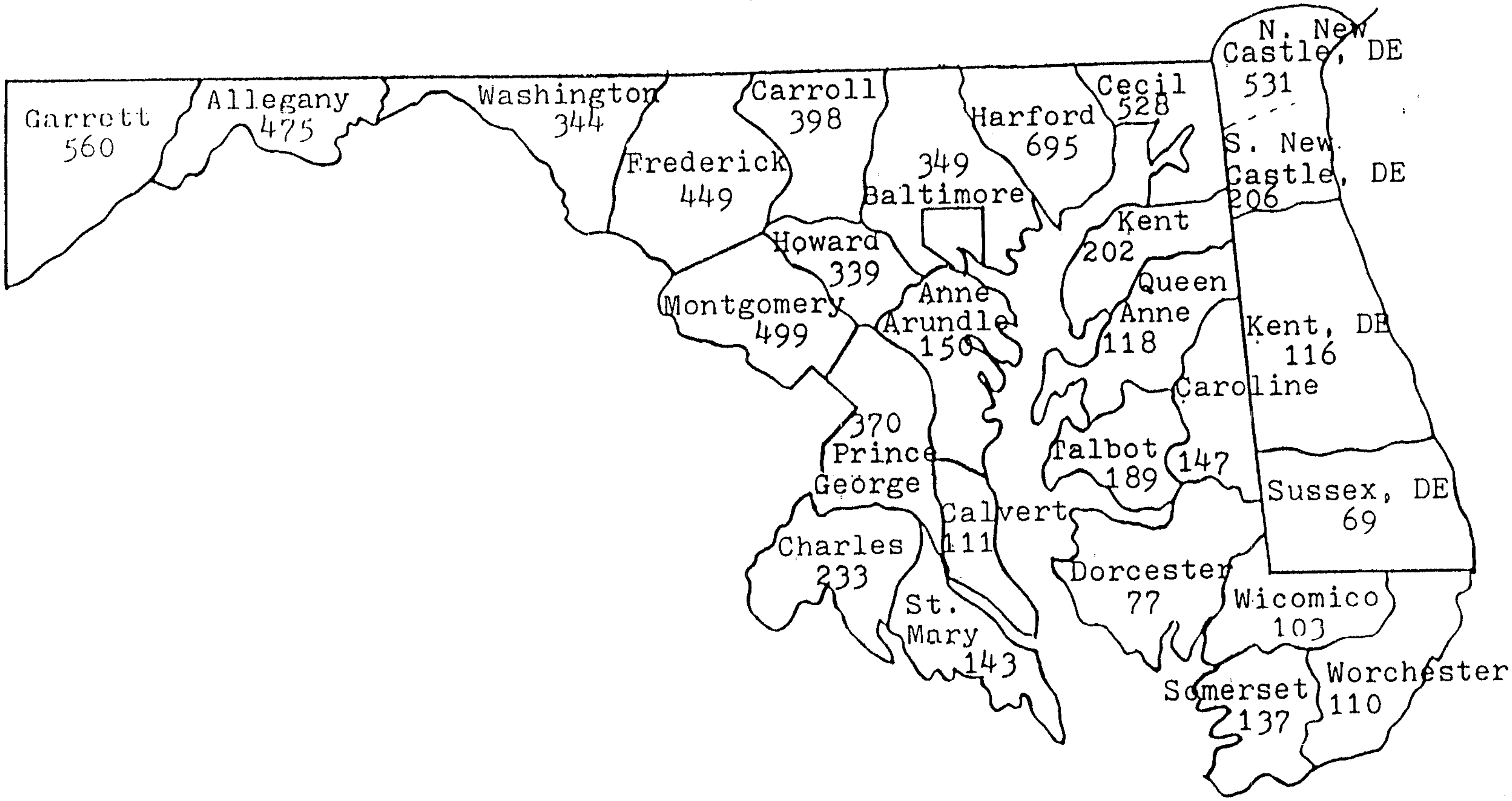


Figure 2. Map of Peak Rate Factors based on Average Curve Numbers for Maryland Counties

As a further study of both watershed-to-watershed and location-to-location variation in the peak rate factor, values of K were estimated for approximately 200 watersheds throughout Maryland and Delaware. The values of K were computed using the log-Pearson type III 2-yr peak discharges obtained from the USGS report for the State of Maryland and estimates of the depth of direct runoff and the time to peak. The resulting values of K were grouped according to county, with a mean computed for each county. The means are shown in Figure 2. Table 5 gives the mean K for each county as well as the extremes for individual watersheds within the counties. Based on the means and knowledge of the general typography, soils, and meteorological characteristics across the state, three regions were identified. For each region mean peak rate factors were computed using the watershed values of K in each region. The coastal region has a mean value of 122. The more mountainous areas of the state have a mean K of 502. The plateau region has a mean K of 445. Figure 3 shows the unit hydrographs for the three regions along with the standard SCS UH (K = 484) and the SCS coastal UH (K = 284).

The peak rate factor for Anne Arundel County is considerably lower than the peak rate factor for the other countries in the central region. This may suggest that this county should be assigned to the eastern region. Watersheds with peak rate factors lower than the county mean were located in the eastern portion of the county, while watersheds with peak rate factors higher than the county mean were located in the western portion of the county. Since the primary drainage pattern is from west to east, it is not advisable to separate the county into two parts. Instead, the larger peak rate factor should be used for the entire county. Thus, the county was assigned to the central region.

To check the accuracy of the regional means, they were used to generate storm hydrographs using measured storm hyetographs and compared with measured hydrographs. The predicted hydrographs showed reasonably good agreement with the measured hydrographs. Figure 4 shows four comparisons, two for the mountainous region (Pony Mountain and Powell's Creek watersheds) and two for the plateau region (two events on Coshocton # 172). The hydrographs suggest that the regional means are realistic. However, there still remains considerable storm-to-storm variation in the peak rate factor.

The coastal-area peak rate factor was tested by comparing computed flood frequency curves with those obtained from USGS files. Welle, Woodward, and Moody (1980) provided frequency curves for peak rate factors of 284 and 484 for four coastal watersheds. They used TR-20 to derive the peak discharges used to construct the frequency curves. In three of the four cases, the frequency curve for the 284 factor is substantially above the USGS frequency curve, especially for the more frequent events. Estimates of the 2-yr and 5-yr peak discharges were computed for the coastal-area peak rate factor of 122. The frequency curves are shown in Figure 5. The county mean values of K seem to give more accurate peak discharges than those based on the 282 factor.

WATERSHED GENERATED UNIT HYDROGRAPHS

The results presented above reveal considerable watershed-to-watershed variation in the peak rate factor. The analyses of different storm events also indicate considerable storm-to-storm

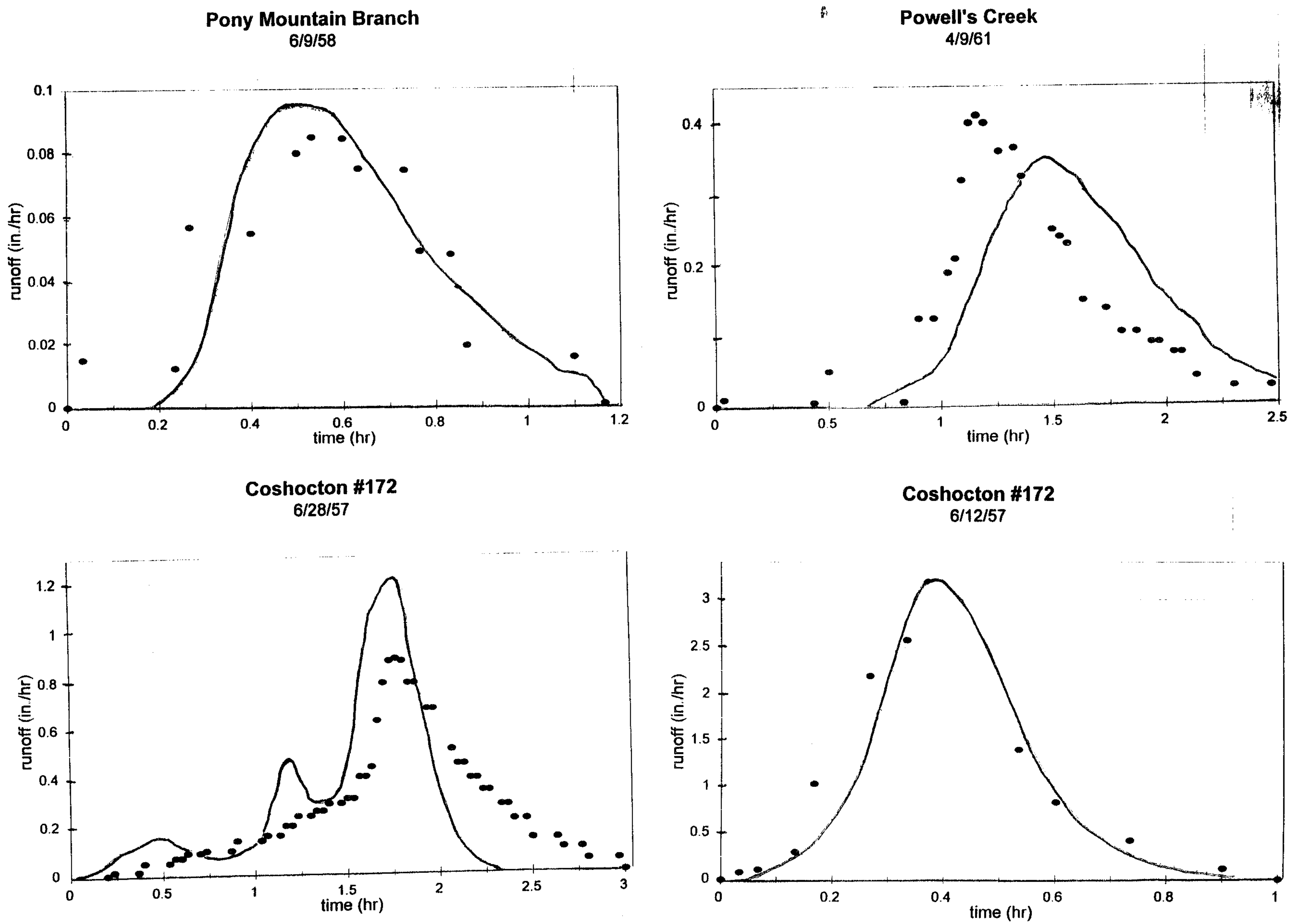


FIGURE 4. Verification of Peak Rate Factors for Mountainous and Plateau Watersheds

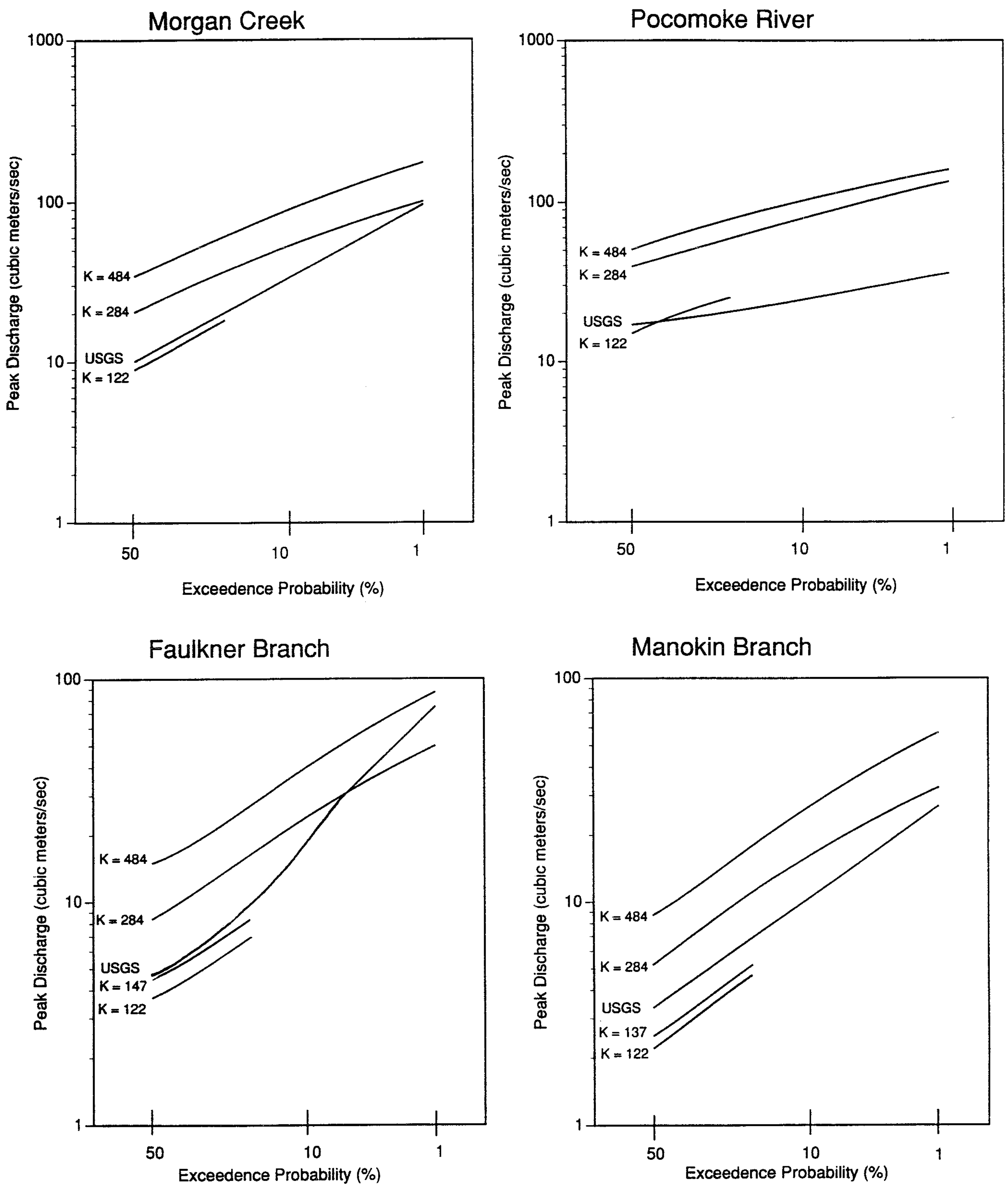


FIGURE 5. Flood Frequency Curves for Coastal Watersheds in Maryland

variation in peak rate factors. Since the unit hydrograph is intended to reflect the effects of watershed characteristics on runoff, the use of the time-area curve as a UH was investigated. While this is not a new idea, the interests were to assess (1) whether or not time-area UH's could be generated with a GIS and (2) the accuracy of the GIS UH.

Data from five small watersheds were analyzed. Two or three storm events per watershed were used to derive mean unit hydrographs for each watershed. Time-area curves were developed using a GIS and also manually using the traditional approach. The time-area curves were routed using a linear reservoir. The proportion under the rising limb and the computed peak rate factor are given in Table 6 for each watershed. Two comparisons can be made from the results.

First, the GIS-generated and manual time-area UH's can be compared. The average difference in the peak rate factor is 9, which is very small; the largest differences was 59 for the Riesel Y-2 watershed. These results suggest that a GIS can produce UH's comparable to those generated manually, which is the traditional approach.

Second, the time-area GIS unit hydrographs can be compared with the storm-event UH's to assess the extent to which the time-area UH's could be used on watersheds where measured rainfall and runoff were not available to derive a watershed-specific UH. The peak rate factors for the five watersheds are shown in Table 6. The mean of the GIS-generated peak rate factors differs from the mean of those derived from storm-event data by 35, with the largest difference for any one watershed being 174 on the Stillwater, OK, watershed. These results suggest that a time-area curve can be very useful in developing a unit hydrograph for an ungaged watershed.

It is also of interest to assess the variation of peak rate factors with the variations derived previously. The mean difference of 35 is quite reasonable when compared with the variations identified for the within-model variation using the data from Ravine A. Thus, the time-area method appears to be a reasonable method for deriving a UH for small watersheds where storm event data are not available.

TIME OF CONCENTRATION ESTIMATION ON SMALL WATERSHEDS

When using a UH based on the peak rate factor, the two inputs to Eq. 1 that are subject to the greatest error variation are the peak rate factor K and the time to peak. The time to peak (t_p) is usually estimated as two-thirds of the time of concentration (t_c) as indicated by Eq. 2. This is usually assumed to be a reasonable transformation. The problem then is the method of estimating t_c. The velocity method, which equates t_c to the ratio of the flow length to the velocity of flow, is most commonly used. Since the flow lengths of watershed segments can be accurately estimated, the velocity appears to be the greater source of error.

For small watersheds where sheet flow, overland flow, and small channel flow dominate the principal flowpath, velocities are usually estimated using a relationship between velocity (V) and the slope (S) of the flowpath:

TABLE 6. Comparison of Unit Hydrograph Characteristics for UH'S Generated from Measured Storm Data and Time-Area Curves.

		Proportion of volume Under rising limb			Peak rate factor		
Location	No. of storms analyzed	Storm event UH	GIS T-A UH	Manual T-A UH	Storm event UH	GIS T-A UH	Manual T-A UH
Cherokee, OK	2	0.395	0.318	0.348	510	410	449
Hastings, NB	3	0.429	0.336	0.331	554	434	427
Riesel W-2, TX	3	0.374	0.329	0.300	483	425	387
Riesel Y-2, TX	3	0.418	0.362	0.316	539	467	408
Stillwater, OK	2	0.212	0.347	0.365	274	448	471
Mean		0.366	0.338	0.332	472	437	428
Standard Deviation		0.088	0.017	0.026	114	22	33

TABLE 7. Summary Statistics for Data Base

Statistic	L (feet)	n	S (ft/ft)	$\frac{nL}{\sqrt{S}}$	Measured Time of Concentration (minutes)
minimum	12	0.0073	0.001	1	1.1
maximum	3033	0.40	0.162	5227	47.3
mean	584.3	0.119	0.057	476	13.9
Standard deviation	747.9	0.10	0.047	870	10.6

$$V = a\,S^b \tag{7}$$

where b is usually assumed to be 0.5, as in Manning's equation, and a is a constant that depends on the type of surface. The problem with this equation is that the constant a is a simplification of part of Manning's equation:

$$a = \frac{1.49}{n}\,R_h^{2/3} \tag{8}$$

with values for n and R_h assumed to construct the figure of V versus S or fit Eq. 7.

For small watersheds, sheet flow can be an important part of the t_c flowpath. One method for estimating the travel time for sheet flow is based on kinematic wave theory. Specifically, the sheet flow travel time is:

$$t_c = \frac{0.93}{i^{0.4}}\left(\frac{nL}{\sqrt{S}}\right)^{0.6} \tag{9}$$

in which L is the flow length in feet, i is the rainfall intensity in inches per hour, n is Manning's coefficient, and S if the slope in ft/ft. One important problem that arises in the use of Eq. 9 is the conditions for which it is applicable. In the 1986 TR-55 document, an upper limit of 300 feet was placed on the use of a similar kinematic wave equation. Some localities that use TR-55 believe that 300 feet is too long and have placed shorter upper limits on its use.

Times of concentration and watershed characteristics from 59 field and laboratory experiments were analyzed to determine a suitable limit for the kinematic equation. The data used in this analysis represented a wide range of watershed sizes, slopes, and ground conditions. Summary statistics of the data are given in Table 7. Data sources included the following. Izzard and Augustine (1943) tested runoff responses to various precipitation intensities on asphalt. Butler (1976) obtained data and results of overland flow on concrete surfaces taken by the U.S. Army Corps of Engineers. Kirpich (1940) studied several agricultural watersheds in Tennessee. Hyetograph and hydrograph data obtained from monitoring stations of the Agricultural Research Service for a variety of watersheds throughout the United States were analyzed to determine travel times; detailed watershed maps and land cover conditions were used to obtain values of L, n, and S. Lane, Woolhiser, and Yevjevich (1975) provided some simulated data on overland flow.

Statistical analyses of three criterion: L, iL, and $nL/\sqrt{S}$ were performed to determine the most reasonable and accurate limit for use of the kinematic wave equation for sheet flow. While increasing the limit of each of these criteria, the bias, standard error, relative bias, and relative error of T_c estimates were computed for all watersheds below the assumed upper limit.

The bias is a measure of the systematic error introduced into estimates, with a negative value indicating under-prediction and a positive value indicating over-prediction. The relative bias, which is the ratio of the bias and the mean t_c for the data, is useful for comparing results from analyses based on different data sets. The standard error is a measure of the accuracy of the estimator. The standard error ratio, which is the ratio of the standard error to the standard deviation of t_c values, is a measure of the relative accuracy and, like the relative bias, is useful for comparing results from analyses on different data sets.

To test the accuracy of the length as the limiting criterion, statistics were computed for all watersheds having lengths less than an upper limit, where the upper limit was successively varied from 75 feet through 1525 feet. For all upper limits less than 525 feet, the biases indicated an average underprediction of about 2 minutes (see Table 8). However, in terms of the relative bias, the average error was decreasing even beyond the upper limit of 525 feet. The standard error ratio decreased from near 1 for small flow-length limits to a minimum of about 0.4 at an upper limit of 525 feet. After 525 feet the standard error ratio increased which indicates poorer accuracy. Placing a limit on flow length, L, of 100 feet or 300 feet resulted in high mean biases and standard errors (see Table 8). The statistics show that any limit on L gives standard errors greater than 2 minutes. The optimum standard error ratio occurs at a limit of 525 feet where the bias is -1.89 minutes and the standard error is 4.21 minutes. Thus the results suggest that the use of flow length as the limiting criterion on the kinematic wave time of concentration equation is inconclusive and, therefore, is not an accurate criterion for limiting the use of the kinematic wave t_c equation.

Izzard's work involved conditions where rainfall intensity and flow length were the dominant variables. Thus, he proposed using the product iL as the limiting criterion with an upper limit of 500. Using the data for the 59 stations, upper limit values of iL were varied from 100 to 8100. The bias and accuracy statistics were computed for all watersheds that had iL values below an upper limit on iL; the results are shown in Table 9. The optimum standard error ratio occurs at an iL limit of 200; however, for upper limits on iL up to 1600 the standard error ratio increases only slightly, which indicates that Eq. 2 may be applicable for situations where iL is as high as 1600. There is a large increase in the standard error ratio, from 0.431 to 0.968, as the limit of iL is increased from 1600 to 1700 (see Table 9), suggesting that the limiting value is probably not 1700 or more. In comparison, Crawford and Linsley (1966) indicated that the transition from laminar to turbulent flow on paved surfaces occurs when iL equals 260, which is roughly comparable to the optimum value of 200 indicated by the computed standard error ratios. Chow (1964) indicated that Izzard's limit of 500 could be extended. The larger limit of 1600 suggested by the data analyzed in this study may reflect Chow's observation. Crawford and Linsley (1966) concluded that the overall response of storage is relatively insensitive to changes from laminar to turbulent flow, which suggests that 1600 may be a reasonable limit for iL.

Finally, the parameter $nL/\sqrt{S}$ was used as the limiting criterion on the kinematic wave time of concentration. The upper limit was varied from 25 to 1525, with the resulting values of the statistics shown in Table 10. At a $nL/\sqrt{S}$ limit of 100, the standard error ratio, equal to 0.344, was a minimum; the bias was -1.027 minutes, the standard error was 1.74 minutes, and the relative bias was -0.145.

TABLE 8. **Goodness of Fit of Computed Times of Concentration Using the Length as the Limiting Criterion**

Upper limit	Sample size	Bias (min)	Standard error (min)	Relative bias	Standard error ratio
75	6	-2.03	2.41	-.415	.858
100	10	-1.76	2.03	-.366	.913
175	15	-2.86	4.10	-.363	.731
250	21	-2.15	3.44	-.289	.701
275	23	-2.09	3.50	-.240	.558
300	24	-2.11	3.46	-.241	.565
350	28	-1.99	3.54	-.209	.518
375	32	-1.98	3.73	-.186	.455
400	35	-1.72	3.64	-.154	.454
450	36	-1.65	3.59	-.145	.451
475	39	-2.15	4.46	-.165	.434
500	43	-1.89	4.25	-.148	.418
525	44	-1.89	4.21	-.145	.414
625	46	-1.35	4.74	-.105	.475
650	47	-1.08	4.96	-.085	.502
675	48	-.57	6.00	-.045	.607
750	49	-.41	6.03	-.032	.615
825	50	-.24	6.07	-.019	.600
875	51	.37	7.43	.028	.736
1225	52	.83	8.12	.064	.808
1250	53	1.85	11.04	.143	1.106
1425	54	2.52	12.14	.196	1.224
1500	55	2.76	12.21	.210	1.220
1525	55	2.76	12.21	.210	1.220

Table 9. **Goodness of Fit of Computed Times of Concentration Using the Product of Intensity (i) and Length (L) as the Limiting Criterion**

Upper limit on iL	Sample size	Bias (min)	Standard error (min)	Relative bias	Standard error ratio
100	7	-1.98	2.65	-.284	.422
200	11	-1.22	2.38	-.120	.281
300	16	-1.65	2.79	-.149	.300
400	17	-1.64	2.72	-.153	.297
500	23	-1.70	3.67	-.146	.322
600	25	-1.63	3.52	-.145	.319
700	28	-2.32	4.28	-.194	.403
800	30	-2.09	4.18	-.169	.404
900	31	-2.26	4.33	-.177	.413
1000	33	-2.13	4.21	-.164	.413
1300	34	-2.41	4.62	-.176	.430
1400	35	-2.40	4.56	-.172	.427
1500	36	-2.30	4.50	-.164	.426
1600	37	-2.36	4.50	-.167	.431
1700	38	-0.86	10.01	-.062	.968
1900	40	-0.89	9.76	-.063	.963
2300	41	-0.67	9.72	-.046	.931
2500	43	-0.30	9.74	-.022	.938
2700	44	-0.13	9.69	.010	.943
2900	46	0.69	10.57	.049	1.047
3000	47	0.91	10.58	.065	1.056
3600	48	0.99	10.49	.072	1.041
3800	50	0.98	10.28	.073	1.022
3900	51	1.26	10.40	.092	1.027
4000	52	1.25	10.30	.092	1.021
4100	53	1.67	10.72	.125	1.061
6300	54	2.10	11.14	.159	1.106
6700	55	2.76	12.21	.210	1.220
8100	55	2.76	12.21	.210	1.220

Table 10. Goodness of Fit of Computed Times of Concentration Using $nL/\sqrt{S}$ as the Limiting Criterion

Upper limit	Sample size	Bias (min)	Standard error (min)	Relative bias	Standard error ratio
25	10	-1.07	1.37	-.261	.813
50	21	-1.22	1.82	-.197	.467
75	23	-1.13	1.75	-.179	.458
100	26	-1.03	1.74	-.145	.344
125	30	-1.53	3.20	-.192	.547
200	34	-1.38	3.19	-.149	.478
250	37	-1.39	3.32	-.133	.424
275	38	-1.32	3.28	-.124	.421
300	39	-1.47	3.45	-.134	.427
325	40	-1.41	3.41	-.126	.424
350	41	-1.42	3.38	-.124	.415
400	42	-1.39	3.34	-.119	.410
425	45	-1.69	4.29	-.130	.426
450	46	-1.42	4.55	-.110	.456
475	47	-1.08	4.96	-.085	.502
500	48	-0.91	5.02	-.071	.512
875	49	-0.41	6.03	-.032	.615
955	50	.09	6.92	.007	.709
1050	51	.24	6.95	.019	.689
1100	52	.53	7.20	.041	.708
1300	53	1.55	10.40	.119	1.030
1525	53	1.55	10.40	.119	1.030

TABLE 11. Comparison of Optimum Goodness-of-Fit Statistics for Three Criteria: L, iL, $nL\sqrt{S}$

Criterion	Limit	Sample size	Bias (min)	Standard error (min)	Relative bias	Standard error ratio
L	100	10	-1.76	2.03	-0.366	0.913
L	300	24	-2.11	3.46	-0.241	0.565
L	525	44	-1.89	4.21	-0.145	0.414
iL	200	11	-1.22	2.38	-0.120	0.281
$nL/\sqrt{S}$	100	26	-1.03	1.74	-0.145	0.344

A comparison of the goodness of fit statistics for the three criterion (i.e., L, iL, and $nL/\sqrt{S}$) shows that the bias and standard error for the optimal limit of the $nL/\sqrt{S}$ criterion are better than those determined using either L or iL as the limiting criterion. The optimum values for the three criterion and their statistics are given in Table 11; the values for lengths of 100 feet and 300 feet are also given as a means of comparing the optimum values to these currently used limits. While the relative biases of the three criteria are essentially identical, the standard error ratios for iL and $nL/\sqrt{S}$ are better than that for the length L.

As another method of analysis, the compiled data base was used to fit the three coefficients of Eq. 9 using numerical optimization (McCuen, 1993) for various upper limits on the $nL/\sqrt{S}$ criterion. As the upper limit on the criterion was increased, the sample size increased since all data points below the limit were used to fit the three coefficients. The results of these analyses are given in Table 12. Except for the analyses based on an upper limit of 125 and 300, the fitted coefficients are different from the theoretical values. All of the analyses show biases that are essentially zero. Values of $nL/\sqrt{S}$ greater than 100 yield relatively poor standard error ratios, and the coefficients are very different than the theoretical values for $nL/\sqrt{S}$ greater than 125. Upper limits of 75 and 100 yield better standard errors and standard error ratios than for larger values of the upper limit. These empirical analyses support a limit for $nL/\sqrt{S}$ of about 100.

Plots of $nL/\sqrt{S}$ versus the error, which is the difference between the predicted ($\hat{t}_c$) and the measured (t_c) times of concentration, were created to see the relationships between increasing $nL/\sqrt{S}$ values and error in t_c. As shown in Figure 6, small errors occurred for small $nL/\sqrt{S}$ values, with the errors increasing with $nL/\sqrt{S}$. Scatter in the data begins to increase as $nL/\sqrt{S}$ approaches a value of 100, further justifying this estimate of a limit.

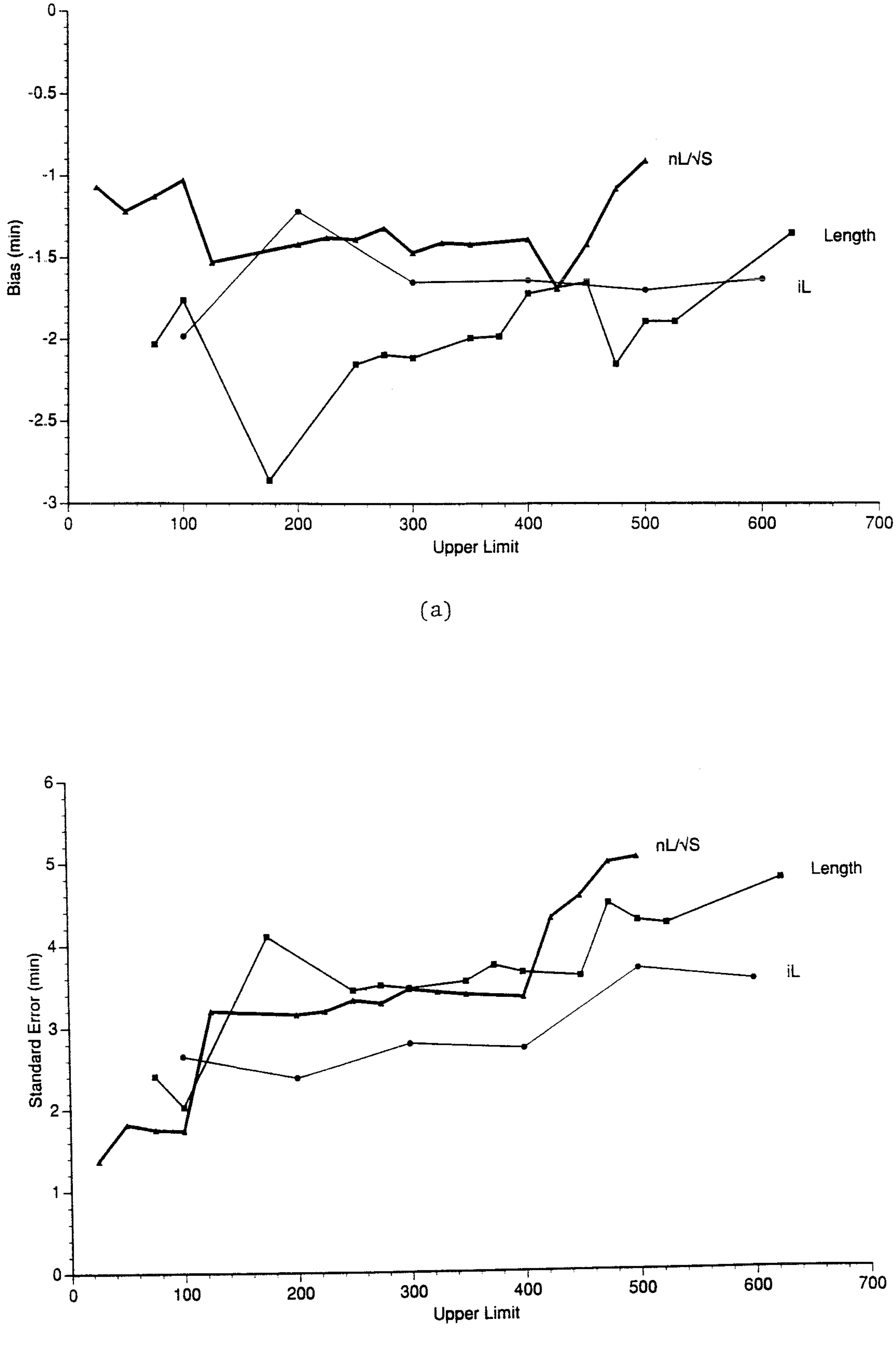
In summary, the $nL/\sqrt{S}$ criterion provided better goodness-of-fit statistics than the length. While the statistics for iL as a criterion were comparable to those of $nL/\sqrt{S}$, the latter is preferred because it is composed of variables that are related to the physical processes that underlie kinematic flow routing. Thus, $nL/\sqrt{S}$ is a more rational criterion than iL for limiting the use of Eq. 9 in estimating sheet-flow travel times. The analyses for an upper limit for $nL/\sqrt{S}$ of 100 provided the best overall results.

As an independent verification of the use of $nL/\sqrt{S}$ of 100 as a criterion for setting the portion of the principal flow path subject to sheet flow, data from ten small watersheds were used. On each watershed, the time of concentration was computed by analyzing rainfall-runoff data. From 2 to 10 storm events were used for each watershed. The times of concentration were computed two ways: (1) the traditional approach using the velocity method and (2) using the sheet flow equation for the portion of the flow path such that $nL/\sqrt{S}$ equals 100, with the velocity method used for the remainder of the flowpath. The results are given in Table 13. While both methods had mean errors of less than 1 minute, the velocity method was far less accurate, with an average absolute error of 98%, while the computations with $nL/\sqrt{S}$ = 100 have an average absolute error of 31%.

TABLE 12. Results of Numerical Fitting of Equation 1 for Upper Limits on $nL/\sqrt{S}$

Upper limit	Sample size	Coefficients of Equation 1* C_1	C_2	C_3	Bias (min)	Standard error (min)	Standard error ratio
50	21	1.77	0.47	0.48	-0.1	1.25	0.32
75	23	1.80	0.47	0.47	-0.1	1.19	0.31
100	26	2.01	0.40	0.44	0.0	1.34	0.27
125	30	1.03	0.32	0.62	0.0	2.74	0.47
200	32	1.46	0.31	0.53	-0.1	2.76	0.43
100	26	2.01	0.40	0.44	0.0	1.34	0.27
225	34	1.57	0.30	0.51	0.1	2.80	0.42
250	37	1.48	0.33	0.52	0.0	3.03	0.39
300	39	1.42	0.32	0.53	0.0	3.13	0.39
350	41	1.49	0.33	0.52	0.0	3.09	0.38

* For comparison, the theoretical values are C_1 = 0.94, C_2 = 0.4, C_3 = 0.6.



(a)

(b)

FIGURE 6. Comparison of (a) Bias and (b) Standard Error for Three Criteria: Length, iL, and nL/S**0.5

TABLE 13. **Verification of $nL/\sqrt{S}$ as a Criterion for Sheet Flow Travel Times**

Watershed/Location	Rainfall runoff t_c (min)	Velocity t_c (min)	Diff. (min)	Rel. diff. (%)	Velocity 100 t_c (min)	Diff. (min)	Rel. diff. (%)
W-1, Stillwater, OK	23.2	12.3	-10.9	47.0	19.6	-3.6	15.5
18-H, Hastings, NB	22.5	6.2	-16.3	72.4	11.7	-10.8	48.0
W-5, Cherokee, OK	33.0	16.4	-16.0	50.3	22.4	-10.6	32.1
UCW, LaCrosse, WI	9.5	4.2	-5.3	55.8	9.3	-0.2	2.1
UPW, LaCrosse, WI	12.0	2.4	-9.6	80.0	9.3	-2.7	22.5
C, LaCrosse, WI	11.6	3.3	-8.3	71.6	9.3	-2.3	19.8
A, Guthrie, OK	19.6	57.1	37.5	191.3	31.0	11.4	58.2
2B, Guthrie, OK	16.0	64.2	48.2	301.2	28.2	12.2	76.3
13, Guthrie, OK	7.8	4.0	-3.8	48.7	10.7	2.9	37.2
J, Guthrie, OK	13.1	4.7	-8.4	64.1	12.8	-0.3	2.3
			-0.7	98.2	means	-0.4	31.4

IMPLICATIONS FOR ENGINEERING DESIGN

The above analyses have shown that the primary inputs to unit hydrograph models, i.e., the peak rate factor and the time of concentration, are not highly accurate. All elements of variation are large. What are the implications of these observations with respect to engineering design?

Does the large variation in these inputs suggest that the UH model is overly simplified? Should the UH model be abandoned in favor of a more complex model? The large variations shown herein are probably representative of any model, whether it is a peak discharge model or a watershed-storage model used with continuous simulation. The U.S. Water Resources Council (1981) showed that UH models provide levels of accuracy that are comparable to those of the uncalibrated peak discharge models. Figure 7 (USWRC, 1981) shows box-and-whisker plots for peak discharges estimated by nine hydrologic models. The calibrated equations of Procedure 1 show the smallest scatter. The UH-based methods (Procedures 9 and 10) have large variations, which is comparable to that of the uncalibrated peak discharge equations. Regression analyses frequently show that models with a very few predictor variables provide almost identical accuracy to models based on many predictor variables (Sauer et al., 1981). Evidently adding more elements to the model does not necessarily improve the prediction accuracy, although it may provide a more flexible model. The comprehensive study by Mandeville et al. (1970) showed that simple models based on two or three independent parameters were as accurate as more complex models. <u>While model complexity may be increased for design problems where there are numerous, hydrologically diverse design objectives, increased complexity cannot be justified on the basis that it will produce significantly greater design accuracy</u>.

If increased model complexity is not the solution, then is it necessary to abandon the unit hydrograph model? The time-area analysis suggested that the estimation of K using on-site information may be preferable to the use of a regional mean K. Similarly, the $nL/\sqrt{S} = 100$ constraint on the use of the sheet-flow equation appears to lead to more accurate estimates of t_c (see Table 13). <u>It appears that improving our understanding of the inputs is a major key to improving design accuracy</u>.

With respect to the peak rate factor, the results presented herein indicate that any value selected to use in a specific design is not highly accurate. Typically, the peak rate factors used are values accepted for the region. They represent mean values that smooth out watershed-to-watershed and storm-to-storm variations. <u>Thus, the need to handle modeling variation in design must be given greater attention, probably more attention than the issue of the type of model to be used</u>.

In the USWRC (1981) report, the calibrated regression equations (Procedure 1 in Figure 7) provided the highest accuracy. The results presented herein for the peak rate factors of Maryland watersheds indicate that calibration of peak rate factors in the form of regional means is preferable to the use of an overall mean value. The results of Table 12 also indicate that calibration of the t_c-equation coefficients can improve accuracy. <u>Thus, it appears that accuracy is affected more by calibration than the type of model used</u>.

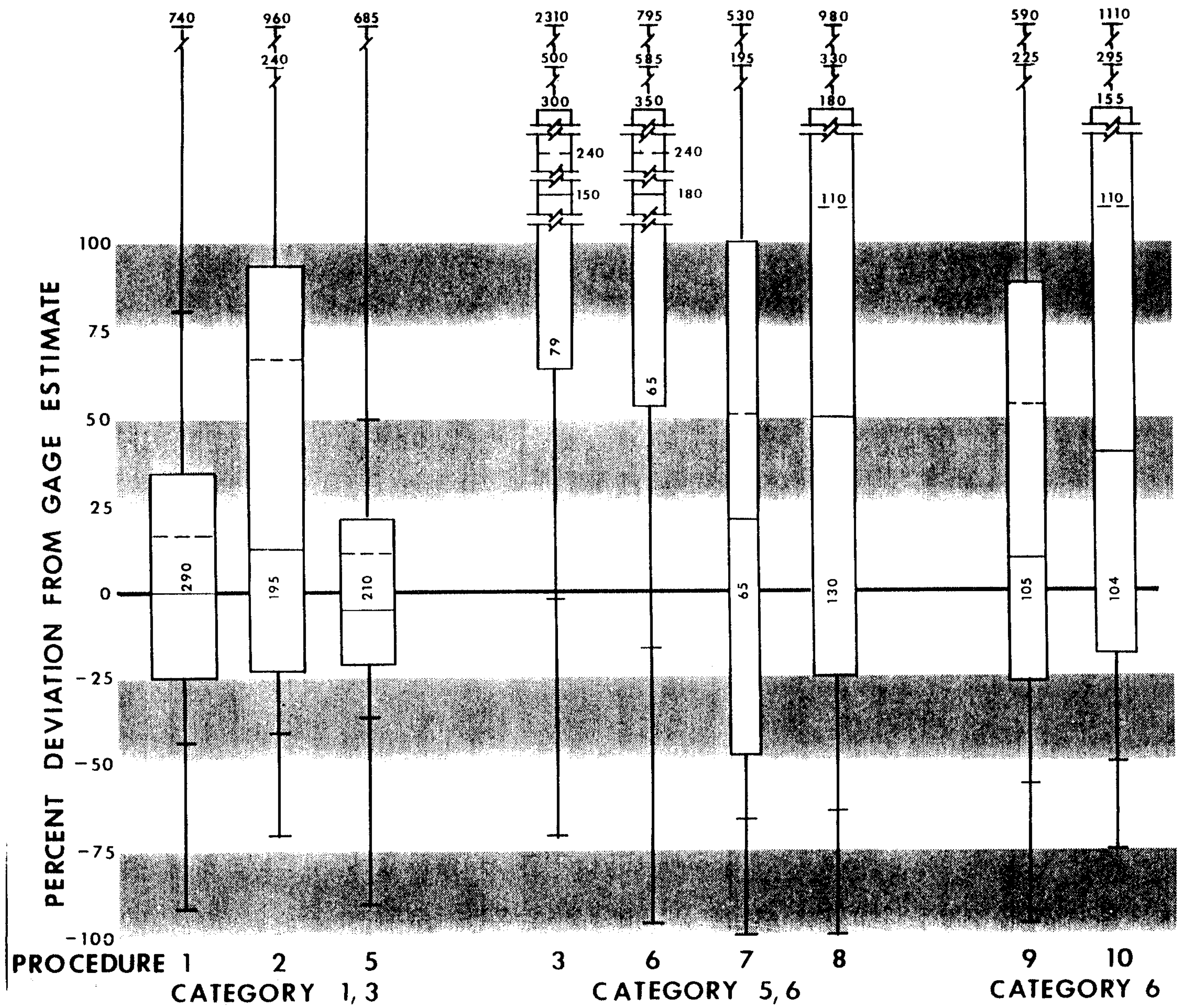


FIGURE 7. Goodness-of-Fit Results for Nine Hydrologic Models

Design risk is represented in small watershed modeling through the return period of the rainfall. The intensity taken from an intensity-duration-frequency curve reflects the variation in the historical record of rainfall. For a design duration, which can be t_c for the Rational method and 24 hours for the SCS methods, the intensity for a given return period reflects the probability distribution of rainfall over the period of record. It does not reflect the variation of watershed conditions, or the nonhomogeneity of land use during the design life of projects. In unit hydrograph models with a peak rate factor, or a similar UH parameter, it would be necessary to use the variation in K to incorporate such uncertainty of watershed conditions. If a mean value of K is used, then it suggests a willingness to accept a 50% risk associated with flood-generating conditions such as a saturated watershed. <u>There is a need to address the issue of incorporating variation in watershed conditions into flood-risk assessment and coordinate this risk with the risk currently handled using the return period of the design storm.</u>

It appears that an initial attempt to address watershed-condition risk was made when the SCS coastal peak rate factor was adopted. They decided to use a peak rate factor of 284 even though this K resulted in overestimates of the flood frequency curves for gaged watersheds in coastal areas (see Figure 5). A frequency curve approximated with a K of 122, the regional value developed using the data of Figure 5, appears to give a better fit to the watersheds used in the development of the coastal area peak rate factor. <u>A problem with incorporating watershed-condition risk into design is the inability to assign a numerical value to the risk that is comparable to the exceedence probability of a design storm</u>. When a method is developed that can quantify watershed-condition variation, then it will be necessary to develop a means of combining design storm risk and watershed-condition risk to obtain an overall, single-valued measure of risk.

Ultimately, a design will be based on a single-valued estimate of, for example, a design discharge. Some structures such as a two-stage riser use two design discharges. But while processes such as continuous simulation can provide knowledge of the distribution of the design discharge, ultimately a single value must be selected. <u>The distribution provides useful information only when there is a pre-established decision rule for selecting the single value to use in the design</u>.

Risk is currently handled using the concept of binomial risk. If a method of combining the rainfall exceedence probability and the watershed-condition probability is adopted, then the binomial risk concept can still be used. This lack of knowledge is currently avoided by assigning failure risk only through the rainfall, with risk associated with watershed conditions ignored. This has been criticized for a couple of decades, although the solution recommended by some has been to do away with the design storm method entirely rather than to fix the problem. <u>Since the hydrologic conditions of the watershed at the time of a rainfall that has the design exceedence probability will significantly affect the hydrologic response, there is a need for a unit hydrograph model that accounts for variation in watershed conditions, thus allowing for combinations of risk</u>.

ACKNOWLEDGMENTS

Parts of this paper have been abstracted from theses and scholarly works of former students. The contributions of the following are significant and gratefully acknowledged: Alice Caponiti, Elizabeth W. Kistler, David Overstreet, and Jill Speiss.

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Role of Hydrologic Modeling within an Interdisciplinary Approach to Urban Watershed Planning and Management-Experiences from King County

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INTRODUCTION

For the last two and a half years I have been intensively involved in applied watershed modeling. However, prior to that I worked in hydrologic process research and model development for over ten years with the USDA-ARS. Given this background, it would probably be helpful for me to share some ideas with those of you currently on the front line of hydrologic model development regarding what our needs are and where current technology is lacking. I propose to do that indirectly by giving you an overview of our current watershed modeling efforts in King County, Washington. In giving this overview, there will be a natural tendency to want to tell a coherent, positive story- everybody does this. So, I may not explicitly expose or even recall all the imperfections or assumptions underlying our rather substantial modeling process-therefore I look to this well-qualified audience to ask some pointed questions and generate some lively discussion after the conclusion of my remarks.

King County has a population of 1.6 million of which approximately 1/3 live in the city of Seattle, 1/3 in smaller cities, and the remaining 1/3 in the unincorporated area. The area of the county is 2,192 square miles of which 35% is public forest lands (mainly U.S. Forest Service administered) located to the east at higher elevations. Four major rivers and a network of tributaries drain a steep, and highly erodible landscape that is a legacy of the area's glacial history (see Figure 1). The drainage system also includes two large lakes, Lake Washington and Lake Sammamish as well as numerous smaller ones. The largely natural drainage system of the county has supported highly productive runs of a variety of natural salmon species; most notably the Cedar River Sockeye. For various reasons, including habitat degradation brought on by urbanization, salmon runs have declined to historic lows in recent years. This decline is a major concern to the government and citizens of King County for a range of legal, environmental, cultural, and economic reasons.

From the point of view of ongoing urbanization, King County can be viewed as consisting of roughly three zones (see Figure 2)- an already fully urbanized north-south band of perhaps 10 miles in width along the shore of Puget Sound, a rapidly urbanizing middle zone next to that of approximately 15 miles in width, and the easternmost rural, forest zone of much slower growth extending to the county border at the crest of the Cascade Mountains. It is the second zone that is the focus of concern with regard to watershed management for it is the not only the region of highest risk with regard to aquatic resource and ecosystem damage, but it is also the region with the highest frequency of flooding and drainage complaints associated with new urbanization.

BASIN PLANNING PROGRAM

I work for the Surface Water Management (SWM) Division of the King County Department of Public Works. I forget what our official mission statement says, but in unglorified terms, our charge is to protect people from flood waters and aquatic resources from people. Over the last 10 years this mission has focused on the western 30% (nearly 700 sq. mi.) of the county with the most active urbanization. The unincorporated portion of this area is known as the SWM service area because property owners within it pay for and receive the full complement of surface water management services including comprehensive basin plans.

Over the last 10 to 15 years, urbanization in this area has generally resulted in the clearing of second growth and even some old growth forest to build roads, subdivisions and shopping centers, and to a lesser though still significant extent to convert farmland to commercial and industrial uses. These actions have often triggered rapid changes in watershed hydrology, severe stream channel destablization, downstream sedimentation and flood damage, degradation of water quality, filling of wetlands, and deterioration or destruction of salmonid habitat. It is the job of the Basin Planning Program to identify and correct current problems and prevent future ones by implementing a long term, coordinated, watershed management plan involving C.P.R.- **C**apital Improvement Projects for flood hazard reduction and habitat enhancement and restoration, **P**rograms of public involvement and education in stewardship and watershed management, and **R**egulations to restrict land use, require special surface water impact mitigation, or best management practices.

Before the CPR plan can be developed for a given watershed, a watershed analysis team is assembled (see Figure 3) to sample, map, describe and analyze current and future surface water problems as well as aquatic resource conditions in the watershed of interest. Hydrologic and often backwater modeling figures prominently in this **Current and Future Conditions** (CFC) analysis. From the point of view of the egocentric hydrologist, all informational pathways lead to and flow from--you guessed it-- the hydrologist whose job is to develop a calibrated watershed model capable of recreating pristine conditions within the basin, characterizing current conditions, and predicting worst-case-scenario future conditions that will occur under full urbanization in the absence of the prescriptive CPR of a basin plan. Subsequently, watershed modeling is also used to develop and test some of the elements of the CPR plan that is developed by the interdisciplinary team. To date, the basin planning program has completed or nearly completed eight basin plans covering nearly half the area west of the service area boundary, and over 65% of the area west of the boundary that is in unincorporated King County (see Figure 4).

OUR MODEL OF CHOICE-HSPF BY DEFAULT

Our watershed planning activities demand a range of detailed hydrologic information that extends beyond the traditional but still important annual flood frequency analysis. Much of this demand arises from the increasing interaction of ecologists, hydrologists, and engineers, in an effort to understand what the impacts of urbanization are, and how to minimize them. In the past our main concern might have been estimation of the 25-yr discharge for road culvert sizing. Now we need to simulate stream flow throughout all seasons of the year to assess impacts on various life stages of salmon, to characterize wetland water fluctuations on which plant and animal communities depend, and to control stream flow durations to pre-developed levels across a range of discharge frequencies in order to prevent accelerated channel erosion downstream of new subdivisions. This level of hydrologic detail is only available via continuous, long term hydrologic simulations. So we need a continuous model as opposed to an event model, a model capable of representing changes in land use, - and one that can represent a range of drainages from small, barely perennial streams draining a square mile or two up to some of our larger rivers draining 100s of square miles. Right now, in Puget Sound Country, for better or for worse, that model is HSPF (Bicknell et al, 1992)- King County uses it, the City of Seattle uses a proprietary version of it, Thurston County uses it, and Snohomish County, and the more advanced, private consulting firms.

THE KING COUNTY HYDROLOGIC MODELING PROCESS

Figure 5 may be boring to look at, but it is a convenient tool for talking about the level and type of effort we expend to build an HSPF watershed model, some of our underlying assumptions in applying the model, and how HSPF itself generates stream and river flow.

Like many rainfall-runoff models, HSPF requires two types of data, spatial and temporal. The spatial data characterizes the watershed in terms of the drainage network and its individual water features, topography, cover, surficial geology or soils. The temporal data are climate data (generally precipitation and pan evaporation) and sometimes stream or river inflows from a gaged or previously modeled upstream basin area. Delineation of a basin plan area is usually done on a watershed basis by selection of a creek, lake, or river with a drainage area of from 10 to 100 square miles. The basin boundary and subcatchments are roughly delineated based on USGS topographic maps and are subsequently refined as a result of stream walks and drainage inspections of the entire area. Subcatchments are delineated by defining hydraulic controls and hydrologic interest points such as outlets of lakes or ponds and confluence points in the drainage network or sites with particular drainage problems. Lands that drain to the this point and that are downstream of any adjacent upstream subcatchment define the subcatchment boundary. In our models, subcatchment areas range in size, but generally fall between 0.5 and 1.0 square miles- usually enough area in average water years to generate small, but detectable perennial stream flow.

Once the subcatchment map has been delineated, a county geologist surveys and maps the surficial geology of the area. The subcatchment maps and geology maps are digitized into GIS data layers. This information is combined in the GIS with LANDSAT- interpreted land cover classes and slope classes derived from USGS DEMs to create the HSPF PERLND/IMPLND composition of each subcatchment. PERLNDS stand for pervious land segments (IMPLNDS for impervious land segments) and are the fundamental, one-dimensional, hydrologic response units from which HSPF calculates surface runoff, interflow, and groundwater discharge. Each PERLND has its own unique parameter set defining its behavior with regard to interception, depression storage, overland hydraulics, infiltration, evapo-transpiration, deep percolation, etc.

For hydrologic modeling purposes in King County, all surficial geologic classes are composited into 4 groups (till, outwash, hydric, bedrock), 3 slopes (flat, moderate, steep), and 4 covers (forest, suburban grass, rural pasture, impervious). Theoretically, if all combinations existed, we would then need to have the model compute the hyrologic response of 48 different PERLNDS, but since many combinations do not occur, we effectively use 17 PERLNDS countywide and often need only 8 or 9 in any given basin study.

In addition to the PERLND areas for each subcatchment produced by the GIS, the model requires routing information for each drainage element. Typically, runoff components (surface, interflow, groundwater) from the appropriate land area of each PERLND within a given subcatchment is routed into a routing reach representing a stream, pond, reservoir, or lake. The model performs a level-pool routing that is based on a user-supplied rating table of storage, elevation, and discharge. This generally requires a rough field survey to determine hydraulic characteristics of lake outlets, culverts, and long channel reaches. For our major creeks and rivers where we have performed HEC-2 backwater analyses, HSPF rating tables will be based on the output from those studies. This approach was utilized in the hydrologic modeling of the Cedar River, one of the two major river systems wholly contained within the county's borders.

HSPF CALIBRATION

Once spatial representation of the watershed is complete, calibration of the model is possible. For this purpose, we generally utilize two years of 15-minute frequency contemporaneous precipitation and stream flow data, usually from our own county-installed gages. Our gaging density is variable, but typically we install a stream gage for every 10 square miles of watershed area and sufficient rain gages such that no point in the watershed is further than two to three miles from the nearest gage.

Calibration of the HSPF model varies from a relatively simple, rational process for 'well-behaved' basins (see Figure 6) to a complex, frustrating, budget-busting odyssey requiring application of some of the blackest arts that the experienced hydrologic modeler is willing or able to conjure (see Figures 7 and 8a-8c). One reason for difficulties in model calibration is the large number of active parameters utilized by HSPF. While related to hydrologic processes, they are not physically measurable in the field and therefore must be calibrated. Convergence to a uniquely optimal parameter set is far from rapid and utilization of any automated optimization routine promises to be hugely consumptive of computer time and probably doomed to failure.

Luckily, in Puget Sound country we start with the benefit of a set of default, generic, PERLND parameters developed in cooperation with the Tacoma office of the USGS (Dinicola, 1990). Locally, these are known as the regional parameters. These parameter values represent global optima from a calibration of several small basins in Pierce, King, and Snohomish counties using two years of site specific rainfall-runoff data from water years 1985 and 1986. While it has been our rule to perform our own site specific calibrations for basin plan areas, we have actually found in some cases that the regional parameters give a very good fit between simulated and measured stream flow hydrographs (see Figure 9). This usually occurs in stream basins strongly dominated by glacial till soils which luckily cover a majority of our watershed areas. Thus, often, with very little adjustment of one to three key parameters, we are able to get a reasonable fit of instantaneous peak discharges, daily, monthly, seasonal, and annual mean flows. Basins containing significant quantities of coarse, glacial outwash soils are often more difficult to calibrate and are not well-represented by the regional parameters. Such basins often have losing or gaining stream reaches, springs, groundwater losses or gains, and other complex phenomena that are not properly characterized by our maps and field data. Such cases not only require more active deformation of the regional parameters, but adjustment of routing networks and possibly reach routing tables if some reasonable physical hypothesis can justify it. These more 'heroic' calibration efforts are risky with only two years of data and often end up being futile or just plain erroneous when checked against data from a subsequent water year. A great deal of judgement, self-control, or sometimes plain exhaustion is required to suspend calibration runs and start to ask questions about the reliability of the gage data, or the ability of the underlying conceptual hydrologic model to represent the complexities of the basin of interest (see Figure 10). Still, HSPF is so flexible, that if you have a hypothesis about how a given basin absorbs, redistributes, stores and releases water, there is usually a way to configure the model to represent it.

PRODUCTION RUNS

Once a basin model is calibrated, we proceed with production runs using National Weather Service (NWS) precipitation data from one of two long term records, (43+ years) one at Seatac Airport and one at the town of Landsburg at the eastern edge of our service area. In addition, 25 years of daily pan evaporation records from the NWS site at Puyallup have been extended and filled for the winter months using a Jensen-Haise relationship to parallel the long term precipitation records. These data are used to drive our basin models for 43 years using an hourly time step. Production run precipitation is adjusted by a constant factor based on correlation of the calibration precipitation data to the closest long term gage record. The raw output from these production runs are 43 years of hourly stream flows, lake, and wetland stages at the outlet of every defined subcatchment. During the problem analysis phase of a basin plan, a set of four different long term simulation runs is made, one reflecting pristine land use conditions in which all constructed impervious is converted back to assumed forest and wetland cover, one current condition run reflecting land use as shown by very recent air photos or landsat image analysis, one future conditions run in which the basin is assumed to be urbanized to the fullest extent allowed by current zoning without the benefit ofany project runoff-control facilities, and another future conditions run with standard runoff control facilities as required by our countywide drainage code and design manual. These raw, simulation data support a variety of hydrologic analyses, drainage design and watershed planning purposes.

EXAMPLES OF WATERSHED MODEL OUTPUT IN BASIN PLANS

Subbasin Prioritization

As mentioned earlier, one of our major concerns are effects of forest clearing and urbanization on all aspects of creek flow, especially creeks used by salmon and trout. One of the primary purposes of these plans is to perform a kind of triage that classifies streams, stream reaches, or other aquatic resources as 'severely degraded', 'degraded but salvageable', 'minimally degraded, but at risk' and 'undegraded and safe'. Figure 11 provides a highly summarized analysis of the effect of current and future urbanization on 25-year return period, peak creek flows for the major tributary streams of the lower Cedar River basin. This graph is based on flood frequency analysis of the HSPF model simulated flows at the confluence of tributary streams with the Cedar River mainstem. Ginger Creek at the bottom of the graph is the most downstream and completely urbanized tributary creek in the basin. Note that a value of 1.0 on the x-axis indicates a low 25-year peak associated with pristine, completely forested conditions. Currently, and in the future regardless of mitigation, the Ginger Creek's 25-year flow is more than 2.5 times its pristine flow. The position of the bars indicates that the creek's watershed is nearly completely built out and that their is little overall value in requiring even standard R/D mitigation- thus Ginger Creek falls into the 'severely degraded' category. At the other end of the spectrum is the Walsh Lake Ditch with a catchment located almost completely within Seattle's protected Cedar River Watershed. In between there are two other types of subbasins- ones where countywide standard R/D ponds largely prevent increases in peak discharges and ones where those standards fail and more prescriptive measures need to be developed for the basin plan. This last category of subbasin occurs because countywide standards excuse small subdivisions and owner-builder projects from providing drainage mitigation for their projects. However, in some of the more rural areas, such projects account for the majority of projected land use change. This type of information is used in combination with aquatic habitat and stream stability surveys to target different tributary areas of a basin for special treatments such as enhanced R/D standards, or where structural measures are inappropriate, forest-retention requirements, or even down-zoning recommendations. The hydrologic benefits of measures such as these are easily analyzed by modification of the basic model and additional simulation runs.

Analysis of Urbanization Effects Mainstem River Floods

Another example from the Cedar River basin plan involves the effect of urbanization on the mainstem Cedar River flood peaks and peak flow durations. These are of keen interest in our region because the river provides habitat to an extremely valuable sockeye salmon run with productivity that has been negatively correlated with high discharges in the river. Additionally, the floodplain of this river near its outlet into Lake Washington is the 'habitat' of the Renton Municipal Airport and Boeing Commercial Airplane Group which begin to sustained down time and/or flood damage during 2-year and 15-year events respectively (King County, 1993). Consequently, it was important to determine what leverage land use and drainage controls might have on preventing further aggravation of flood peaks and durations. Note that the upper two thirds of this 188 square mile river basin lie within Seattle's water supply watershed, are forested and presumably protected from future development. Figure 12 shows flood frequency curves at the outlet of the river for the four previously described scenarios. In these four HSPF simulations of the lower basin, inflows from the upper basin are kept constant assuming rule-curve operation of Seattle's Masonry Dam. While the curves do show that peak flows have increased and will increase moderately in the future as the lower basin continues to urbanize, it also shows that R/D ponds do not help reduce peaks at the mouth of the river. In fact, as shown in in Figure 13, R/D ponds can even aggravate flow durations above certain thresholds. It was concluded from these results that R/D ponds would not be effective in solving flooding problems at the mouth of the river regardless of their effectiveness in protecting the river's tributaries.

EXAMPLES OF WATERSHED MODEL USE IN COUNTYWIDE ANALYSIS

In addition to basin plans, hydrologic modeling is also being utilized on a county-wide basis through the King County Surface Water Design Manual (1990) and in comprehensive resource analysis taking place as a cooperative effort with the municipalities within out county.

Development of KCRTS

One of the most important changes that will be incorporated in our revised 1995 manual is the switch from the SCS-SBUH methodology of design-storm analysis to a continuous modeling approach using a database of different PERLND unit area discharges generated from HSPF with regional parameters. The new computer program known as the King County Runoff Time Series (KCRTS) will be used by land developers to determine the downstream impact of their projects and design appropriate mitigation. The method and software improve on previous techniques because it results in R/D pond designs that more accurately meet stated performance criteria, allows the design of flow duration-matching ponds, and generally allows for more flexibility in the types of hydrologic analyses that can be performed. Because of an increasing interest and understanding of the interaction between land runoff and aquatic habitat health, we simply need to know more, and mitigate for more than the peak discharge generated by a 10-year, 24-hr, design storm.

County-wide R/D Standard for Sensitive Lake Catchments

Continuous hydrologic modeling using HSPF is also being used to develop and test a special R/D design standard for projects discharging to lakes, ponds, closed depressions, and open water wetlands where increases in the frequency and durations of flood stages and discharges can not be tolerated because of damage to shoreline or downstream aquatic resources, public or private property.

County-wide Assessment Urbanization, Structural, and non-Structural Mitigation Alternatives

We have carried out a series of generic urbanization simulations using HSPF to try to define the relationship of R/D ponds, forest retention, and percentage catchment impervious area on the health of aquatic resources in King County. This analysis is based on field work that shows that when the duration of flows above the pristine 2-year return period flow are increased more than 100% by urbanization, small streams in King County tend to become unstable, erode their bed and banks, and degrade aquatic habitat (Booth, CFC, 1993). With some additional thresholds assumed for loss of summer base flow and water quality impacts, a very approximate screening-level view of active and passive mitigation alternatives is possible. What results are a series of nomographs that can be used by policy makers and planners to rapidly assess the current state of aquatic resources and plan future structural and non-structural drainage mitigation measures. Figure 14A shows aquatic resources may be preserved in the absence of detention ponds, but only if greater than 50% of forest cover in a stream's catchment is retained and if impervious area is less than 15% of the total catchment area. The lower line of positive slope is the locus of impervous/forest ratios that according to the model produce the field-observed threshold condition for stream stability. The higher line of positive slope defines the boundary above which summer base flows are at least 80% of their pre-developed, pristine value. Figure 14B is a similar nomograph except that a series of development scenarios with the county's base conveyance standard that requires post developed flows to be no greater than pre-developed flows for all peak annuals between the 2-year and 10-year return period discharge. Clearly the potential for excellent aquatic resources is much greater if developments are required to build R/D ponds. However, good aquatic resources still depend on the maintenance of at least 30% forest cover in a stream's catchment, and increasing impervious area above 30% depletes base flows to below 70% of their pristine condition levels. Actually, based on our experience, very few streams or ponds support good habitat when their catchment areas contain more than 10% impervious surfaces- probably as a result of nonpoint pollutant loadings. This suggests a vertical line at the 10% as a water quality threshold that might be extended to 20% if BMPs and AKART were implemented to control pollutant loadings (Lorin Reinelt, Personal Communication, 1994). Finally, as shown in Figure 14C, with enhanced R/D ponds that are large enough to control the peaks and durations of flows between half the pre-developed 2-year and the 50-year return period discharge, stream stability appears to be maintained even with high levels of impervious and low levels of forest retention; however, depletion of base flows and water quality degradation would certainly occur before a catchment was built out to 30% effective impervious area.

SUMMARY AND CONCLUSIONS

In my remarks today, I have attempted to give you an appreciation of why we use a continuous watershed model in King County, how we develop our watershed models, and the extent to which our modeling affects surface water management from technical design to countywide policy and planning. Do we think we have a perfect tool? Not at all. It is difficult to learn and costly to apply and it may contain some fundamental, alogorithmic weaknesses. Although we are currently heavily vested in

HSPF technology, we certainly welcome efforts to develop improved watershed models and are anxious to assist those efforts in any way we can including sharing data or assisting in watershed model testing.

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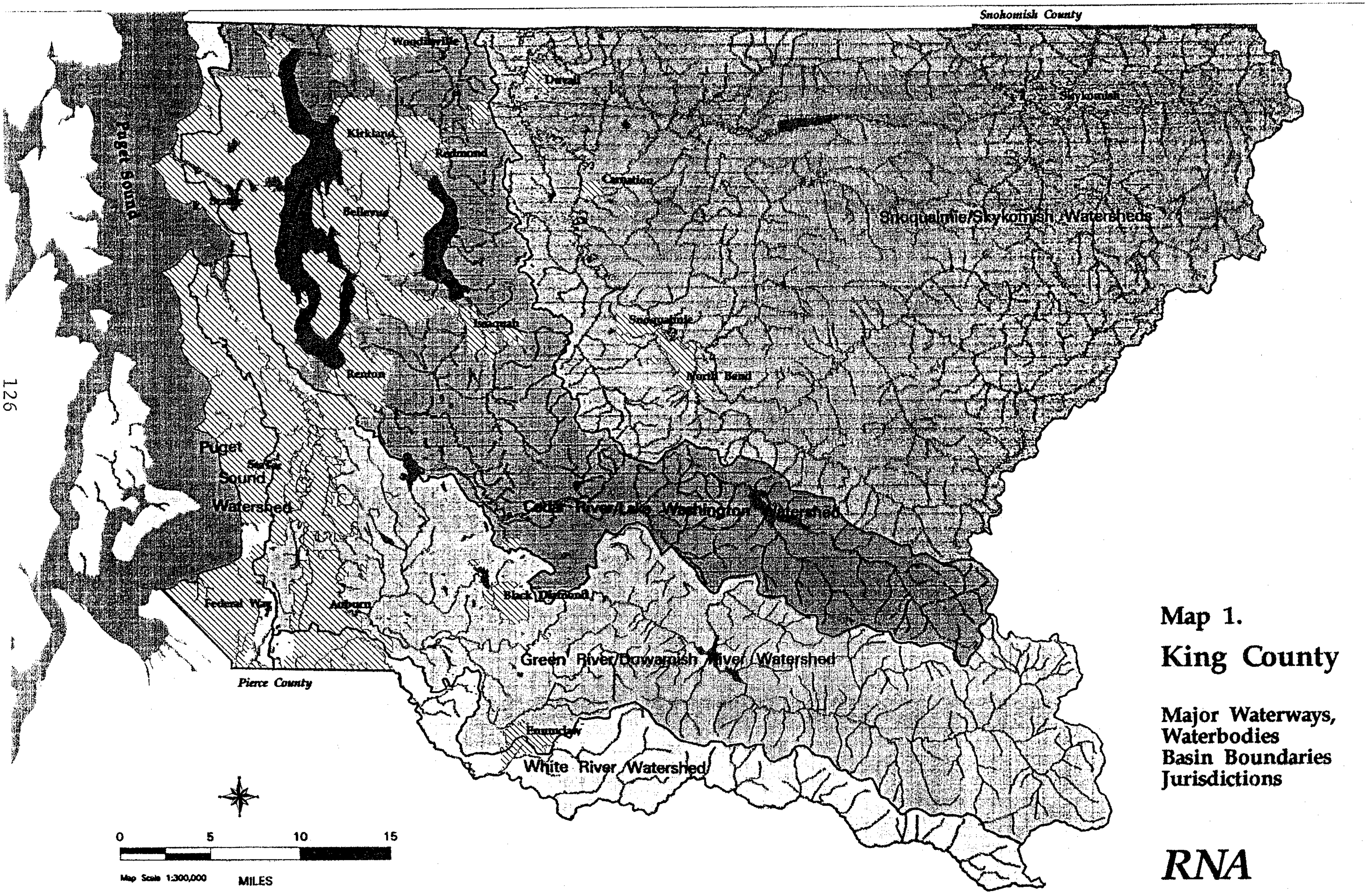


FIGURE 1. MAJOR RIVER BASINS IN KING COUNTY

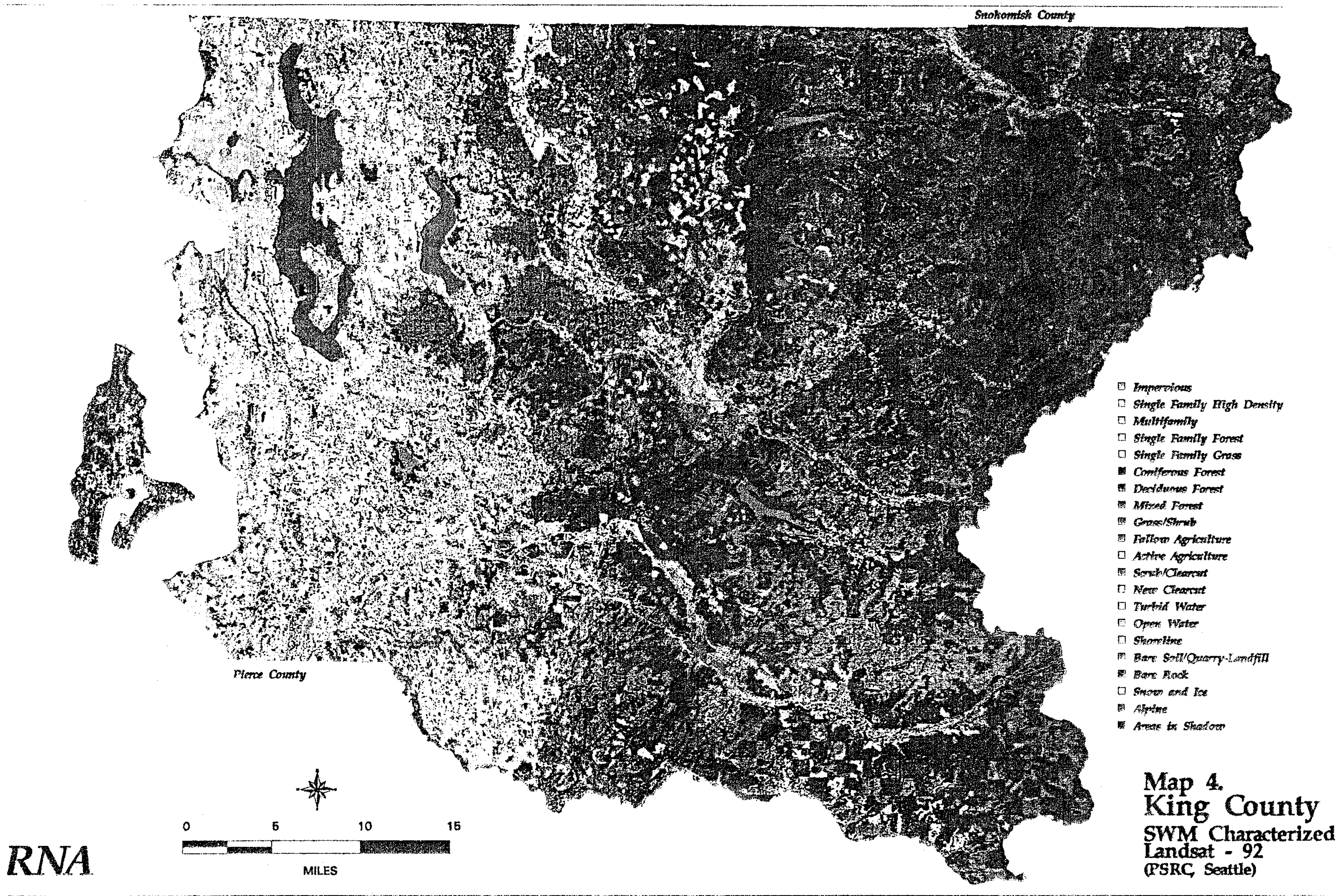
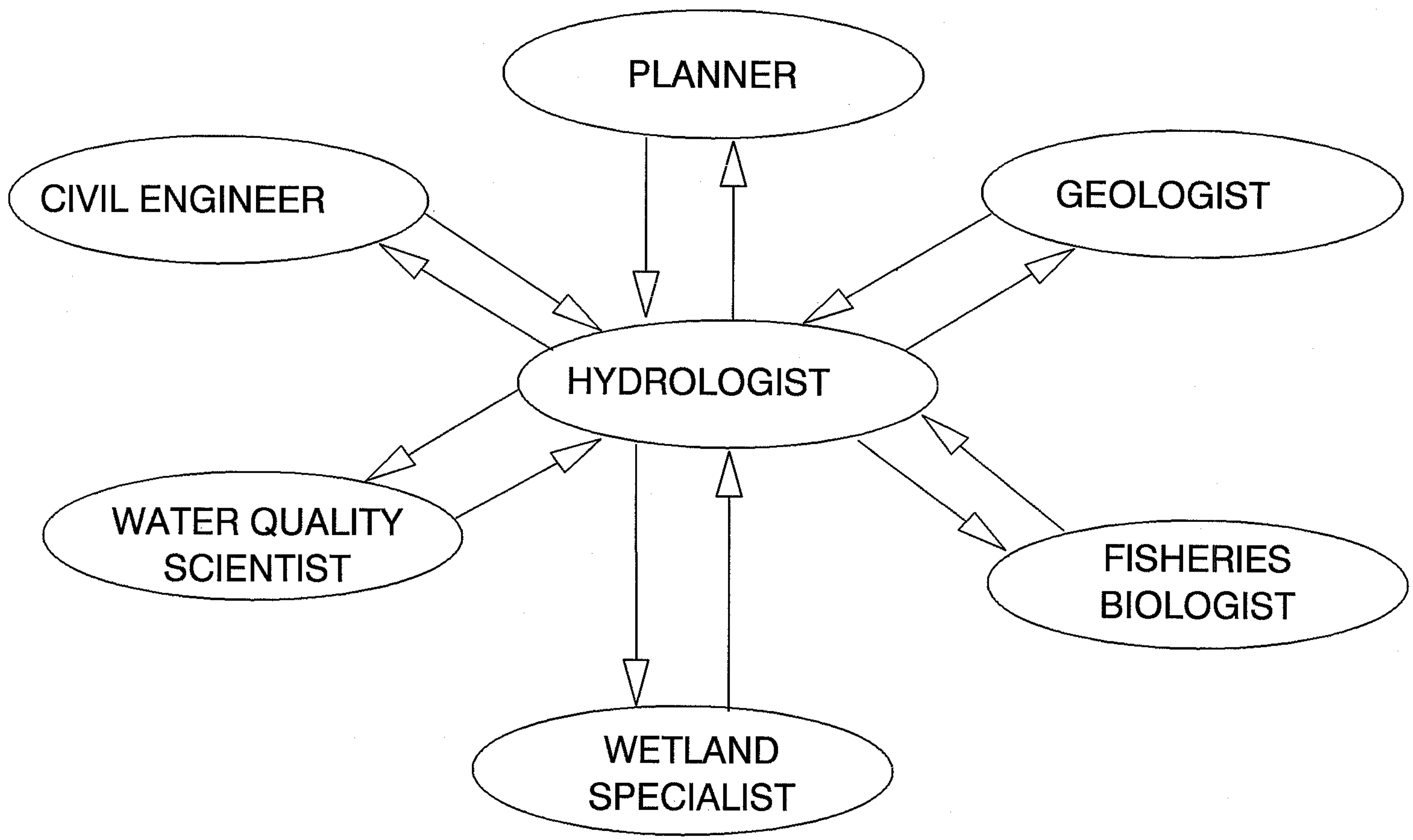


FIGURE 2. KING COUNTY LAND COVER INTERPRETED FROM A LANDSAT IMAGE

FIGURE 3. HYDRO-CENTRIC VIEW OF THE KING COUNTY BASIN PLANNING PROCESS



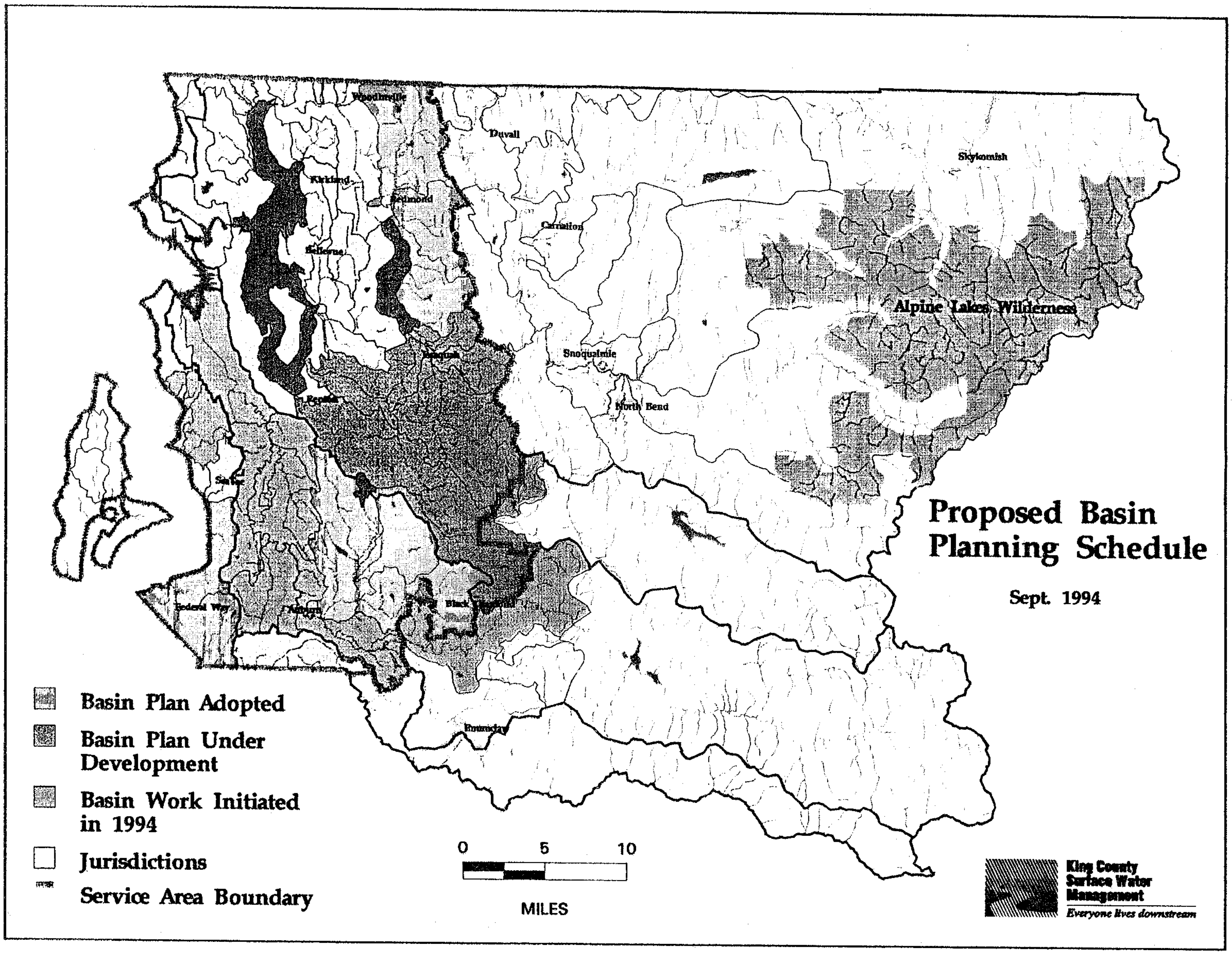
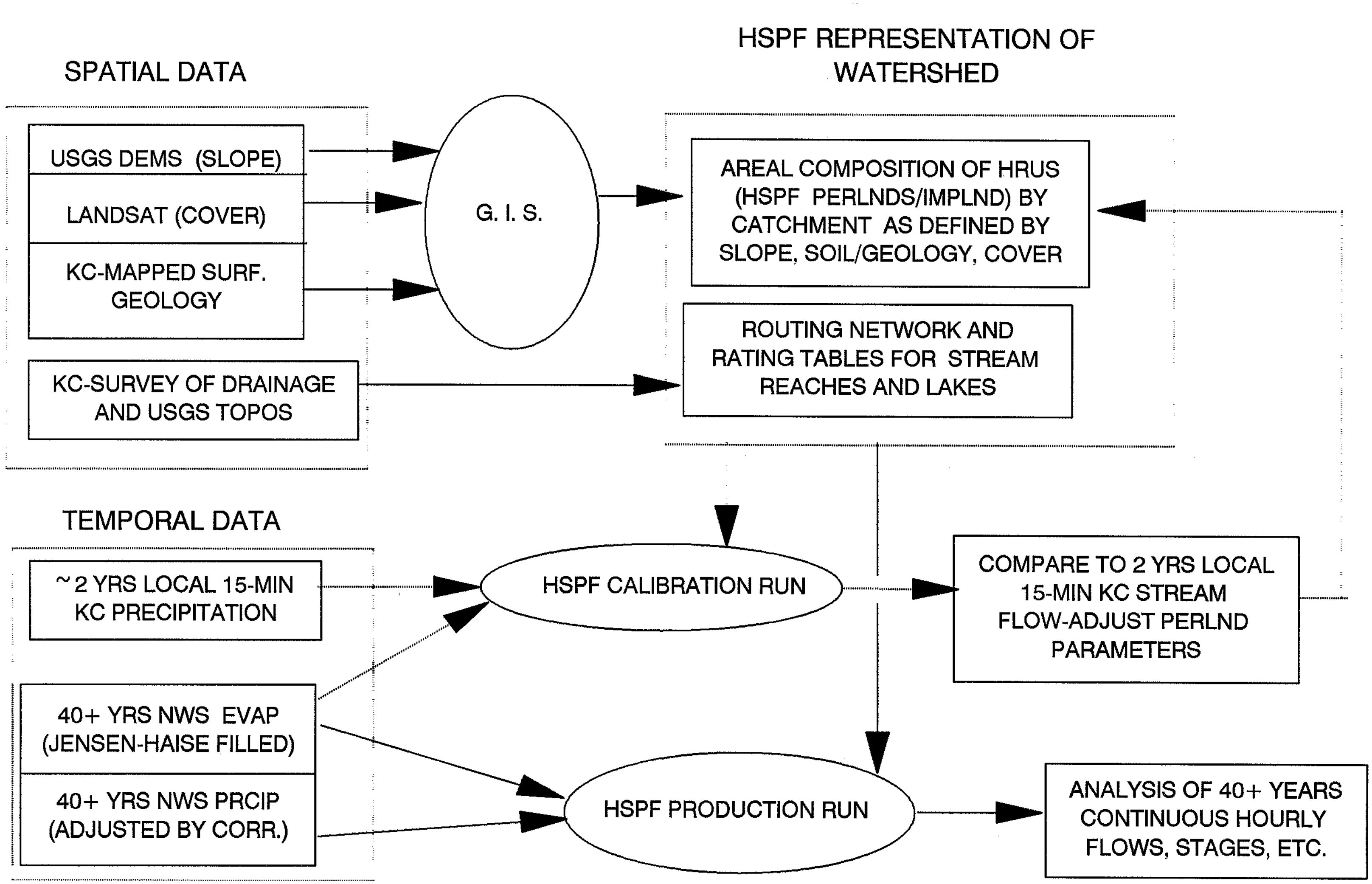


FIGURE 4. KING COUNTY SURFACE WATER MANAGEMENT SERVICE AREA.

FIGURE 5. KING COUNTY-HSPF WATERSHED MODELING SCHEMATIC



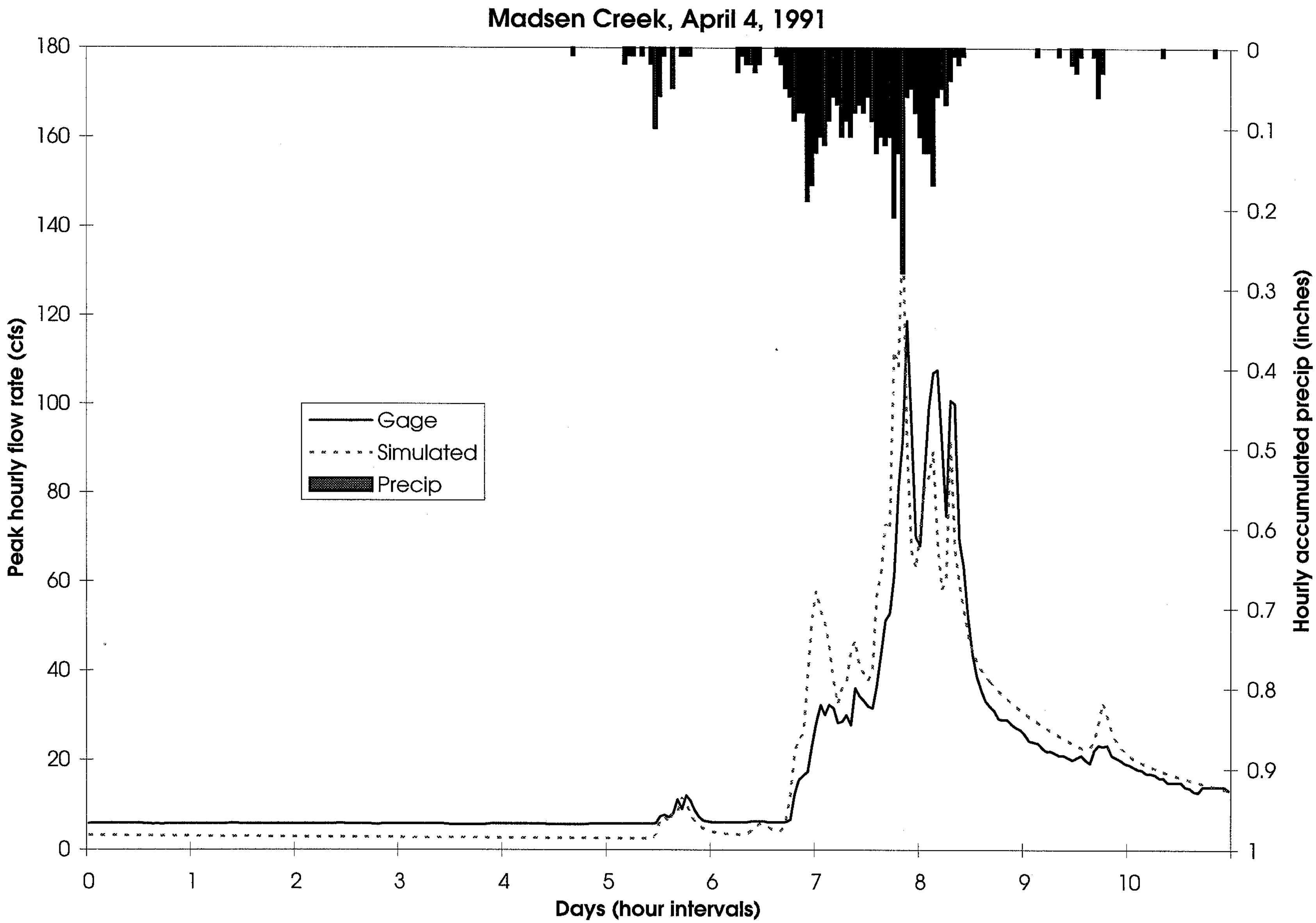


Fig. 6- Example of Good Calibration in 2.0 sq. mi. (Till Basin)

Madsen creek precip gage used

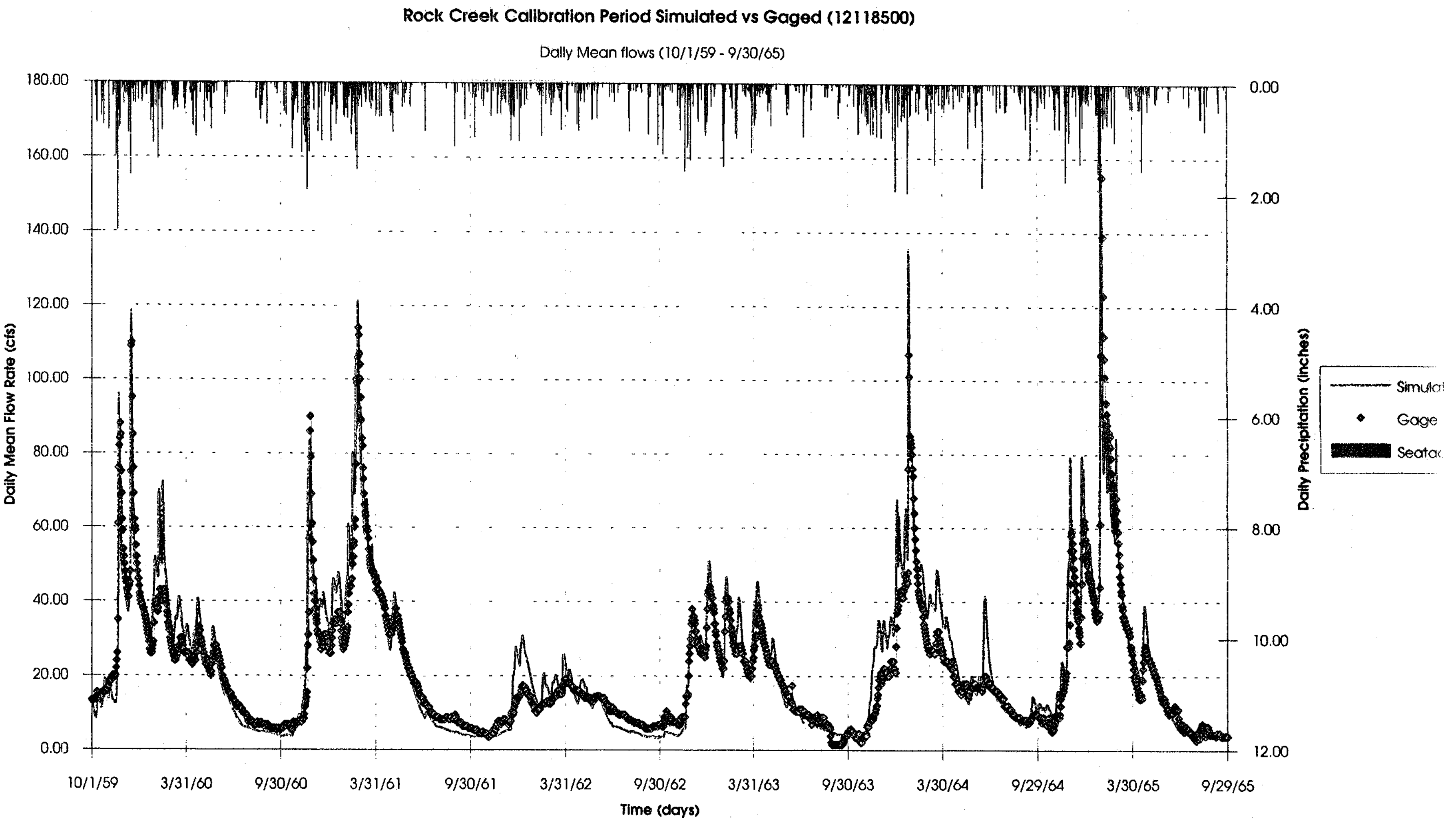


FIGURE 7.- EXCELLENT CALIBRATION OF OUTWASH-DOMINATED BASIN

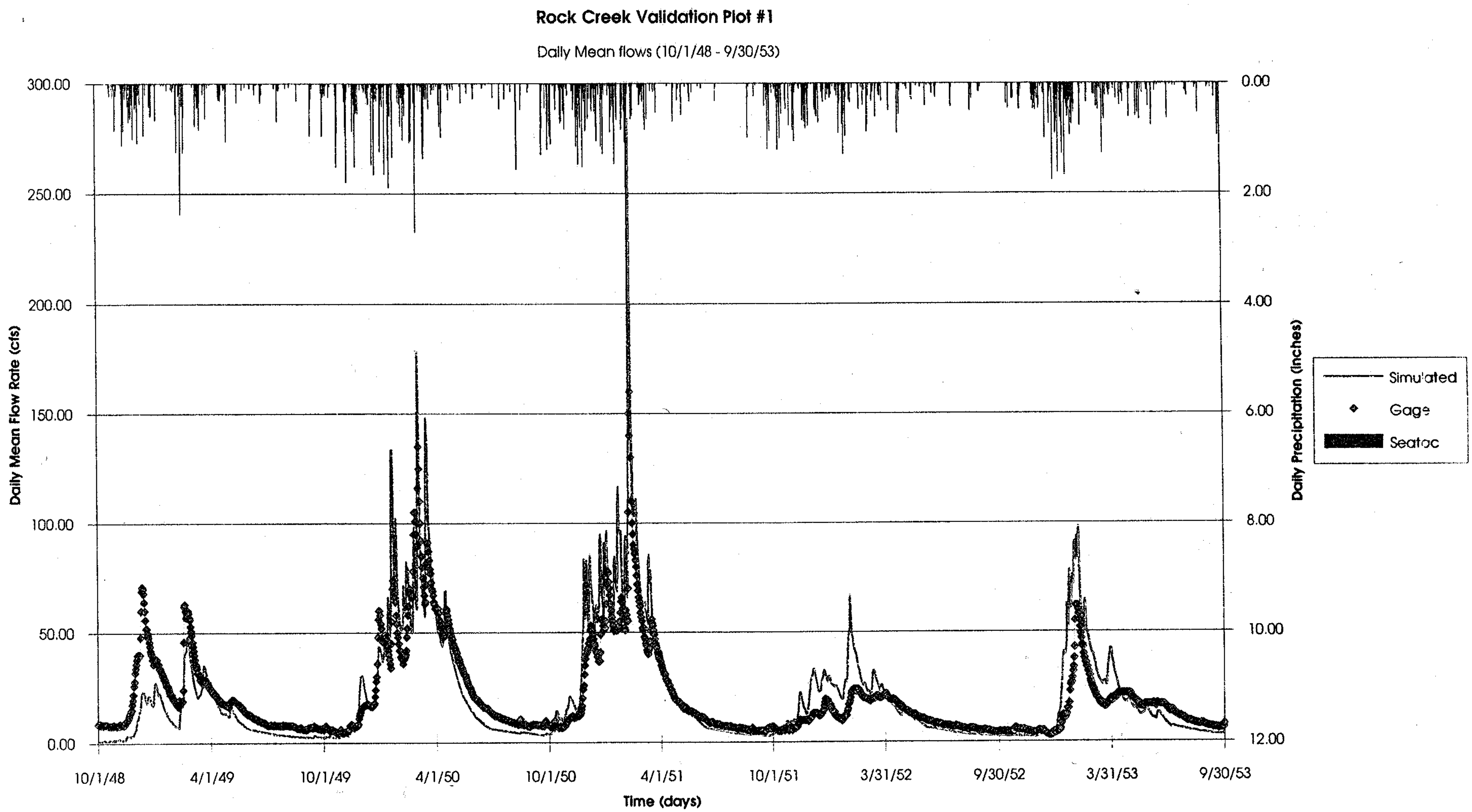


FIGURE 8a. IMPRESSIVE VALIDATION

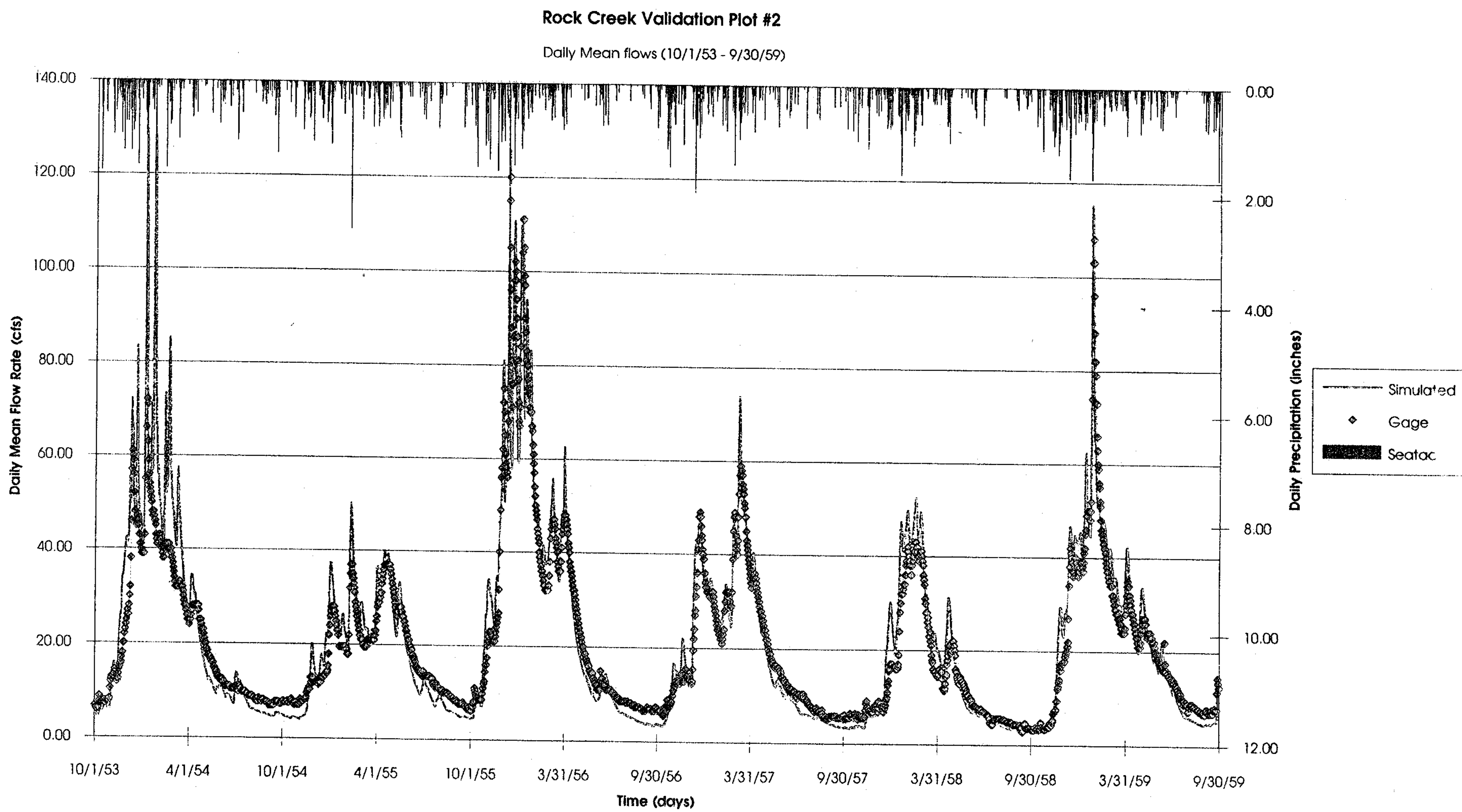


FIGURE 8b. IMPRESSIVE VALIDATION

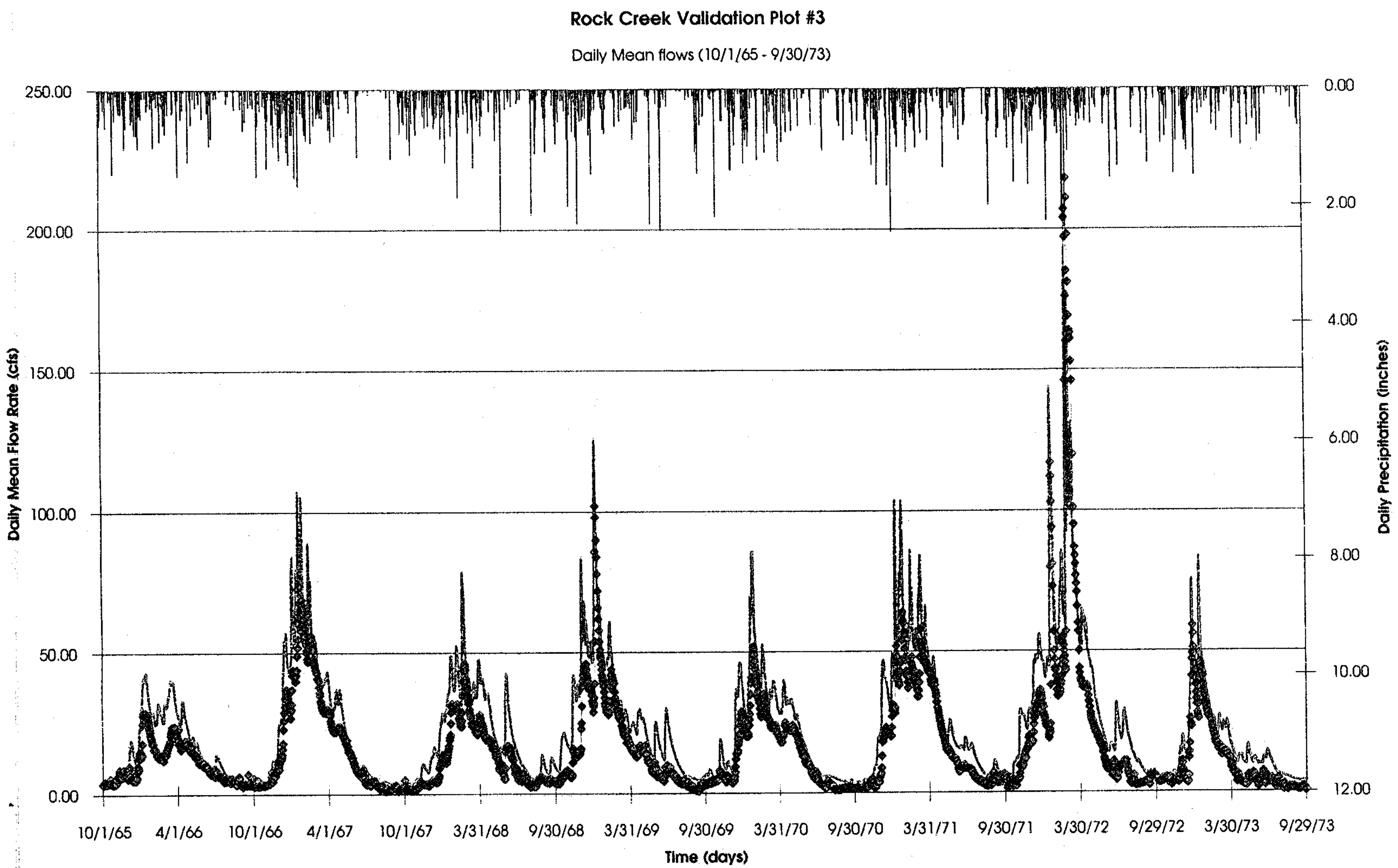


FIGURE 8c. IMPRESSIVE VALIDATION, continued.

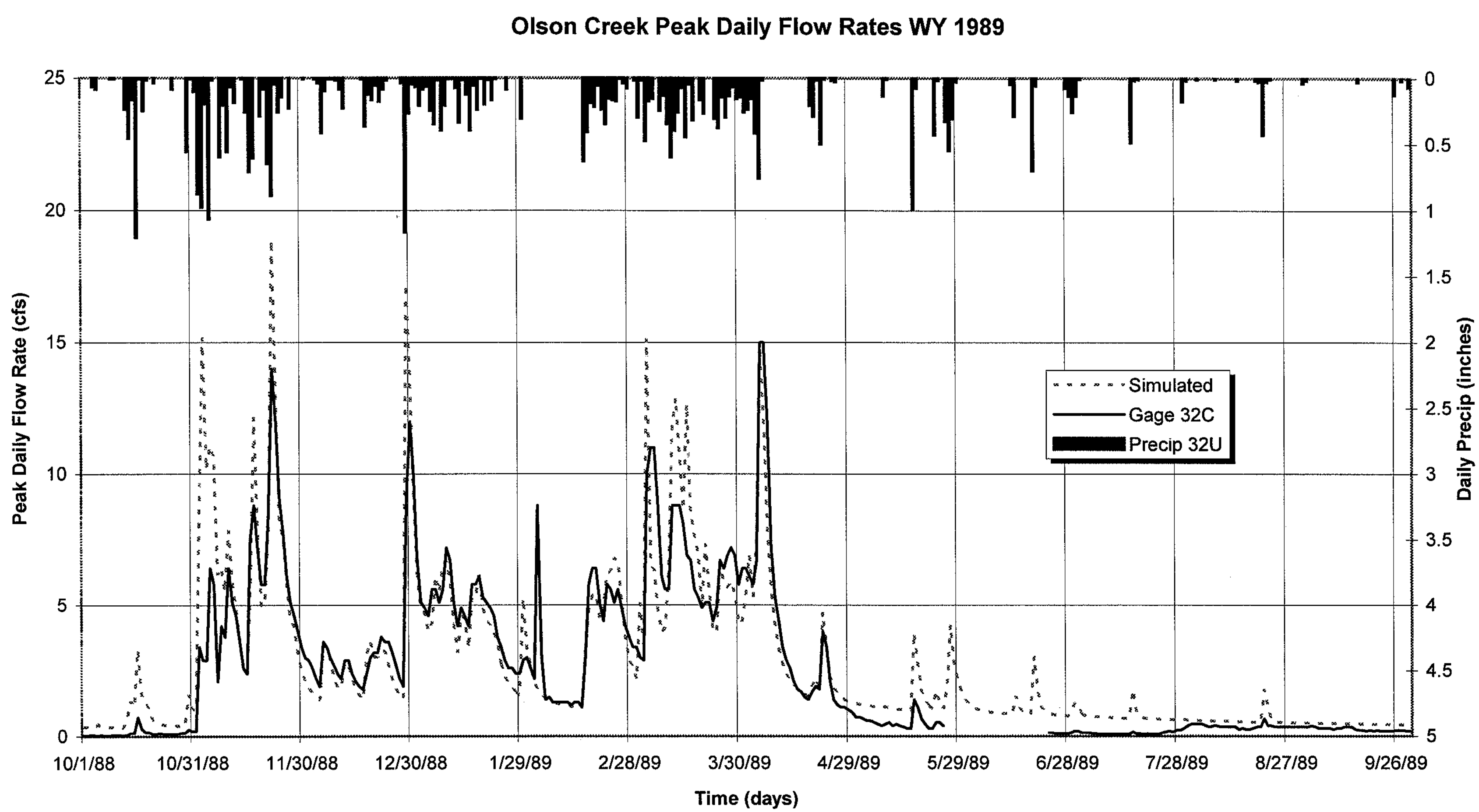
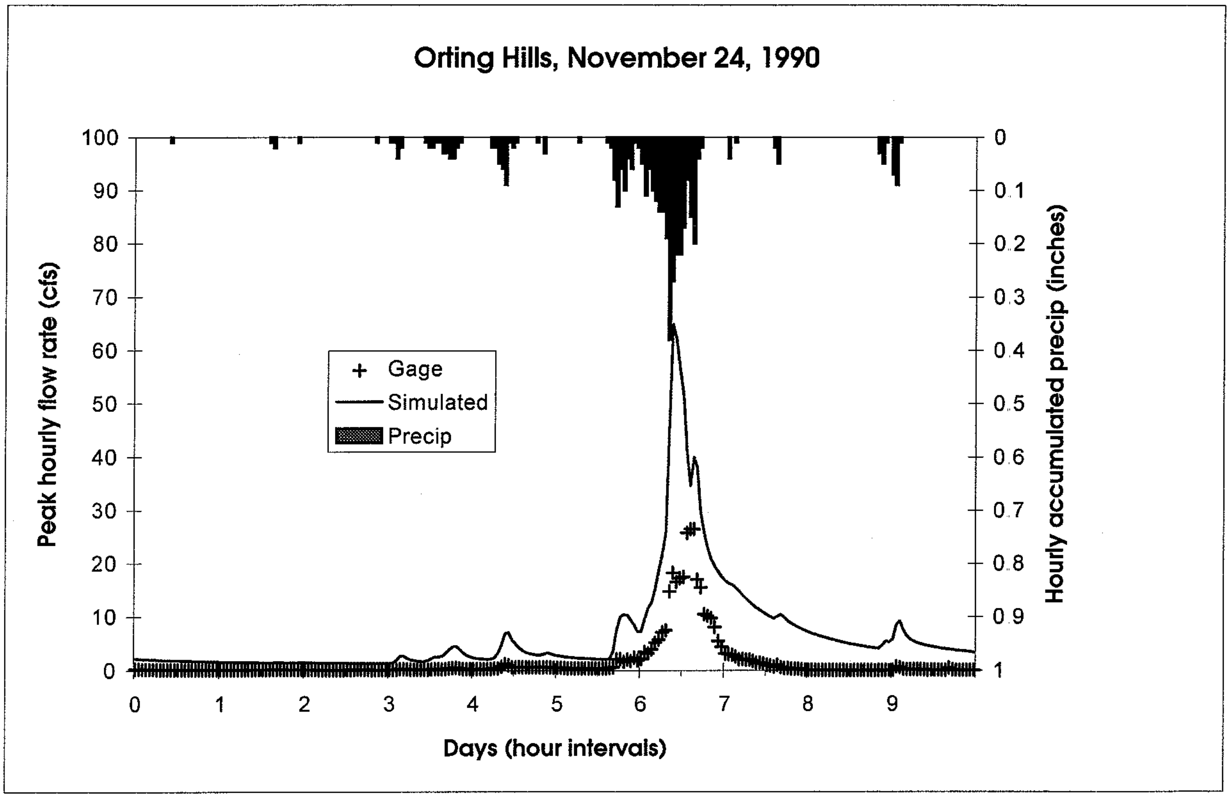


Fig. 9- Validation of Regional Parameters (Till Basin)



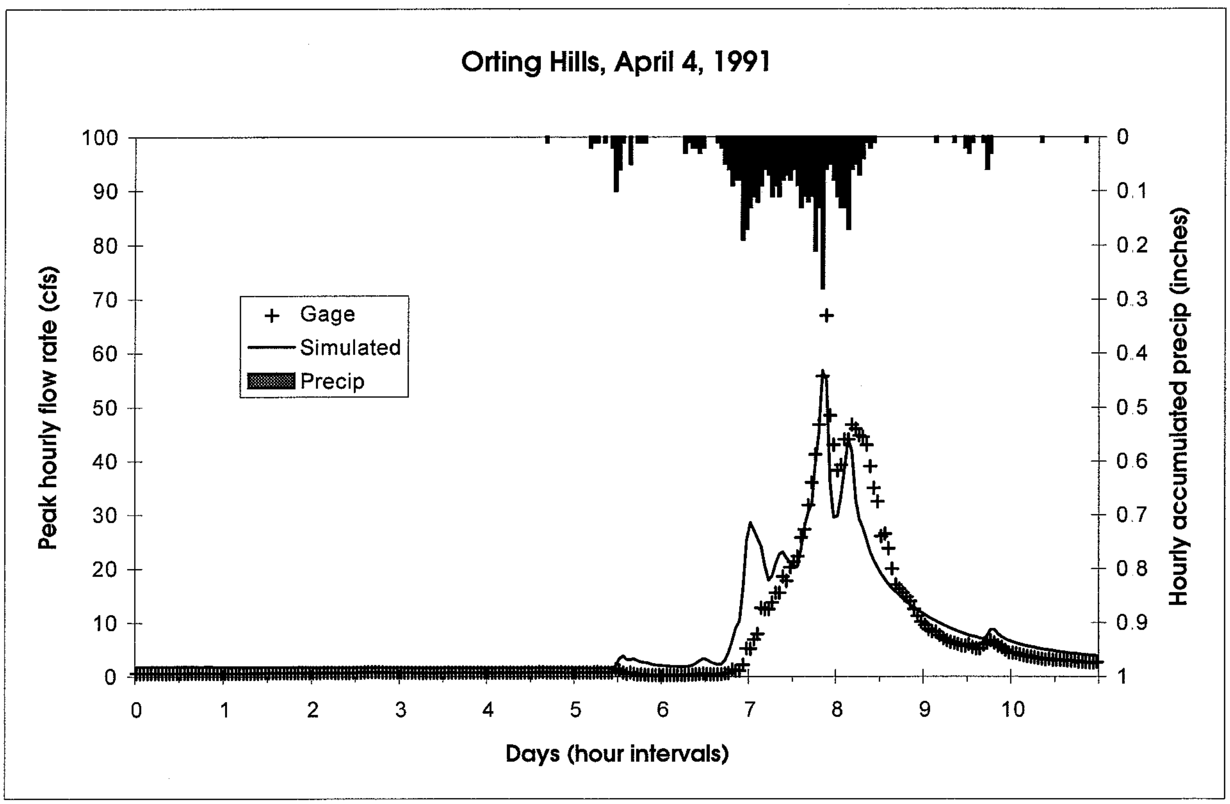


Fig. 10- Mixed Message from 30% Outwash basin "Poor" calibration

Fig. 11. Subbasin Peak Flow Comparison
(25-YEAR Q)/(FORESTED 25-YR Q)

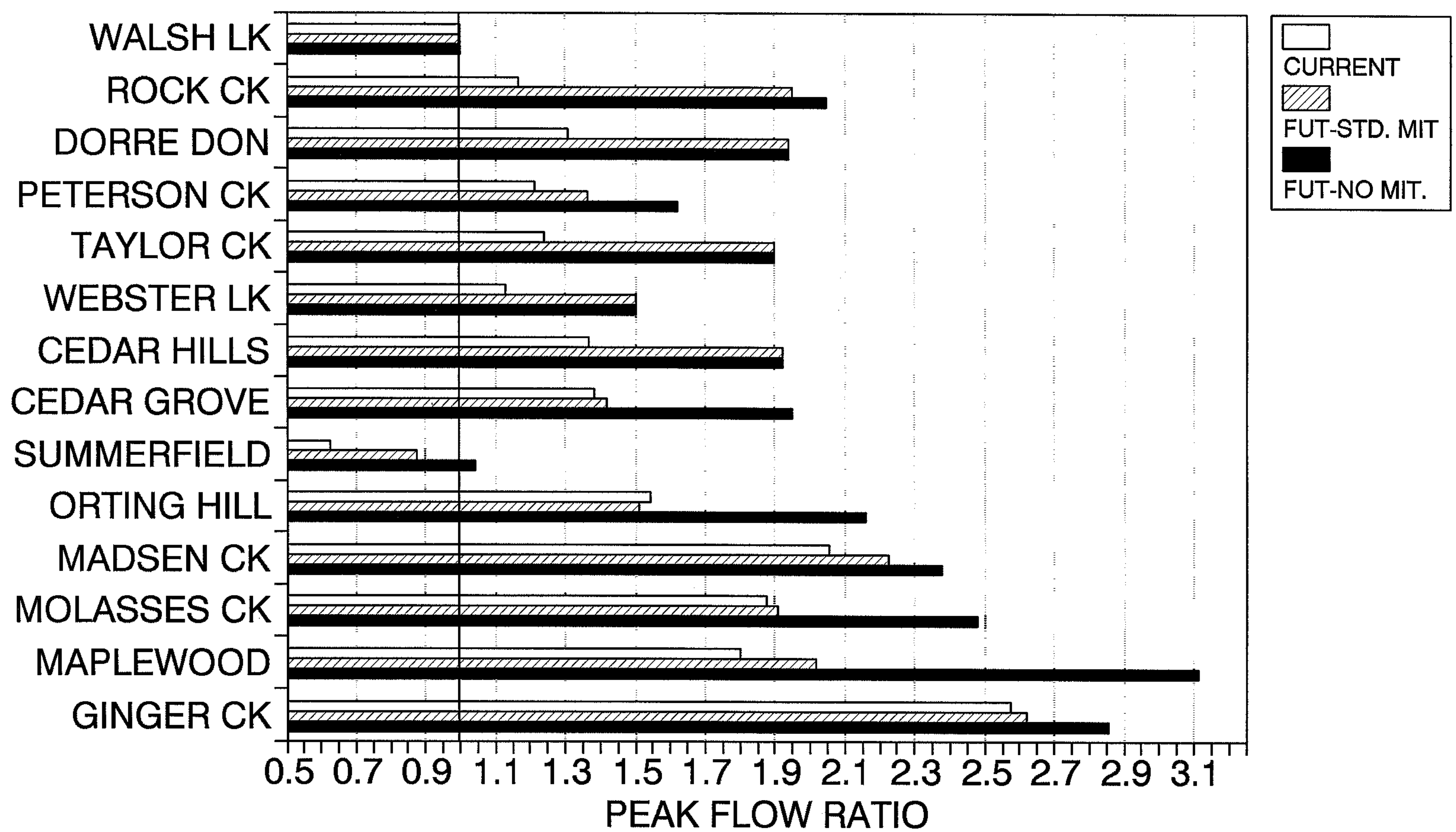


Fig. 12. Urbanization and Renton Floods

SEAFM AND HSPF SIMULATION 10/48-9/89

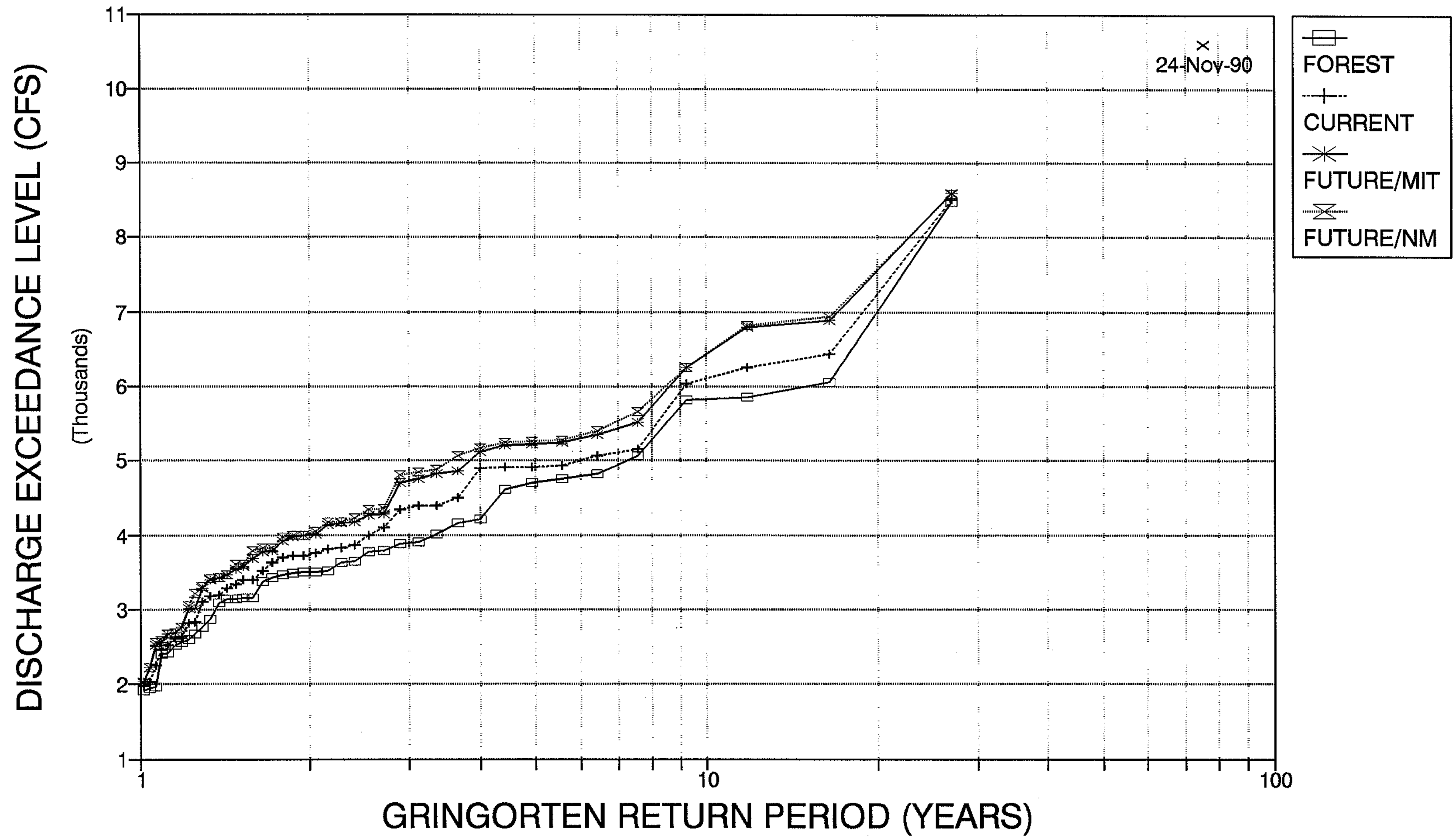


Fig. 13. Urbanization and Durations

CEDAR RIVER AT RENTON (HSPF)

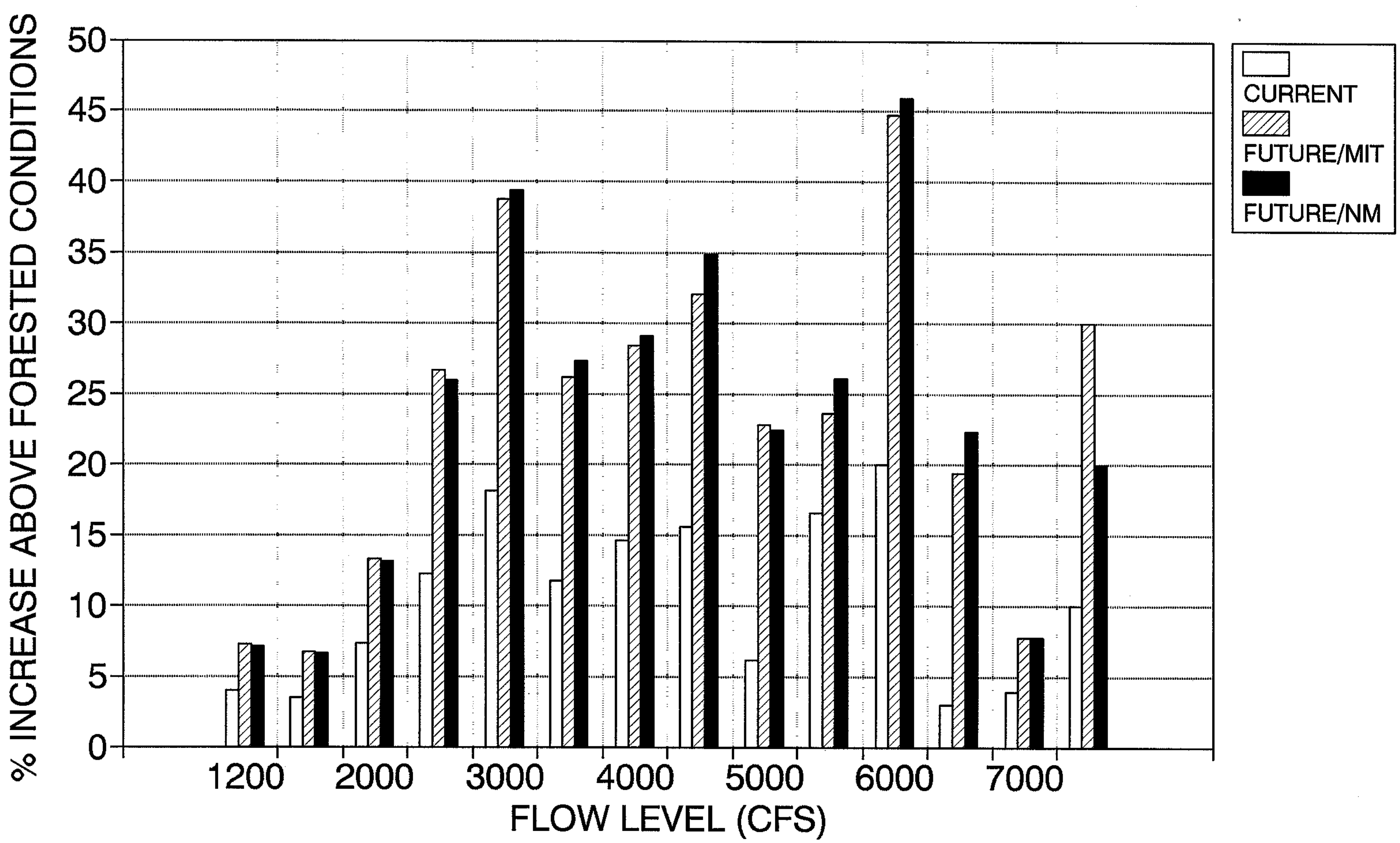


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Fig. 14A. STREAM RESOURCE CONDITION
NO DETENTION FACILITIES

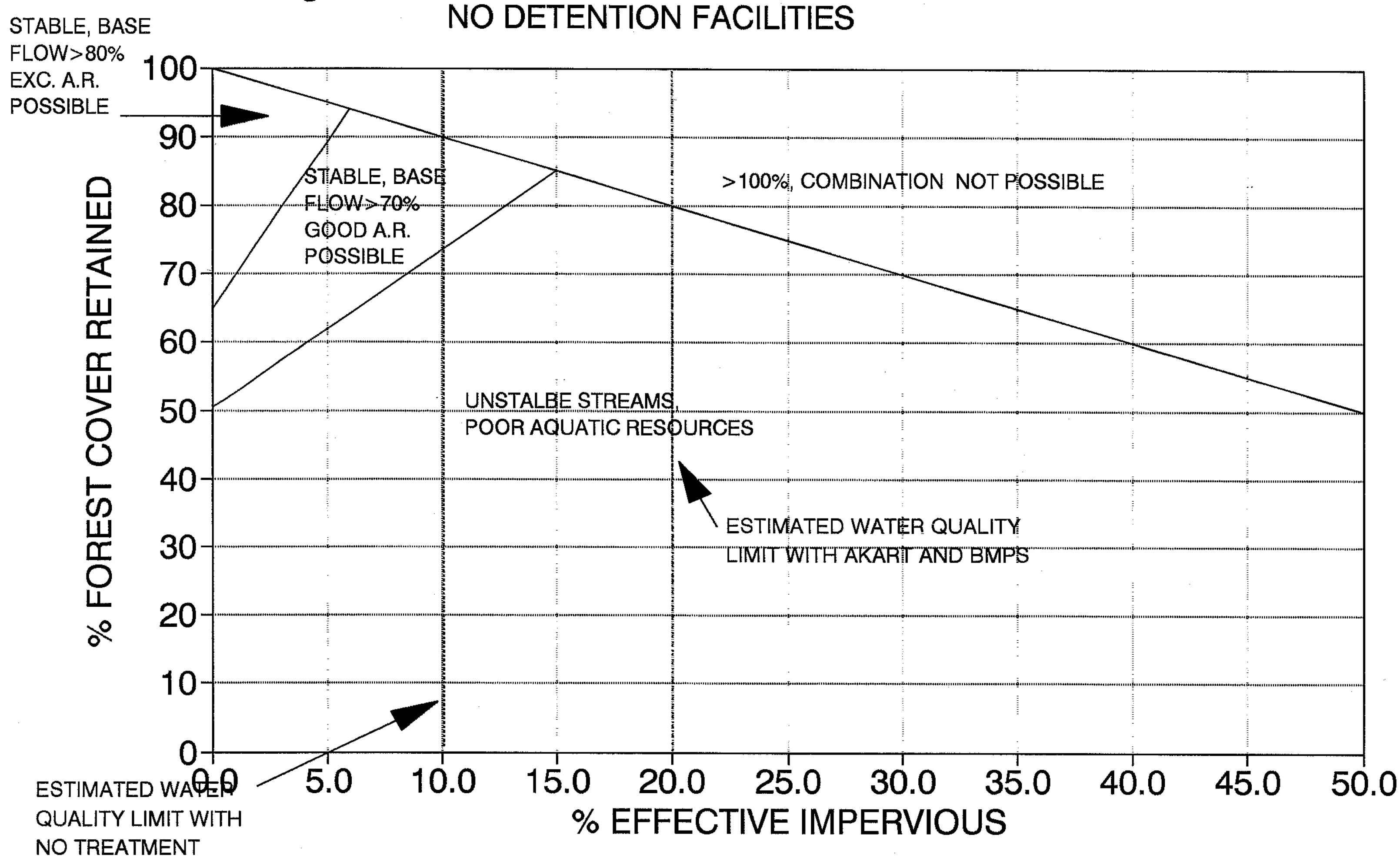

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Fig. 14b. STREAM RESOURCE CONDITION
2-10 STANDARD DETENTION (KCRTS)

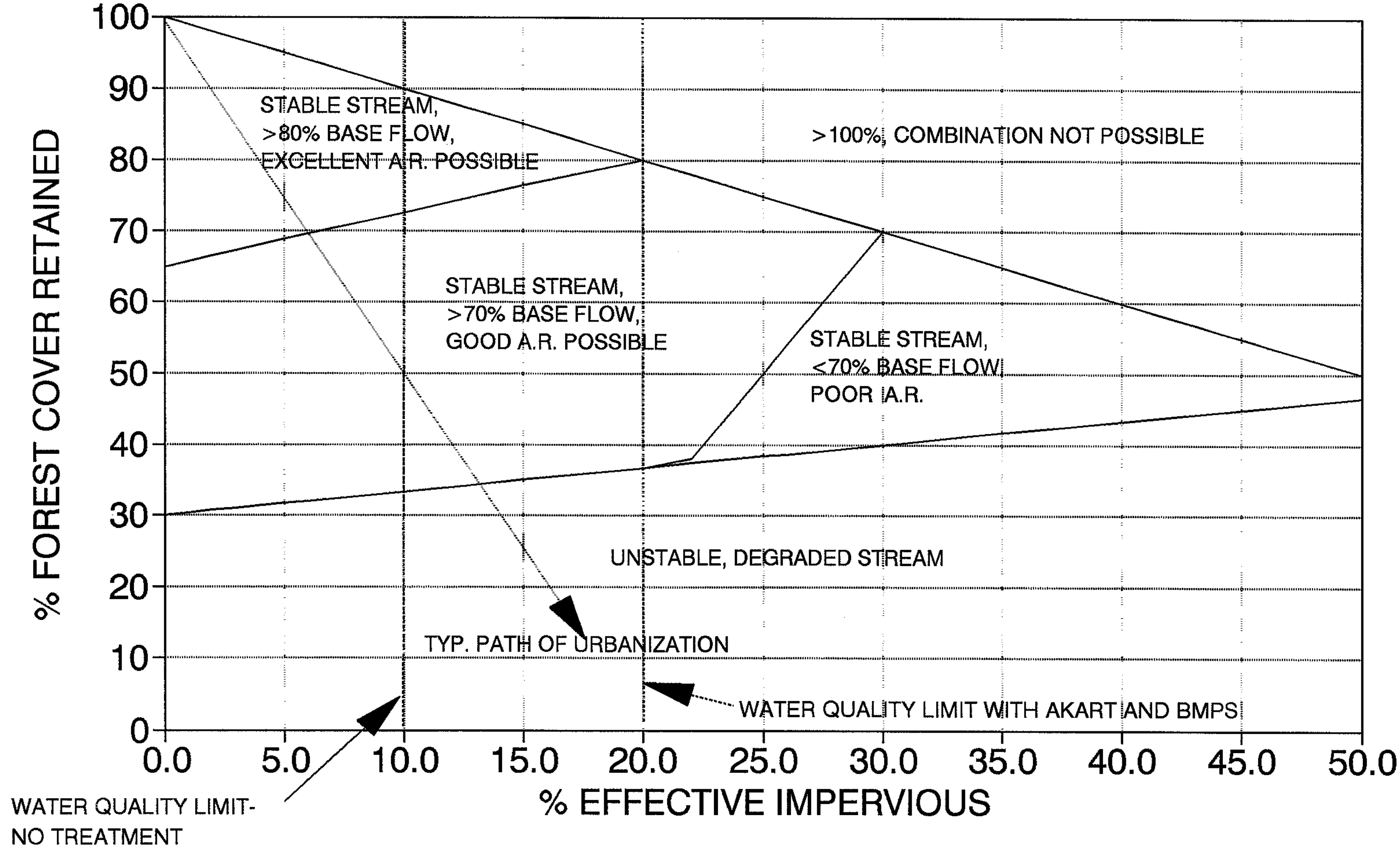

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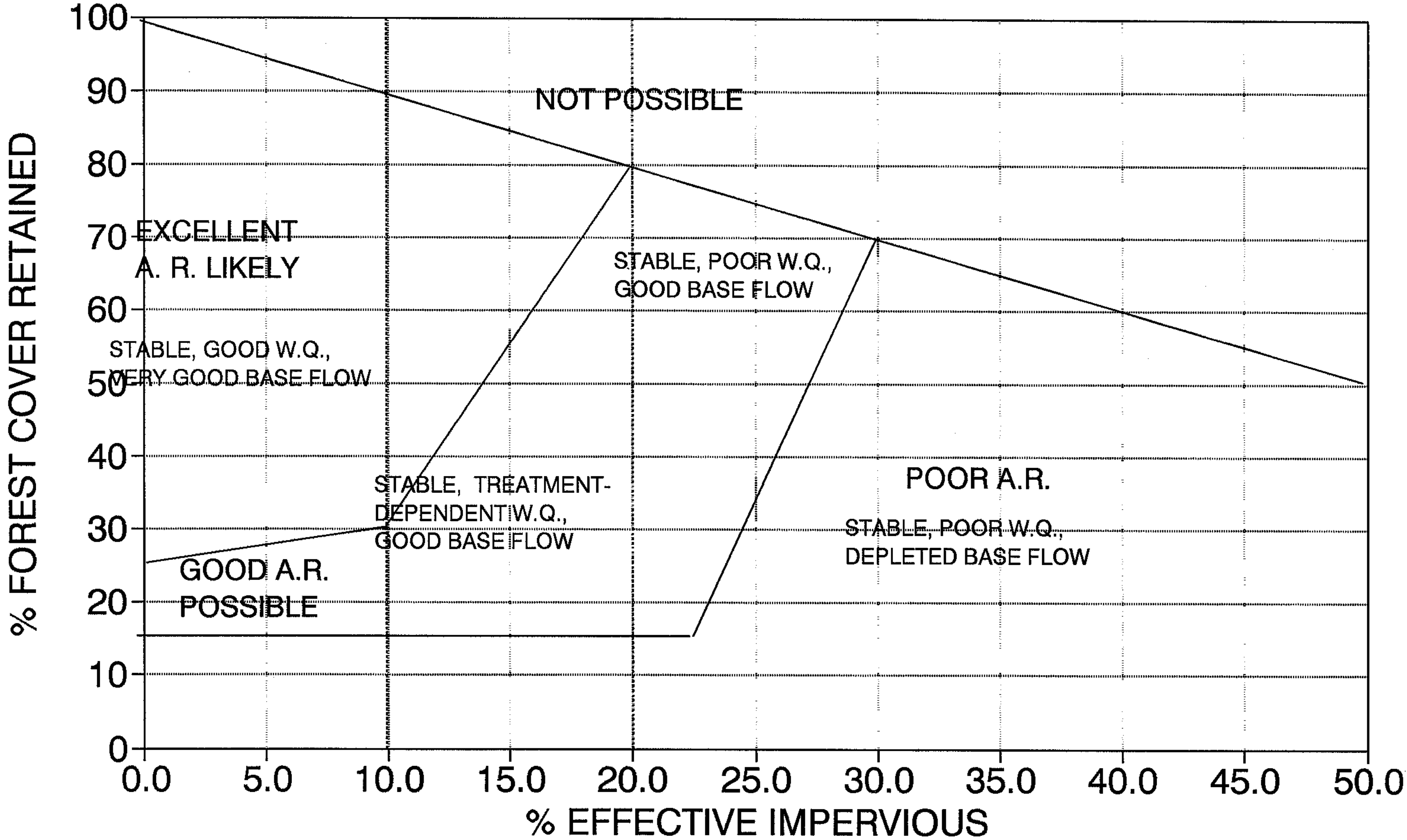
Fig. 14c. RESOURCE STREAM CONDITION
STREAM STABILITY STANDARD DETENTION



DEVELOPMENT OF HYDROLOGIC STANDARDS IN SACRAMENTO COUNTY USING HEC-1

by

Peter M. Hall[1]

ABSTRACT

Sacramento County has been designing pipe drainage systems for thirty years using design curves based on the Rational Method, the most common procedure used by public agencies for small watershed hydrology. Recent updates to the County hydrologic procedures led to the use of HEC-1, the selection of a unit hydrograph method, and a balanced design storm based on rainfall statistics. The new method produced peak flows for very small watersheds (less than 100 acres) that reflect the short duration rainfall intensities, substantially exceeding current design flows. Stream gages were placed in pipe systems to determine appropriate rainfall-runoff parameters, and the applicability of HEC-1 to model small urban watersheds. **The gage information indicated that hydrologic parameters should be verified locally, and that HEC-1 can adequately model urban watersheds, where the typical subbasin areas range from 200 to 2000 acres (0.3 to 3 square miles).** Design of pipe systems will continue to be accomplished by the current design curves; HEC-1 will be used for drainage master planning and facility design. Subdivisions will be included in the HEC-1 models as single subbasins, thereby eliminating the need to model very small subbasins.

INTRODUCTION

Sacramento lies at the confluence of two main rivers, the Sacramento and the American. The local tributaries that drain to these rivers, through Sacramento County, generally range in size from a few square miles to less than 200 square miles. They drain into the main river systems by gravity, pumps, or a combination of both. Except for the most downstream reaches, these tributaries are above backwater influences from the main river systems. Current land use varies from primarily agricultural to nearly fully developed. Certain stream groups are undergoing intense development.

In November 1989, the City and County of Sacramento jointly undertook the first comprehensive review of drainage design standards since the mid-1960's. These standards are scheduled for completion by the end of this year, a five year process. During this time, the new standards have been used for drainage master planning efforts, and the various parameters and techniques have been refined.

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INTRODUCTION (CONTINUED)

For the past thirty years the County has been designing drainage facilities using curves based on the Rational Method and regional equations. Current development regulations require a more sophisticated hydrologic method to determine the impacts of new development, and the improvement projects and policy measures required to mitigate for these impacts. HEC-1 was selected because of its wide usage and support, and because of local familiarity with HEC-1 models developed by the Sacramento District of the Corps of Engineers for two large stream groups in Sacramento. Even though HEC-1 has not, for the most part, been widely used for subdivision-sized subbasins (less than 500 acres), it was felt that with appropriate calibrations, a relatively smooth transition from the existing hydrologic procedures to the implementation of HEC-1 could be accomplished.

A computer program called SACPRE, patterned after the Corps Los Angeles District's LAPRE1, was developed to serve as a preprocessor to HEC-1. This provides for ease of input and consistent application of local hydrologic information. This paper describes the issues encountered in using HEC-1 to model Sacramento watersheds, and the efforts taken to determine appropriate parameters and to insure adequacy of results. It is an update of two papers that were presented at the 1993 International Symposium on Engineering Hydrology in San Francisco, California. (Brown and Hall, 1993, and Hall et al., 1993)

CURRENT HYDROLOGIC METHOD

Since 1961, when the current hydrologic procedures were developed by George S. Nolte, Consulting Engineers (Nolte, 1961), the County has sized drainage facilities by two methods, depending on drainage area. For watersheds less than 160 acres, peak discharge versus area curves were used based on land use and three rainfall zones. (These curves were derived from the Rational Method with frequency varying on any given curve from 2- to 10-year as the area increased.) For watersheds larger than 160 acres, regional equations were used to determine peak discharge for the mean annual, 10-, 25- and 100- year frequencies in two hydrologic areas. Factors were applied for urbanization and channelization for areas larger than 2 square miles. Smooth transition curves were used between 160 acres and 2 square miles.

HYDROLOGIC METHOD USING HEC-1

The selection of HEC-1 led to the development of local hydrologic information used in SACPRE. (Much of this information has been documented in technical reports and are included as an appendix to the new hydrology standards.) SACPRE, the local hydrologic information and related comments are described as follows:

<u>SACPRE.</u> The main points of SACPRE are that it:

- facilitates the creation of an HEC-1 input file through interactive input.

HYDROLOGIC METHOD USING HEC-1 (CONTINUED)

- contains four main input menus: general information, subbasin information, hydrograph routing and combining.
- provides recommended parameters and checks for unreasonable values.
- produces a summary file of input values for review purposes.
- creates an intermediate file that can be edited by text editors and can be run in batch mode. The intermediate file is transformed into a HEC-1 input file.
- provides Modified Puls reservoir and channel, Muskingum Cunge and Muskingum routing techniques.

Design Storms. These are based on an annual maximum analysis of 88 years of record at the NWS downtown Sacramento gage. Return intervals of 2-,5-,10-,25-,50-,100-,200- and 500- years are used. There are two types of design storms built into SACPRE, short and long duration.

- Short Duration (6-, 12- and 24-hours). These are based on a synthetic, single peaked, centrally balanced event that includes all maximum intensities for a given return interval. These are used for most applications.

- Long Duration (36-hours, 5- and 10-days). These are multi-peaked and are based on an analysis of historic storms. These are used in certain cases when volumes are of primary interest. (Use of continuous simulation in the forthcoming HEC-HMS may supercede these design events). In certain areas, the February 1986 historic storm, which almost overtopped the Sacramento River levees in this area, has been adopted as the long duration event.

Loss rates. Initial and constant loss rates are used. Initial losses are based on local reconstitutions and reflect higher losses for the more frequent events. Constant losses are calculated based on soil type (SCS runoff classification) and land use (percent impervious).

Base Flow. Analysis of gaged data indicated that base flow does not occur to any extent in Sacramento and can be omitted. This simplified calculations of detention volume required to mitigate for development.

Unit Hydrograph. This is the method preferred by FEMA to calculate subbasin hydrographs. The Bureau of Reclamation dimensionless urban unit hydrograph (Cudworth, 1989) was selected. Two important factors are the lag time and the unit duration:

- Lag Time. This is basically the time that the runoff lags the rainfall. Two methods were developed to determine lag time:

    - Subbasin "n" Equation. The equation was originally developed by Snyder and later revised by the Corps Los Angeles District and the Bureau of

Reclamation, and is:

$$Lag = 1560n[(LL_c)/S^{0.5}]^{0.33}$$

where,

Lag = Lag Time in minutes

L = Length of longest watercourse in miles

L_c = Length on longest waterourse to a point opposite the centroid in miles

S = Slope in ft/mile

n = Subbasin "n"

The subbasin "n" values are determined from a table based on land use and condition of the subbasin's drainage system (natural or pipe/channel). The derivation of this table is discussed in more detail later.

o Travel Time Component. This is the sum of the travel times through the individual components (overland, gutter, pipe, channel). Standard values were established for overland flow, and the total travel time was set equal to the lag.

Theoretically both methods should give approximately the same results. In the end, after calibrating the subbasin "n" values, and fine-tuning the travel time method, they did give similar results. This tended to verify the methodology, but also eliminated the need to compute lag time by the more time consuming travel time method in most cases.

• Unit Duration. This is the computation time interval, NMIN on the IT card, used in HEC-1. To provide adequate definition of the unit hydrograph near the peak, the unit duration should approximate the lag divided by 5.5 (Cudworth, 1989). A planning level study for a typical urban watershed in Sacramento has subbasins ranging from 200 to 2000 acres (0.3 to 3 square miles). The developed lag times may vary from 30 minutes to 2 hours. The typical minimum lag time (30 minutes) divided by 5.5, gives an appropriate NMIN value of 5 minutes. A sensitivity analysis has shown that using the smallest allowable value of 1 minute for NMIN will give the most accurate definition of the unit hydrograph, however other factors may be involved in the selection of a time step, such as the storm duration, the time of concentration of the entire watershed, version of HEC-1 (300 or 2000 maximum hydrograph computation points), routing parameters or user preference.

The smaller the lag time, the more the short duration rainfall intensities are reflected in the computed peak flow. The rainfall-runoff model can be used to model an entire subdivision, however, and thereby eliminate the need for very small subbasins, and very short lag times.

Lag/Frequency Factor. In order to account for the increase in lag time in pipe systems where the design capacity is exceeded, and where overland release is provided (new subdivisions are required to provide overland release), the following multiplication factors are applied to **pipe systems** based on return interval:

TABLE 1. Lag/Frequency Factors

Return Interval	2,5,10yr	25yr	50yr	100yr	200yr	500yr
Lag Factor	1.0	1.1	1.2	1.3	1.4	1.5

The 100-year factor was determined by comparison with a computed hydrograph that combined pipe and street hydrographs routed through a typical subdivision using Modified Puls storage routing. The other factors were estimated based on the amount of street flow.

Where overland release has not been provided, or where a more detailed analyses of the hydrograph at the outlet from a subdivision is required, the Lag Factor is replaced by a Modified Puls storage/discharge routing that reflects the pipe design capacity and storage, surface storage, street routing, and outlet conditions.

IMPACT OF REVISED HYDROLOGIC METHOD

For total drainage areas of a few square miles or more, peak flows derived using the new HEC-1 preprocessor, SACPRE, were generally comparable to those using the Nolte regional equations. For these larger watersheds, the new computed flows also compared well to the Sacramento District's hydrologic studies, as well as recent County studies that used available stream gage data.

However, for smaller drainage areas the proposed methodology produced peak discharge-per-acre rates substantially exceeding the County's current standards. (The very small basins virtually reproduced the short duration rainfall intensities - cfs/acre being roughly equivalent to in/hr.) While the County expected some increase in design flows from smaller watersheds, the magnitude of the increase raised both technical and policy questions. First, there was no history of systemic flooding to suggest that pipe systems installed in the past 30 years are substantially undersized. Second, very little gaged peak flow data wasavailable to validate the higher flows predicted for small basins using SACPRE. Finally, if the higher flows are valid, the County must resolve such issues as: What degree of in-street detention best balances the cost of public drainage infrastructure (larger pipes, channels, bridge openings, and detention basins) with public safety and nuisance impacts? How much of an impact would larger pipes have on increasing flows to downstream facilities? Is it more desireable to continue using the Nolte design curves to design pipe systems, even though the design return interval is less than thought?

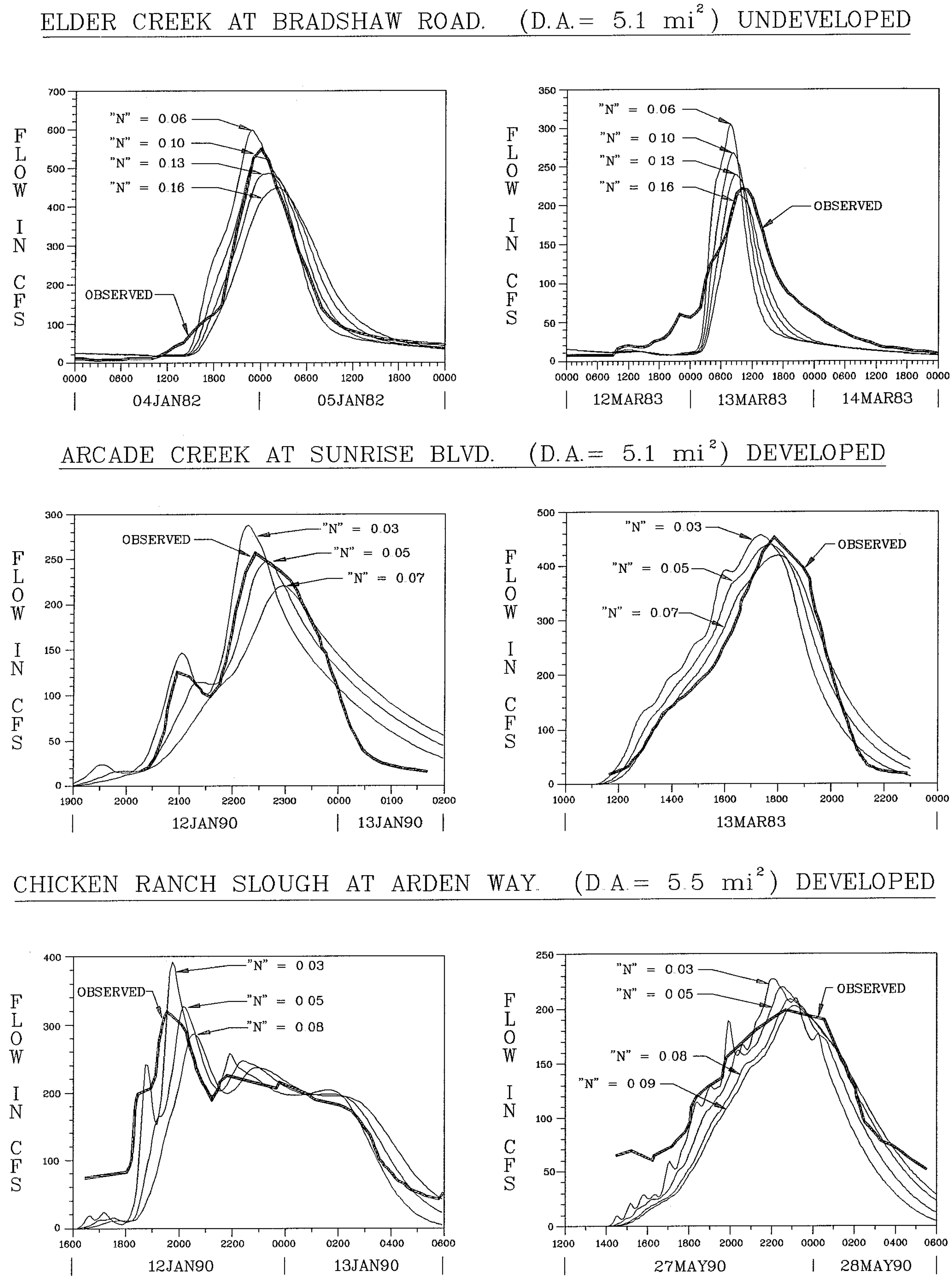
Considering these technical, economic, and policy implications, the County concluded that the SACPRE parameters had to be thoroughly tested before the new hydrology standards could be adopted.

SUBBASIN "N" LAG PARAMETER CALIBRATION

The central issue of the calibration effort became the selection of a method and the appropriate parameters to compute lag time. Lag is a critical factor in determining the peak discharge from a subbasin. The original subbasin "n" parameters programmed into SACPRE were based on the Bureau of Reclamation's compilation of reconstitutions done by the Los Angeles District and others. These values produced very high peak flows for small developed areas. There was also an inconsistency between results using the two lag methods.

There was general agreement that stream gages provided the best information for determining the actual magnitude of flows from small areas. Initially, currently available stream gage information in small areas was used to determine applicable subbasin "n" values specific to Sacramento. For the long term, a program was initiated to install temporary in-pipe gages on small watersheds (100 to 300 acres) as well as additional permanent stream gages in larger areas designated to be master planned. This information would be used to provide refinement to the hydrologic procedures as well as to calibrate models for master planning purposes.

Available Gages. Sacramento County currently has 13 stream gages and 26 precipitation gages as part of its ALERT system. Three stream gages with the smallest drainage areas (4 to 5 square miles) are located in developed areas of low to medium density residential land use. A fourth gage, temporarily installed by the Sacramento District in 1982/1983, recorded storm events from an undeveloped, 5 square mile watershed. Several storm events were reconstituted at these gages specifically to derive subbasin "n" values applicable to similar areas in Sacramento. These reconstitutions are shown on Figure 1. **The subbasin "n" values derived were approximately two times the original values for developed areas taken from the Bureau of Reclamation's manual (Cudworth, 1989)**, and slightly higher for undeveloped areas. A sample of the revised subbasin "n" table is given in Table 2.



SACRAMENTO HYDROLOGIC STANDARDS

FIGURE 1. Calibration of subbasin "N" values by storm reconstitutions at three stream gages. (5 mi² Drainage Areas)

SUBBASIN "N" LAG PARAMETER CALIBRATION (CONTINUED)

TABLE 2. Revised Subbasin "n" Parameters

Land Use	Percent	Subbasin "n"	
	Impervious	Developed	Undeveloped
		Pipe/Channel	Natural
Commercial/Parking	90-95	0.030	0.070
Apartments/Office	80	0.033	0.072
Residential 4-6 units/acre	40	0.042 *	0.084
Residential 1-2 units/acre	20	0.053	0.093
Open Space/Natural	1	0.075	0.120 *

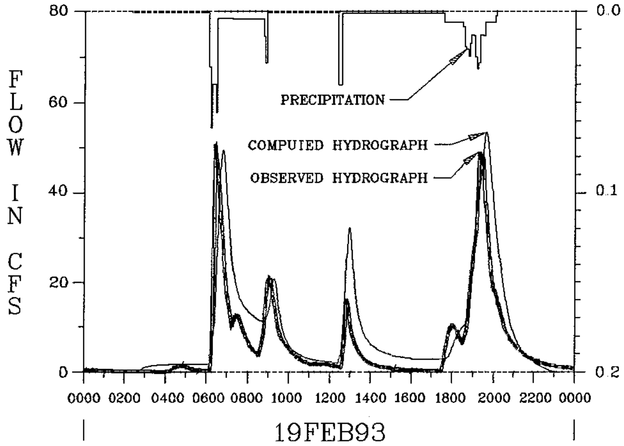
* Calibrated from available gages. The other values were estimated based on % impervious.

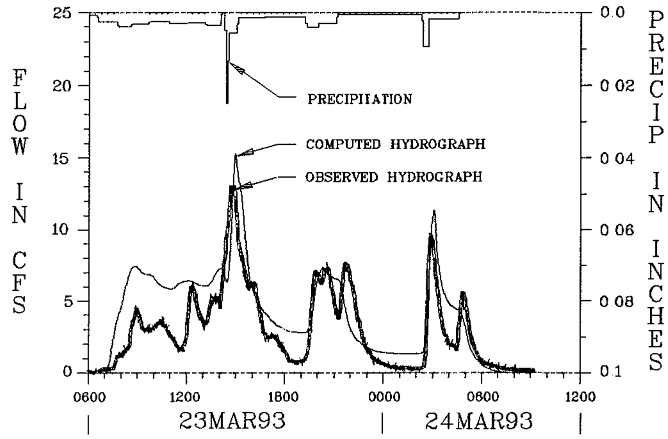
Portable Gages. A project was initiated for the 1992/1993 storm season that installed three ISCO ultrasonic gages in three piped watersheds, in manholes, with drainage areas of approximately 150, 200 and 290 acres. The gages recorded depth versus time, with a two minute sampling interval. Since flow was not measured, a depth versus flow rating curve was needed at each gage. In order to simplify development of the rating curve, the gages were located in pipe systems above areas effected by backwater from the outlet into the channel. There were three storms recorded during the sampling period. None of these events exceeded the capacity of the pipes. A sensitivity test indicated that a "n" value of approximately 0.04 gave the best reconstitutions, verifying the results indicated by the gages at 5 square miles. The reconstitution of these events using the above Subbasin "n" values is shown on Figure 2. They indicate a good reproduction of observed peaks, timing and volumes. **These results indicated that the proposed method is valid for the range of subbasin areas normally encountered in drainage master planning.**

The portable gages were placed at three different locations for the 1993/1994 storm season, in order to provide information for other land uses and drainage conditions. That year was a dry year and, together with operational difficulties, provided no new information during the sampling period. The sampling effort will continue.

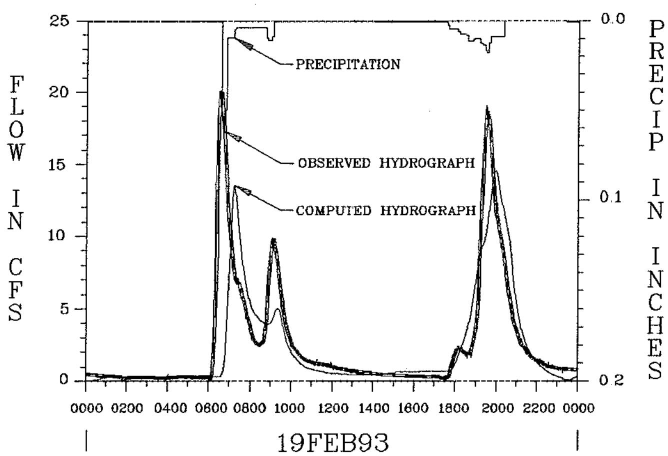
Permanent Gages. The County is currently developing drainage master plans for areas that are experiencing rapid growth. Stream gages at strategic locations in the watersheds provide the most accurate means of calibrating the hydrologic models. Additionally, gages placed near the County/City boundary provide a basis for agreement on the magnitude of flows, and go a long way toward resolving jurisdictional disputes concerning such issues as increases in peak discharge or volume due to upstream development. Finally, these gages will provide an additional check on the basic parameters used in the hydrologic model.

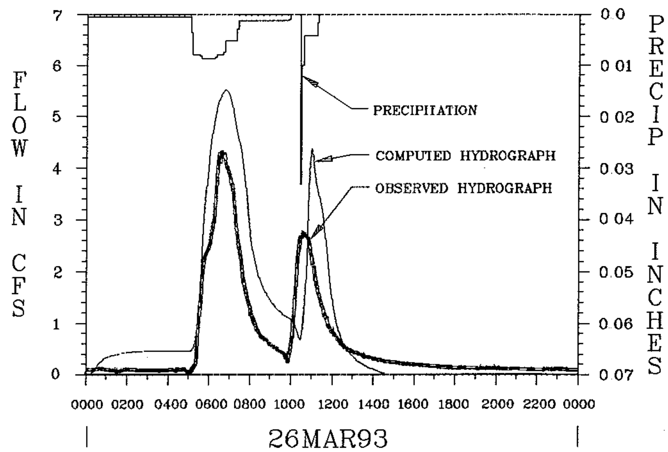
KIEFER BLVD. (Tributary to Mayhew Drain) (D.A.= 290 Acres)



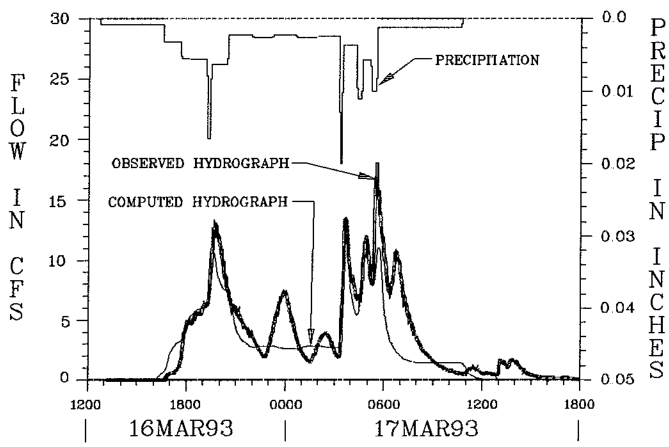


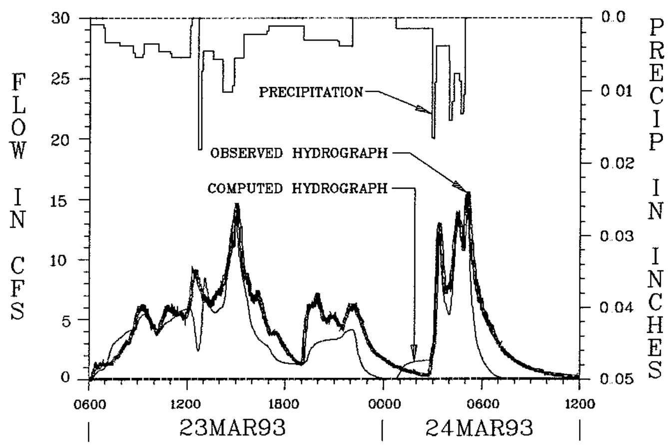
COUNTRYSIDE ESTATES (Tributary to Strawberry Creek) (D.A.= 150 Acres)





FAIR OAKS BLVD. (Tributary to Arcade Creek) (D.A.= 200 Acres)





SACRAMENTO HYDROLOGIC PROCEDURES

FIGURE 2. Verification of revised subbasin "N" method in three small piped watersheds, (150, 200 and 290 acres), by storm reconstitutions.

REFINING THE TRAVEL TIME COMPONENT LAG METHOD

Information from five existing subdivisions was analyzed to determine the relationship between the peak discharges produced by the two lag methods, as well as the Nolte regional equations, and the Rational method. The Rational method produced the lowest flows for all cases. The travel time flows were inconsistent and generally produced high flows. The original travel time method was based on the relationship Lag = 0.7xTc, where Tc (time of concentration) is the sum of the travel times through the various components (overland, street, pipe and channel). Sensitivity tests were performed on these factors and the following revisions were made. The equation was changed to Lag = Tc, which is more applicable to small developed watersheds, the only proposed use for the travel time component method. Additionally, the overland flow component is by far the most sensitive component. Small changes in flow distance can dramatically effect travel time. Standardized overland flow parameters were developed and are given in Table 3.

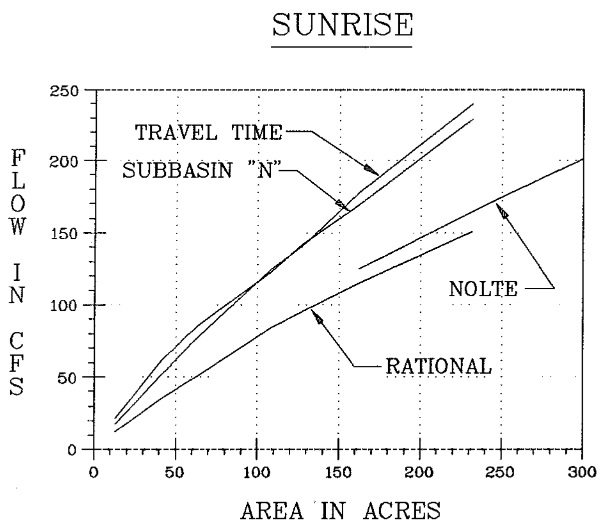
TABLE 3. Standard Overland Flow Parameters

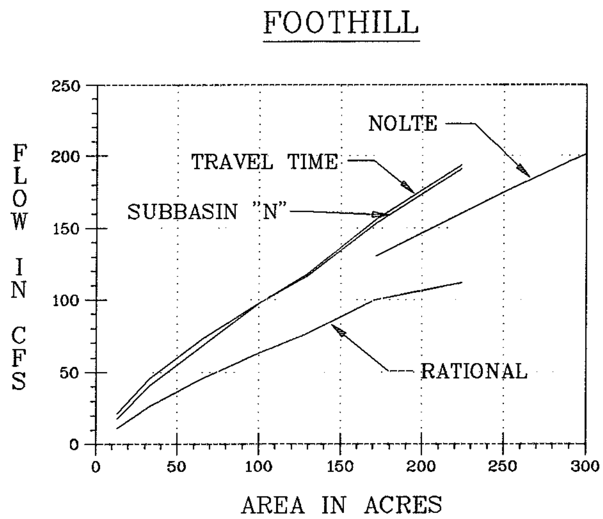
Land Use	Percent Imperv.	Overland "n"	Slope (ft/ft)	Distance (ft)	Tc (min)
Commercial	70-90	0.02	0.01	200	3
Residential	25-60	0.20	0.01	100	9

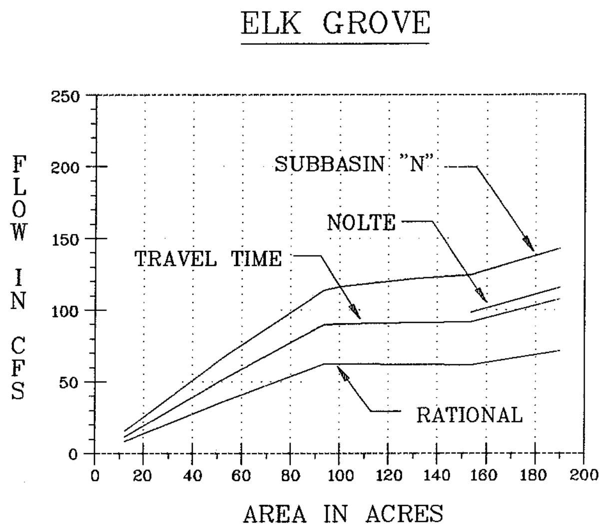
Flows from the revised travel time component method were plotted for the five subdivisions and compared to the revised subbasin "n" method, the Nolte regional equation, and the Rational Method. These are shown on Figure 3. The peak discharges from the two revised lag methods were consistently close to each other. This seems to be mutually supportive of the results from the revised lag methods.

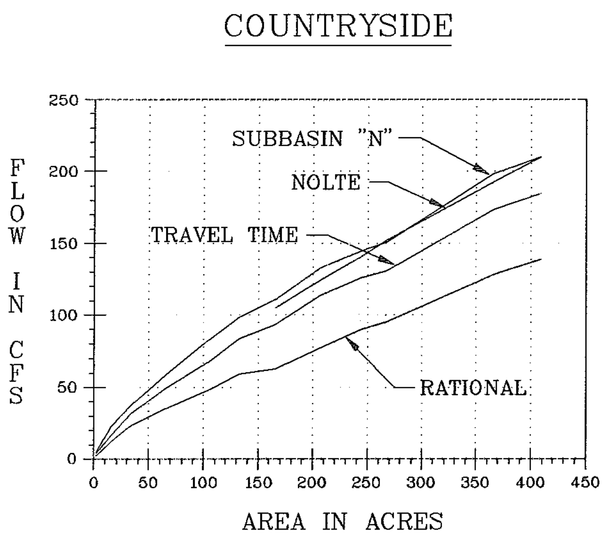
HYDROLOGIC METHODS PROPOSED FOR SACRAMENTO COUNTY

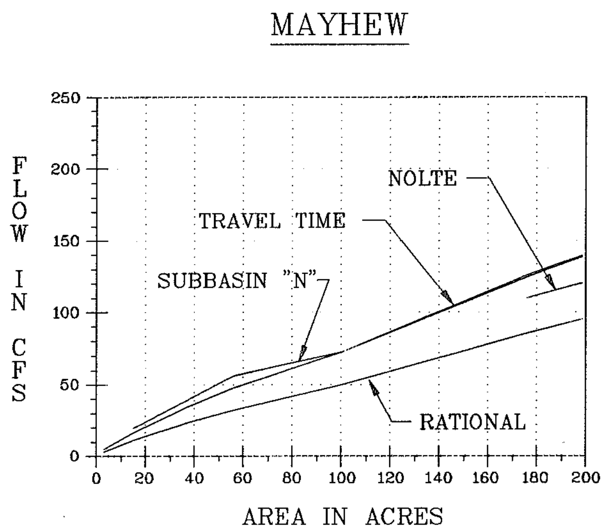
Pipe Design. An analysis of gage data indicated that the Nolte curves were based on approximately 2-year rainfall, less than the 10-year return interval that had been previously thought. A cursory analysis of combined pipe and street flows, however indicated that only nuisance street flooding results in the 10-year event. Based on this, and the fact that there was no substantial history of flooding due to undersized pipes, the County decided to continue using the Nolte curves for pipe design (as well as street and culvert design), for drainage areas less than one square mile.











NOTE: Nolte curves are based on local regional equations.

SACRAMENTO HYDROLOGIC STANDARDS

FIGURE 3. Comparison of results of computing 10-year peak flows for existing subdivisions by various methods.

HYDROLOGIC METHODS PROPOSED FOR SACRAMENTO COUNTY (CONTINUED)

Master Planning, Facility Design. Since HEC-1 using SACPRE adequately reproduces gage information, it will be the underlying hydrologic method used in Sacramento County. It will primarily be used for drainage master planning and facility design, including open channels, bridges and flood control basins. It will also be used for pipe and street design for special cases, such as streets designated as emergency evacuation routes, commercial or high public use areas, or areas with potential loss of life or substantial damages.

Compatibility. Many agencies use different hydrologic methods depending on drainage area size. By using the Nolte design curves for pipe design, yet accounting for the pipe capacity in the HEC-1 model, the County continued the use of a procedure that has provided an acceptable level of drainage service, while providing a means of adequately modelling urban basins that, except for the smallest subbasins, reflects appropriate rainfall-frequency relationships.

FUTURE EFFORTS

The County will be involved with the following efforts in the near future regarding the new hydrologic standards:

- Continuation of the stream gaging program.
- Development of a technical report to simplify and standardize as much as possible, storage routing techniques that reflect actual outfall hydrographs from subdivisions.
- Working with the Corps Sacramento District to arrive at mutually acceptable and verifiable hydrologic procedures for Sacramento.

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Role of Design Storms in Urban Hydrology

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INTRODUCTION

Flood management systems are designed using a flood discharge of a specific return frequency. Stream discharge records can be used to develop a flood flow frequency relationship when these records are available. The problem that most hydraulic engineers and hydrologists face is that stream flow records rarely exist at the required locations. To overcome this problem designers use hydrologic models with rainfall of the desired return period T (or frequency, $f = 1/T$) as a means of computing a flood of a specific return period. There is a major problem associated with this, since runoff is a function of ***effective*** rainfall, or the total rainfall minus losses. The losses are a function of antecedent rainfall and have their own frequency distribution. However, most hydrologic analyses are based on the assumption that the calculated flood flow frequency is the same as the frequency of the rainfall used for the computation of the discharge.

To incorporate a factor of safety into the design process a conservatively low value for the loss rate is usually chosen, and consequently the discharge is over-predicted in these cases. The design engineer is usually more comfortable with a design discharge that is over-predicted rather than under-predicted.

For return periods that are greater than 50 to 100 years, the most intense period of rainfall that produces the peak runoff discharge often occurs after a period of moderate rainfall. This sequence tends to produce low loss rates during the highest rainfall periods so that the effective rainfall during this period is the largest. Thus, a rainfall event with a 25-yr return period may produce a flood with a 5-year return period or possibly a flood with a 40-year return period. However, in most cases it can be expected that a rainfall with a 100-yr return period will produce a flood of approximately the same frequency.

A design storm must provide the following (Viessman *et al.* 1989): storm duration, point rainfall depth, areal depth adjustment, storm intensity, time distribution, and areal distribution pattern. These parameters are developed from historical records of rainfall in the region to which the storms are to be simulated. The analysis becomes a process of evaluating observed rainfall at appropriate rainfall stations and computing the frequencies of occurrence associated with rainfalls of specific durations.

Commonly the design storm hyetograph is developed in the form of an alternating block of rainfall values with the greatest rainfall within a duration Δt at the center of the storm and successive amounts in additional time increments arranged in alternating fashion on each

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side of the maximum (HEC 1982). An example of this type of design storm (also termed a "nested design storm" is shown in Figure 1. An equation can also be used to fit the hyetograph (Chow, Maidment, Mays 1988). However, using an equation for computing the rainfall intensity as a function of time may lead to using unrealistically high rainfall intensities for the shortest rainfall periods.

In urban hydrology the total catchment area is usually broken down into subareas, and hydrographs are computed for each subarea. A single design hyetograph is usually used as the design storm for the total watershed, which in effect assumes that the point rainfall occurs simultaneously over the full watershed. When small subcatchments are used, it is necessary to use small computational time increments in order to get an appropriate characterization of the runoff hydrographs. It is not uncommon to see computational time increments as small as 1 minute (which is the lower limit for most hydrologic models) to be used. Extrapolating point rainfall data to very short durations and then applying these very intense point rainfall values to even relatively small area is not appropriate in many cases, however.

Inspection of actual rainfall processes show that rainfall at high intensities does not occur uniformly over areas that are larger than single subareas in a watershed. Storm cells exist within the larger storm system. These adjacent cells grow and decay over relatively short time intervals, and as a result of the integrating effect of the runoff process, produce an average runoff response to rainfall excess. It is probably only for the smallest watersheds that hyetograph increments on the order of 5 minutes are appropriate. As larger catchment assemblages are analyzed, the hyetograph increments should be made progressively larger. Also, current practice in watershed analysis is to break up the watershed into many small subareas and then to link the subareas with routing elements. In most cases the routing parameters are not known, and it might be more appropriate to treat the entire area above a collection point as a single catchment. Making the watershed structure more complex does not make it more accurate, and in fact, may increase the uncertainty of model predictions.

AN EXAMPLE OF THE DEVELOPMENT OF A DESIGN STORM

Background. Local agencies are responsible for hydrology studies and the review of flood studies for development. In California flood control responsibility is given to county flood control agencies usually under the jurisdiction of the county's department of public works. Many of the counties in California have developed guidelines and criteria for hydrologic studies in the form of county hydrology manuals. These manuals provide data on loss rates, rational method and unit hydrograph parameters, and rainfall data.

In most counties there are very few stream gaging stations, and therefore, rainfall-runoff modeling is used to determine flood flow frequency relationships. The assumption is usually that the rainfall frequency is the same as the calculated stream discharge frequency.

Conservative values for loss rates are used so that in most cases calculated flows are greater than the actual flow of a particular frequency.

Boyle Engineering Corporation has recently developed a hydrology manual for San Joaquin County, California (Hromadka *et al.* 1994). The hydrology analyses for the county are based on a design storm approach. The following paragraphs describe the development of the San Joaquin County design storm.

Rainfall Data Base. Records for twenty-one rainfall gages with lengths of record from 5 to 101 years were found for San Joaquin County. An additional seven gages located adjacent to the county were also used. Most of the twenty-eight gages had daily rainfall data. Five of the gages had rainfall data for short durations, down to 5 minutes. These were used to define a relationship between the 24-hour rainfall values and shorter rainfall durations.

Based on the work of Goodridge for the California Department of Water Resources (DWR 1981, Goodridge 1990) and for other agencies (Goodridge 1991, 1992), it has been shown that for many locations in California there is a high degree of correlation between the average maximum 24-hr rainfall at a station and the mean annual precipitation (M.A.P.) at that station. A plot of the Mean Annual Precipitation contours (M.A.P.) is shown in Figure 2. A plot of the relationship between the *average maximum 24-hr rainfall* and the M.A.P. is shown in Figure 3. The daily rainfall is recorded at a specific clock time each day by the observer. The factor used to correct this value so that it corresponds to the rainfall that can be expected to occur within a given period of 24 hours is 1.14.

Goodridge has performed extensive statistical analyses on rainfall throughout the state of California (Goodridge 1990). His work indicates that an appropriate frequency distribution for rainfall is the Pearson Type 3 distribution. To fit data to this distribution requires the mean, the standard deviation, and the skew. With samples of small size (essentially all rainfall records are small samples) skew cannot be reliably determined from the sample statistics, and regional skew values are used. Goodridge has determined that the regional skew for the Central Valley region is 0.11, and this value was used in the analyses for San Joaquin County described here.

The rainfall of a particular frequency and duration can be determined by using frequency factors with the following relationship (Chow 1964).

$$x = \bar{x} + Ks \qquad (1)$$

where x is the value of the parameter, x is the mean, s is the standard deviation, and K is the frequency factor. For the Pearson Type 3 distribution, K is a function of the skew and recurrence interval.

Development of Mean Annual Precipitation Map. Application of this method requires the determination of the mean annual precipitation (M.A.P.) to be used the calculation of the average maximum daily rainfall. The San Joaquin County mean annual rainfall map shown in Figure 2 has contours of equal annual rainfall which were developed using San Joaquin County rain gage data plus data available from rainfall maps for adjacent counties (Sacramento County 1993, Goodridge 1991, 1992). The general rainfall contour pattern was initially developed by interpolating contours from the measured data at the rainfall stations. There were some anomalies, where stations quite close together had significantly different measured rainfall amounts. It was recognized that the catch of a rain gage is quite sensitive to gage location and exposure. The position of the contours were therefore adjusted to give smooth contours based on the gages with in the county as well as with the published rainfall contour maps from adjacent counties.

Rainfall Duration. Durations Less than 24 Hours--Rainfalls of durations of less than 24 hours were correlated with the 24-hr values (the average maximum rainfall for a given duration vs. the average maximum 24-hr rainfall). This relationship is log-linear, so that the points can be fitted to a straight line when they are plotted on log-log graph paper. The relationship is given by the equation

$$P_T = (P24_j)\left(\frac{T}{1440}\right)^{0.401} \qquad (2)$$

where T is the duration of the rainfall in minutes, P_T is the precipitation in the interval in inches, and the exponent 0.401 is the slope of the line on the log-log plot.

Actual individual storms have durations of varying length. For design purposes hydrologists frequently use storms of three, six, and twenty-four hours as representative of storm durations that are appropriate for design of urban storm water management systems. The rainfall intensity varies within these periods so that a distribution of rainfall within the period must be developed. This is done by constructing a design storm from the design rainfall for the various durations from 5 minutes, 10 minutes, 15 minutes, 30 minutes, 1 hour, etc. up to the total storm period of 6 or 24 hours.

Point Rainfall Depth and Areal Depth Adjustment. The rainfall data are from point measurements at a rain gage and require adjustment for the size of the area to which it is to be applied. Depth-area reduction curves similar to those used by the National Weather Service (Hershfield 1961) are provided in the San Joaquin County Hydrology Manual. For areas less than ten square miles this adjustment is usually neglected.

Storm Intensity and Time Distribution. When applied to a watershed using a rainfall-runoff model the rainfall must be distributed in time. This is done by tabulating rainfall amounts for a storm of given return interval for the specified durations (5-min, 10-min, 15-min, 30-min, 1-hr, 2-hr, 3-hr, 6-hr, 12-hr, and 24-hr). The appropriate time interval is selected and the incremental rainfall in each time period is determined. This gives a table of rainfall amounts for a constant time interval, from the greatest amount in the interval to the smallest. This rainfall now must be distributed in time to produce an actual storm pattern. For the San Joaquin County Hydrology Manual this design storm is arranged to position the rainfall with the maximum intensity two-thirds of the way through the storm period. This will give a more conservative rainfall pattern for the analysis of detention storage than a symmetrically shaped storm pattern.

A typical design storm for San Joaquin County is shown in Figure 1. This storm represents a 100-yr, 3-hr design storm for the Stockton area where the MAP = 15 in. The Average Maximum Daily Rain = 1.69 in., and $P24_{100}$ = 3.53 in. The 1-hour, 100-yr rainfall is 0.98 in., while the 10-minute, 100-yr rainfall is 0.48 in.

Data Format. The precipitation data are provided in the hydrology manual in the form of tables of Intensity-Duration-Frequency for specified return periods (2-yr, 5-yr, 10-yr, 25-yr, 50-yr, and 100-yr) for Mean Annual Precipitation (MAP) values from 9 inches to 19 inches since this spans the total variation of rainfall within the county. No elevation adjustment factor was provided because the effect of elevation is included in the MAP value. It is only in the very north-eastern and south-western parts of San Joaquin County that any significant elevation effects are seen. Graphs giving intensity-duration-frequency data are also provided.

DISCUSSION

Design Storm Applications. Figure 4a and 4b show runoff computations for a small (0.20 mi^2) watershed using design rainfall specified at 5 minute intervals using the HEC-1 model for computational intervals of 2.0 and 5.0 minutes respectively. The basin lag used in this analysis is 0.4 hr which corresponds to a time of concentration of 30 min (assuming LAG = $0.8t_c$). The basic hydrographs are quite similar, with the computation using a Δt = 2 min giving a peak discharge about 5 percent higher than the computation using Δt = 5 min.

Figure 5 shows the same basin with the rainfall defined for 10-min intervals and a 10-min Δt. The peak discharge for this run is significantly lower (about 10 percent) the peak computed from rainfall specified at 5-minute intervals. The 5-min rainfall interval is probably more appropriate for this small basin. For larger basins it could be argued that a longer rainfall hyetograph increment and computational time increment would be more appropriate.

SUMMARY

Because discharge data for specific locations in a watershed are lacking, design storms are used with hydrologic models with rainfall of the desired return period T (or frequency, f = 1/T) as a means of computing a flood of a specific return period.

As an example of the design storm development procedure the design storm for the San Joaquin County Hydrology Manual is described. The design storm was developed to provide the data for storm duration, point rainfall depth, areal depth adjustment, storm intensity, and time distribution pattern. Historical records of rainfall in the San Joaquin County region were used, and records for twenty-one rainfall gages with lengths of record from 5 to 101 years within the county and seven gages located adjacent to the county were used. Five of the gages had short duration rainfall data.

In the developed procedure rainfall for specified frequency and duration is calculated from the average maximum daily rainfall, which in turn is determined from the mean annual precipitation. The data are arranged into a pattern to form a design storm which is used with a computer model of the rainfall-runoff process.

The question of appropriate rainfall hyetograph time increment is discussed. Using too short a time interval for the rainfall can lead to over-prediction of runoff peaks. Inspection of actual rainfall processes show that rainfall at high intensities does not occur uniformly over areas that are larger than individual subareas commonly used in watershed modeling. Storm cells that exist within the larger storm system grow and decay over relatively short time intervals, and as a result of the integrating effect of the runoff process, produce an average runoff response to rainfall excess. It is probably only for the smallest watersheds that hyetograph increments of 5 minutes or less are appropriate. As larger catchment assemblages are analyzed, the hyetograph time increments should be made progressively larger. Also, current practice in watershed analysis is to break up the watershed into many small subareas and then to link the subareas with routing elements. In most cases the routing parameters are not known, and it might make more sense to treat the entire area above a collection point as a single catchment. Making the watershed structure more complex does not make it more accurate, and in fact, may increase the uncertainty of model predictions.

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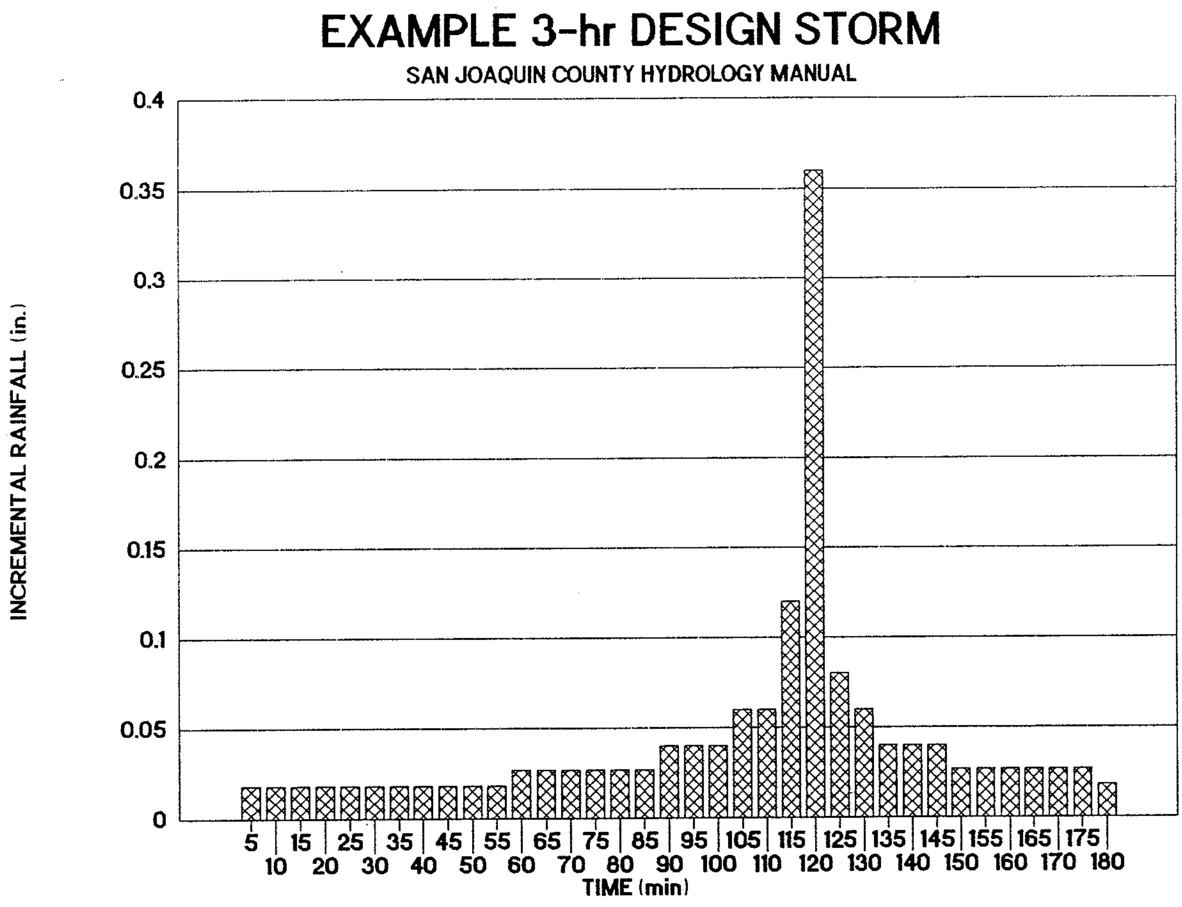


Figure 1. Example 3-hr Design Storm for San Joaquin County

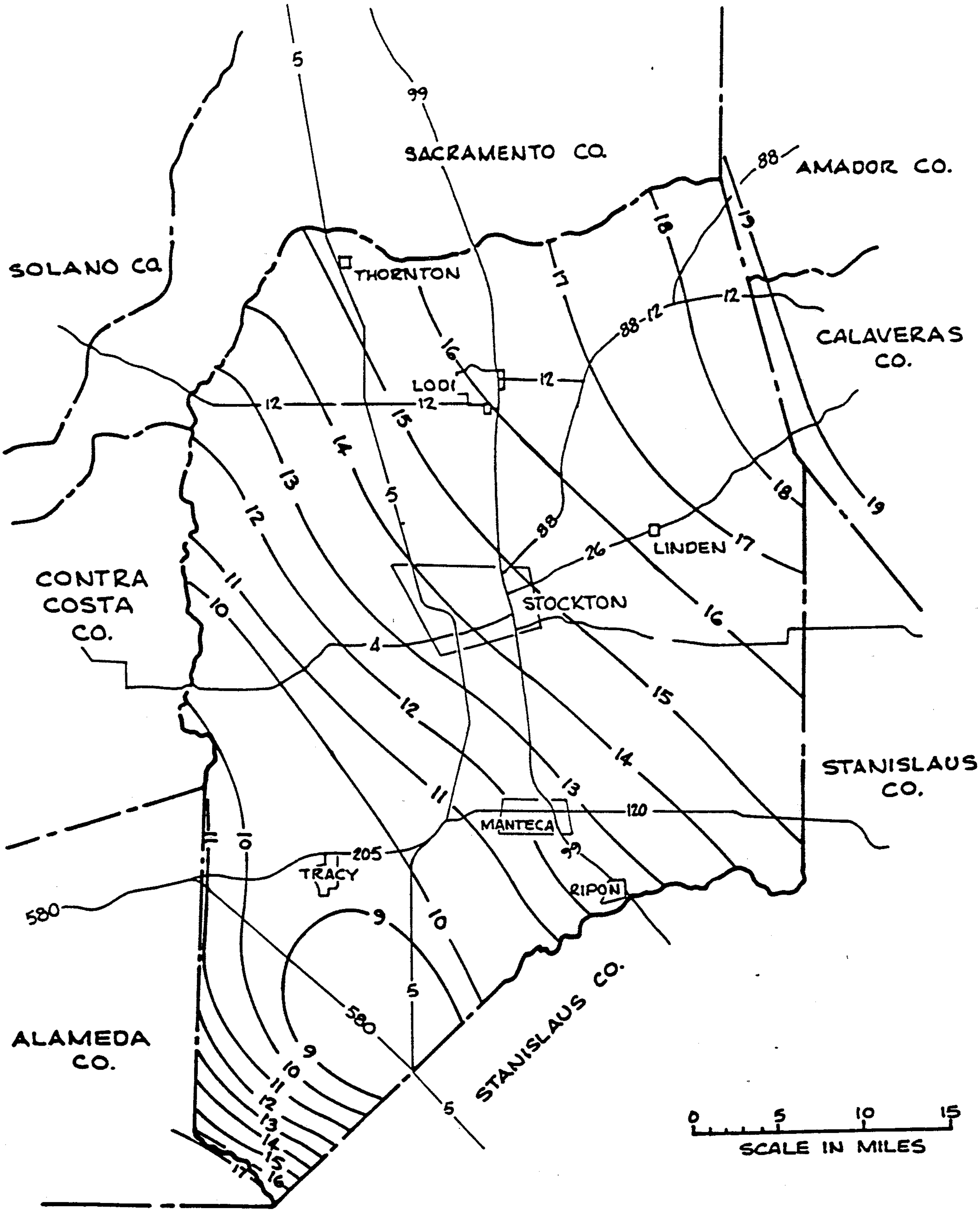


Figure 2. Mean Annual Precipitation for San Joaquin County

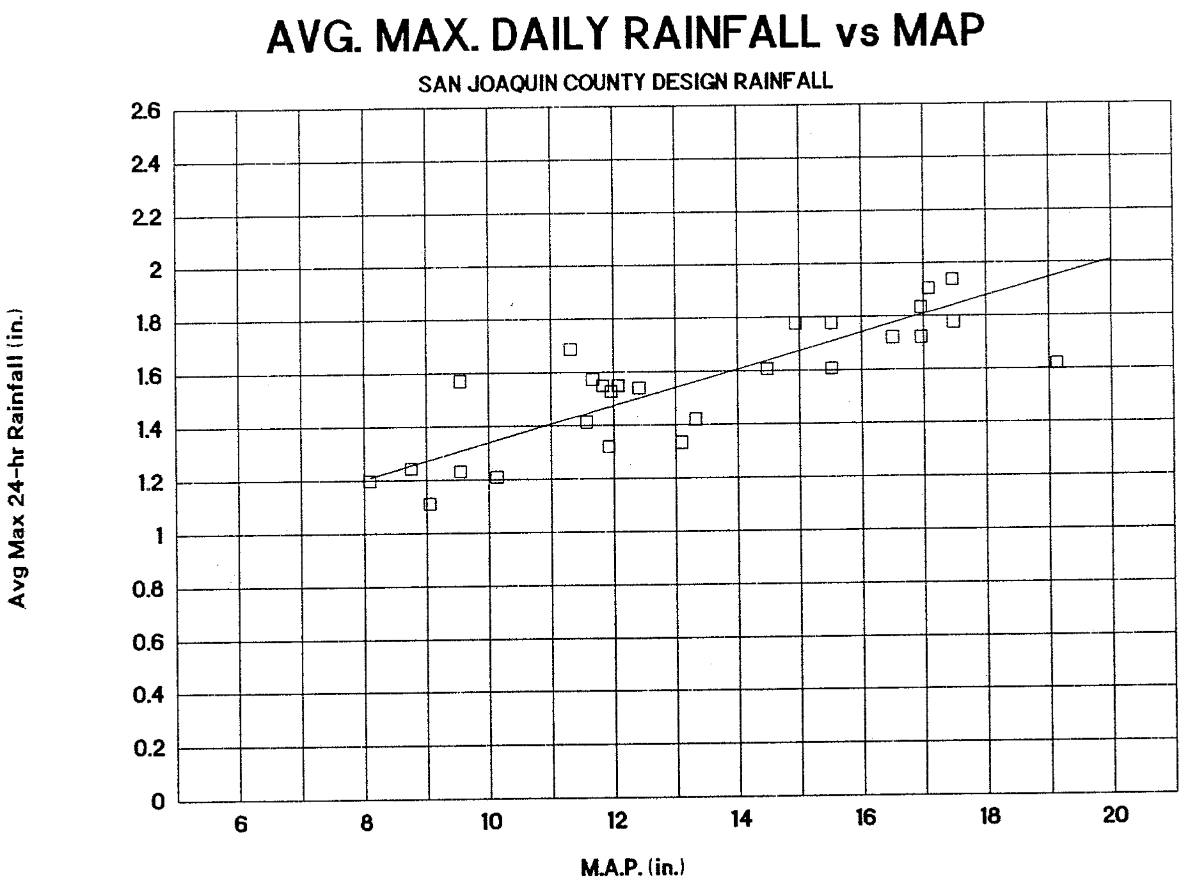
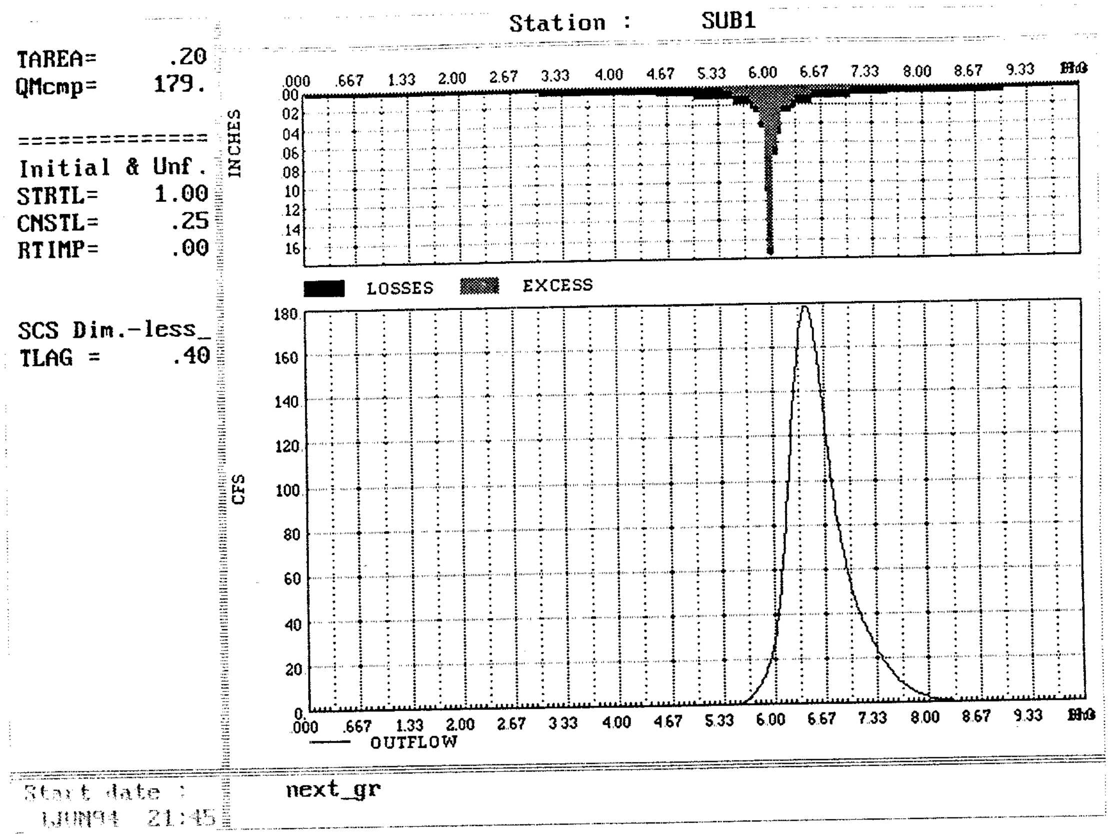
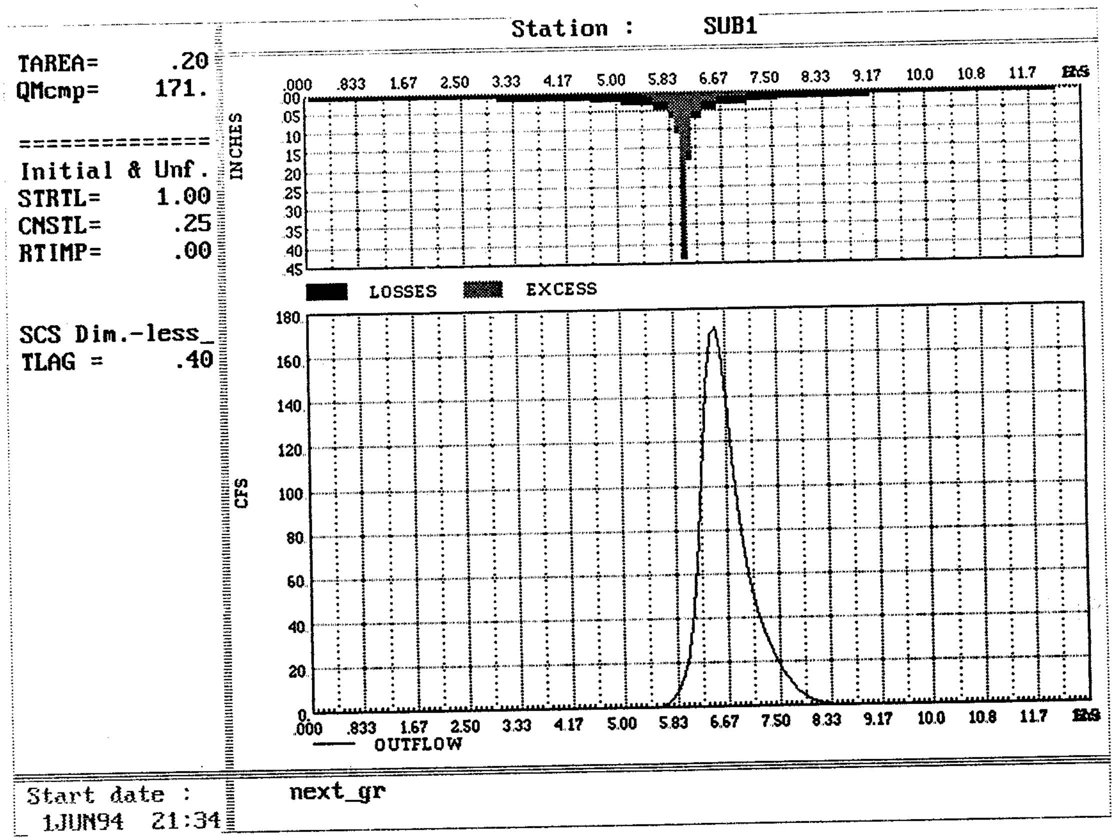


Figure 3. Average Daily Maximum Rain vs MAP -- San Joaquin and Adjacent Counties



a. Computational Time Increment Δt = 2 min.



b. Computational Time Increment Δt = 5 min.

Figure 4. Example Runoff Computations Using Design Rainfall (Rainfall Hyetograph Time Increment = 5 min)

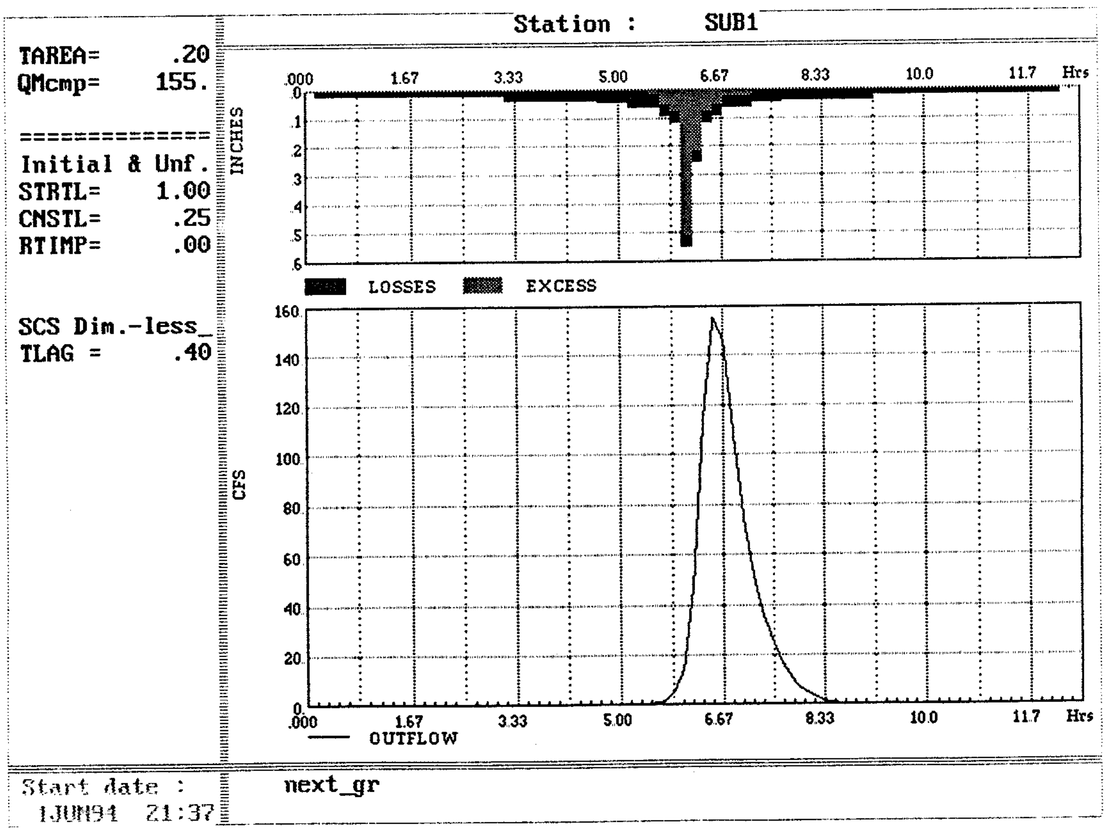


Figure 5. Example Runoff Computations Using Design Rainfall (Rainfall Hyetograph Time Increment = 10 min, and Computational Time Increment Δt = 5 min)

ON REASONS WHY TRADITIONAL SINGLE-VALUED, SINGLE-EVENT HYDROLOGY (TYPICAL DESIGN STORM METHODOLOGY) HAS BECOME SIMPLE-MINDED, DISHONEST AND UNETHICAL

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ABSTRACT

The position taken in this paper is decidedly *anthropo-centri-fugal*, to coin a new word; it avoids placing society at the center of concern. Proceeding from a discussion of human *populution*, it describes some dimensions of ethical design for sustainable ecosystems. Arguments are presented that event modeling, and its associated design methodology, at best contributes to the destruction of aquatic ecosystems: the principal argument in favour of design storm methods is design economy, but cheap stormwater drainage design is an avoidance of consideration of the inevitable long-term ecological impacts of that design, tantamount to a deliberate decision to remain ignorant of the impacts of urban drainage system design. Eco-sensitive design on the other hand demands the adoption of continuous modeling.

Even though computer hardware, software and expertise is now more than capable of supporting long-term continuous modeling, current engineering design manuals do not support this position. Nevertheless, professional engineering associations all over North America have recently adopted new ethical codes that require ecosystem-sensitive design. To effect timely adoption of eco-ethical design, all who suffer from impaired aquatic ecosystems, including engineers constrained to practice conventional hydrology exclusively, should seek or demand a comprehensive paradigm shift in stormwater management, away from traditional design storm methods, and towards the adoption of fuzzy, continuous models.

Originally presented with a large number of visual illustrations, not reproduced here, the style of this paper is somewhat colloquial.

INTRODUCTION

Efforts of the writer's research group are directed towards the enhancement of modern continuous computational methods, such as the US EPA's Stormwater Management Model (SWMM, Huber and Dickinson, 1988), enhancements which have been seriously under-funded in the last ten years, a situation that continues. Nevertheless, the responsible and proper management of surface water pollution in North America

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and indeed, globally, is fundamentally important to all living things, and thus it is appropriate for us all to actively participate in the enhancement and wide adoption of methodology such as SWMM, especially together with the US Army's Hydrological Engineering Center, an office more renowned for software that is practical than eco-sensitive.

Our work is graduate student research, funded basically by the Canadian national Natural Sciences and Engineering Research Council, and from sources where-ever there is a common interest. One example of our disparate activities, is our newsletter *SWMM News & Notes* (for which, incidentally, we are always soliciting contributions). We also sponsor an annual conference in the Toronto area on modeling the management of the impacts of urban stormwater, alternating six-monthly with a conference somewhere in the United States. Annually, the Canadian proceedings are published as a peer-refereed, archival book in the style of a hard-cover textbook, with extensive bibliographic tools. As well, we support three Internet list servers, including SWMM-USERS[@UOGUELPH.CA], which is moderately active, and has an extremely high signal-to-noise ratio.

Before continuing the logical thrust of this paper, readers need to have the usage of certain terms clarified. *Modeling* has the simple purpose, in this discussion, of evaluating the likely performance of potential arrays of various best management practices (BMPs) in an impacted watershed, and determining the "best" array, according to some presumed, prevailing values. If we had prior knowledge of all such performance, sufficient to determine reasonably precise optimal BMP capacities, then modeling would of course be a waste of time. The corollary to this is that, if we had available prior field observations of the performance of all arrays of all types of BMPs for all aquatic systems and physiographies, again we would not need to model. *Because these two conditions are infeasible, given the inherent complexity, we may conclude that modeling will remain necessary for the forseeable future.*

Processes underlying the BMP performance are imperfectly known, even for the ***as-is*** system, the best-known of the arrays to be modeled, and thus the computed model output is uncertain. The ***to-be*** (proposed interventions, the so-called *what-if?* scenarios) and ***as-was*** systems (pre-development, especially if pre-forest-clearance or pre-agricultural) are even more uncertain. This is especially true of surface water pollution processes. Field data monitoring is required to validate the models; if the model were certain for all arrays then monitoring would of course be a waste of time. *Because even the best available codes are uncertain models of proposed BMP arrays, field data monitoring will also remain necessary for the forseeable future.*

In surface water quality modeling, models that disregard all dry-weather processes are referred to as *event* models, while models that include code for processes that are active during dry weather, such as pollutant build-up, evapo-transpiration, storage depletion, recovery of loss rates, and so on, are termed *continuous* models. Continuous models also usually include processes associated with winter seasons. Event modeling suffers the limitation that every run is governed by arbitrary assumptions of start-up conditions, which are themselves seldom subject to careful modeling scrutiny, such as sensitivity-, calibration, and error analysis (SCEA). Event models are obviously only run for short durations. To the extent that the effect of these initial conditions persists through the model run, the computed results may be unreliable. There is a great deal of hydrology literature asserting that these start-up effects are indeed important (James and Robinson, 1986a, provide a review). Of course, the fundamental premise of event hydrology is that the design storm probability is exactly equal to that of the associated flood event. However, studies have shown that this is not true. Klemes (1987) eloquently condemns event modelers who claim to be able to compute risk and reliability, referring to the association of a fixed probability to a design event as *wishful thinking*.

Event modeling evolved in bygone times before computing, and it is simply no longer appropriate to adopt such simplistic methodology (James and Shivalingaiah, 1986). Modeling should always be continuous, rather than event-oriented, for all design inference. However, short runs for both dry and wet events, and events that are combinations thereof, are recommended for SCEA. In these cases there is no start-up error, because the initial state is given by the observed record.

James (1993) argues that 75-year continuous modeling has now become feasible, indeed desirable, in order to address concerns of sustainability. In a landmark case, the Supreme Court of Canada upheld a decision in favour of downstream riparian land owners suffering fluviological impacts resulting from the urbanization of a large city. Arguments that helped to convince the judges were based on 47-year continuous modeling, whereas the losing side based their analyses on event hydrology and that for very few events, only one point on one event being used for calibration (James, *in press*). Event hydrology cannot be used to evaluate fluvial morphology downstream (James and Robinson, 1986b). Readers should be aware that this case is precedent-setting in the US as well.

"POPULUTION" ISSUES

In this section, we attempt to build a case against anthropocentrism in engineering design, based on empirical evidence that engineering design very often leads to landscape interventions that increase the human population locally, and that the process seems to have no imposed limits.

Writing for the moment not as a technical expert, but as a full-time resident drinker of water downstream of major US conurbations, I am perhaps qualified to challenge some uncharted premises of municipal-engineering political-correctness: viz. that further growth and development is *necessarily and unconditionally good*. I can and do question whether we bipeds are not already too numerous in North America. Overall, the population density in the US part of North America is probably about an order of magnitude greater than that of the Canadian part. Significant problems arise when populations of 25 thousand (say) to 25 million people congregate in urban areas (Mexico City's population will soon be larger than all of Canada, and comparable to California). Modelers of best management practices for the management of the impacts of surface water pollution must, at some point, probably sooner rather than later, ask a leading question: *are we doing enough in North America about containing the numbers of large mammals, especially humans, that directly cause a deterioration in the natural environment?* When will there be development enough? Is a population of 7 million sufficient in SE Michigan, or Southern Ontario? Why not 70 or 700 million? Or is 700 thousand more reasonable? Why not 70 thousand for that matter? These seem like questions that are *too big* for engineers to answer (even, one suspects, army engineers), but we *are* involved because we write the computer codes that relate pollution to population activities, and that lead to development controls. What would be the basis for the answer to these questions of carrying-capacity vs. sustainability, other than the so-called sustainable quality-of-life? I have not yet met any individual who feels that local surface water quality, and concomitantly her/his quality of life, would be *better* if the local region *developed* from housing and employing, say, 7 thousand, to, say, 7 million. In fact the opposite seems clearly to be the human experience. This truth is evidently not universally acknowledged by the councils of our elected representatives, however, who very often make indirect use of the results of our water-environment models to *encourage* further development, and thus population growth.

In this particular respect our models are not functioning: *as they do not explicitly examine carrying capacity, they do not test all reasonable alternative management strategies*. It is the large numbers of us people, especially the very large numbers in our North American cities, that destroy natural habitats. Pet and farm animal populations are also causing serious problems at several locations in our part of the globe. Processes now incorporated in our stormwater and water quality models do not make the connection (between population carrying capacity or land-use controls on the one hand, and aquatic ecosystem changes or pollution on the other) sufficiently pointedly; modelers need to be able to convince decision-makers that negative growth is a valid strategy that ought to be routinely examined.

COMING TO TERMS WITH SUSTAINABILITY ISSUES

In this section we attempt to develop the argument that continuous modeling is ecosystem-friendly, and that certain concerns can be met, especially if the simulation period is very long, say 75 years or so.

The biggest problem in North America, as I have ventured to suggest, is *loss of natural aquatic habitat*. In my judgment, in the water environment, this is determined by three derivative environmental factors, above all others:

1. Modulus of the rate of change (and magnitude) of the mean discharge, which becomes very high as a result of urbanization. During flood events, the rapid increase in stage washes away ecosystems and changes the fluvial morphology. Bank-full flows are increased perhaps by two orders of magnitude over three generations. And during dry events, it dries out channel beds and riparian areas, increasing pollutant concentrations, soon displacing the original cold water ecosystems. Flow variations kill relentlessly; they are furthermore sensitive to most anthropogenic activities.
2. Sediment and sediment-attached pollutants, and their impacts on the substrate, the habitat, and aquatic organisms. Sediment loads are also sensitive to increased anthropogenic activities, including, of course, flow variations.
3. Thermal enrichment is a factor which is seldom considered: as a result of the actions of our fellow engineers, huge tracts of land are being covered by black asphalt mixtures, and act as excellent solar receptors with excellent water conveyance, a deadly combination, because water is an excellent conductor of heat. Most population centers with say 50,000 people or more, when located on small creeks, have so severely degraded the aquatic thermal environment that riparian habitat immediately downstream becomes totally changed, e.g. from cold water fisheries to a coarse warm water system. Of course, this effect depends on location, the relative size of the creek catchment and the area developed. Suffice it to say, it's fundamentally important in Canada.

Such sustainability and eco-restoration factors are first and foremost computable *by continuous modeling*, or *period-of-record modeling*.

Lack of progress in continuous modeling methodology has been truly amazing. Having attended these and related conferences and workshops, between five and ten per year for the past 30 years, I have heard and seen copious event modeling. It is as though event modeling still makes sense, or as though event modeling is easier than continuous modeling, or cheaper. It can be demonstrated that none of these assertions is true anymore. Norm Crawford published his dissertation on the Stanford Watershed Model in 1962, which is more than a generation ago, a long time given modern science and technology. *Read my lips: that's* ***32 years!*** In this time, continuous modeling has again and again proven effective, reliable and cost-effective.

Backing up a bit, we should define more carefully the term *model* (here a deterministic surface water quality model). For our purposes, a model of a drainage system is taken to be the combination, such as a SWMM application, of both (a) a program, together with (b) its input datafiles. Thus a model may have a useful life extending over decades. In this sense, *SWMM* is a misnomer. It is important to come to terms with the fact that both the program and the model evolve over the years. Thus models may become very complex, gradually integrating encyclopedic knowledge of component processes as it becomes available, applying it to vast databases as the databases build over time, examining potentially thousands of arrays of best management practices (BMPs) as they are put forward from time-to-time, re-running each array for say 75 years of hydrometeorologic and physical topo-hydrographic data. A further rule may now be posed: *A model can and should be tested for SCEA and the uncertainty always reported as a part of the computed output. Indeed not to do so, is likely a dereliction in design engineering ethics. In other words, it is simply dishonest not to report the uncertainty associated with any computed response.*

By comparison, the very concept of *event* modeling has become obsolete. Originally proposed in the late 19th century, and developed in the first half of the 20th century, long before automatic computers, the need for such simplistic logic has long gone. Even the use of the term implies that the (only) event is a wet event; everything else is an non-event, and nothing happens between rainfalls. Yet many processes that are active during dry periods are extremely important: recovery of infiltration capacity, recovery of storage, build-up of pollutants, to name a few. Some models have been developed recently that set up a series of alternating wet and dry events, as a kind of compromise with tradition. But this also leads to complications: how can the fuzzy periods between wet and dry events be accommodated by such continuous-event-hydrology? For significant periods of natural time there is very light precipitation and/or saturated humidity levels; during these periods it is not clear whether pollutants are building-up or washing-off. Process disaggregation into an artificial wet/dry dichotomy is unnecessary today, given modern computing.

Of course, continuous modeling brings difficulties, primarily associated with data management. Fortunately interfaces with GIS systems, and with time series management systems such as HEC-DSS and ANNIE, are being developed in the stormwater management modeling arena. The integration of ANNIE with SWMM, making the interaction between SWMM and HSPF transparent, is long overdue.

This is not a trivial challenge. Model developers must resist the pursuit of the simplistic. It is not true that simplistic models are easy to understand or apply: subsuming complexity into a simple coefficient is often difficult to explain. Too many of our colleagues still seem to be teaching, particularly in our civil engineering schools, that simplistic representation is itself somehow a worthy pursuit. It is difficult to convince practicing engineers that rolling infinite complexity into a simple fudge-factor is sometimes anti-intellectual. On the other hand, end-users should understand that only complex modeling, along with sensitivity analysis, parameter estimation, error analysis, and field data collection programs, can help us determine which processes are important, but only if all potentially relevant processes are represented in our models. Surely this is the correct intellectual approach. We must fight the notion, spawned from our BC (before computing) heritage, that the most simple representation possible is intellectually desirable. I remind you that this is 1994 in the United States (California at that) supposedly one of the most highly educated spots on earth - we ought not to be supporting the blinkered pursuit of the simplistic *per se*.

My research group has decided to take sustainability, or, more accurately, modeling for sustainability, seriously. We believe that long-term modeling, of the order of 50 or 100 years, say three generations, is very important. We believe that this approach is timely, and credible, and in this we find support in publications by several modern writers, who contend that society understands and is concerned about changes over three generations. Families have an in-built memory of this length, because grandparents talk to grandchildren. Moreover, we believe that component model processes such as sedimentation and fluviology, and perhaps habitat destruction, can be computed over simulated times of (say) 75 years. One-hour time-steps for moderate problems (40 sub-catchments and conveyances, 15 pollutants) can readily be computed for such durations in an eight-hour working day on inexpensive personal micro-computers.

Certainly three generation modeling (3GM) with SWMM is readily available, although we still need quality code for a six-minutely rainfall generator. It is not unusual in North America to find rain records of 50 years and longer, although the available time step is rather coarse. A suitable FORTRAN program for rainfall rates at fine time steps has been developed at Monash University in Australia.

It seems likely that 3GM will at least start to make a significant difference to the way developers do business in this arena, and thus to our environments and ecosystems.

SOME ECO-ETHICAL CONCERNS

In this section we proceed to some new dimensions of eco-sensitive design, not yet associated with continuous modeling, but impossible to reconcile with event methodology.

Apart from human population control, or reduction, there can be no activity more urgent to the future of the world than to reverse the loss of natural habitat (Kennedy, 1993). Habitat loss is the result of anthropogenic activity in watersheds, such as urbanization. Lazaro (1990) describes the physiology, anatomy and the relentless morphology of cities, with their consequent degradation of aquatic environments. By urbanization, we mean the concentration of people into urban settlements, and the change in land-use first from indigenous, original forest or prairie, to rural or agricultural, then to urban commercial and industrial. All land-use changes affect the hydrology of an area, but urbanization is by far the most forceful (Leopold, 1968). There is no doubt that the contemporary population intensification into urban areas will continue through the next few generations, and that the associated hydrological problems of aquatic habitat destruction will become increasingly more acute.

There also can be no doubt that conventional design-storm, or event-hydrology methodology, focuses on extreme simplification, precisely to reduce the cost of the design phase of landscape interventions in, or changes to, watersheds. That is why simplistic methods have been strongly supported by some pro-development forces, such as land developers, construction industries, government departments of agriculture, and consulting engineers who profit from such design studies and construction. They advocate extremely fast, short-cut and cheap design, design that is not concerned with, but may even denigrate, the study of long-term ecosystem impacts. Design costs, whether cheap or expensive, are eventually passed on to homeowners, of course. Recent consumer surveys, however, evidently suggest that homeowners in North America are prepared to share the higher cost of eco-sensitive design.

Long-term continuous water quality modeling on the other hand leads naturally to consideration of impacts on aquatic ecosystems. Necessary information is brought into focus in the foreground. As an example, salmonid (cold water fish) habitat requires seasonal and growth-phase upper and lower limits on flow depth and flow velocity, a continuous low upper limit on turbidity (for sight feeders), a mean summer low upper limit on mean water temperature, very small deposition of suspended material in spawning areas and seasons, and all this in addition to the established limits on transported chemicals. Event hydrology does not provide such information, whereas output from continuous SWMM, for example, begs to be fed into the In-stream Flow Incremental Method (IFIM) for evaluating fish habitat (Navarro et al., 1994).

Features of an unpolluted river are: (a) ecosystem diversity, (b) flow and water quality stability, and (c) self-purification. In a few minutes it is possible to collect two dozen visible plant and animal species, and 100 microorganisms. Proponents of ecological sustainability regard nature in this complexity not just as a set of limiting concentrations, but as a better model for the design of housing, townships, neighbourhoods and regional economies. Sustainability depends upon replicating the structure and function of natural systems with their far-reaching inter-connectedness. Orr (1992) suggests a number of concerns for design that is sensitive to sustainable eco-systems:

- the *living world* is the matrix for all design,
- design should follow the *laws of life*,
- *biological equity* must determine design,
- design must reflect *bioregionality*,
- projects should use renewable energy systems,
- design should *integrate living systems*,
- projects should *heal the planet*, and
- design should follow a *sacred ecology*.

(The emphases are those of the present writer; not all design precepts were cited).

Wet-event-hydrologists (such as HEC-1 users) will raise objections to the above position. And while they would agree that traditional design has led to numerous minor eco-disasters, such as the loss of cold-water fisheries locally, it is true that these have been more the result of ignorance than of deliberate eco-vandalism. Conventional, narrow design does not care about downstream ecosystems; the case was neatly posited by Field and Lager almost two decades ago (1975): *....simply stated, the problem is as follows. When a city takes a bath, what do you do with the dirty water?* [This suggests that there is no alternative but to degrade aquatic and riparian ecosystems downstream of urban creeks.]

Serious questions should be asked not only about the quality of the designs proposed, but also about the quality of the design study itself. By now we all know that simple engineering solutions have very often led to ecosystem nightmares. Wedell Berry (cited by Orr, 1992) states that a *bad solution is bad because it acts destructively upon the larger patterns [of nature] in which it is contained. It acts destructively upon those patterns, most likely, because it is formed in ignorance or disregard of them.* On the other hand, a solution is good if it is in harmony with those larger patterns. As I understand it, and further paraphrasing Berry, this means that a good design will have to meet a number of requirements more appropriate to a post-modern society. Among these a number may be mentioned, even though few of them, if activated at all, could be attributed directly to continuous modeling philosophy. Among other requirements, a good design will:

- accept the *limits* of the discipline of engineering;
- improve and restore the *natural balances* and *bio-diversity*;
- *correct the human behaviour that caused the problem to the ecosystem*;
- *imitate the structure of the natural, native or indigenous system*;
- *be good for all parts of the natural system*;
- *not enrich one individual or group to the distress or impoverishment of another*; and
- *be in harmony with good character, cultural value, and moral law.*

(Not all points have been listed; only the first five would probably be implicated by continuous modeling, assuming that the continuous output would be further processed by aquatic ecologists; the last two distinctly relate to post-modern society, and are included from the larger list out of interest; the emphases are those of the present writer.)

The point is that continuous modeling makes it difficult to avoid ecosystem concerns, while the use of event hydrology makes it difficult to consider them. This point has also been developed by Abbott (1993). Opting for event hydrology, then, given the computational environment of late 1994, is akin to a deliberate decision to choose ignorance all-the-way-down-the-road, to invite eco-disaster. In this sense, perhaps wrecklessly, we may say that it has become *simple-minded,* in the words of the title of this paper.

PROCESSES RELEVANT TO ECOSYSTEMS

In this section, we mention by way of example, four sets of processes or procedures that are not commonly found in popular event models used in design applications for managing the impacts of urban stormwater. All four sets rank at the top priority for future enhancements, in the writer's judgment, and the discussion helps develop an argument for new code that may be added to existing codes, perhaps through suitable shells.

Ecosystem restoration

In my judgment, the most important BMPs (how many, one wonders, could simultaneously be *best* when so many are demonstrably detrimental?) have not yet been coded into the widely-distributed and supported surface water quality models, and hence these BMPs are not appearing in alternative water pollution control strategies. The most important BMPs are those which redirect us, back to the ecosystems which existed before so-called "white-man's" agriculture or urbanization, BMPs which by their design objectives seek to direct runoff back into the ground, to remove pollutants at the source, and add feed-stock for aquatic ecosystems. Deciduous canopy is an important BMP in urban areas, to cool overheated cities. Similarly, infiltration BMPs, particularly those with sand filters, would help reduce the loss and destruction of cold water fish habitats. We have not yet developed design methods for BMPs that add the *"contaminants"* that are essential for aquatic life, such as flies that have aquatic larval stages.

Thermal enrichment

Temperature modeling is very inadequate in our existing models. Not even HSPF can model heat accumulation in blacktop paving and roof tiles, nor do any urban runoff models cover the wash-off of thermal energy from paving and roofs. Temperature is the number one determinant of ecosystem types downstream of urbanization. Available models do not deal with solar thermal enrichment, yet all aquatic chemical and biochemical processes are temperature-dependent. Areas such as Detroit are effectively enormous, exposed, black-body solar receptors, and rainfall/runoff is an efficient transporter of heat from hot pavement and roofs. In this discussion, *thermal enrichment* is taken to mean *the increase in thermal energy, carried from the existing or proposed development to receivers, over what it would have been had there been no significant anthropogenic activity, including agriculture and clearance of deciduous canopy.* Enrichment should not merely denote potency in terms of *existing* conditions - sediment constituents, for example, because the sediments have been seriously degraded since the time of the original mixed forest canopy. My group has done some studies on paving, parking lots and on the city of Guelph as a whole, modeling the thermal enrichment due to storm water.

Rain cell dynamics

In the Great Lakes region we experience summer convective rains approximately once every four days. We do not make enough use of readily available data that deals with the inherent spatial and transient variability of thunderstorms. Yet it is the hot weather and the associated short-duration, high-intensity rain that causes the flooding, thermal enrichment and erosion downstream. Convective summer thunderstorms are dynamic: they age, have preferred trajectories and preferred directions, they are not stationary on the average, and they are multicellular. In fact there is still almost no modeling of thunderstorm dynamics, even though all surface water quality modeling is wrong if the rain is wrong. *All of it.* Modelers end up calibrating for bad rain data by fiddling with infiltration rates, which is really funny, when you think of it.

In North America, weather radar data is readily available. Weather radar units are becoming economical, compared with networks of recording rain gages. Urban runoff in sewers is now being controlled in Japan by small-scale radar. Software exists for converting radar reflectivities to rainfall intensities at a scale of 50 meters and in time steps of 1 to 6 minutes, approaching the resolution and averaging that we need for urban hydrology. My group is using ArcInfo to convert radar reflectivities to rainfall intensities directly, and to model storm cell dynamics, as a part of the spatial data management for the SWMM program.

Inherent fuzziness

Fuzzy logic is being quite widely applied in hydrology these days. We certainly need to present computed urban pollutants as fuzzy output. Since the fundamental equations in surface water quality models are not analytical, we should not strive as we do, to present computed results as a smooth, single-valued relationship, especially for pollutant build-up and wash off. Modellers are not relaying the inherent fuzziness of the processes in their results. Fuzzy logic will require a re-examination of fundamental principles. Simple elementary measures, like thermal enrichment, will no doubt benefit from the approach.

DECISION SUPPORT SYSTEMS

Most, if not all, of the above processes are relevant to ecosystem concerns. As argued before, they also seem to imply a need for continuous modeling, which in turn requires better shells and decision support code. What are the requirements of decision support systems (DSS) that favor such continuous modeling? For a start, such DSS would need, at a minimum, to provide:

- spatial data and facilities management systems,
- time series management systems,
- communications to remote, integrated, distributed databases,
- long-term six-minutely rain generators,
- analysis of the number and duration of exceedances and deficits in the output time series,
- presentation tools for very long time series analysis,
- data compression for long duration time series (e.g. 75 years of 6-minute data for hundreds of stations and dozens of chemicals),
- integration with downstream, dependent models such as fluviology,
- sensitivity analysis,
- calibration,
- error analysis,
- tools to display model reliability and optimal complexity, and
- GUI WIMP presentation tools that display the inherent fuzziness of the computed output.

Such DSSs must become a normal part of the modeling procedures, so that uncertainty due to poorly defined processes and information are duly and responsibly revealed to the end-users. Beyond the topic of this paper, calibration and error analysis are important attributes of such a shell. In passing, it should be remembered that calibration requires only short term records of input and objective functions, and thus a short-term monitoring effort; design however requires long-term input functions that may in fact be reasonably transposed from elsewhere.

There seems to be no doubt then that our deterministic surface water quality models could benefit from more statistical tools. For design purposes, very-long-term input functions may be synthesized from shorter records using stochastic rainfall generators, for example. Considering the inherent variability of rainfall, more statistical manipulations are necessary to build 75 years of data, and to present the results of 75 years of flow and many pollutants at many points. End-users of 3G modeling could not comprehend such results without a fair amount of statistical manipulation. The melding of statistics with deterministic models will have another advantage: it will facilitate the removal of the artificial distinction between data gatherers (field personnel) on the one hand and data consumers (modellers) on the other (or between monitoring people and analytical laboratory people on the one hand, and the modellers on the other). Provided such models incorporate suitable sensitivity analyses, very long term complex deterministic models with comprehensive statistical tools provide useful management tools for data collection programs. They are the best means for filling in missing data. Ranking the sensitive parameters helps rank priority for selecting chemicals, sampling frequencies and locations, and accuracies of determination.

Widely distributed, integrated databases are about to become useful. Our surface water quality models should be tied in with suitable data and user networks, such as the Great Lakes Information Network (GLIN) and the Consortium for International Earth Science Information Network (CIESIN). We also need to encourage widespread use of our models, in education, decision making, as well as engineering and research.

For instance, we should write code that helps make our models available in different languages. Not all North American modellers speak English. Apart from minority groups, there is a great need to translate HSPF and SWMM into Spanish and French, because about 60 million people who drink the water that about 300 million of us are all polluting today, speak those two languages, rather than English, and could benefit from using good SWMM programs just as much as we do.

PCSWMM FOR WINDOWS[2]

PCSWMM for Windows is a ***shell*** - code that is written around existing code, usually to provide better human interaction. In this case the existing code is *SWMM4.3* - a collection of programs comprising hundreds of files, hundreds of routines, and scores of thousands of lines of FORTRAN source, object and executable code; more specifically SWMM4.3 refers to the U.S.EPA official issue, compiled and linked using Lahey or Ryan-McFarland products, and downloaded during or after June, 1994. It makes little sense to test a *program* for SCEA.

Written for a drag-and-drop Windows environment, the shell *PCSWMM for Windows* (James, 1994) aims especially at sensitivity, calibration and error-analysis for design applications using large-scale, long-term, continuous modeling at high spatial and temporal resolution. SCEA is rendered semi-automatic for version 4.3 of the U.S.EPA's Storm Water Management Model. Fuzzy logic is used to manage the complexity, and to interpret the ranked parameter sensitivity in various RUNOFF state-variable spaces.

So far as is known, this is the only shell to focus especially on SCEA for design applications using large-scale (say 1000 and more elements) and long-term (say 75 years) continuous water quality modeling at high resolutions (elements of say 100 ft length, 1 acre extent and integration intervals of 60-seconds). Contrariwise, several papers in the peer-reviewed literature, claim that these specifications are impossible to

[2]This part of the paper has been abstracted from the user documentation for *PCSWMM for Windows* - see James, 1994.

achieve.

Of course the shell also includes the usual graphics user interfaces that are windows, interactive, menu-driven and pointing-device oriented (GUI-WIMP; note that these gooey wimps are not what they sound like - they were created to be *robust*). Pre- and post-processors for data input and output interpretation are included. But most significantly, SCEA is rendered semi-automatic for approximately 10^2 different hydrologic parameters and a total number of parameters in the order of 10^4, depending on the processes active, and the spatial resolution. James and James (1993) describe the genesis of *PCSWMM*.

Various objective functions, performance evaluation functions and error functions are used in *PCSWMM for Windows*. Since the error analysis is linear, with first and higher order sensitivity gradients, the procedures depend on user-intervention at various levels. Intensive, interactive user-dialog demands that the terms used are carefully defined, especially if they differ in important ways from much of the literature, which has heretofore been understandably loose. That is why, for clarity, the terms used have once again been defined throughout this paper.

CONCLUDING DISCUSSION

Principal among the purposes of the modeling effort, is to design an optimum array of BMPs - landscape interventions that temporally divert, store or treat urban stormwater runoff to remove pollutants, reduce flooding or provide other amenities. The recommended way to design them in the future is by using continuous 3GM. *But....* despite almost 30 years' of arguments in favour of continuous modeling, the method has not yet become routine in design offices. Amazingly, most stormwater and flood design manuals that have been published over the past year or two, still do not recommend continuous over event modelling. Thus recourse to existing engineering design manuals does not help.

In any case, adoption of a long-term time-series should not be the insurmountable challenge that it seems to have been: *because of:*

- the modern wide distribution of inexpensive computers;
- freely available knowledge and information on continuous modeling;
- the urgent need to develop ecosystem sensitive-methods; and
- the informed engineering community, itself the product of excellent higher educational institutions, and an informed society;

- there can no longer be any case for event-hydrology methods.

The old argument that continuous modeling is not computable is no longer true: 75 years at one-hour time-steps for 40 subcatchments, 40 conveyances, and 15 pollutants can be handled by a 486DX2 easily within an 8-hour day. Thus, instead of using the *one-in-fifty-year-storm*, simply use *one fifty-year-storm*.

For continuous, fuzzy modeling, appropriate decision support systems are recommended: *PCSWMM for Windows* is such a decision support system - code that manages the simulation system, as opposed to the internal process codes (collectively called the *engine*). This methodology also involves:

error analysis - the computation of the likely error that a computed response may incur;

dissaggregation - the degree to which the physical components of a system are modeled by increasing the number of defined processes;

discretization - the number of spatial components selected to represent the physical system that has been dissaggregated into processes, and the degree to which the physical parameters are averaged (lumped) spatially and temporally;

model complexity - a measure of the number of uncertain parameters in the model. Models should be neither too complex nor too simple for the problem and problem-solving environments. By *environment* we mean the space with its objects that surrounds a thing that is considered to be more important.

In the final analysis, the method must be *computable* - a simulation that can be performed in a working day (eight hours) using, and providing computed output that can be contained on, typical engineering office workstations, comprising (say) 500 megabyte (Mb) hard disk and a 486-66 motherboard with 16 Mb dynamic memory (DRAM).

Finally, in keeping with our philosophy of continuous modeling for ecosystem concerns, we have to redefine potency factors and enrichment, so that we can relate our results back to pre-forest-clearance conditions, since impacts of the change from indigenous forest to agriculture were often as bad as the change from agriculture to urbanization.

In closing, we need code for cost-effective, comparative evaluation of eco-restorative BMPs, and the decision support systems that encourage their evaluation, preferably *their adoption*. And remember: never use event hydrology for design.

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Small parts of this paper have been taken and modified (here and there) from a paper given this summer at the ASCE Hydraulics 94 conference in Buffalo, NY. Also much of this presentation was given orally (but more graphically) at a workshop sponsored by the US EPA at Heidelberg College, in Tiffin, Ohio, in September 1993.

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METHODS AND DATA CONSIDERATIONS IN URBAN STUDIES
BY THE U.S. GEOLOGICAL SURVEY

by

ALAN M. LUMB[1]

INTRODUCTION

This paper provides an overview of selected activities of the U.S. Geological Survey (USGS) related to urban hydrologic studies with an emphasis on modeling techniques. The USGS collects and disseminates data on water resources and conducts and reports results of scientific investigations needed for the management of the water resources by all levels of government and the private sector. Some of the scientific investigations have produced hydrologic models and techniques that are useful for analyzing flooding problems in urban environments. Two of the techniques, regional regression analysis and watershed modeling, are described in this paper.

REGIONAL REGRESSION ANALYSIS

For many years, the USGS has developed regional regression equations for estimating flood magnitude and frequency at ungaged sites. These equations have been developed on a statewide basis as part of a cooperative program with State or local agencies. In a recent effort (Jennings and others, 1994), all the current regression equations have been compiled and placed in a computer program titled the National Flood Frequency Program (NFF). Regression equations are available for estimating the magnitude and frequency of peak discharges for both urban and rural watersheds. This program includes techniques for estimating a typical flood hydrograph for a given recurrence interval of peak discharge.

For urban areas, local regression equations are available for the following cities, metropolitan areas, or States:

Alabama	Statewide Urban
Florida	Tampa Urban Leon County Urban
Georgia	Statewide Urban
Missouri	Statewide Urban
North Carolina	Piedmont Province Urban
Ohio	Statewide Urban
Oregon	Portland-Vancouver, Washington Urban
Tennessee	Memphis Urban Statewide Urban
Texas	Austin Urban Dallas-Ft. Worth Urban Houston Urban
Wisconsin	Statewide Urban

For other urban areas, the nationwide equations developed by Sauer and others (1983) are used. The urban equations from Sauer and others (1983) are based on the estimated rural peak discharge for the selected region and up to six other variables. These variables are contributing drainage area; main channel slope; 2-hour, 2-year rainfall; basin storage; basin development factor; and the percentage impervious area. The rural and urban equations are available for peak discharges with 10-, 25-, 50-, and 100-year recurrence intervals. For many regions of the country, equations are also available for peak flow with 2-, 5-, and 500-year recurrence intervals.

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The flood hydrograph estimation procedures in NFF use the peak discharge and a dimensionless hydrograph. Basin lagtime is used to scale the dimensionless hydrograph. For urban areas, regression equations for estimating basin lagtime require values for main channel length; basin development factor; basin storage; percentage impervious area; 2-hour, 2-year rainfall; and main channel slope (Sauer and others, 1983).

WATERSHED MODELS

River basin (watershed) models are commonly used to estimate the timing and distribution of runoff before and after a planned activity of man or as a result of a natural phenomenon. River basin models used in the USGS are generally categorized as distributed-parameter, continuous-simulation, physically-based watershed models. Two fully-supported models used for urban hydrology in the USGS are the Distributed Rainfall Runoff Routing (DR3M) (Alley and Smith, 1982) and the Hydrological Simulation Program—Fortran (HSPF) (Bicknell and others, 1993) (jointly supported with the U.S. Environmental Protection Agency). A third model, the Rainfall-Runoff Modeling System (RRM) (Carrigan and others, 1977), is a lumped-parameter, continuous-simulation model only used to extend streamflow records for flood-frequency estimates.

These distributed-parameter models take into consideration the variation in slope, soils, vegetal cover, and land use in the study areas. In most cases, a lumped-parameter model cannot appropriately reflect physical changes in the study area. The three models are continuous-simulation models to provide a more complete assessment of the effects from constantly changing wet and dry periods. Also, with a long simulated time series of flow or storage of water at a location in the river basin, the probabilities of exceedances of flow and storage can be computed directly without the assumptions required by event-based simulations to combine the probability of a watershed condition with the probability of a meteorologic event. Because the three models are physically based, there is reasonable assurance that the modeled response to a land use or other change is a reasonable representation of the actual response that would be observed in the study area. The size of a typical study area is usually a few square miles to several thousand square miles so that detailed, micro-scale representations of the physics of the processes must be replaced with a physically-based macro-scale representation. Such macro-scale representations of the physical system include concepts of variable-source area, even within the smallest land segment simulated with a unique parameter set.

SOFTWARE DEVELOPMENT FOR WATERSHED MODELING

Several projects in the USGS have the objective to decrease the cost and improve the efficiency of using watershed models. Software shells are being created to help the modeler create the input necessary to simulate hydrologic processes in a river basin, edit the input to the model, and produce graphic visualizations of the model output. These shells are also being developed for different users: the researcher that wants to investigate different process algorithms, the scientist or engineer that wants to calibrate and validate the models for a river basin or region, and the manager that wants to use a calibrated and validated model to assess the effects of change. One such shell or system used in several urban studies is based on HSPF (Lumb and others, 1994, and in fig. 1).

Continuous simulation of hydrographs for long periods at many locations in a study area requires the management of large volumes of time-series data. The Watershed Data Management (WDM) file and associated data management program (ANNIE) assist the user with tasks of screening, plotting, editing, and tabling the data. The program METCMP is used to compute meteorologic time series and estimate values for periods of missing meteorologic data. The WDM file is very similar to the Hydrologic Engineering Center's Data Storage System (DSS) (Hydrologic Engineering Center, 1992). The batch version of HSPF can use both WDM and DSS files, but the interactive expert system for HSPF calibration (HSPEXP) and the interactive program to generate and simulate scenarios (GENSCN) use the WDM file.

Geographical Information Systems (GIS) provide an important tool in watershed modeling through the development of input for watershed models. Such tools are useful in automating the disaggregation of the river basin into land-surface units that have a similar hydrologic response to meteorologic inputs and in linking those units to the drainage system of tributaries and main channels (Jenson and Stannard, 1988). The automation becomes most effective when numerous land-use and disaggregation conditions are being investigated (Jeton and Smith, 1993). GIS can also be useful for graphic visualization of the results of simulations, but it is most powerful for development of input for the models.

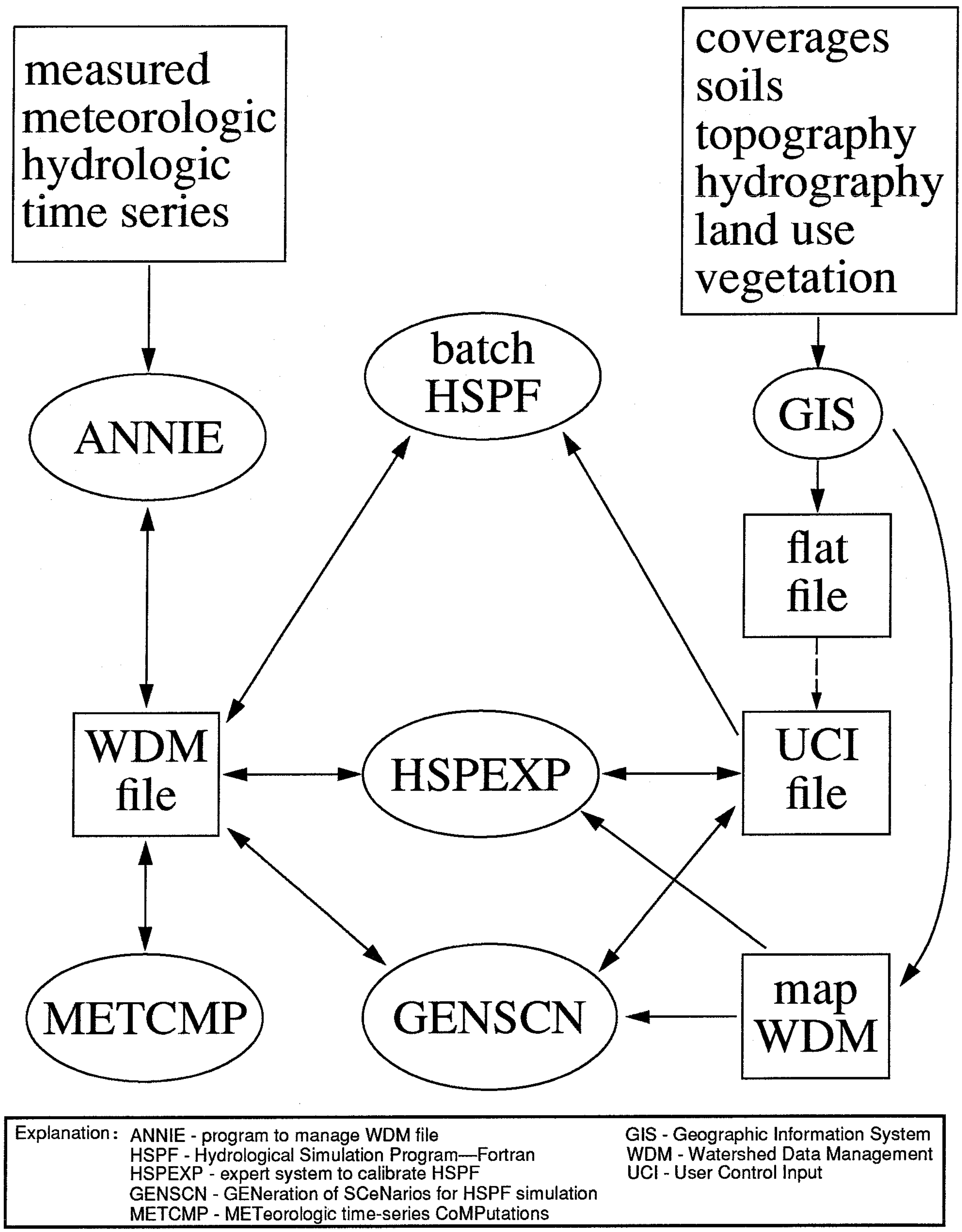

Figure 1. Software and data flow for river basin simulation.

Current efforts to interface complex modeling and complex GIS will extend the effort of Fisher (1989) and involves the development of a data model and format for data to be exchanged between GIS and a river basin model. In that way, any one of several GIS systems could be used for any one of several watershed models.

WATERSHED MODEL CALIBRATION/VALIDATION

Although input parameters for some watershed models can be estimated from characteristics of the drainage basin, the ability of a watershed model to more accurately simulate streamflow in a basin can be improved significantly by calibration. However, calibration requires additional data and more resources. For a single-purpose, single-site investigation, the costs of calibration commonly exceed the benefits. However, the USGS has several investigations in Washington, Texas, Illinois, Kentucky, and Maryland where HSPF applications are or will be numerous in the urban areas in these States. In such applications, a watershed model can be calibrated by associating the model parameters with measurable characteristics of watersheds, such as vegetation, soils, land use, and basin slope. The observed data for some of the watersheds are used for calibration and some are used for validation; the calibration process develops the associations or parameter sets for the major categories of land use and soils, and the validation process checks the associations of watershed characteristics with model parameters for the validation watersheds. The errors resulting from simulations for the validation watersheds provide an estimate of the errors that can be expected when the model is applied to ungaged watersheds in the region.

Dinicola (1990) completed a regional calibration and validation effort in the Seattle metropolitan area. Model parameter sets were developed for the following land categories:

till soils, forest cover, flat slopes
till soils, forest cover, moderate slopes
till soils, forest cover, steep slopes
till soils, non-forest cover, flat slopes
till soils, non-forest cover, moderate slopes
till soils, non-forest cover, steep slopes
outwash soils, forest cover, all slopes
outwash soils, non-forest cover, all slopes
Custer-Norma soils, forest cover, all slopes
Custer-Norma soils, non-forest cover, all slopes
saturated soils, all covers, all slopes
effective impervious areas, all slopes

HSPF parameter sets have been developed for each land segment type so the model can be applied to ungaged areas.

To assist with model calibration and validation, HSPEXP was developed to capture the expertise of the experienced modeler as a set of rules for parameter adjustments, which are based on computed discrepancies between simulated and observed streamflow. The system also summarizes results in several tables and 11 types of graphs. HSPEXP is an interactive program that uses a set of menus and forms for the selection of options and the modification of specifications and parameters. The major menu options include: select a basin, simulate, compute errors, display summary tables, produce graphics, get calibration advice, modify model parameters, and modify error criteria. The steps and procedures to calibrate HSPF with HSPEXP are shown in figure 2.

A TOOL FOR MANAGERS

How can hydrologic models and modeling methods be efficiently used to solve urban flooding problems? A vision shared by many in the USGS, as well as those in other agencies, universities, and the private sector, is to provide the water manager software to display maps and schematics of the river basin to easily and graphically identify alternatives, to execute the simulations, and to provide meaningful graphs and tables of the effects of the selected alternatives. The vision is being realized with modeling software that is categorized as decision support systems.

To produce effective decision support systems, greater communication and cooperation are needed between the managers, practicing hydrologists and engineers, research scientists, software engineers, and

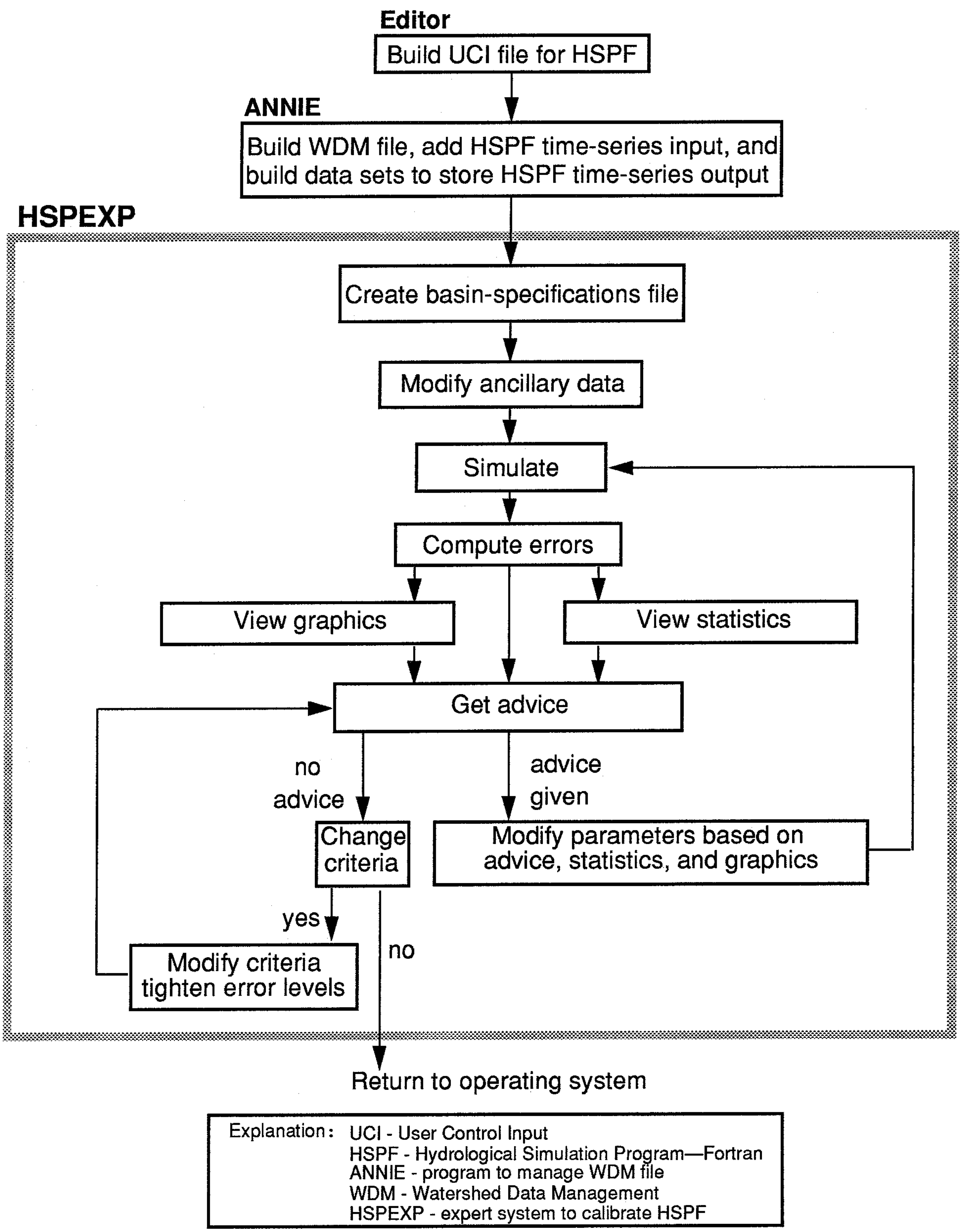

Figure 2. Detailed steps to calibrate HSPF with HSPEXP.

geographic information system specialists. The technologies of each are quite complex and difficult to learn but need to be combined to meet the demands of managers of urban drainage systems. Successes are emerging, but much remains to be done.

Within the USGS, the process of developing effective decision support systems has begun. First, an objective for an assessment of change is identified for a river basin or region. Data availability is evaluated and additional data networks are established as needed. The model parameters are defined in part by use of GIS. The regional calibration and validation of models are completed using modeling shells. Then the data files and model are placed within an application shell for managers. This application shell displays maps of the area and provides facilities to describe interactively the change to be evaluated and to identify locations where hydrologic information is required. The evaluation requirements provided by the manager are then translated by the shell to detailed changes to model inputs. The model is executed by the shell and the required information is displayed or tabulated in a form directly usable by the manager.

One of the shells that has been developed is shown on figure 1 as GENSCN, a program that has been developed for the Patuxent River in Maryland to better assess the relative impacts of urban and agricultural best management practices on the nutrient loads to the Chesapeake Bay and for the Truckee River in California and Nevada for the assessment of water allocations and management of the basin. GENSCN provides for the creation of new scenarios (best management practices as implemented by model parameter changes), simulation of the scenario with the model, and production of tables and graphs comparing scenarios.

SUMMARY

USGS program activities to provide hydrologic information for urban areas include regional regression equations and the development of hydrologic modeling systems for use by water-resource managers to more fully assess the effects of (1) detention storage, (2) land-use change, (3) implementation of best management practices, and (4) modifications to the drainage system. These needs are being addressed by the use and integration of simulation models of hydrologic processes, data-collection networks, principles of software engineering, and application of geographical information systems, and by a greater communication between the research scientist, practicing hydrologist and engineer, geographer, computer scientist, and the water-resource manager.

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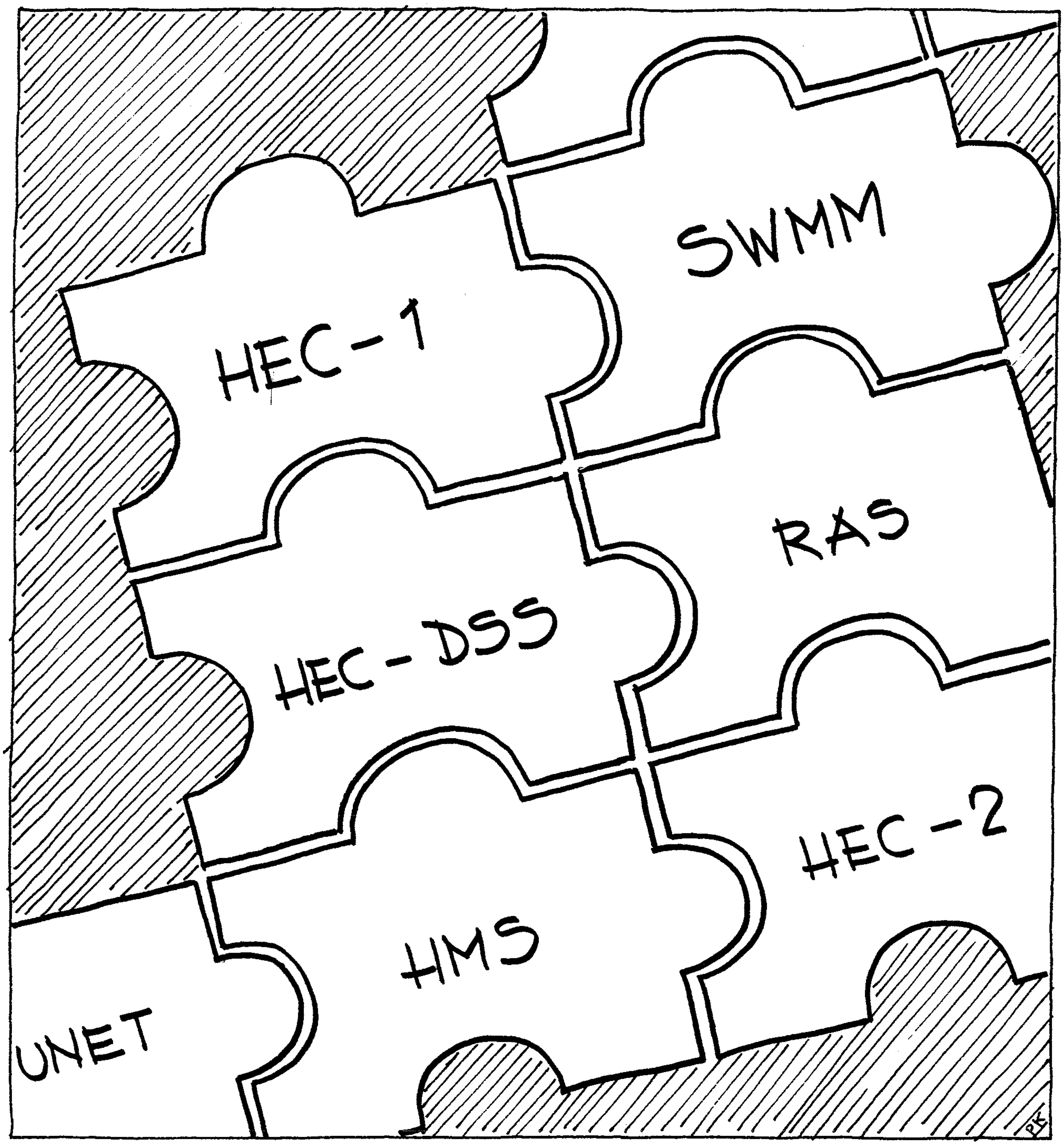
Session 3:

Modeling Approaches



SUMMARY OF SESSION 3: **Modeling Approaches**

Overview

This session covered a broad spectrum of topics. A central theme was the notion that there exists a wide variety of models, from simple to complex and from well established to newly developed, and that for each study undertaken, the selection of models is an important consideration. This selection of models, along with their coordinated use, is in fact what defines a modeling approach. Of course, as some participants pointed out that the thinking that occurs before models are considered can be the most crucial step in a modeling approach. Related topics were discussed including the comparison of individual models, differing levels of detail, and solution techniques. A second theme was the pragmatic issue of developing, supporting and maintaining models, along with the importance of coordinating such efforts amongst agencies.

Paper Presentations

Paper 8. Jerry Webb, Chief, Hydrology Section, Huntington District, presented a paper titled "Interior Drainage Analysis, West Columbus, OH." Jerry described a complex, multi-model interior hydrology/hydraulics study. First of all, calibration was attempted for SWMM with large storms, before Jerry got involved with the study. Because of the difficulties of volume accounting in such a scenario, the calibration had gone poorly, with such problems as answers which didn't make sense from a volume standpoint, and instabilities in the model. Subsequently, a different tack was taken whereby small storms - which did not cause ponding - were used for calibration. In this case, stable, reasonable results were obtained. The resultant maximum pipe flows were then used by subtracting them from the HEC-1 computed overland flow hydrograph. Regarding runoff computation, it was decided to lump the small runoff areas into larger subbasins. This decision was based on sensitivity analysis performed on a representative subbasin. It was found that subdividing required a great deal of effort and produced similar results as lumping the areas together. Modeling issues brought out by Jerry include: 1) running SWWM was difficult, tricky, and unstable, 2) maintenance and support of SWWM, as well as dissemination of SWMM experience is important, 3) interconnection between models via HEC-DSS was a crucial element, 4) the big picture should be looked at before any detailed work is begun (calibration for example), and 5) that an integrated modeling package capable of above ground and below ground analysis is needed. During the discussion, several SWMM spokespeople, including Huber, Schmidt, and Heaney pointed out that many of Jerry's problems with SWMM have been rectified, or are being worked on.

Paper 9. Harry Dotson, Hydraulic Engineer, Hydrologic Engineering Center, presented a paper titled "Case Study for Analysis of Interior Flood Damage Reduction Measures - Napa River, Napa, California." He described the various phases of performing an interior hydrology study using HEC-IFH. During the development of the runoff portion of the model, he was faced with the dilemma that HEC- IFH does not use kinematic wave method, the method for which parameters were available from previous modeling efforts. He was able to extract information from the kinematic wave model by calibrating a unit hydrograph based HEC-1 model against it.

Two other time saving ideas were used in Harry's study. Because the interior and exteriors flows were likely coincident, determined through simplified continuous simulation, hypothetical event modeling was satisfactory in the interior area. Also, because the interior area is well sewered, the surface flows were assumed to get to the line of protection; No sewer routing was performed.

Paper 10 Ben Urbonas, Chief, Master Planning and South Platte River Programs, Urban Drainage and Flood Control District, Denver, presented a paper titled "Municipal Stormwater Computer Modeling Program Needs" in which he outlines his agencie's model selection process. He described analysis of several models: HEC-1's kinematic wave, SWMM, HSPF, and CUHP. The conclusion of his analysis is that the more distributed methods are only capable of giving good results if they are calibrated and used for events on the order of the calibration events. The trend was for these methods, when uncalibrated, to overestimate peaks. As he points out, overestimating peaks, and its resultant impact on budgets, is frowned upon by municipalities who both control money and flood control decisions. He goes on to make several recommendations for future focus in the development of stormwater runoff models. During discussions, several participants supported Ben's ideas but questioned his particular model comparison because some were calibrated, while others were not.

Paper 11. Peter Kaufmann, Lecturer, Burgdorf Engineering School, Burgdorf, Switzerland, presented a paper titled "Urban Hydrology in Switzerland." Peter gave a review of past Swiss practices, up to the present, and outlined the way of the future. Switzerland's conditions are unique and resulted in some features such as 99% connection to combined sewer systems. The combined system approach was natural for many years because the systems were underutilized. However, with ever increasing population, inefficiencies became pronounced. Also, increasing environmental concerns, along with tightening budgets resulted in an acute need for more efficient systems. The result was new goals for urban planning, as well as new methods to accomplish these goals. Some of the goals mentioned by Peter were: 1) Local infiltration of rainfall, if possible, before it enters the system, 2) reduction of unpolluted water from sources like creeks entering the system, and 3) adoption of a "whole watershed" view of urban drainage planning. To achieve these goals, a top down approach is described whereby the important aspects of a study are identified, and possible solutions proposed, as the first step. Subsequent steps include data acquisition, and documentation, and ultimately detailed analysis. A distinguishing feature, brought out by questions, of the Swiss approach is that these steps are actually mandated for all urban drainage studies. In fact, Peter pointed out that other European countries are also adopting the guidelines.

Paper 12. J.S. Wang, Bechtel Corporation, San Francisco, Presented a paper titled "Problems Encountered and Solved in the Application of SWMM." He shared experience in the application of SWMM's Runoff and EXTRAN blocks. The emphasis was on adjusting the model setup to overcome model deficiencies and bugs, as a stopgap measure in lieu of code modification. Problems covered include: 1) The limit on the number of subcatchments permissible at an inlet, 2) flooding at junctions, 3) weir/flap option, and 4) in-line pump option. Additional advice is given on output evaluation and dealing with large numbers of computed flow data. The above observations and advise are given with the thinking that even though SWMM is very powerful, no model can be perfectly suited for every application scenario or completely error free. During the discussion, it was pointed out that many of the problems encountered by J.S. have been fixed. However, new ones will surface and, as J.S. points out, the user must be able to work around deficiencies by understanding the system and the model.

Paper 13. David Kibler, Professor of Civil Engineering, Virginia Tech, Blacksburg, presented a paper titled "Model Choice in Urban Flood Frequency Analysis." In order to gain insight into how various models respond to similar data inputs, David compared several models for a test watershed in western Washington. Recognizing that rainfall distribution methods can also play an important role, especially considering that the test basin response is fifteen minutes, he also used several standard distribution patterns. The models compared are: 1) Rational, 2) USGS 7 and 3 parameter urban flood regression, 3) SCS TR-55 and TR-20, 4) PSRM, and 5) USGS regression developed from Bellevue sites. Several rainfall distributions were used including NOAA Atlas, Yarnell, and various schemes based on evaluation of rainfall from within the basin. The outstanding conclusion is that the choice of model is of second importance to the choice of the rainfall distribution; Different models, with the same rainfall gave similar results.

David also examined the impact of the solution technique for the kinematic wave method. The finite difference scheme used in HEC-1, and the Newton-Rapson used in SWMM were compared against the characteristic method, which was assumed to be a quasi-analytic solution. He concluded that having a variable distance step is the key to improving results. Also identified as important is a variable n value for laminar flow that occurs early and late in the hydrograph. David concluded that hydrologic-physical effects must be separated from the artificial impacts of model choice.

Paper 14. Jim Heaney, Professor, Department of Civil, Environmental, and Architectural Engineering, and Faculty Associate, CADSWES, University of Colorado at Boulder, presented a paper titled "Towards Integrated Urban Water Systems Management." Jim reviewed model development and support in the U.S. during past decades, and goes on to present the current status as dormant, underfunded, and disintegrated. During the review of past efforts, Jim describes the accomplishments of many individuals and agencies. He especially makes note of their integrated, systems approach to looking at urban hydrology and hydraulics. With decreasing funding starting in the 80's, research efforts in urban hydrology have suffered. Jim outlines the options for the future; do nothing in which case other countries will take the lead, increased federally funded research for specific needs, or create new funding sources such as those conceived in the sixties to provide support for a systems approach to research. Jim calls for the development of a new agenda to achieve these goals.

Paper 15. Wayne Huber, Professor and Head, Dept. of Civil Engineering, Oregon State University, presented a paper titled "Updating, Maintaining and Improving Available Models." He illustrated the successful mechanisms which have existed for development, support, and maintenance of models by highlighting some of the models which have been around for some time. He attributes model longevity to their being non-proprietary, having available source code, and good documentation. He points out that these qualities usually result from a model having federal support, either in the form of extra-mural funding such as has existed for SWMM, or in-house control as has occurred at HEC. The issue of what makes a good model was also addressed. Arguing from a regulating agency point of view, he gave two criteria for a good model: 1) Established track record and the accompanying confidence in its use, and 2) consistency. By consistency, he means that similar results are obtainable from different engineers. He mentioned the SCS loss method as an example of a consistent method, but points out that any method can be made to produce consistent results with enough guidance. He also pointed out that the key to good guidance for a method is that its algorithms match the true physics of the rainfall-runoff processes.

After Wayne established the past modelling successes and outlined what future models should strive for, he addressed the current options available for their development, support and maintenance. In particular, the virtues of building on the old - shorter development time, proven routines - were compared to the virtues of starting fresh - advantage of new paradigms and easier to maintain final product. Also, internal versus external development was compared, and the information highway was held up as an aid in all model support and development activities. Subsequent discussion focussed on the funding side of model development and support. It was pointed out that the mechanism whereby HEC has historically supported and maintained models is about to change. James recognized this as a loss not only for U.S. engineers but for those of Canada who have gained from U.S. federally supported models.

INTERIOR DRAINAGE ANALYSIS, WEST COLUMBUS, OH

by

Jerry W. Webb[1]

INTRODUCTION

Purpose. The interior drainage analysis of the West Columbus, Ohio area poses a challenge to standard techniques and methodologies of hydrologic investigation. Located along a long meandering bend of the Scioto River, this highly urbanized area is drained by an extensive storm and sewer system that provides a relatively low level of protection against interior storm events. The Corps of Engineers has designed a levee and floodgate system which will provide protection against flooding from an exterior source, but has not resolved the issue of residual flooding associated with an interior storm event. The analysis performed by the Corps of Engineers addressed coincidental frequency of flooding, flood warning systems, existing capacity of storm and sanitary systems, existing pump station capacities, and routing of flows across a maze of geometric controls. The modeling efforts included use of the Storm Water Management Model (SWMM) developed by the Environmental Protection Agency; HEC-1, Flood Hydrograph Package and HECIFH, HEC Interior Flood Hydrology, both developed by the Corps of Engineers Hydrologic Engineering Center. The interaction of these models, strengths, weaknesses, and limitations of application to this complex watershed will be presented. The problems encountered during this study clearly indicate the need for development of a comprehensive model that can accommodate the variety of drainage conditions associated with urban drainage systems.

Project Description & Flood Conditions. The project described in this paper is located on the right bank of the Scioto River, in the western part of the City of Columbus, Ohio, generally bounded by the Scioto River on the north and east and Interstate 70 on the south and west. Without the proposed Corps flood control project, the West Columbus area, sometimes referred to as the Franklinton area, is subject to flooding from the river by overtopping and/or possible breaching of existing levees and elevated railroad embankments on the north side, and backwater flooding through the Interstate 70 underpasses on the south side. Localized areas could also be flooded by significant rainfall events that would exceed the capacity of the existing storm sewer system. For these events, overland flow is impeded by existing topographic features, such as the elevated road and railroad embankments that subdivide the interior area. The proposed plan of improvement that is recommended by the West Columbus, Ohio LPP Interior Flood Control DM # 5, dated October 1992, and shown on Exhibit No. 1 consists of a levee/floodwall combination for protection against flooding from the river, and a collector/interceptor and pump station system to remove interior flood waters. The levee/floodwall project baseline is approximately 5.3 miles long and protects approximately 1300± acres.

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[1]Chief, Hydrology Section, Huntington District, U.S. Army Corps of Engineers

INTERIOR FLOOD CONTROL DESIGN PROCEDURES

General. The Interior Flood Control analysis was conducted, using three mathematical models to simulate the operation and response of each interior flood hydrology system considered. The three models that were used are the U.S. Environmental Protection Agency's Stormwater Management Model(SWMM), the Flood Hydrograph Package(HEC-1) and the Interior Flood Hydrology Package(HECIFH). Previous modeling efforts did not account for the underground drainage network. This limitation resulted in the need to perform this study to a higher level of detail than previous investigations. A more detailed computational scheme which would account for underground capacities for various operational scenarios was necessary.

Storm Water Management Model (SWMM) . Due to the complex nature of the storm sewer network and surface drainage patterns, careful consideration was given to the selection of the mathematical model that would be used for the interior flood control analysis. The existing drainage network is extensive and modeling limitations were established before proceeding with the computer model. A meticulous process was involved in ascertaining each sub-catchment size with regard to each runoff intrusion node in the stormwater system. West Columbus storm sewer profiles and 1-foot contour mapping of the project area were used to develop this data for the SWMM model. RUNOFF block hydrographs were then used to generate hydrographs that were then entered into the EXTRAN block at these predetermined nodes. The EXTRAN block then computed network flow routings. In order to improve the functionality of the model, it was decided that only those storm sewers of 24 inches in diameter or larger would be used in this model.

Rainfall data was entered into the RUNOFF block of SWMM for the Standard Project Flood centered over Columbus. The intent was to deal with the worst situation involving runoff first and calibrate SWMM to that condition. Time was spent developing an EXTRAN surface drainage model. The effort was eventually abandoned because of program instabilities and the need for runoff volume accountability. A new approach was taken which was based on using the 5, 10 and 25 year hypothetical rainfall events. The underground network results from the SWMM model were reasonable and a new modeling procedure was adopted using the total maximum underground discharge at subarea boundary exit points. These discharges were applied to the HEC-1 model developed for West Columbus.

HEC-1, Flood Hydrograph Package. It became evident, after developing a surface network model, that runoff volume accountability would not be easily accomplished with the SWMM computer program. A prudent solution to this situation was to pursue the development of a HEC-1 model to route and combine the surface runoff. HEC-1 is a computer program that models the precipitation-runoff process. The model is limited to single event analysis and routing techniques that do not account for downstream backwater conditions.

Surface runoff in this project generally flows in a southern direction. The floodwall and elevated railroad and highway embankments are the controlling boundaries that divide the interior into hydrologic subareas. Each subarea functions as a small reservoir with respect to overland flow, with the highway and railroad underpasses serving as spillways. The SWMM model provided the peak discharge capacity of the underground system for incorporation into the HEC-1 surface model. It should be noted that each of the subareas has a major ponding area. This reasoning governed the development of the HEC-1 model for the West Columbus LPP. The overland flow discharge capacities of the railroad and highway underpasses were estimated by normal depth computations.

A sensitivity analysis was conducted of the HEC-1 computation of runoff for a sample subarea. The subarea was subdivided into smaller drainage areas, similar to the sub-catchment areas defined in the SWMM model. The discharge hydrographs were extracted for each sub-catchment from the RUNOFF block, and combined and routed with HEC-1 to the location of low point ponding in the subarea for comparison with the hydrograph that was computed with the drainage area defined as the entire subarea. The results of this study show that the ponding elevations computed by either method are comparable, and that the timing effects produced by the single area computation with HEC-1 are more representative of anticipated timing. Because the ponding elevations were comparable and because the sub-catchment computations were extremely time consuming, the single area computation was retained as the adopted method for development of flood frequency data for the remaining areas.

HECIFH, Interior Flood Hydrology Package . Existing conditions were evaluated using the HEC-1 model. A problem was encountered when a need developed to examine the proposed plan of improvement in the West Columbus project. These plans consisted of a combination of interceptors and pump stations designs. HECIFH was used to evaluate the numerous alternatives. The HEC-1 model was used prior to HECIFH because of the multi-basin drainage scheme of the West Columbus project.

HECIFH is a menu driven computer program that can be used to determine runoff into a ponding area adjacent to a levee and then route the inflow though the levee utilizing gravity outlets and/or pumping capacity. The rainfall-runoff process, streamflow routing, auxiliary inflow, diversions, and seepage can be simulated as well as complex configurations of gravity outlets and pumping facilities. Period-by-period, monthly, annual and total analysis summaries are generated for all applicable parameters during simulation. Interior area elevation-frequency relationships can be determined for various alternative plans by using continuous simulation or hypothetical storm event analysis.

DESIGN CRITERIA

Rainfall. Rainfall for West Columbus was obtained from Technical Paper No. 40, Rainfall Frequency Atlas of the United States published by the National Weather Service, for durations from 30 minutes to 24 hours and return periods from 1 to 100 years. This study addressed the 2-year, 5-year, 10-year, 25-year, 50-year and 100-year rainfall amounts based on a 24-hour duration. Total rainfall amounts were applied to a triangular precipitation distribution, with the maximum rainfall depth occurring during the central part of the storm. The Standard Project Flood rainfall data was developed by distribution methods, outlined in EM 1110-2-1411, Standard Project Flood Determination, for a 96 hour period with an isohyetal pattern centered directly over West Columbus.

Infiltration Losses. As with the drainage area, the percentage of impervious area was extracted from the contour mapping during preparation of sub-basin data for the SWMM model. The sub-basin impervious area data within each subarea was totalled, and the resulting percentage of impervious area ranged from 52% to 30%. The Green-Ampt infiltration loss rate method was selected to compute rainfall loss rates for the project area. The first component of this method predicts the volume of water which will infiltrate into the soil before the surface becomes saturated. Then, infiltration capacity is predicted by the Green-Ampt equation. Thus, infiltration is rated based on the volume of water infiltrated as well as the moisture conditions in the soil surface zone. Data for the Green-Ampt input parameters, were obtained from a report entitled "Soil Survey of Franklin County, Ohio", published by the Soil Conservation Service, and from information in the SWMM user's manual.

Hydrographs. Unit hydrographs were developed from the Soil Conservation Service(SCS) unit hydrograph method. Values of 1.51, 0.96, 2.35, 3.25,and 1.11 hours were estimated for time of lag for the five subareas included in the model. The 10-year, 25-year, 50-year, 100-year and Standard Project Storm rainfall was applied to each of the designated subarea unit hydrographs based on a 5 minute incremental time period.

Coincidental Frequency Considerations. Sufficient records for extreme historic events on the Scioto River with the associated local interior storms are not available, whereby a coincident statistical analysis could be performed that would produce meaningful results. Therefore, a method had to be developed for the determination of the joint probability, or coincidental frequency, associated with interior and exterior events. This was accomplished by determining interior ponding frequency information both for low and high river conditions, to establish the upper and lower bounds of the joint probability curve. The coincidental frequency in each subarea was determined through engineering judgement and graphical analysis of results of the two conditions. In the absence of statistical approaches, graphical analysis dictates that the upper and lower bounds of the curve are controlled through the operational scenario that predominantly produces ponding.

Low river conditions would most likely occur coincident with frequent interior storms such as a 10-year event, or less. The capacity of the existing storm sewer system is exceeded and interior flooding begins between a 5- and 10-year event. Therefore, based on SWMM results, the low river condition was used to represent the coincidental curve, up to the 10-year event.

The high river scenario involves the operation of a complex arrangement of gates and closures that begin operation at various frequency storm events. The existing storm sewer system begins to become ineffective with a 10-year water surface on the Scioto River and would be totally ineffective with a 25-year water surface on the river. High river conditions would most likely control interior events in excess of the 50-year event. This means that a 10- to 25-year water surface profile on the Scioto River would occur coincident with a 50-year or greater interior storm event. Therefore, with the upper and lower bounds of the curves for the individual subareas established, the portion of the curve between the 10- and 50-year storms was graphically smoothed to fit the boundary conditions. Acknowledging the subjective nature of this procedure, an envelope curve was developed to evaluate the sensitivity of these assumptions on the economic analysis. The curves were fully developed and furnished for use in the economic analysis of project benefits and damages. An example of this comparison is provided as Exhibit No. 2.

Low River Existing Ponding Elevations. The existing condition ponding elevations were derived by using SWMM to determine the maximum underground discharge capacity at the boundaries of each subarea evaluated in this study. After comparing the computed discharges of the individual conduits at each subarea boundary for the 5-year, 10-year and 25-year events, it was observed that peak discharges do not increase significantly for the increasing rainfall amounts as shown on Exhibit No. 3. The maximum underground discharge from each subarea was then established by the summation of each of the individual conduit discharges at each subarea boundary. This discharge was then incorporated into an HEC-1 model for the final determination of the existing condition elevations by removing it as a constant flow rate from the bottom of each of the reconstituted hydrographs for each subarea. The HEC-1 model was developed using existing facilities between each subarea and rating the outflow areas based on computed ponding.

Maximum Existing Ponding Elevations. The "worst case" condition for the interior was derived by using the HEC-1 program with no outlet at Renick Run and only existing pumping facilities at ST-2 and ST-8. Reconstituted hydrographs for each of the subareas were routed and combined through the existing interior area. The hydrographs represented the worst possible condition since the underground storm sewer capacity was not eliminated. HEC-1 calculated elevations in each ponding area. These elevations represent instantaneous existing condition elevations and do not represent the steady-state situation. In order to evaluate this situation, rainfall excess from each frequency and theoretical event for each subarea was changed to a volume. These amounts in each subarea were accumulated and converted to final static elevations. The largest elevation between the static and steady-state conditions was then used to derive the final maximum existing ponding elevations in each subarea.

RESULTS OF INTERIOR FLOOD CONTROL ANALYSIS

Existing Storm Drainage and Collection Systems. The existing storm water collection system in the interior of the proposed local protection project included in the analysis consists of a complicated network of gravity flow conduits and two storm water pump stations. Generally, this system, along with the existing surface topography, collects and transmits storm runoff toward the central and southern region of the interior. Then, large underground conduits provide relief from the interior by transmitting storm water to the south under Interstate 70 to the existing Renick Run storm and sanitary sewer pumping facilities, located approximately two miles to the south of the proposed project area, for disposal into the Scioto River.

Recommended Interior Flood Control Plan. The proposed interior flood control plan consists of two storm water pump stations. Both are associated with gatewells and sluice gates that are required as a part of the proposed levee/floodwall system that will provide a positive cut off during Scioto River flood conditions. The Dodge Park facility will have a capacity of 100,000 gpm. A section of the existing 72-inch storm sewer between the proposed pump station and the river will be replaced along with an existing headwall. A 72-inch pipe with a network of inlets along Rich Street will also be provided to improve the supply capability of the sewer system to the proposed pump station.

The Cypress Avenue pump station will have a capacity of 180,000 gpm. A 60-inch collector/interceptor will be provided to divert flow from the existing storm sewers at Nace, Glenwood and Yale Avenues to the proposed pump station. The existing storm sewer from the proposed pump station to the existing junction box south of Mound Street will be replaced by two 8 foot by 7 foot box culverts.

Project Cost of NED Plan. The costs associated with construction of the West Columbus local protection project, interior flood control features have been estimated to be $9,409,000 in October 1992 price levels. This estimate includes the design and construction of two pumping facilities, (100,000 GPM at Dodge Park and 180,000 GPM at Cypress Avenue), other appurtenant items and required relocations and real estate.

Benefits of Recommended IFC Features. The selected plan would alleviate approximately 75% of the $1,391,000 of average annual flood damages resulting from interior flood events. In excess of one million dollars in average annual benefits would be directly attributable to the recommended Interior Flood Control features. The recommended features would produce $218,000 in net NED benefits and have an incremental benefit-to-cost ratio of 1.3.

Of the 1160 structures damaged by the 100-year interior event only 274 would experience damage with the recommended features in place. Of the 274 structures damaged only twenty two would experience flooding above the first floor. The remainder would suffer only basement or foundation related damage. Under baseline conditions approximately 324 acres of the study area would be inundated by the 100-year interior event. The recommended plan would reduce this area of inundation to less than 30 acres.

Residual Flooding. The recommended plan will not completely eliminate interior flooding as shown on Exhibit No. 4. The project area will continue to experience the nuisance flooding associated with the minor interior low points and incidental street flooding. These pockets of stored water in most cases eventually drain into the stormwater system. The technical analysis involved in determining the volume of water associated with each of these low points would have been time consuming and would not alter the formulation of the recommended plan. It is for this reason that resources were not used to determine the elevations in these ponding areas.

The following table provides flood depths and the extent of inundation produced by the 100-year frequency flood for various operational conditions and with the recommended plan of improvement.

FLOOD DEPTHS
AREA SUBJECT TO INUNDATION
(100-YEAR FREQUENCY FLOOD)

Subarea Description	DEPTH IN FEET			
	High Scioto River w/ No Project	Low Scioto River w/ No Project	Coincidental w/ No Outflow	NED Plan
Western	5.0	2.3	3.8	3.8
West-Central	10.7	4.0	5.1	3.0
East-Central	9.3	3.9	6.2	2.9
Eastern	5.7	2.9	3.3	1.4
Inundation Area	870 Acres	75 Acres	324 Acres	30 Acres

The proposed pump stations were designed strictly for relief of interior flooding during high river events for which levee/floodwall closures will be required. However, it appears that the proposed Dodge Park facilities could be used to relieve interior flooding along Rich Street during moderate river rises, such as occurred on 13 July 1992. This is due to fact that the invert of the flap gates at the outfall of the 72-inch storm sewer are positioned at normal pool formed by the Greenlawn weir, and even a moderate rise in the river closes the gates. Therefore, the only outflow capability from the eastern section of the project is through the 36-inch by 48-inch combined sewer and by backflow into the 42-inch pipe from the junction box at Davis Avenue, where the existing 72-inch storm sewer begins. Therefore, outflow from the eastern section of the project during a significant interior rainfall with moderate river conditions will occur though the combined sewer. If the proposed pump station could be activated during an event similar to the 13 July 1992, outflow capability from the eastern section would be significantly improved. It is reiterated that this condition would occur coincidentally with the design for high river conditions. Since low river condition improvements were not considered to be a part of the scope of this study, and the potential was not recognized until late in the study, the potential improvements have not been analyzed or guaranteed.

SUMMARY

The problems encountered with the interior hydrology analysis of the West Columbus, OH LPP are typical of many urban areas throughout the country. There appears to be a need for a comprehensive model that can account for underground and surface drainage in a more efficient manner. This study utilized three independent models that required a significant amount of engineering judgement and experience to apply to the site specific requirements of the project. Each model had it's strong points and weaknesses. None of the models are capable of performing the variety of analysis necessary to assess the flooding problem experienced in the West Columbus area. A summary of the analytical method used in this study is shown on Exhibit No. 5.

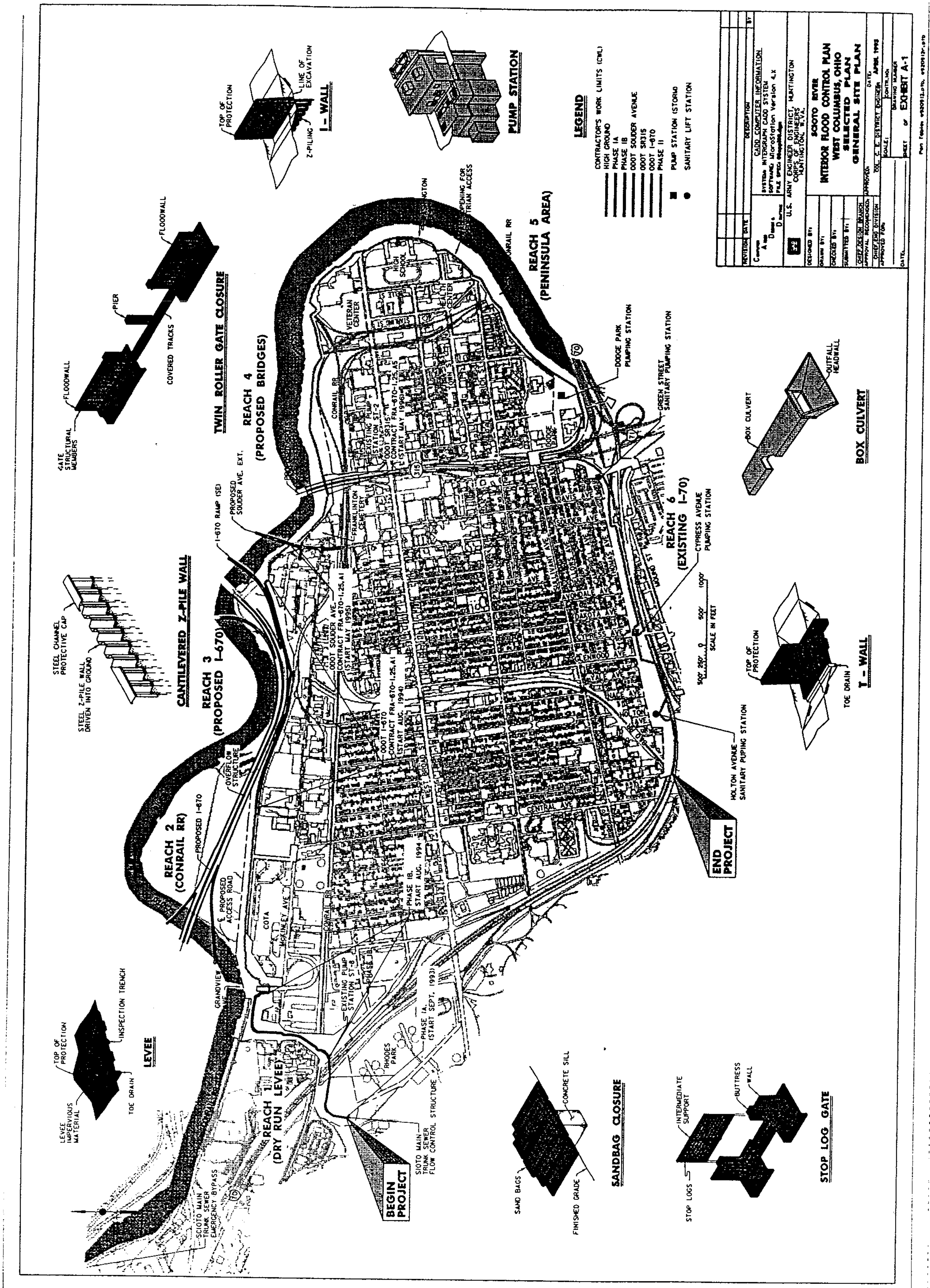

SCIOTO RIVER
INTERIOR FLOOD CONTROL PLAN
WEST COLUMBUS, OHIO
SELECTED PLAN
GENERAL SITE PLAN
EXHIBIT A-1
U.S. ARMY ENGINEER DISTRICT, HUNTINGTON
CORPS OF ENGINEERS
HUNTINGTON, W.VA.
CADD COMPUTER INFORMATION
LEGEND
CONTRACTOR'S WORK LIMITS (CWL)
HIGH GROUND
PHASE IA
PHASE IB
ODOT SOUDER AVENUE
ODOT SR315
ODOT I-670
PHASE II
PUMP STATION (STORM)
SANITARY LIFT STATION
I - WALL
TOP OF PROTECTION
Z-PILING
LINE OF EXCAVATION
PUMP STATION
TWIN ROLLER GATE CLOSURE
FLOODWALL
PIER
COVERED TRACKS
GATE STRUCTURAL MEMBERS
CANTILEVERED Z-PILE WALL
STEEL CHANNEL PROTECTIVE CAP
STEEL Z-PILE WALL DRIVEN INTO GROUND
LEVEE
INSPECTION TRENCH
TOE DRAIN
LEVEE IMPERVIOUS MATERIAL
SANDBAG CLOSURE
CONCRETE SILL
SAND BAGS
FINISHED GRADE
STOP LOG GATE
INTERMEDIATE SUPPORT
BUTTRESS WALL
STOP LOGS
BOX CULVERT
OUTFALL HEADWALL
T - WALL
REACH 1 (DRY RUN LEVEE)
REACH 2 (CONRAIL RR)
REACH 3 (PROPOSED I-670)
REACH 4 (PROPOSED BRIDGES)
REACH 5 (PENINSULA AREA)
REACH 6 (EXISTING I-70)
BEGIN PROJECT
END PROJECT
DODGE PARK PUMPING STATION
GREEN STREET SANITARY PUMPING STATION
CYPRESS AVENUE PUMPING STATION
HOLTON AVENUE SANITARY PUMPING STATION
EXISTING PUMP STATION ST-8
SIOTO MAIN TRUNK SEWER FLOW CONTROL STRUCTURE
SCIOTO MAIN TRUNK SEWER EMERGENCY BYPASS
PHASE IA (START SEPT. 1993)
PHASE IB START AUG. 1994
ODOT I-670 CONTRACT FRA-670-1.25.AI (START AUG. 1994)
ODOT SOUDER AVE. CONTRACT FRA-670-1.25.AI (START MAY 1993)
I-670 RAMP (SE)
PROPOSED SOUDER AVE. EXT.
PROPOSED I-670
PROPOSED ACCESS ROAD
OVERFLOW STRUCTURE
GRANDVIEW AVE
FRANKLINTON CEMETERY
VETERAN CENTER
HIGH SCHOOL
CONRAIL RR
CENTRAL AVE
RHODES PARK
COTA
SCALE IN FEET

EXHIBIT NO. 1

WEST COLUMBUS(SUBAREA 4)
INTERIOR DRAINAGE ANALYSIS
COINCIDENTAL FREQUENCY

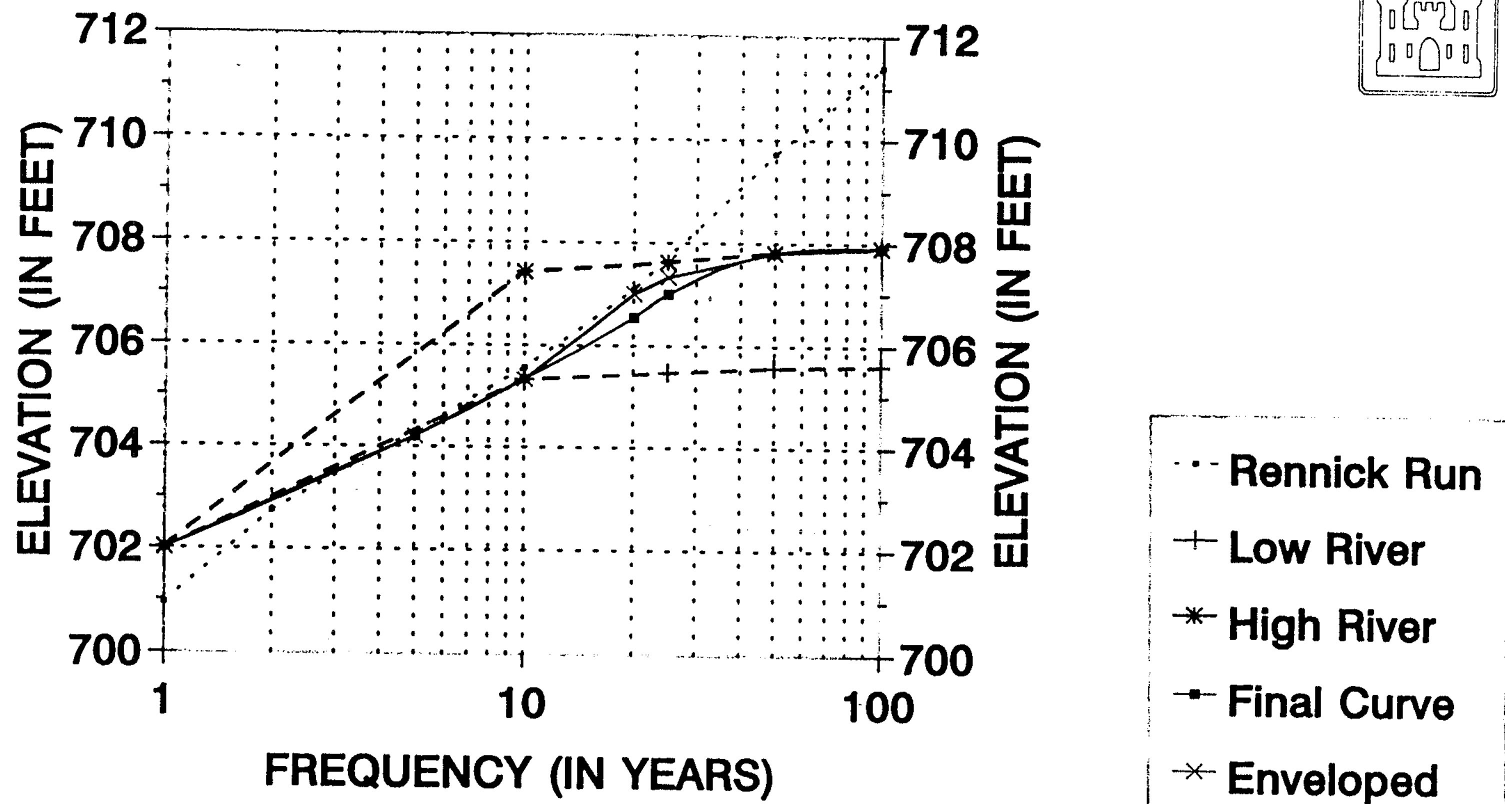


EXHIBIT NO. 2

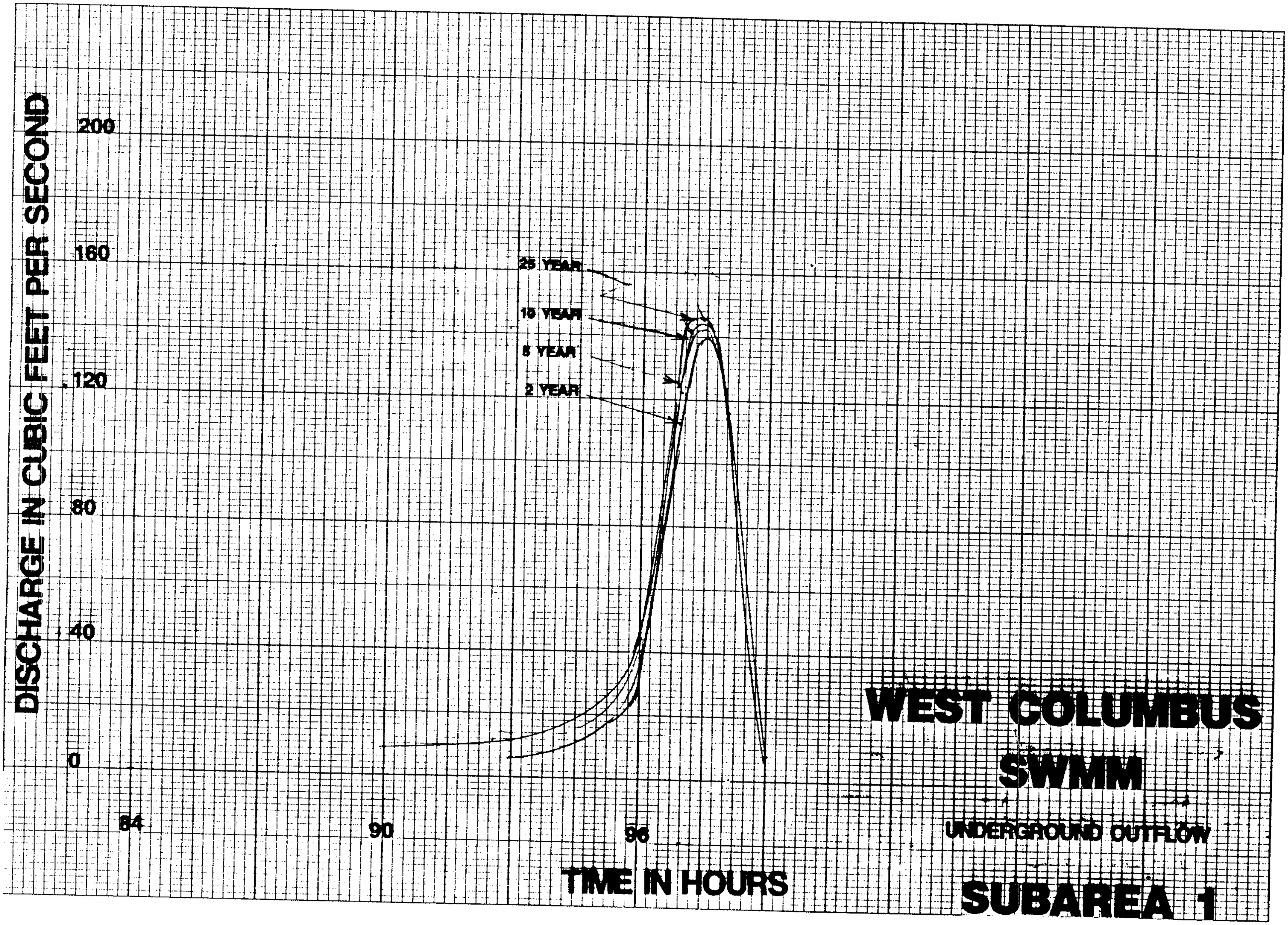
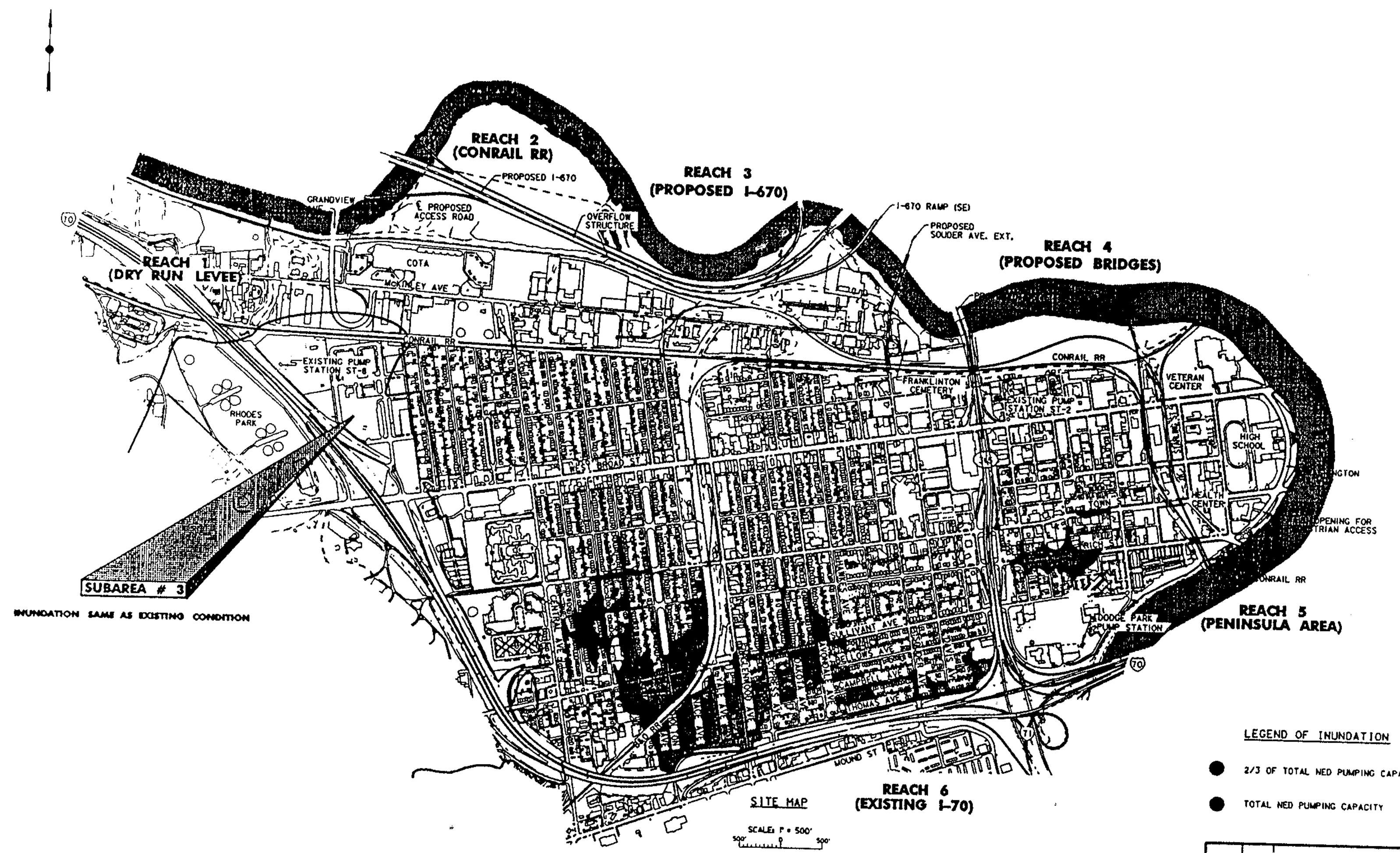

DISCHARGE IN CUBIC FEET PER SECOND
200
160
120
80
40
0
25 YEAR
10 YEAR
5 YEAR
2 YEAR
84
90
96
TIME IN HOURS
WEST COLUMBUS
SWMM
UNDERGROUND OUTFLOW
SUBAREA 1

EXHIBIT NO. 3



LEGEND OF INUNDATION

- 2/3 OF TOTAL NED PUMPING CAPACITY
- TOTAL NED PUMPING CAPACITY

100-YEAR FREQUENCY INUNDATION

REVISION	DATE	DESCRIPTION	BY

CADD COMPUTER INFORMATION
SYSTEM: INTERGRAPH CADD SYSTEM
SOFTWARE: MicroStation Version 4.X

U.S. ARMY ENGINEER DISTRICT, HUNTINGTON
CORPS OF ENGINEERS
HUNTINGTON, W.VA.

SCIOTO RIVER
COLUMBUS, OHIO
WEST COLUMBUS L.P.P.
NED PLAN
100 - YEAR INUNDATION

MAY 1993

PLATE 6

EXHIBIT NO. 4

WEST COLUMBUS
ANALYTICAL METHOD

SYNTHETIC RAINFALL

- HYDROMETEROLOGICAL REPORT 35, NATIONAL WEATHER SERVICE
- TECHNICAL PAPER NR. 40, FREQUENCY ATLAS OF THE UNITED STATES, NATIONAL WEATHER SERVICE
- STANDARD PROJECT FLOOD EM 1110-2-1411

STORMWATER MANAGEMENT MODEL(SWMM)

- RUNOFF BLOCK - CALCULATES UNIT HYDROGRAPHS BASED ON BASIN PARAMETERS
- TRANSPORT BLOCK - ROUTES AND COMBINES FLOWS
  - MODELS UNDERGROUND SYSTEM
  - MODELS PUMP STATIONS
  - EXTERIOR OUTLET CONTROLS

UNDERGROUND CAPACITY

HEC-1 MODEL

- SCS UNIT HYDROGRAPH DEVELOPED FOR 5 MAJOR SUBAREAS
- INTERIOR MODEL DEVELOPED
  - GREEN AND AMPT LOSS RATES (LG)
  - ROUTE AND COMBINE FLOWS (RD/HC)
  - REMOVE UNDERGROUND FLOWS (RL CARD)
  - PASS UNDERGROUND FLOWS INTO NEXT SUBAREA (WP CARD)
  - STORAGE ROUTINGS AT RAILROAD UNDERPASSES (SA/SE/SQ)

DATA STORAGE SYSTEM(DSS)

DSSMATH - MANIPULATE/CORRECT DATA

INTERIOR FLOOD HYDROGRAPH PACKAGE(HECIFH)

Case Study for Analysis of Interior Flood Damage Reduction Measures Napa River, Napa, California

by

Harry W. Dotson[1]

1. Introduction

This case study presents part of the results of the hydrologic engineering analysis of interior flood damage reduction measures for the City of Napa, CA conducted by the Hydrologic Engineering Center (HEC) for the Sacramento District Corps of Engineers. The objective of the hydrologic engineering analysis was to determine: 1) the minimum outlet facility associated with the proposed line-of-protection; 2) the stage-frequency relationships for the without project conditions; and 3) the stage-frequency relationships for a range of gravity outlet and pumping station sizes and configurations for the interior areas.

This case study presents the results of applying the HEC-IFH program for evaluation of one of the several interior area involved in the overall investigation. The case study includes a description of 1) the study area, 2) the Napa River proposed flood damage reduction project, 3) interior area data and information, 4) without-project conditions analysis for minimum facility analysis 5) minimum facility analysis, and 6) stage-frequency for interior flood damage reduction plans. The Sacramento District was responsible for developing data for the without-project conditions, including stage-damage relationships, cost estimates of the flood damage reduction measures, and other data required to perform the economic analysis of each plan. The design requirements for conveyance systems, inlet and outlet works, and the economic analysis of project components are beyond the scope of the case study presented herein.

2. Description of the Study Area

The Napa River basin is located about 50 miles north of San Francisco, CA. The basin is about 50 miles long on a north-south axis, varies between five and ten miles in width, and has a drainage area of about 426 square miles. (See Figure 1.) The north, east and west limits of the basin are formed by portions of the north coast mountain range. The southern limit is bounded by the San Pablo Bay.

[1]Hydraulic Engineer, Hydrologic Engineering Center, 609 Second Street, Davis, CA 95616

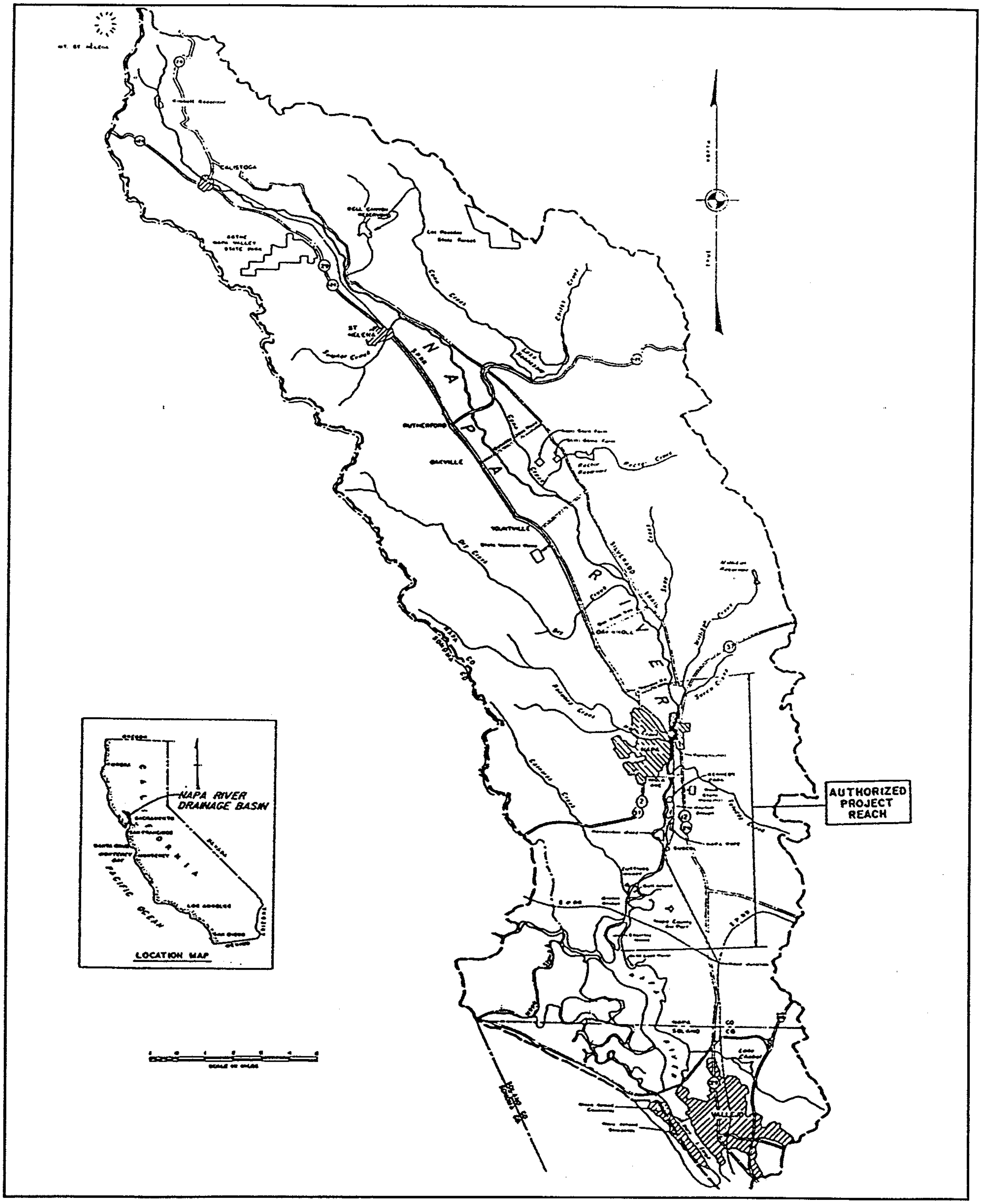


Figure 1. Napa River Basin

The Napa River originates near Mount St. Helena and empties into the Mare Island strait which flows into the tidal marshlands and sloughs of San Pablo Bay. The city of Napa, CA is located in the lower third of the basin and has a population of about 60,000. Basin land use consists mainly of vineyards in the valley area north of the City of Napa and limited mixed use in the marshlands and reclaimed tidal lands south of the city.

3. Description of the Proposed Flood Damage Reduction Project

a. Napa River and Napa Creek. The current recommended plan for the City of Napa, CA provides for protection against the one-percent chance event from the Napa River and Napa Creek. The proposed plan consists of channel excavation, sheetpile walls, concrete flood walls, set-back earthen levees, a bypass channel, and related environmental mitigation measures.

b. Interior Area Measures. The interior flood damage reduction measures will consist of replacing approximately twenty-one existing storm sewers in six identified interior areas with minimum gravity outlets through the Napa River planned line-of-protection. Additional outlet capacity by gravity or pumps will be provided where economically justified. The proposed improvements for Napa Creek consist of channel excavation only and therefore, will not include interior measures. The case study presented herein will describe the analysis of interior measures for one of the areas.

4. Interior Area Data and Information Assembly

a. General. Hydrologic data and other information required for the analysis of the interior area were assembled. It includes data for both the interior and exterior (Napa River) areas. The information is applicable for any analytical method, but was specifically targeted for application of the HEC-IFH computer program. Appropriate information was assembled to permit analyses using continuous simulation analysis (CSA) with period-of-record historical data and hypothetical event analysis (HEA) with synthetic storm event data.

CSA is attractive because it preserves the relationship between Napa River stages at interior outlet locations and interior area runoff. A drawback of CSA is the difficulty of defining rare flood events when only a relatively short historical period-of-record is available, as is the case for the Napa area. Therefore, HEA was adopted for this study in order to define the full range of flood events. The stage-frequency relationships from HEA and CSA were compared to help substantiate the reasonableness of the HEA results. Hydrologic data and other required information is described in the same manner as an analyst would assemble and enter the data into the HEC-IFH program. Data sets and module information are shown by including representative program screens as figures, where appropriate.

b. Rainfall Data. Historical rainfall records were assembled for continuous simulation analysis (CSA) and hypothetical depth-duration-frequency relationships were developed for hypothetical event analysis (HEA).

(1) Historical rainfall records of nearby recording raingages were used to develop a continuous period-of-record rainfall record for Napa River interior areas. Recorded hourly incremental rainfall data were adjusted by the ratio of mean annual precipitation at the gages to that for Napa River interior areas. A composite precipitation record for Water Year (WY) 1949 through WY 1989 was determined in this manner for use in CSA. The computed composite record was written to HEC-DSS and then imported into the HEC-IFH program. After importing the composite record, incremental rainfall can be plotted on a yearly, monthly, or daily basis. Figure 2 shows daily total daily precipitation for WY 1986.

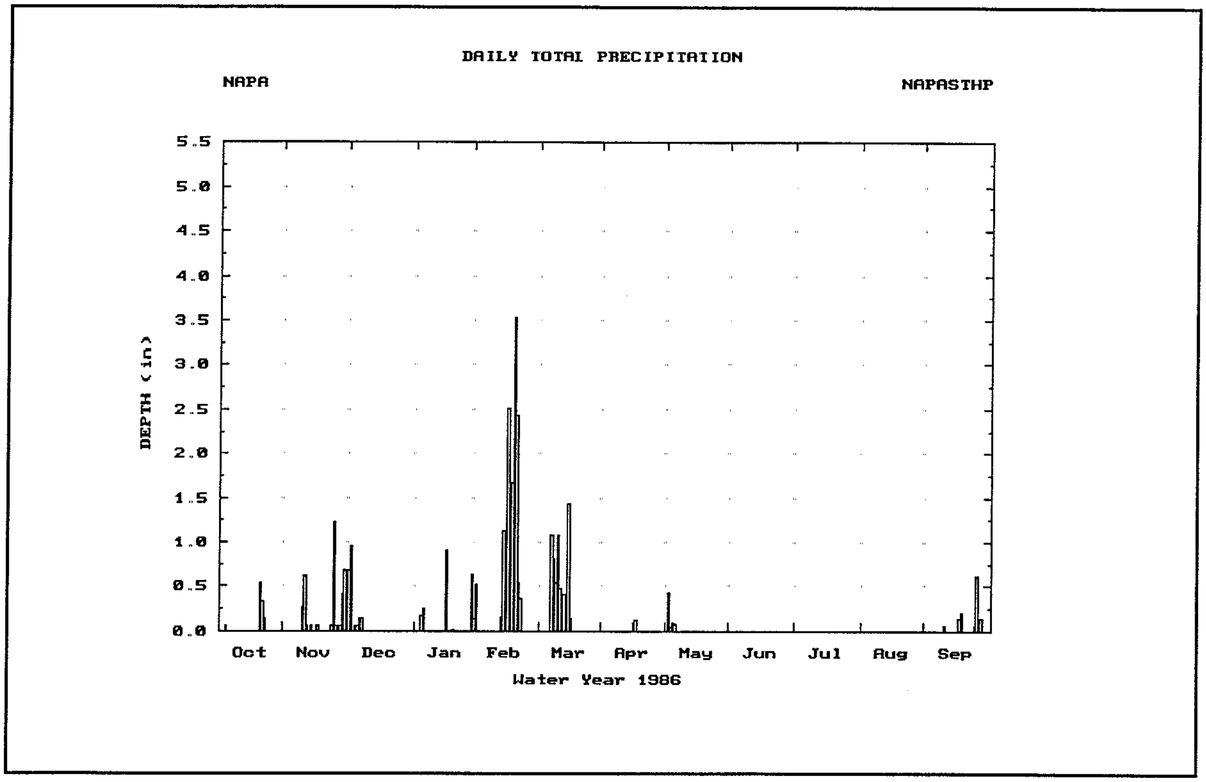


Figure 2. Interior Area Composite Historical Precipitation Data

(2) Hypothetical frequency storm depth-duration-frequency relationships for general rain and local storms were developed from rainfall frequency data that was available for the Martinez 3S and Napa State Hospital gages. Depths were adjusted by ratios of the mean annual precipitation (MAP) for the gages and the MAP for the Napa River interior area estimated from a MAP isohyetal map. The adopted depth-duration-frequency rainfall relationships for a general rain storm are shown in Figure 3. The development of precipitation data for computing exterior period-of-record discharge hydrographs is described in Section 4,f., Exterior Stage Data.

c. Delineation of Interior Areas. Interior areas were delineated based on alignment of the line-of-protection, minimum facility requirements, runoff topology, topography of local ponding areas, present and potential future storm sewer and water collector/conveyance systems.

Interior Area 5 is located on the right bank of Napa River just upstream of the mouth of Napa Creek. (See Figure 4.) This 1.5 square mile area is bounded by Napa River on the east, Highway 29 on the west, approximately Trancas St. on the north, and Napa Creek on the south. The area was divided into an upper and lower portion to accommodate the previously developed HEC-1 basin model. Runoff parameters and the existing storm sewer layout are described in subsequent sections.

HEC-IFH

HEA 01.04.00
Study ID NAPA
Module ID PRECHLOC

Basin Average Precipitation (PRECIP)

Enter Partial-Duration Rainfall Depth-Duration-Frequency Data

Duration	Rainfall Depth (in) for each Hypothetical Event						
	50%	20%	10%	4%	2%	1%	0.2%
5 minutes	0.15	0.20	0.25	0.30	0.32	0.36	0.45
15 minutes	0.27	0.38	0.46	0.56	0.62	0.69	0.85
1 hour	0.57	0.80	0.97	1.16	1.30	1.44	1.78
2 hours	0.82	1.16	1.39	1.66	1.87	2.07	2.55
3 hours	1.02	1.42	1.70	2.04	2.29	2.54	3.14
6 hours	1.50	2.12	2.53	3.03	3.40	3.76	4.67
12 hours	1.98	2.79	3.33	4.00	4.48	4.96	6.15
24 hours	2.45	3.44	4.12	4.95	5.56	6.14	7.60
2 days	3.12	4.51	5.42	6.63	7.49	8.33	10.48
4 days	4.03	5.77	6.94	8.38	9.44	10.45	13.01
7 days	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10 days	0.00	0.00	0.00	0.00	0.00	0.00	0.00

1Help 2PrtScr 3 4 5 6DOS 7 8 9 10Exit

Press <F10> to Save Data and Continue

Figure 3. Interior Area Hypothetical Precipitation Data

d. Runoff Characteristics.

(1) Unit Hydrographs. The Sacramento District developed an HEC-1 rainfall-runoff model for simulating historical flood events for Napa River interior areas during previous studies. The HEC-1 model used the kinematic wave technique of transforming rainfall to runoff. The HEC-IFH program does not use kinematic wave and therefore it was not possible to reproduce the modeling effort in HEC-IFH. It was important to preserve the timing of the interior runoff and the detail of the HEC-1 model where interior areas were divided into many subareas and reaches to represent urban runoff. Therefore, the kinematic wave HEC-1 model was used with one-inch of runoff to generate composite unit hydrographs for each interior area. Clark unit hydrograph parameters TC and R were estimated from the kinematic wave unit hydrographs using the parameter estimation capability in the HEC-1 program. These unit hydrograph parameters were used in the HEC-IFH program for computing runoff from the interior area during hypothetical event and continuous simulation analysis.

Figure 4. Napa Interior Area 5

(2) Loss Rates. The initial and uniform loss rate model was used for both CSA and HEA. There are no stream gages in the interior area so calibration of runoff parameters was not possible. Other methods were used to insure the reasonableness of the parameters as described below.

(a) CSA. For CSA, the initial loss was 0.4 inches and the uniform loss was 0.02 inches per hour. The monthly initial loss recovery rate for CSA was 0.04 inches per day. Test simulations with different initial loss recovery rates for CSA showed that peak interior runoff was not sensitive to this parameter. Examination of monthly precipitation, loss, and percent loss is possible in HEC-IFH and helps verify the reasonableness of selected loss rates. (See Figure 5.)

(b) HEA. For HEA the adopted initial loss was 0.2 inches and the uniform loss was 0.02 inches per hour. These loss rates were held constant for all hypothetical events. The loss rates were consistent with those used by the district had in previous studies and were considered reasonable for the highly urbanized areas. As expected, the HEA loss rates, which are representative of rare single events, are lower than the CSA rates.

Peak interior runoff using the described adopted loss rate parameters were compared for CSA and HEA. Peak interior flow-frequency relationships for CSA and HEA are shown in Figure 6 and compared closely for moderately rare events. This further substantiates the reasonableness of adopted parameters.

(3) Base flow. No base flow was specified for either CSA or HEA. Base flow was considered to have little or no impact on peak runoff or volume for these small interior areas.

HEC-IFH

CSA 01.04.00
Study ID NAPA
Plan ID PLAN5-3A

Hydrologic Analysis Summaries

Begin 01OCT1948/0015
End 30SEP1989/2400

K. Monthly Summaries - Average Monthly Rainfall

Month	Lower Sub-Basin			Upper Sub-Basin			Exterior Basin		
	Precip (in)	Losses (in)	Percent Loss	Precip (in)	Losses (in)	Percent Loss	Precip (in)	Losses (in)	Percent Loss
Oct	1.28	0.49	38.25	1.28	0.49	38.25			
Nov	3.37	1.09	32.30	3.37	1.09	32.30			
Dec	4.38	1.34	30.60	4.38	1.34	30.60			
Jan	5.02	1.47	29.21	5.02	1.47	29.21			
Feb	3.96	1.17	29.53	3.96	1.17	29.53			
Mar	3.43	1.20	34.90	3.43	1.20	34.90			
Apr	1.72	0.70	40.89	1.72	0.70	40.89			
May	0.41	0.25	61.46	0.41	0.25	61.46			
Jun	0.16	0.10	59.73	0.16	0.10	59.73			
Jul	0.06	0.03	43.31	0.06	0.03	43.31			
Aug	0.06	0.04	67.54	0.06	0.04	67.54			
Sep	0.38	0.19	49.00	0.38	0.19	49.00			

1Help 2PrtScr 3 4 5 6DOS 7 8 9Plot 10Exit

Press <F10> to Return

Figure 5. Precipitation, Loss and Loss Percent for Interior Area 5 - CSA

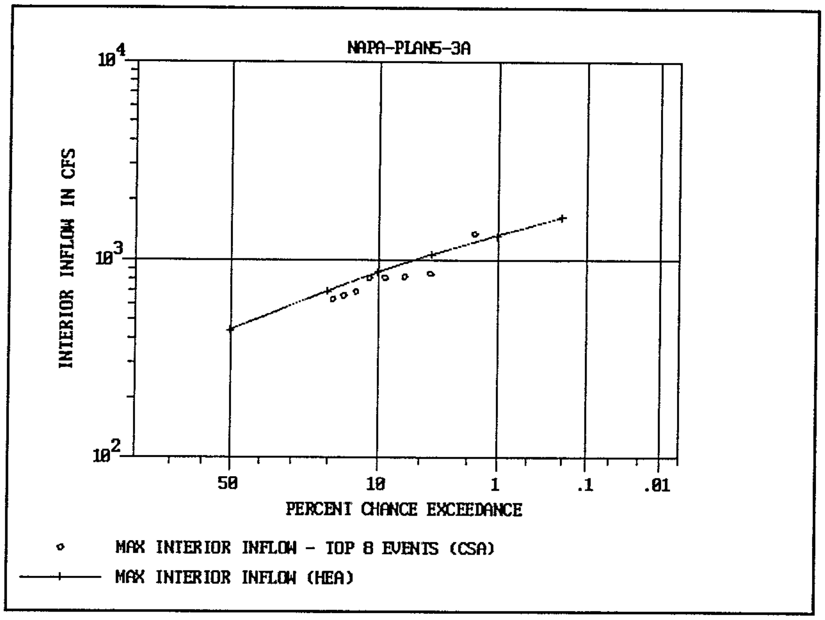


Figure 6. Interior Runoff Discharge-Frequency Relationships - CSA and HEA

Some of the runoff parameters for interior Area 5 are shown in Figure 7.

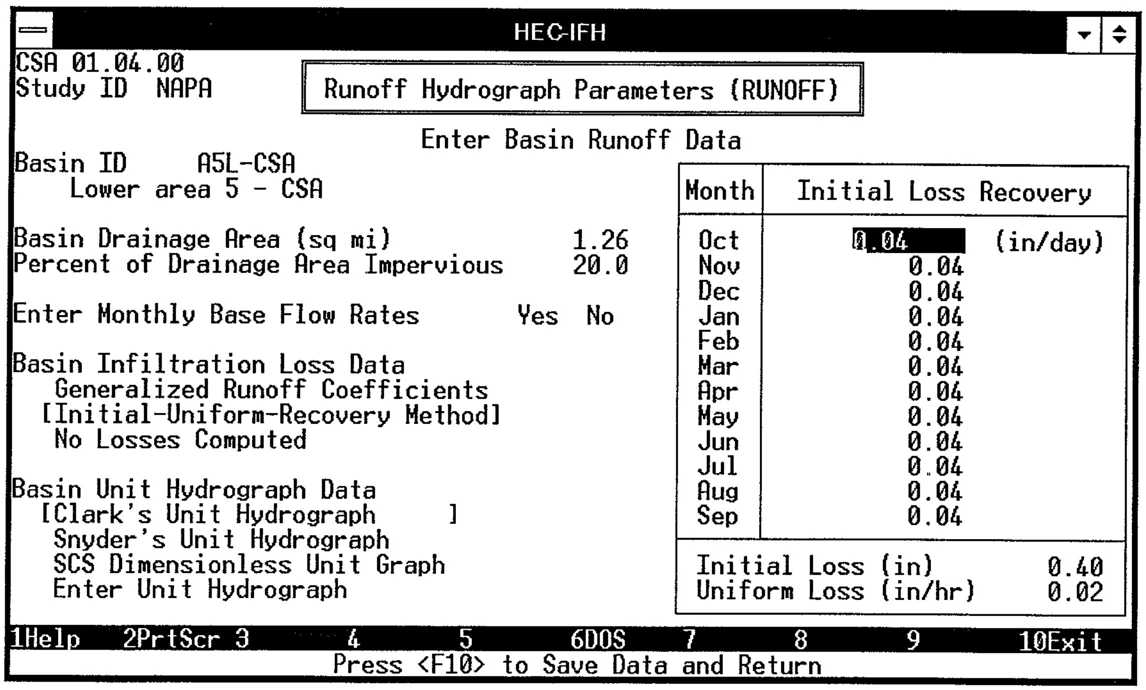


Figure 7. Runoff Parameters - Area 5 Lower Subbasin, CSA

(4) Streamflow Routing. No routing was used between the upper and lower subareas for interior Area 5 due to the short travel time and the fact that the area is heavily sewered.

(5) Interior Runoff Computation Time Interval (Δt). The interior runoff computation time was 15 min. for CSA and 5 min. for HEA. The time of concentration for the upper and lower subbasins for Area 5 were 0.79 and 1.1 hours, respectively. Accordingly, these time intervals were considered adequate to define the runoff hydrographs at the outlets.

e. Interior Ponding Area. Elevation-area relationships were delineated for each ponding area adjacent to the line-of-protection at the flow concentration points. The relationships were taken from elevation-area tables generated from computerized topographic data of the project area. The elevation-area data were entered into the HEC-IFH program which automatically generates the storage values from end-area approximations. The minimum value was established from the lowest invert elevation to be analyzed for interior Area 5. The maximum value was established from the highest stage anticipated in the analysis, which in this case is the top of the levee embankment at the line-of-protection. A portion of the pond elevation-area-storage relationship for interior Area 5, as implemented in the HEC-IFH program, is shown in Figure 8.

HEC-IFH

CSA 01.04.00
Study ID NAPA

Interior Pond (POND)

Enter Surface Areas for Computing Volumes

Storage Table ID AREA5

Description
Area 5 natural pond at Soscal Ave.

Pond Elevation (ft)	Surface Area (ac)	Storage Volume (ac-ft)
-5.30	0.0	0.0
0.00	0.1	0.4
2.00	0.2	0.7
5.00	0.2	1.4
7.00	0.3	2.0
8.00	0.4	2.3
9.00	0.5	2.8
10.00	0.6	3.4
12.00	0.8	4.8
14.00	1.9	7.4
15.00	4.9	10.8
16.00	7.2	16.9
17.00	42.5	41.7

1Help 2PrtScr 3 4 5 6DOS 7 8 9 10Exit
Press <F10> to Save Data and Return

Figure 8. Interior Pond Elevation-Area-Storage Relationship for Area 5

f. Exterior Stage Data. Exterior stage hydrographs were required to establish the exterior conditions for both CSA and HEA.

(1) Exterior Conditions - CSA. Exterior stage data for period-of-record CSA include continuous stage hydrographs that represent the historic patterns of Napa River discharge at the outlet locations of each interior area. A continuous discharge hydrograph was developed for the exterior from rainfall-runoff analysis. Historical rainfall records of nearby recording and nonrecording raingages were used with the PRECIP program to develop a continuous, period-of-record, composite rainfall record for the Napa River basin. Runoff parameters for the exterior basin were derived by calibration with the computed SPF hydrograph, the estimated peak discharge of the February 1986 flood event, and the project design discharge-frequency curve for Napa River below Tulucay Creek. The computed exterior runoff hydrographs were used with Napa River rating curves to determine continuous exterior stage hydrographs during CSA. The rating curves were defined at the outlet locations based on project channel water surface profiles provided by the district. Rating curves were adjusted slightly so that the peak flow of each hypothetical flood hydrograph matched the water surface elevation from the water surface profiles for the corresponding event. Figure 9 shows some of the runoff parameters for the exterior basin.

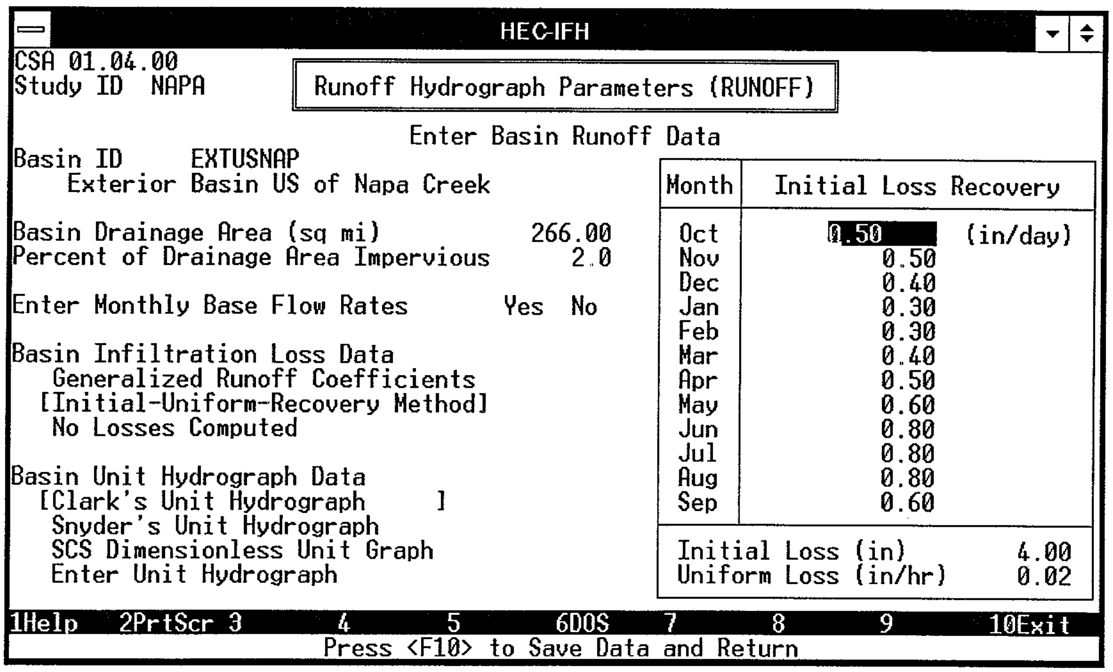


Figure 9. Runoff Parameters for the Exterior Basin - CSA

(2) Exterior Conditions - HEA. Hypothetical storm analysis was conducted using general rain 96-hr local storms centered over the interior for unblocked, low Napa River

HEC-IFH

HEA 01.04.00
Study ID NAPA
Module ID EXHYPUSN

Exterior Stage (EXSTAGE)

Index Location

Enter/Import Exterior Discharge Hydrographs (cfs)

Time Interval 15MIN

Number of Intervals 800

Da/HrMn	Hyp.Frq 50%	Hyp.Frq 20%	Hyp.Frq 10%	Hyp.Frq 4%	Hyp.Frq 2%	Hyp.Frq 1%	Hyp.Frq 0.2%	SPF
1/0015	81.	113.	139.	167.	191.	216.	296.	0.
1/0030	81.	112.	139.	167.	191.	216.	296.	0.
1/0045	81.	112.	139.	167.	191.	216.	296.	0.
1/0100	81.	112.	139.	167.	190.	216.	295.	0.
1/0115	81.	113.	139.	167.	191.	216.	296.	0.
1/0130	81.	113.	140.	168.	192.	217.	297.	0.
1/0145	82.	115.	141.	170.	194.	219.	301.	0.
1/0200	84.	117.	144.	173.	197.	223.	309.	0.
1/0215	86.	120.	147.	177.	202.	229.	322.	0.
1/0230	89.	124.	152.	183.	209.	237.	343.	0.
1/0245	92.	128.	158.	190.	217.	248.	370.	0.
1/0300	95.	134.	164.	199.	228.	261.	421.	0.
1/0315	100.	140.	172.	208.	240.	276.	471.	0.

1Help 2PrtScr 3 4 5 6DOS 7 8 9 10Exit

Position with <PgDn>, <PgUp>, <Dn>, <Up>, <Home>, <End>

Figure 10. Portion of Hypothetical Flood Hydrographs for Exterior Basin - HEA

conditions. For hypothetical interior and exterior analysis the general rain 96-hr hypothetical storms were centered over both the interior area and the Napa River basin.

Hypothetical storm flood hydrographs at the outlet locations of each interior area were developed from HEC-1 data sets provided by the district. The data consists of a S-curve unit hydrograph rainfall-runoff model upstream of the Oak Knoll streamgage and a kinematic wave model downstream to Imola Avenue in Napa. The hydrographs were determined by taking ratios of the SPF. These HEC-1 rainfall-runoff models were used by the district to develop project discharge-frequency relationships for the Napa River. Therefore, the HEC-1 model developed hypothetical flood hydrographs were used for exterior conditions during HEA. The flood hydrographs were imported into the HEC-IFH program and used with rating curves to compute exterior stage hydrographs at interior outlet locations during HEA. Figure 10 shows a portion of the imported hypothetical flood hydrographs for the exterior basin.

g. Existing and Proposed Storm Sewer Design and Configuration. The details of existing and any proposed storm sewer layout, discharge design capacities, including elevation of the inverts was required to define drainage areas, minimum facilities, gravity outlet inverts, pumping station on-off elevations, and design criteria for inlet and outlet works. The layout and design of existing and proposed future storm runoff conveyance systems was obtained from the Napa Public Works Department. The information provided included storm sewer location, length, size, and invert elevation. These data were provided on a 1 inch equal 100 feet scale areal photo, with 2-foot contour intervals.

Area 5 is well sewered and has several existing gravity outlets that cross the line-of-protection and/or convey portions of the runoff to the Napa River. The outlets are shown in Figure 4 and are described in the following subparagraphs. Numbered outlets refer to the primary and secondary outlet locations as shown in the figure.

(1) 54 inch pipe at Truncus St. A major storm sewer system runs easterly along Trancas Street and discharges into the Napa River via a 54 inch circular pipe just downstream of the Truncus Street Bridge. This outfall is above the upstream limit of the project and therefore will not be disturbed. The outlet invert is not subject to blockage from high river stages due to the relatively high outlet invert elevation. It was estimated by the City of Napa that this outfall would pass a maximum of 50 CFS into the Napa River during flooding. This was simulated in the HEC-IFH program by diverting this flow from the upper subbasin to the river. (See the subsequent section on Auxiliary Flow.)

(2) 72 inch pipe near Soscol and Pueblo Sts. intersection. The next downstream major storm sewer is a 72 inch circular pipe which enters the river at the north end of the Lake Park leveed area just east of the intersection Soscol and Pueblo Sts. It serves a major portion of the upper subbasin under pressure flow. This outlet is just upstream of the upper limits of the flood control project and therefore, will be left undisturbed. The capacity of this pipe was estimated to be 300 CFS and this flow was diverted from the upper subbasin to the river for HEA and CSA. (See the subsequent section on Auxiliary Flow.)

(3) Lake Park/Edgewater area. This leveed area and its associated existing gravity outlets and pump station are considered independently and are not part of the interior Area 5 analysis.

(4) Location 5.0. Overflow ditch and 42 inch pipe north of the confluence of Napa Creek and Napa River. This system includes a 72 pipe that empties into an overflow ditch that enters the Napa River just upstream of the confluence of Napa Creek and Napa River. At the outfall there is a 42 inch circular pipe that runs beneath the overflow ditch. This outfall location is the flow concentration point for Area 5 and was designated as the ponding area (see Section 4.D., above) and the primary gravity outlet location for this interior area.

(5) Additional existing outlets. There are 3 additional existing outlets that cross the line-of-protection and are to be replaced with new gravity outlets with drop inlets. They are all upstream of the primary gravity outlet and are designated and analyzed as secondary outlets for HEA and CSA. These outlets are described below:

(a) Location 5.1. 1-24 inch pipe located at Imperial way

(b) Location 5.2. 1-18 inch pipe located at North Bay Drive (to be replaced by a 24 inch drop inlet)

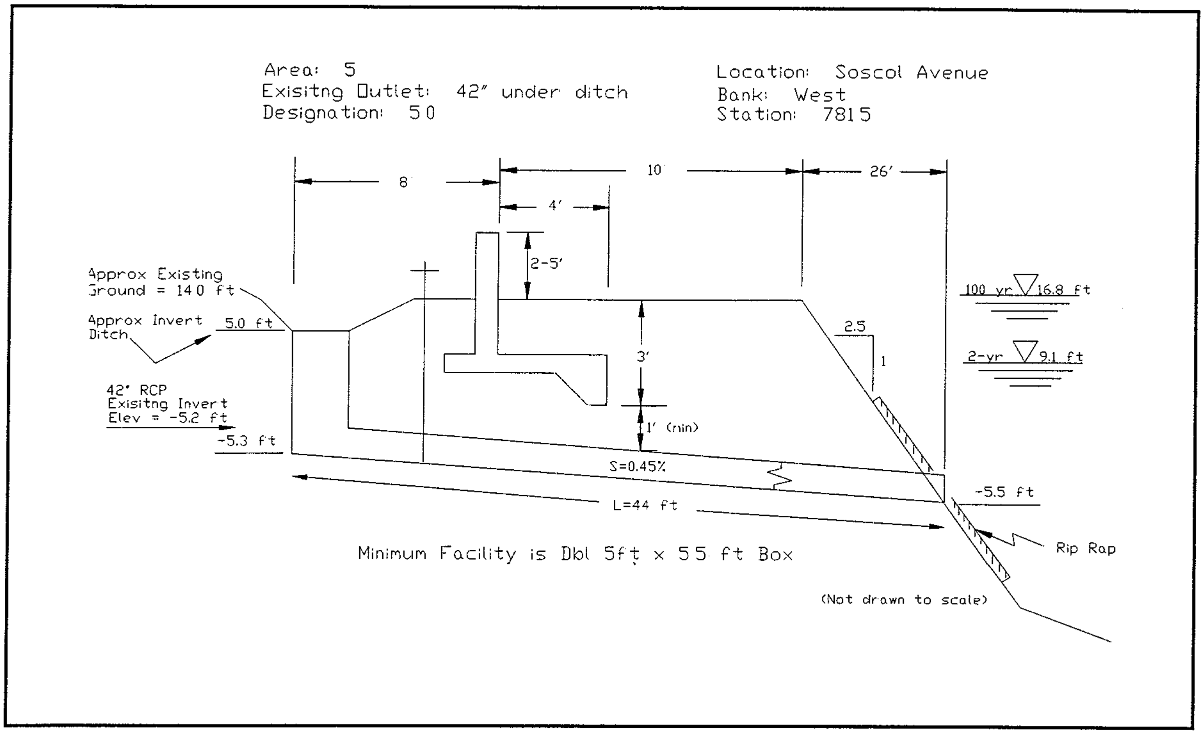
(c) Location 5.3. 1-30 inch pipe located at Lincoln Avenue.

There are a few small outlets that convey a minor portion of interior runoff from Area 5 into Napa Creek from the left bank (north side). These outlets will not be cut off by the project since they are upstream of the Napa River tie back levee were channel excavation is the only project feature. The effect of these outlets were considered to be negligible in the analysis of Area 5.

h. Field Reconnaissance. Two field trips were made to locate outlet inverts and ditches that will be cut off by the line-of-protection, bridges, hydraulic structures, and flood plain channels and overbank areas. Several meetings were held with the Napa Department of Public Works and Sacramento District to discuss existing and proposed storm conveyance systems and proposed interior features that would convey storm runoff through the line-of-protection.

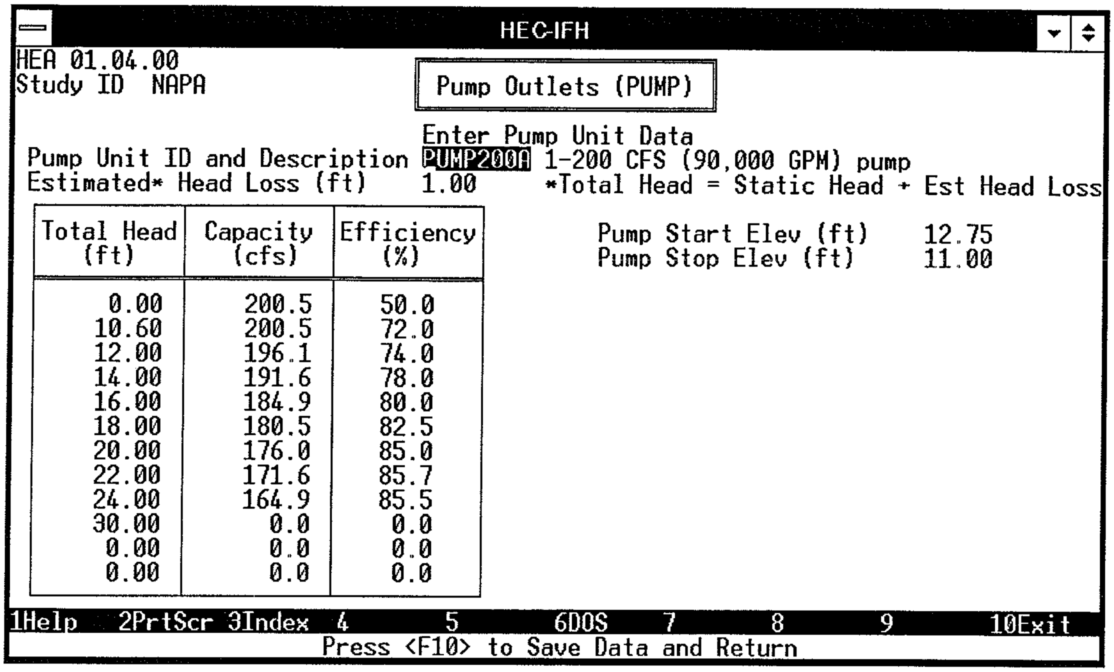
i. Gravity Outlets. The characteristics and configuration of typical new gravity outlets were defined to establish gravity outlet parameters and for developing rating curves for the outlets. This information included 1) culvert length, size, etc., 2) invert elevations and slopes consistent with existing storm sewers, 3) culvert type (box or circular, concrete or CMP, etc.), and 4) entrance and exit configurations.

The typical outlet through the line-of-protection was defined as a concrete box culvert with a grated drop inlet after coordination and agreement with the study manager.. The outlet inverts of the drop inlets are established by the existing storm sewer inverts entering the drop inlets. Lengths of the box culverts were dependant on whether the line-of-protection consisted of a set back levee, sheetpile wall, or concrete flood wall at the outfall. Slopes of the box culverts were set to maintain the slopes and outlet invert elevations of the existing outlets as close as possible. Required information was taken from project drawings provided by the district and existing storm sewer layouts provided by the City of Napa. Manual gate closure valves as well as flap gates will be included as part of each new outlet. The minimum head differential required for gravity flow was specified as 0.5 feet. No special gate closure requirements were established. A typical layout of a drop inlet box culvert at the primary location for interior Area 5 is shown in Figure 11.



11. Typical Layout - New Box Culvert with Drop Inlet Area 5

j. Pumping Stations. Typical pumping station configuration and operation was determined through coordination with the district. The criterion for number of pumps and pumping station capacity was that each pumping station would have a total of 3 pumps, each having two-thirds of the total designated station capacity. Two of these pumps would be operating as needed and one would be for back up incase one of the other pumps went out of service. For example, a 300 CFS pumping station would include 3-200 CFS (90,000 GPM) pumps, two of which would be operating. Pump head-capacity-efficiency relationships were determined from pump performance curves provided by the district. Figure 12 shows the relationships for a 200 CFS (90,000 GPM) pump unit.



HEC-IFH

HEA 01.04.00
Study ID NAPA

Pump Outlets (PUMP)

Enter Pump Unit Data
Pump Unit ID and Description PUMP200A 1-200 CFS (90,000 GPM) pump
Estimated* Head Loss (ft) 1.00 *Total Head = Static Head + Est Head Loss

Total Head (ft)	Capacity (cfs)	Efficiency (%)
0.00	200.5	50.0
10.60	200.5	72.0
12.00	196.1	74.0
14.00	191.6	78.0
16.00	184.9	80.0
18.00	180.5	82.5
20.00	176.0	85.0
22.00	171.6	85.7
24.00	164.9	85.5
30.00	0.0	0.0
0.00	0.0	0.0
0.00	0.0	0.0

Pump Start Elev (ft) 12.75
Pump Stop Elev (ft) 11.00

1Help 2PrtScr 3Index 4 5 6DOS 7 8 9 10Exit
Press <F10> to Save Data and Return

Figure 12. Pump Unit Head-Capacity-Efficiency Data

Pump on and off elevations were determined so that the pumps come on to effectively reduce damaging stages and turn off when stages drop below damaging levels. However, pumps should not cycle on and off over very short periods of time. Pump on/off elevations were determined based on the "zero damage" elevation and rate of rise for the ponding areas in each interior area. Pump on/off elevations may need adjusting depending on the final design configuration of the pumping station. Preliminary on/off elevations for the two operating pump units for a 300 CFS station are shown in Figure 13 and are based on a "zero damage" elevation of 14.0 feet for interior Area 5.

k. Auxiliary Flow. Auxiliary flow includes auxiliary inflow to the interior subbasin, diversions out of the system, seepage inflow from the exterior (Napa River) to the interior area, and overflow out of the interior area. As indicated in Section 4.f., the effect of the existing 54 inch and 72 inch pipes located upstream of the upper limits of the flood protection project was represented by a diversion from the upper subbasin in interior Area 5. Specified diversions for Area 5 are shown in Figure 14. Seepage was not considered a factor for the Napa River interior study because the minimum time earthen embankments would be inundated and the extensive use of sheetpile and concrete flood walls along the line-of-protection.

HEC-IFH

HEA 01.04.00
Study ID NAPA
Plan ID PLAN5-4C

Hydrologic Analysis Summaries

Begin 01/0005
End 07/0600

D. Analysis Input Summaries - Pump Station Data

PUMP Module ID PMOD300 300 cfs station (2-200 cfs pumps oper.)

Pump Number	Pump Unit ID	Maximum Capacity (cfs)	Pump Start Elevation (ft)	Pump Stop Elevation (ft)	Maximum Total Head (ft)
1	PUMP200A	200.5	12.75	11.00	30.00
2	PUMP200B	200.5	13.25	11.75	30.00
3					
4					
5					
6					
7					
8					
9					
10					

1Help 2PrtScr 3 4 5 6DOS 7 8 9 10Exit
Press <F10> to Return

Figure 13. Pumping Station Data for Interior Area 5

HEC-IFH

HEA 01.04.00
Study ID NAPA

Auxiliary Inflow/Outflow (AUXFLOW)

Enter Diversion Rate for Upper Sub-Basin

Diversion Table ID AREA5U

Description
Outflow to Napa R via Pueblo & Trancas.

Runoff + Aux. Inflow (cfs)	Diverted Flow (cfs)
0.0	0.0
10.0	10.0
50.0	50.0
100.0	100.0
200.0	200.0
350.0	350.0
10000.0	350.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0
0.0	0.0

1Help 2PrtScr 3 4 5 6DOS 7 8 9 10Exit
Press <F10> to Save Data and Return

Figure 14. Diversion Rate for the Upper Subbasin - Interior Area 5

l. Water Surface Profile Data. Water surface profiles for with-project conditions were developed by the district using the HEC-2 program. These profiles were used to determine rating curves for the Napa River at interior area outlet locations. The water surface profiles were also used to determine exterior stage transfer relationships for transferring the computed exterior stage at the primary outlet location to the secondary outlet locations. The rating curve for Napa River at the Area 5 primary location is shown in Figure 15.

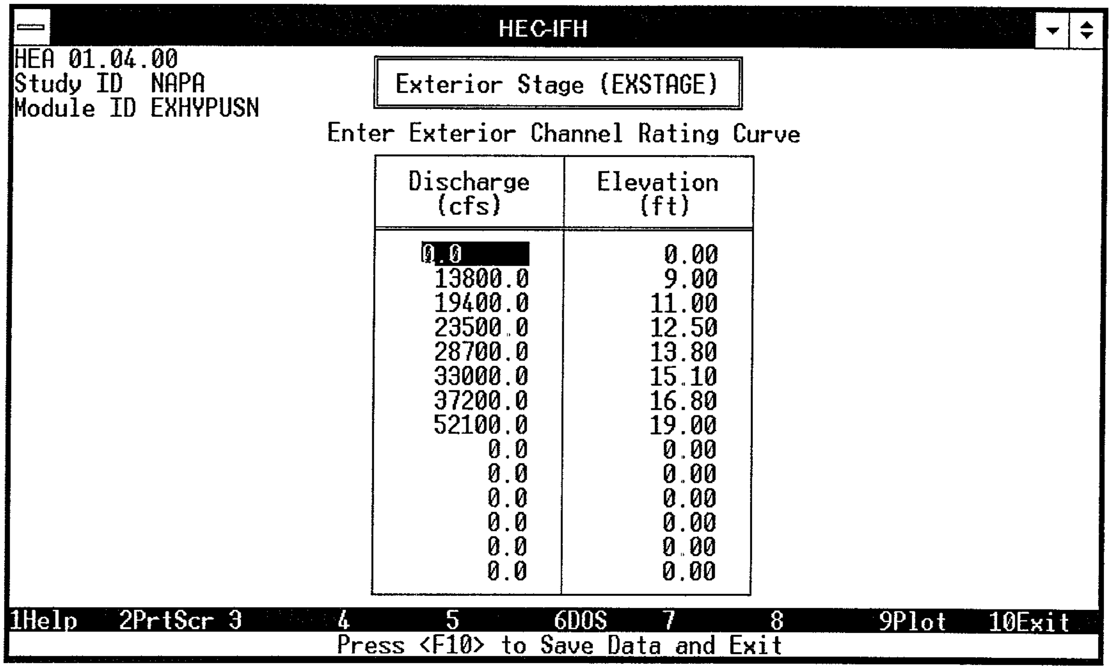


Figure 15. Exterior Rating Curve for Interior Area 5

m. Stage-Damage Relationships. Representative stage-damage relationships for the interior areas at runoff concentration points are required for economic analysis and identification of interior plans which maximize net flood damage reduction benefits. Economic analysis is not part of this investigation, and therefore, complete stage damage relationships were not required. The elevation where significant damage begins or "zero damage" was required to establish the size of the minimum facility and to set pump on/off elevations. These elevations were provided by the district.

5. Without-project Conditions Analysis for Minimum Facility Evaluation

a. General. The without-project analysis involves evaluation of conditions with and without the line-of-protection in place. Degree of flooding for these conditions are needed to select a minimum facility. The without-project condition used to formulate and evaluate the interior flood damage reduction measures will assume that the adopted minimum facility is in place and is described in Section 6, Minimum Facility Analysis.

b. Napa River Flooding Without Line-of-Protection. The source of serious flooding in the City of Napa is the Napa River and to a lessor extent Napa Creek. The recommended flood damage reduction project protects the city from flooding up to the one-percent chance flood for both the Napa River and Napa Creek. The basis for sizing the minimum facility is to assure that flooding from local storm runoff, when the Napa River and Napa Creek are below bank full, is not more frequent with the line-of-protection in place than without the line-of-protection in place.

c. Local Runoff Flooding Without Line-of-Protection. Local flooding was evaluated without the line-of-protection in place, assuming the present storm sewer system in place, and Napa River and Creek below flood stage. Stage-frequency relationships for this condition were not developed due to lack of data. Storm sewer system design criteria for the City of Napa, used for existing and new systems, were well documented and were used to establish the target condition for the minimum outlet facility analysis. The first criterion used was that only minor street and gutter flooding should occur up to the 10-percent chance (10-year) flood event. Minor street and gutter flooding in this case is defined as not exceeding a depth that would result in flooding more than 10 feet from the street gutter. The second criterion was that no significant damage from flooding will occur in residential and commercial areas from floods up to the 4-percent chance (25-year) flood event. This second criteria was interpreted as meaning that the interior stage resulting from the 4-percent chance event should not exceed the start of significant damage elevations determined by the district office. Based on the past performance of the existing sewer system and the overall reasonableness of the criteria, the storm sewer system design criteria were adopted for sizing the minimum facilities.

d. Assess Future Without Project Conditions Impacts. Future conditions that could affect Napa River interior area local runoff flooding were considered. Hydrologic and/or hydraulic conditions are not expected to significantly change over the project life, and therefore, no changes needed to be incorporated into the analysis. The interior areas are fully urbanized and limited future urbanization would have minimal effect on watershed runoff. Proposed and planned improvements in the existing storm sewer system, as described by the City of Napa, were evaluated and incorporate in the interior areas where appropriate. There were no planned changes to the existing storm sewer system in interior Area 5.

6. Minimum Facility Analysis

a. General. The adopted minimum facility, sized according to the criteria described in Section 5.c., is a justified part of the line-of-protection. The stage-frequency relationships for the with minimum facility in place condition becomes the without condition for evaluating potential interior flood damage reduction measures beyond the minimum facility. The residual damage with the minimum facility in place becomes the target for damage reduction of proposed additional interior flood damage reduction measures. As described previously, the minimum facility was sized to provide interior flooding relief so that during low exterior stages (unblocked gravity outlet conditions) the local interior area runoff will pass the design storm sewer outflow without an increase in interior stages over natural or without line-of-protection conditions.

b. Selecting the Minimum Facility for Interior Area 5. A series of gravity outlet capacities and configurations using local storm hypothetical events analysis (HEA) and assuming unblocked conditions were evaluated using the HEC-IFH program. The physical characteristics of the gravity outlets were described in Section 3.i. A new plan was defined for each gravity outlet capacity to be evaluated and the interior stage-frequency relationship was developed for each outlet. The plan components as defined in the HEC-IFH program for one of the plans evaluated for interior Area 5 is shown in Figure 16.

The stage-frequency relationships of gravity outlets was compared to the storm sewer design criteria described previously and the outlet size which came closest to meeting the criteria was selected. For interior Area 5 the selected minimum facility was a double 5 X 5.5 ft box culvert. The "zero damage' elevation is 14.0 ft. and the 4-percent chance elevation is 13.55 ft. based on the results of the HEA unblocked condition simulation. The stage-frequency relationship with the minimum facility in place is shown in Figure 17. The 10-percent chance stage is below the criterion elevation for street flooding and therefore this minimum facility is adequate.

HEC-IFH

HEA 01.04.00
Study ID NAPA

Perform Interior Analysis

Plan ID PLAN5-1D Description Area 5 Min. Fac. 2-5X5.5 boxes-HEA unblk

Module	Module ID	Description
Basin Average Precipitation	PRECHLOC	General rain interior area hyp. precip.
Runoff Hydrograph Parameters	A5-HEA	Area 5 (U & L) runoff for HEA
Interior Pond	POND5	Pond surface area-elevation for Area 5
Gravity Outlets	OUTMOD5B	New Dbl 5X5.5 box at primary- Soscal Ave
Pump Data		
Exterior Stage	UNBLOCK	Low Napa River exstage (unblocked cond.)
Auxiliary Flow	AREA5	Outflow to Napa R via Pueblo & Trancas

ANNUAL series
Computation Time Interval (e.g. 1HOUR, 1DAY, ...) 5MIN
Number of Time Intervals 1800

1Help 2PrtScr 3 4 5 6DOS 7 8 9 10Exit
Press <F10> to Proceed to the Menu

Figure 16. Plan Components, Minimum Facility - HEA, Unblocked

c. Without Project Condition Stage-Frequency Relationship with the Minimum Facility in Place. After the minimum facility was selected, it was evaluated using general rain hypothetical event analysis (HEA). A new general rain HEA plan (Plan 5-2A) was defined using precipitation depth-duration-frequency data for general rain events occurring over the Napa River watershed as well as the interior area. Exterior stages were computed from imported hypothetical flood discharge hydrographs and an appropriate stage-discharge rating for the Napa River at the interior area outlet, as previously described. The results of the analysis were used to test the effectiveness of the minimum facility gravity outlet by assessing local runoff flooding that occurs during blocked conditions (e.g., with general rain storms centered over the interior and exterior basin causing flooding on both the interior and exterior). The resulting stage-frequency relationship is shown in Figure 18. Plan 5-1D is HEA with unblocked exterior conditions and Plan 5-2A is HEA with interior and exterior flooding conditions.

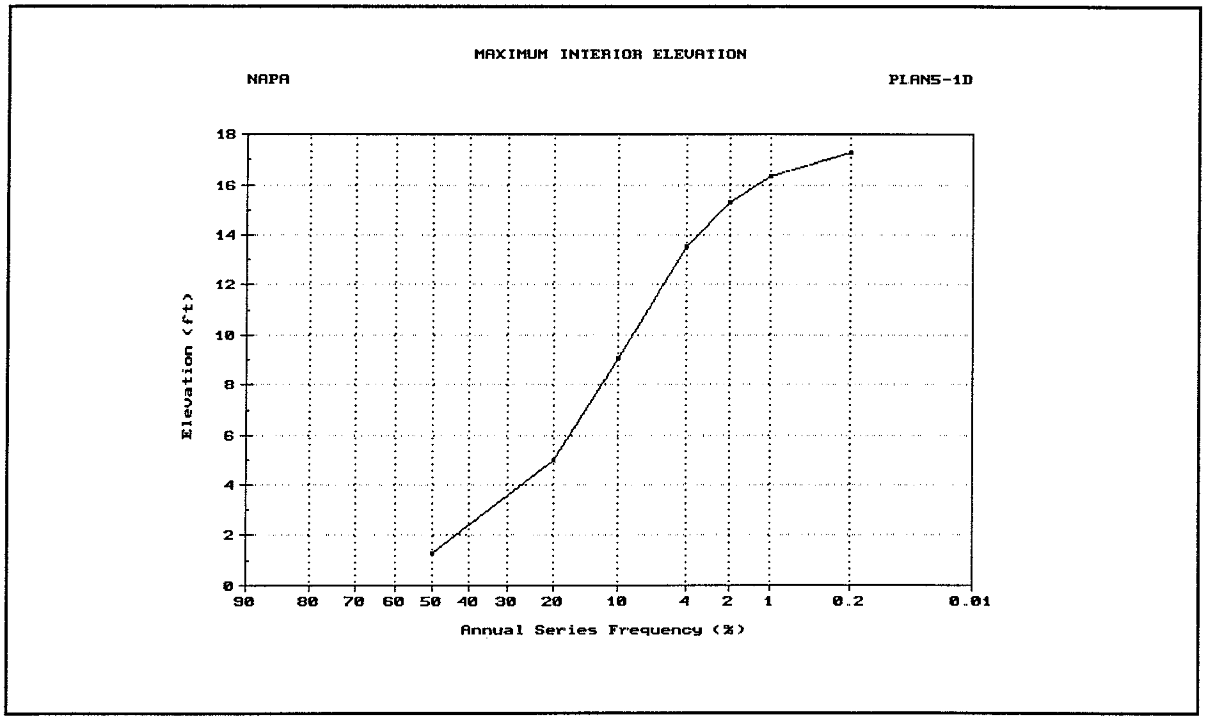


Figure 17. Stage-Frequency for Minimum Facility - Area 5, HEA, Unblocked

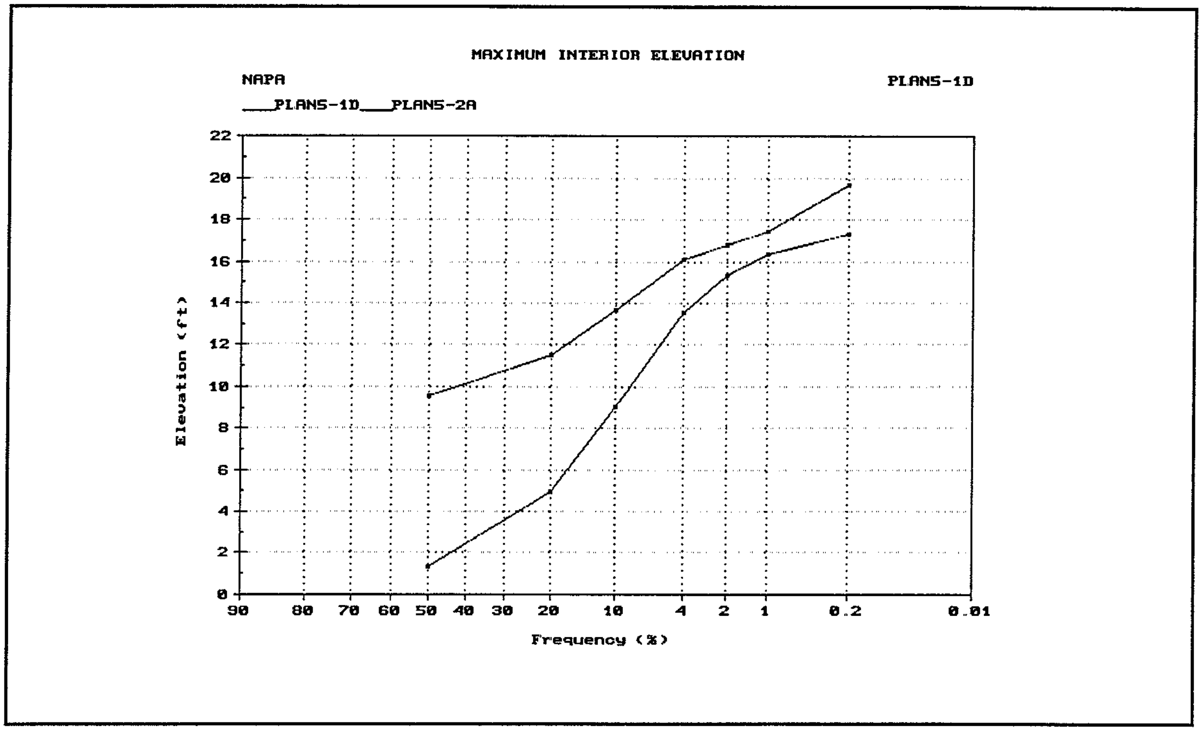


Figure 18. Stage-Frequency Relationships - HEA Unblocked and Interior/Exterior

Continuous simulation analysis (CSA) was performed using previously described period-of-record composite rainfall. The purpose of evaluating CSA in addition to HEA is to compare the resultant stage-frequency relationships. CSA captures the relationship between interior runoff and exterior stage whereas HEA assumes interior and exterior flooding is coincident. Examination of CSA results for several historical events shows that interior and exterior flooding is typically coincident, as illustrated in Figure 19 for the February 1986 event. An exception to this was the January 1973 event where the 41-year record interior rainfall and resultant runoff occurred while Napa River stages were very low. (See Figure 20.) Timing of the peak interior runoff and the maximum exterior stage is critical in the Napa study due to the small ponding area storage available. Due to this fact and the fact that the historical CSA shows that the peak interior runoff can occur before, after, or simultaneous to the exterior peak stage, HEA stage-frequency relationships were adopted for the evaluation of interior features. HEA captures the critical combinations of interior runoff and exterior stage that can occur but are not always well represented in the historical record. Figure 21 shows a comparison of the stage-frequency relationships for CSA and HEA. The differences in stage are relatively minor considering a 2-foot difference in stage (17.0 minus 15.0) is equivalent to less than 0.25 inch of runoff from the interior area. The relatively good comparison between the relationships helps substantiate the reasonableness of the HEA developed stage-frequency relationship. The HEA stage- frequency relationships were adopted for the without condition and used for evaluating additional interior flood damage reduction measures, as described in Section 7.

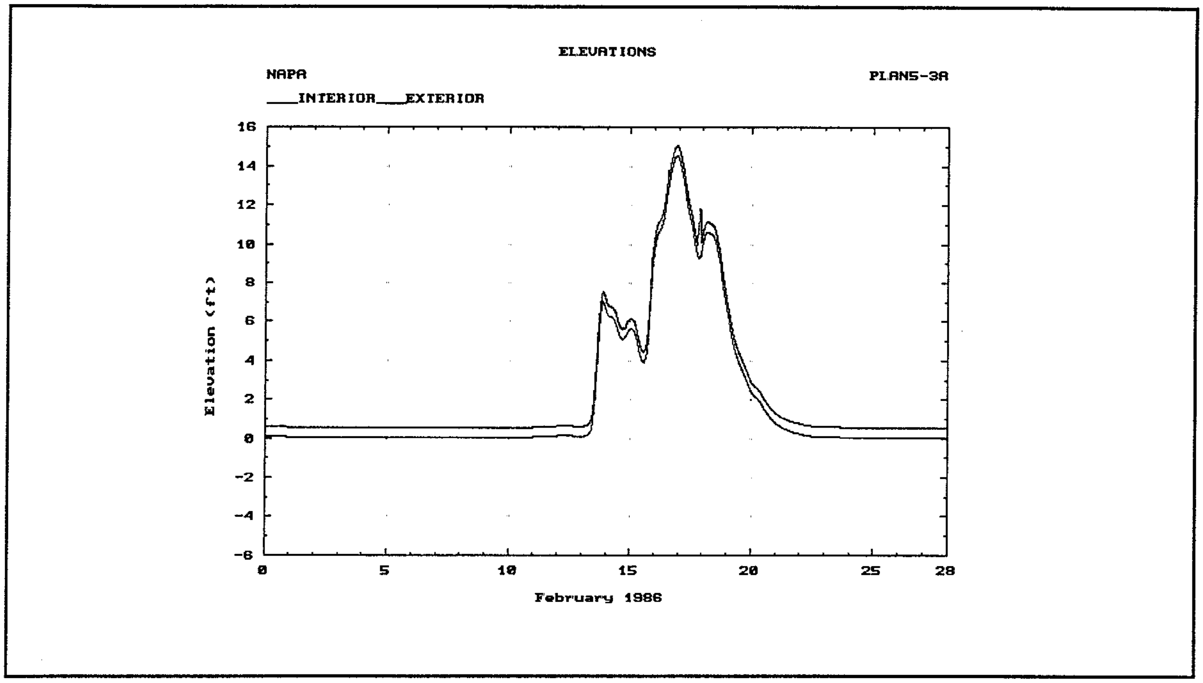


Figure 19. Interior and Exterior Elevation - February 1986, CSA

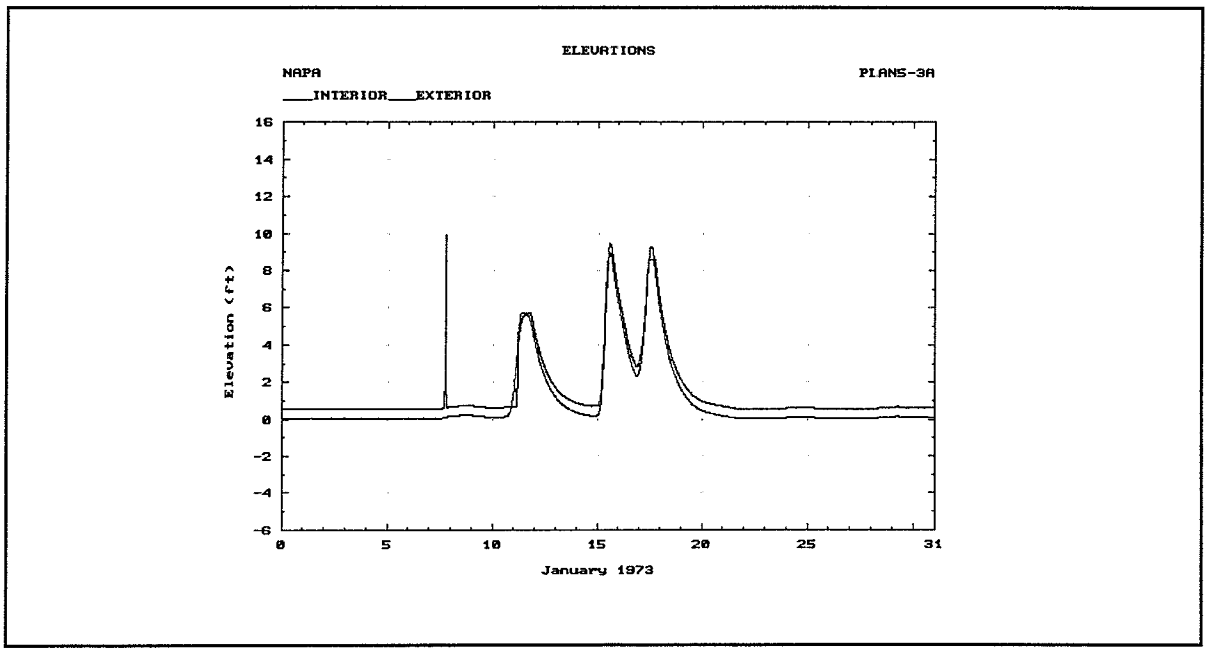


Figure 20. Interior and Exterior Stages - January 1973 Event, CSA

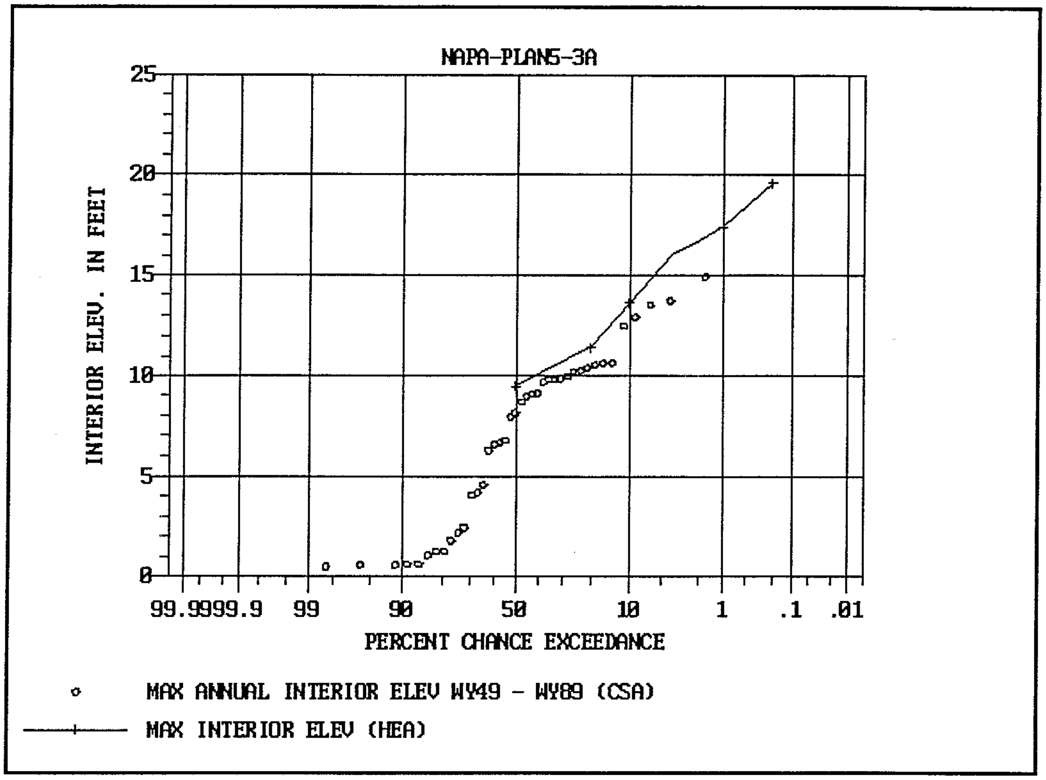
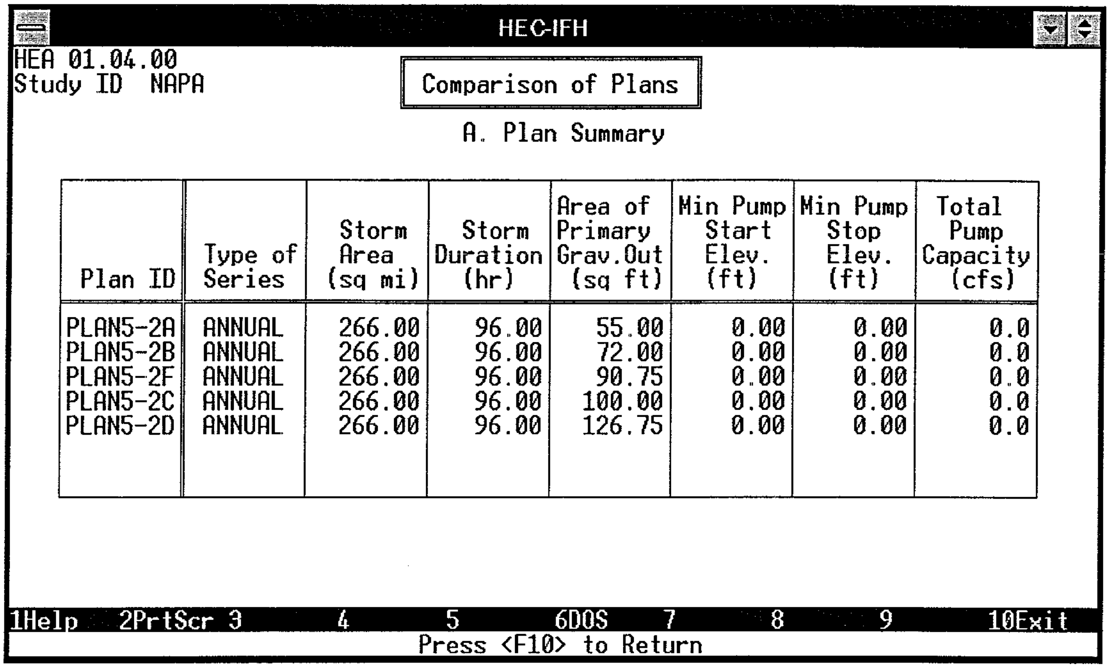


Figure 21. Interior Stage-Frequency Relationships for CSA and HEA - Area 5

7. Stage-Frequency for Interior Flood Damage Reduction Plans

a. General. The objective of this task is to develop stage-frequency relationships that can be used to formulate a set of flood damage reduction plans for each interior area. The condition with the line-of-protection and the selected minimum gravity outlet in place becomes the without project condition for evaluating additional features such as additional gravity outlets, pumping stations, additional ponding area storage, and nonstructural measures.

b. *Stage-Frequency Relationships for Additional Gravity Outlet Capacity.* New plans for evaluating additional gravity outlet capacity using data previously developed for the HEA with the minimum facility in place were defined. Only the gravity outlet data needed to be changed to define plans with a range of outlet sizes. Four or five gravity outlet configurations (modules), with one or more gravity outlets in addition to the minimum facility outlet, were defined. Each module represents an incremental increase in total outlet capacity. Several plans which incorporated the gravity outlet modules were defined and interior stage-frequency relationships were developed for each plan. The HEA results were adopted for final stage-frequency relationship for each gravity outlet plan. These relationships will be used in the economic analysis to select an optimal plan. A plan summary for the four different plans analyzed for Area 5 are shown in Figure 22. Figure 23 shows a comparison of the plan stage-frequency relationships.



HEC-IFH

HEA 01.04.00
Study ID NAPA

Comparison of Plans

A. Plan Summary

Plan ID	Type of Series	Storm Area (sq mi)	Storm Duration (hr)	Area of Primary Grav.Out (sq ft)	Min Pump Start Elev. (ft)	Min Pump Stop Elev. (ft)	Total Pump Capacity (cfs)
PLAN5-2A	ANNUAL	266.00	96.00	55.00	0.00	0.00	0.0
PLAN5-2B	ANNUAL	266.00	96.00	72.00	0.00	0.00	0.0
PLAN5-2F	ANNUAL	266.00	96.00	90.75	0.00	0.00	0.0
PLAN5-2C	ANNUAL	266.00	96.00	100.00	0.00	0.00	0.0
PLAN5-2D	ANNUAL	266.00	96.00	126.75	0.00	0.00	0.0

1Help 2PrtScr 3 4 5 6DOS 7 8 9 10Exit
Press <F10> to Return

Figure 22. Summary of Plans for Evaluating Outlet Capacity - HEA

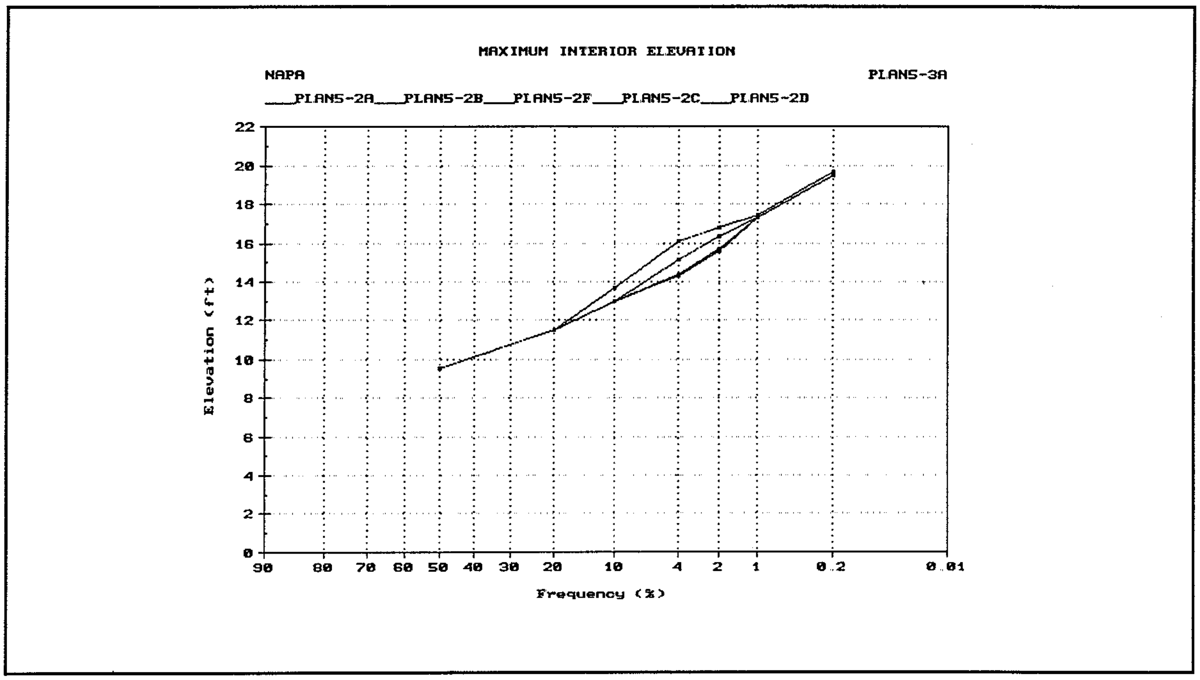


Figure 23. Stage-Frequency Relationships for a Range of Gravity Outlet Sizes

c. Determine Stage-Frequency for Added Pumping Capacity

(1) General - The analysis for Area 5 shows that additional gravity outlet capacity is not effective, due to considerable coincidence between interior runoff and high exterior stages. Residual damages may be significant, and pumps may be justified. The same steps described for evaluating additional gravity outlet capacity are appropriate for evaluating added pumping capacity. Some differences in the analysis are described below.

(2) Base condition - The base condition for evaluating pumping capacity is with the minimum facility and, most likely, the economic optimal gravity outlet configuration in place. Several plans are evaluated against the base plan, each with an incremental increase in pumping capacity. At the time of this writing the preliminary economic optimal gravity outlet was selected as 4-5 X 5 ft. box culverts (Plan 5-2C). HEA plans for Area 5 with the selected outlet and three different size pumping stations were defined and analyzed. The plan configurations are shown in Figure 24 and the stage-frequency relationships are shown in Figure 25. These relationships will be used to define the optimal pumping station size for interior Area 5.

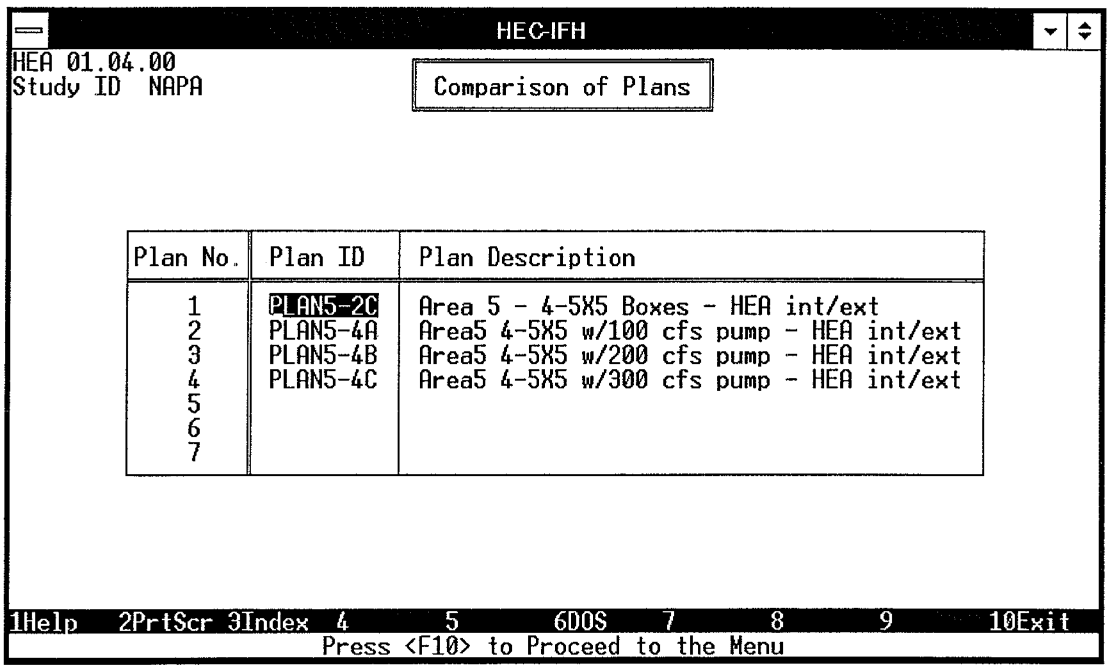


Figure 24. HEA Plans for Evaluating Pumping Capacity

HEC-IFH

HEA 01.04.00
Study ID NAPA

Comparison of Plans

B. Maximum Interior Elevation-Frequency

Plan ID	Peak Elevation (ft) vs. Percent Chance Exceedance							
	50%	20%	10%	4%	2%	1%	0.2%	SPF
PLAN5-2C	9.54	11.51	13.01	14.33	15.62	17.29	19.49	0.00
PLAN5-4A	9.54	11.51	13.01	13.80	15.05	16.48	19.15	0.00
PLAN5-4B	9.54	11.51	13.00	13.56	14.52	15.93	18.79	0.00
PLAN5-4C	9.54	11.51	12.99	13.54	14.15	15.54	18.57	0.00

1Help 2PrtScr 3 4 5 6DOS 7 8 9Plot 10Exit

Press <F10> to Return

Figure 25. Stage-Frequency Relationships for Evaluating Pumping Capacity

d. Nonstructural Measures. Temporary evacuation, relocation, flood proofing, and other non-structural measures that reduce susceptibility to damage, as well as the increase in available storage, will be evaluated by the district and considered in the final recommended plan.

e. Final Plan Selection. Other social, institutional, and environmental issues, including the management of future development, and flood warning and preparedness programs, will need to be evaluated in the final plan selection for each interior area.

MUNICIPAL STORMWATER COMPUTER MODELING PROGRAM NEEDS

Ben Urbonas*

INTRODUCTION

Stormwater runoff modelling needs of the municipalities in the United States have evolved over the last 30 years. It used to be sufficient to use the Rational Formula because the only need was the design of storm sewers. Then the municipalities began to insist on the use of detention to control runoff peaks which mandated that a runoff volume and/or hydrograph be also addressed. When municipalities began doing system-wide planning, total system hydrologic response had to be understood and addressed. Hence, the use of distributed routing models became the state-of-practice.

The ever-evolving development of distributed routing models, in combination with the availability of powerful desktop computers, has resulted in widespread use of these models among engineers and hydrologists. Unfortunately, a very large percentage of model users have little or no understanding of the computational processes that occur within the "black boxes" or, better stated now, "colored VGA boxes" they are using. The tools are easy to use, often have a graphic representation of the results and give the impression that stormwater runoff is actually behaving as "seen" on the screen. But how accurate are the results?

MODEL SELECTION FOR DENVER

In the 1977-78 period the author investigated for the Urban Drainage and Flood Control District (UDFCD) four stormwater models, SWMM, HSPF, HEC-1 and the Colorado Urban Hydrograph Procedure (CUHP), for adoption as a standardized model for the Denver metropolitan area (i.e., the area within UDFCD). Rainfall and simultaneous runoff data collected over an eight to ten year period at eight separate urban sub-watersheds was used for this evaluation. The goal was to have a model that everyone in this metropolitan area could use with confidence and obtain relatively consistent results despite the users' level of expertise. During this investigation we discovered that HEC-1, SWMM and HSPF gave

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excellent results when they were properly calibrated and were used to model a range in runoff events similar to the events that were used for initial calibration.

In almost all cases, the use of SWMM, HEC-1 and HSPF without calibration produced runoff peaks that were often 50 to 200 percent off from the recorded values. Also, since most of the data were from smaller events (i.e., 2-year storms or less), the calibrated models, especially the kinematic wave models, performed very well for smaller events but tended to over-estimate the larger floods such as one 10-year and one 25-year flood for which we also had data. In almost all cases the bias was towards over-estimating of the peak flows by HEC-1 and SWMM. Overestimating of runoff peaks and volumes are a major concern to municipalities. It is not only of academic interest that the estimates be accurate. They are used by municipalities to size storm sewers, culverts, bridges, detention facilities, etc., all of which affect local budgets and billions of dollars throughout the United States.

Further evaluation of the four models led to a decision by UDFCD to update and recalibrate the CUHP model in use at that time for District's continued use. The reasons for this decision, among others, included:

- Local engineers were already familiar with CUHP

- CUHP was relatively easy to use

- CUHP results related well to known and forecast land use and geometry

- CUHP gave consistent results without calibration (i.e., 80% of the calculated flow peaks were within 20% of the recorded peaks)

The CUHP model, which is a parameter-based runoff model for an individual sub-watershed, was calibrated using the locally available data. In addition, design storms were also developed for the 2-, 5-, 10-, 50- and 100-year return periods. These were designed to produce runoff peak rates using the CUHP model that match the cumulative probability distribution of peaks for the test sub-watersheds. The latter were found by simulating runoff using 73 years of recorded rainfall at the Denver NWS gage. The final results are summarized in Figure 1 which shows a comparison between the recorded peak flows at the eight test sub-watersheds and the calculated peak flows for these same storms. Although the eight test sites represent a diversity of land uses, a range in tributary areas of 0.4 to 3.04 square miles and in storm types, the CUHP model produced an excellent agreement with the recorded data.

The calibrated version of CUHP is described in detail in the Urban Storm Drainage Criteria Manual (UDFCD, 1984). The procedure was computerized and the software can produce an unlimited number of sub-watershed hydrographs for a given design or a recorded storm. The resultant hydrographs can be linked to the COE Missouri River Division's Runoff Block of SWMM. This version was adopted by UDFCD and, after some debugging, was converted in 1986 to operate on a desktop computer.

The combined use of CUHP-PC and UDSWM permits the UDFCD, the municipalities, land developers dealing with municipalities and their consultants within the UDFCD to generate regionally consistent and reliable stormwater runoff estimates for urban sub-watersheds. The resultant hydrographs can be routed through a system of conveyance, storage and diversion elements that represent the watershed's drainageway system.

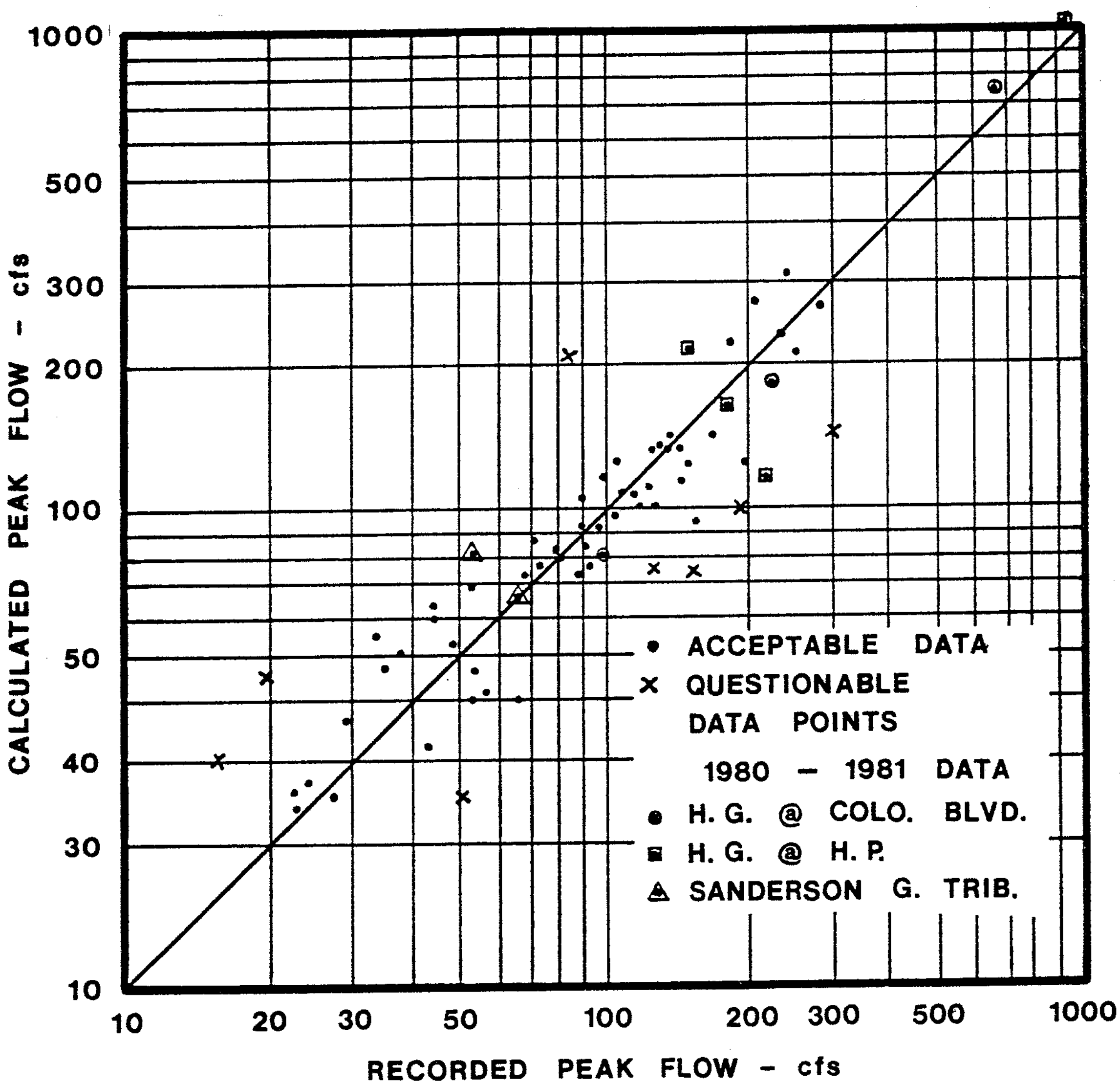

Figure 1. Calculated vs. Recorded Peaks for the CUHP Model. (UDFCD, 1984)

MUNICIPAL MODELING NEEDS

Despite the fact that much attention is given to the development of more complex models with user friendly interfaces, some obvious shortcomings remain in urban stormwater surface runoff computer models. The greatest shortcoming and concern is that the more complex models do not provide accurate results unless they are calibrated. On the other hand, most municipal stormwater system design and planning takes place without the benefit of simultaneous rainfall and runoff data for the study watershed. This is especially true for land development projects. Also, even if data is available, the changing land use within these watersheds makes calibration moot.

In addition to the author's experience described earlier, a recent Master of Science of Civil Engineering thesis (Small, 1993) at Virginia Polytechnic reported on the evaluation of various stormwater models and methods. Figure 2 is a graphical summary of the results for seven of these models. Apparently, Ms. Small ranked the performance of each model in its ability and accuracy of simulating several aspects of hydrology, storm conditions and land uses, assigning a score of 100 for excellent performance and a score of zero for poor performance. Without making a judgment of how he obtained these rankings, two models, HEC-1 kinematic wave version and SWMM (ver 4.05), showed excursions towards poor performance. Both models showed poorest ratings in simulating urban sub-catchments and larger watersheds with extensive overland flow areas, while the kinematic version of HEC-1 also did poorly in simulating runoff peaks and short duration storms. Other HEC-1 sub-models, such as SCS UH, Snyder UH and Clark UH performed well.

In a separate blind test of six public-domain stormwater models conducted by a committee of ASCE (the results of which are yet to be published by the investigators), both HEC-1 and SWMM showed a tendency to grossly overestimate peak runoff rates. The blind tests involved the use of data from two relatively small urban watersheds, each from totally different meteorological regions of United States. Each model was coded by a modeler experienced with the model being tested. Watershed information and five to six recorded rainfall events were provided to each modeler and the results were compiled by the chief investigator who compared them against the recorded hydrographs. As was the case in Small's findings, the simpler, parameter-based runoff models produces best estimates of runoff peaks. Clearly, neither the model's sophistication or complexity or the user's expertise assure accuracy when these models are not calibrated.

The expertise of an average, or the majority of model users working the municipal system, is often not equal to the models being used or to the expertise of the modelers involved in the "blind" test conducted by ASCE. The more complex the model, the more input parameters are required, the more judgment is required of the user to select each parameter and the likelihood of inaccuracy increases. For example, the *tributary width* parameter in the runoff block of SWMM is often assumed to be the physical dimension of the sub-watershed. If the watershed is a perfect plane of constant slope and roughness and does not have a complex network of streets, parking lots and buildings, this assumption may be valid. In reality this is a variable requiring calibration using either field recorded data, a target discharge value or tremendous expertise to select values that will yield accurate runoff response simulation.

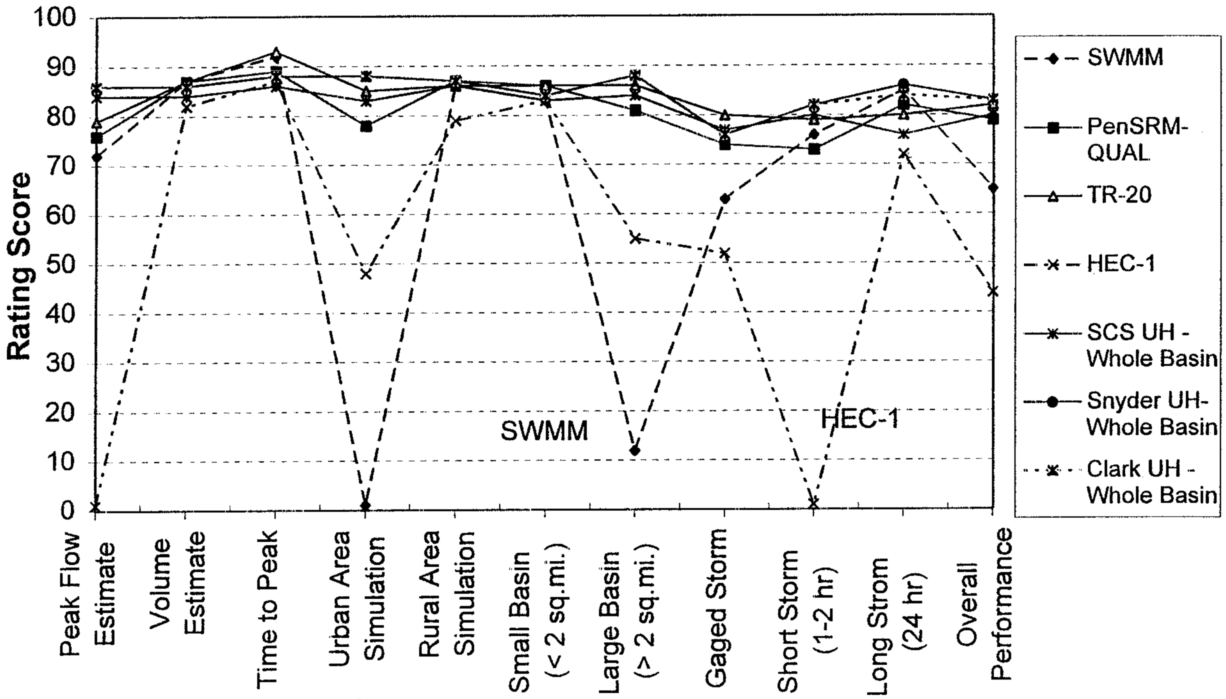

Figure 2. Summary of Model Ratings by Small (After Small, 1993)

Observing how computer models are used by hundreds of engineers serving municipalities, some of whom are excellent modelers, the author offers the following suggestions of what should be the new focus in the development of stormwater runoff models that serve urban systems:

- Simplicity of use.
- Minimum number of input parameters that may yield reasonably accurate results.
- Ability to override and input more parameters by more experienced users when calibrating data are available.
- Clear guidance on parameter sensitivity and how to test for it.

- Clear guidance for calibration of models and what to do when data is not available.

- Clear discussion on each model's limitations and areas of concern such as:
  + Rainfall spatial distribution.
  + Variability of spatial and temporal rainfall distribution by,
    - meteorological region,
    - intensity and duration of storms,
    - frequency of storm occurrence (i.e., 2-year or less vs. 100-year).

- Guidance to avoid additive or multiplicitive conservative factors to prevent overestimates of flow rates and volumes.

- Acceptance and use of locally or regionally calibrated hydrologic procedures, either for direct use or for the use of their results as calibrating targets for more complex distributed routing models.

CONCLUSION AND RECOMMENDATIONS

The municipal stormwater modeling needs are varied and need to be designed for a broad range of user expertise in modeling and hydrology. There are a variety of models available on the market that range in complexity from simple parametric, unit-hydrograph, based models to fully dynamic and kinematic wave based distributed routing ones. The level of sophistication and complexity of the model, despite its user-friendly interfaces, does not assure better accuracy. In fact, unless the more complex models are well calibrated, their accuracy in predicting runoff peaks appear to be less than what can be produced by the simpler, parameter based models.

It is recommended that more attention be given to local and regional differences when developing guidelines in how to use the newer models. Many local and regional efforts have taken place in the United States that are based on local data and conditions. To ignore them tends to discredit both the model and the organization that is using it, especially when the results conflict with existing local studies. Also, more effort is needed to develop reasonable calibrating or "target value" procedures for use by less experienced users that can easily be understood. The possibility of having two levels of input should be explored, one for inexperienced users and for users without calibrating data, the other for experienced users with data or "targets" available for calibration. Building only more exacting mathematical representations of the rainfall/runoff processes is not the answer since we do not have sufficient data to calibrate them. In urban areas, when we need to predict the effects of changing land uses, data is not available for almost all of the watersheds we need to model.

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Urban Hydrology and Hydraulics Workshop
13 - 15 September 1994
Hydrologic Engineering Center (HEC)
US Army Corps of Engineers
Davis, California

Urban Hydrology in Switzerland

By

Peter Kaufmann, Troy Nicolini and Arlen Feldman

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1. Review of Swiss Urban Drainage Practices

Nearly 100% of the urban population in Switzerland is connected to a sewer system. The traditional system servicing most of these urban areas is the combined-sewer system, shown below in Figure 1. The alternative to the combined sewer is the separate-pipe system. This approach is used only in certain areas across Switzerland, as most Swiss homes have basements that make installation of multiple pipes difficult and costly.

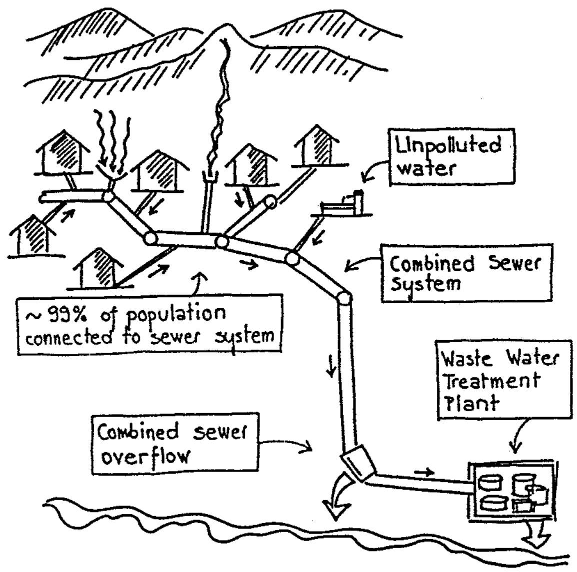


Figure 1 :
Actual Swiss Urban Drainage Practices

The combined-sewer system reflects sentiments of the "old theory," where the goal of drainage was to collect as much water as possible in a short amount of time. Its design is based on the rational method, using a return period of five to ten years. This long return period made the combined-sewer system an effective safeguard against floods, as sewers in such a system were seldom overloaded. The downfall of the combined-sewer system, however, is that these systems collect wastewater, rainwater, snowmelt, industrial waste, and water from small streams and creeks, and forced wastewater treatment plants to be much larger than necessary. Thus, a large treatment facility in a period after construction had a difficult time finding enough water to stay in operation, and would treat water regardless of whether it was polluted or not. Also, the combined-sewer system relies on a significant storm event every few weeks to flush accumulation of deposition from the system. Such reliance on mother nature often yields unsatisfactory results.

All wastewater, regardless of the system used, is treated in a wastewater treatment plant. As a general rule, a facility's capacity is equal to twice the dry weather flow for the given area. The wastewater treatment facilities use both physical and biological processes, and, in special cases, filtration, not disinfection, of the effluent is also used. During wet weather, and other times of unusually high flow, combined-sewer overflow is treated in sedimentation basins or screened.

Many of the combined-sewer systems were built more than 50 years ago. As urban areas grow, so too must the sewer systems. The old solution of merely diverting water away from the city is no longer a valid solution. The lack of planning for alternate methods in the past make accepting new ideas very difficult today.

2. Shortcomings of the Existing Urban Drainage Practices

An analysis of the current system yields the following observations (illustrated in Figure 2):

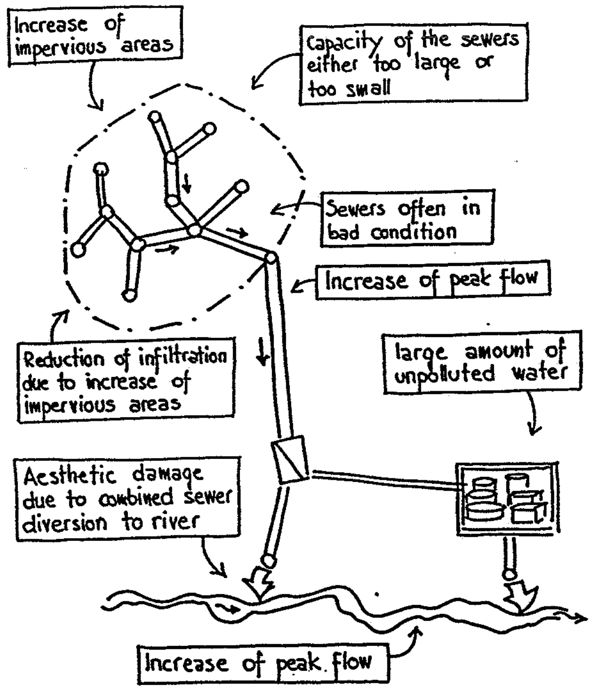


Figure 2 :
Analysis of the current Swiss urban drainage system

1. Increased urbanization leads to increased impermeability and peak flow in the sewer systems and rivers. New sewer systems are necessary, and in many areas the current system needs expansion.

2. Many sewers are not in proper serviceable condition and must be upgraded or replaced soon.

3. The old method of draining all water to the sewers results in a large amount of unpolluted water channelling to the drainage system during both "wet" and "dry" weather periods. This, by itself, does not lead to hydraulic problems in the sewers, but it does cause great inefficiency in the operation of the wastewater treatment plants. The dilution of polluted water with unpolluted water forces wastewater facilities to constantly operate, raising costs and wasting energy.

4. Combined-sewer overflows create several problems:

   * Hydraulic problems; watercourse capacity never checked or verified

   * Danger to wildlife; fish and other creatures depending on the streams and rivers for survival are poisoned by substances from human sewage

   * Degradation of aesthetics; waste, such as paper and fabric, litter banks and shores of parks and public river systems

5. Most urban areas do not have the financial resources to continually fix environmental problems.

6. Urban expansion has been mis-managed. New urban developments are being added in areas where the sewers are already being burdened and cannot support a new neighborhood; and other sewer systems in the same area are only being used at a fraction of their capacity.

7. Increasing impervious areas diverts more water to the rivers, depriving groundwater sources of recharge. Without a recharge source, local groundwater levels fall.

3. Objectives for the New Approach for Planning of Urban Drainage

Urban drainage must now meet the following requirements:

* Wastewater from residential and industrial centers must be removed from the urban area immediately for prompt treatment at a wastewater plant.

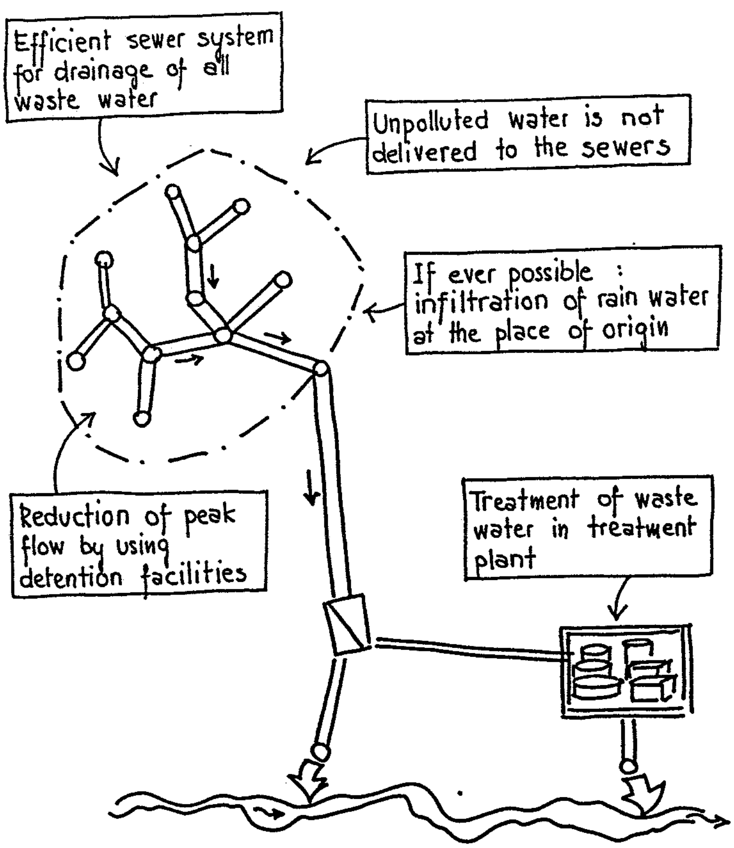
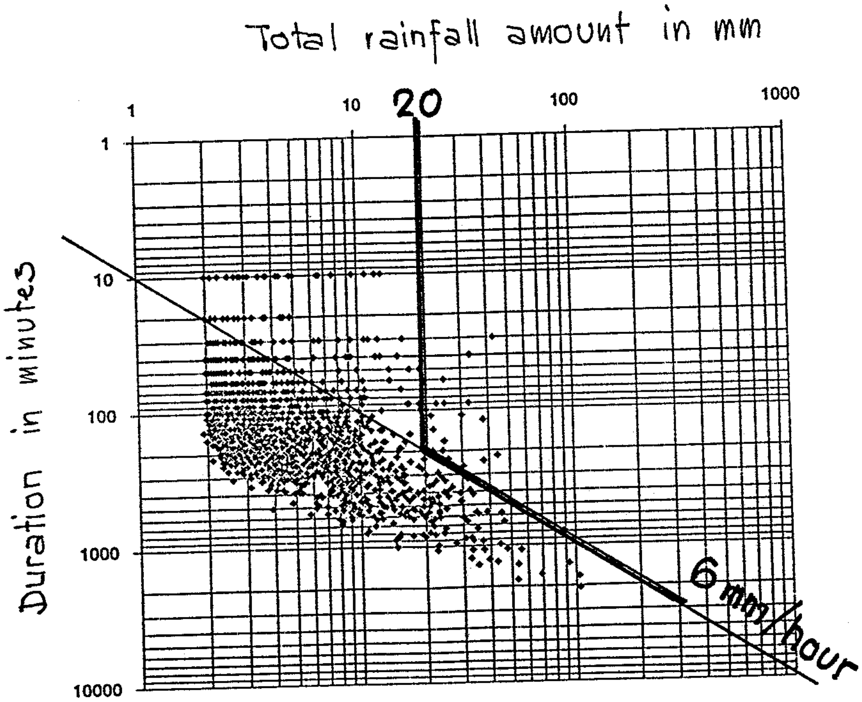


Figure 3 : Objectives for the new approach

* Precipitation must infiltrate at the place of contact with the ground. If infiltration is not possible, unpolluted water must be transported to a river or lake. Only if no other method is physically or financially feasible will precipitation be routed with wastewater to a wastewater facility.

Comparison of many rainfall event magnitudes to infiltration capacities shows on site infiltration can be very succesful with regards to water quality. As shown in Figure 4 for the historical rainfalls of the city St. Gallen (represented by their amount of rainfall and duration) only a few storm events are too large to be infiltrated. For example for 20 mm of soil storage capacity and an available infiltration capacity of 6 mm/hour, only 2 % of events produce runoff. For flood concerns this conclusion is not as important as for water quality where the small events carry most polutants.

Figure 4 :
Rainfall diagram for the city of St. Gallen,
1982 - 1991,
1408 historical events



* Peak flow for runoff must be reduced when possible using detention facilities.

* Unpolluted water from springs, creeks, groundwater, fountains, etc., must be eliminated from the combined-sewer system.

* Future urban drainage plans must consider the catchment, drainage system, receiving water, groundwater, soil, and wastewater treatment. The interface of these components is shown in Figure 5.

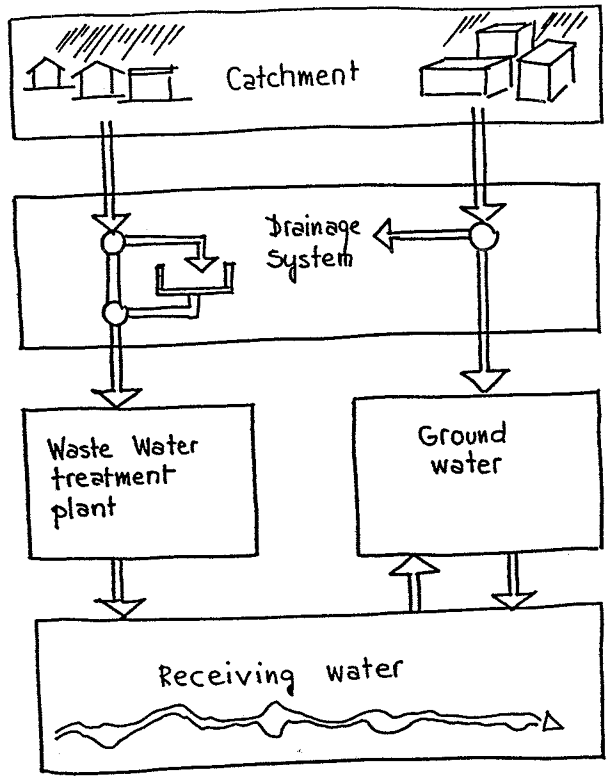


Figure 5 :
Interface of the components
for future urban drainage plans

The modern urban hydrologist must abandon the "old school" thought of working alone, and work as a member of a team, as is outlined in Figure 6 Such interaction between agencies and specialty disciplines results in the new approach, as depicted in Figure 3.

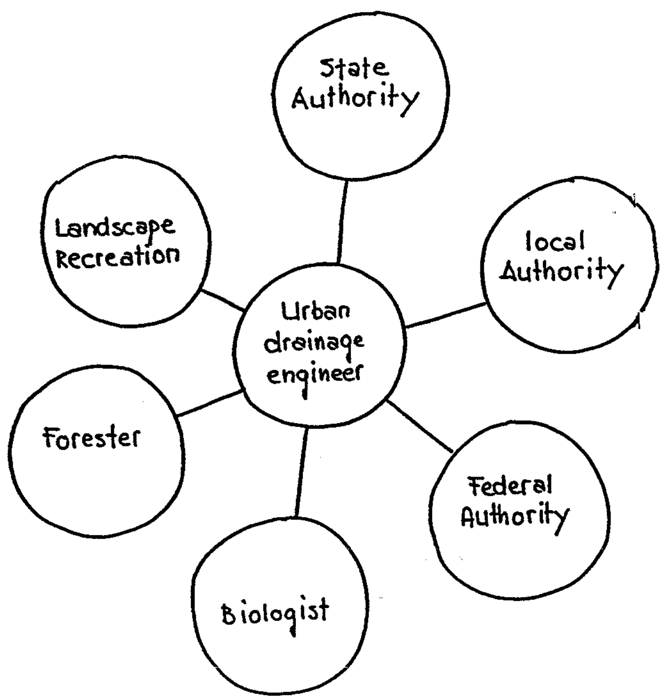


Figure 6 :
Interaction between
agencies and speciality
disciplines

4. Implementation of the New Approach

The new guidelines developed by the Professionals of Waste Water and Environmental Protection in Switzerland (VSA) requires a stepwise procedure [1,2]:

1. Definition of goals
2. Elaboration of the main topics
3. Description of tasks, and definition of level of effort to attain goals
4. Acceptance by authorities
5. Data acquisition and documentation, as well as physical system description and literature review
6. Conceptual analysis of existing system and recommendations for drainage systems components
7. Design of drainage system components

The new methodology, as it is described in the new guidelines for the Urban Drainage Master Plan, also requires:

* The level of effort for a particular task should be commensurate with the significance of the task to the overall study objectives, as shown in Figure 7.

For Example: if problems arise in the receiving water due to wastewater overflow, the actual conditions of the receiving surface's wastes must be documented thoroughly. If the concerned drainage area contributes to a large river, however, small amounts overflow from combined wastewater drainage systems will not have a significant effect on the water quality; no benefit will come from an in-depth study.

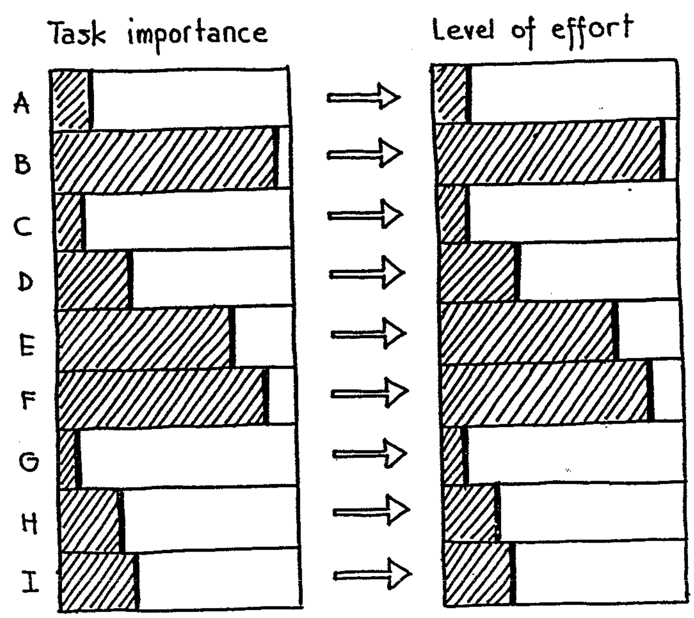


Figure 7 : Level of effort as a function of the task importance

* As urban drainage systems become increasingly complicated, it is more and more important for involved agencies to collaborate towards the solution. Agencies that play a critical role in such a project are governmental offices and non-engineering disciplines like geology and biology.

* The planning process must be done in a stepwise, methodical way. Such a process is shown in Figure 8.

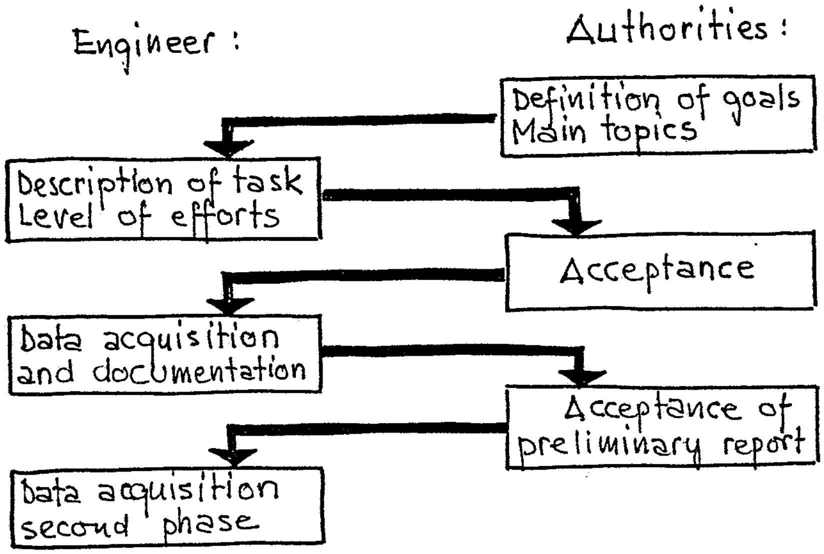


Figure 8 :
Stepwise planning process

* The choise of the method to calculate runoff and flow through a drainage system has to be done considering different conditions as illustrated in Figure 9.

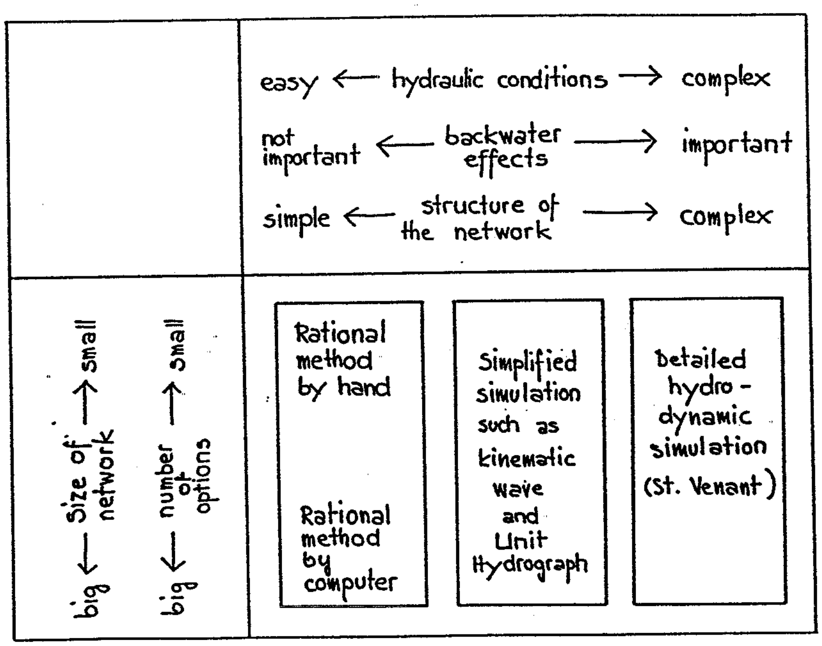


Figure 9 :
Table for chosing a hydraulic calculation method

4.1 Data Acquisition and Documentation, Physical System Description, and Literature Review

Urban drainage master planning begins with an extensive monitoring of the existing urban drainage system. This includes:

* reports on demographic, economic, and water resources development.

* storm water infiltration and detention.

* structural properties of the sewers.

* infiltration analysis.

* hydraulic capacity of the sewer system.

* hydrology, morphology, and ecology of the receiving waters.

All evaluated topics must be documented in separate reports.

Report 1: Receiving Surface and Groundwater

All topographical, hydrological, morphological, chemical, limnological, and biological data must be collected and evaluated. In addition, consideration must be given to human activities such as swimming, water supply, and fishing when receiving waters are evaluated.

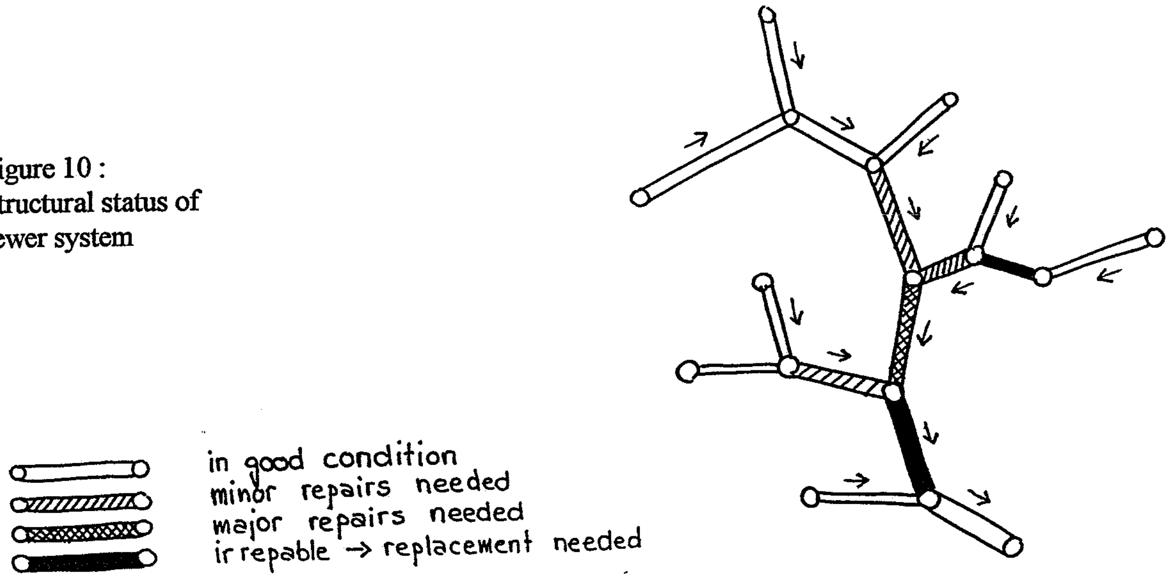
Report 2: Unpolluted Water

Unpolluted water origins and rates of flow leading to combined sewer systems must be specified. In Switzerland, for example, many fresh water fountains, foundation drains, and small creeks are connected to the sewer system. A useful tool for identifying such sources is a video of the inside of the sewers that shows all input sources for the sewer.

Report 3: Sewer System

Sewers must be evaluated in terms of structural, operational, and hydraulic data. Figure 10 shows an example of a sewer's structural status.

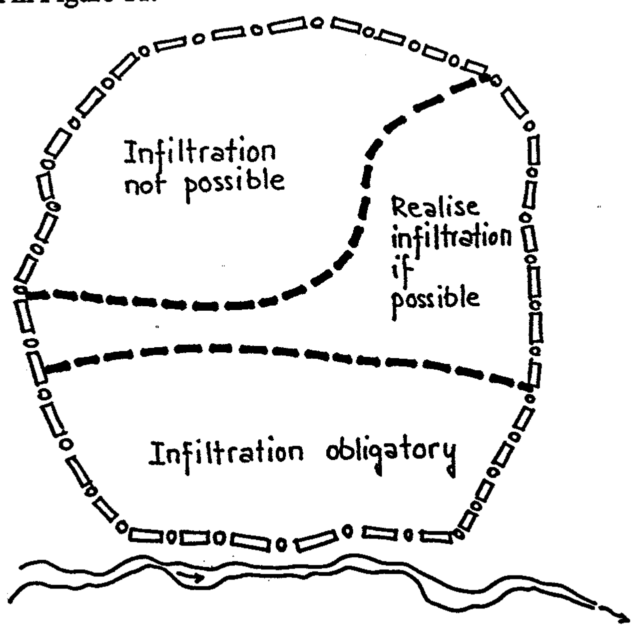
Figure 10 :
Structural status of
sewer system



Report 4: Storm water Infiltration

Hydrologists must identify possible infiltration sites and capacities. These sites must then be evaluated with respect to pollution of storm water, hydraulic conductivity of the soil, and groundwater protection zones (areas). This is illustrated in Figure 11.

Figure 11 :
Zones for storm
water infiltration

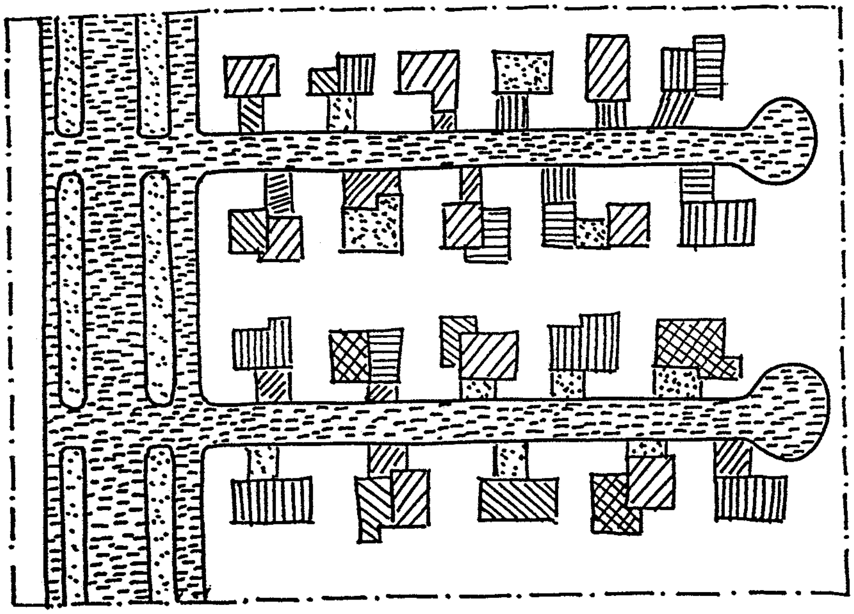


This information must be available during the subsequent phases so that the true impact of infiltration can be assessed, and possibly designed for. Since, in Switzerland, almost all houses are directly connected to the drainage system, the potential impact of induced infiltration is high, but depends on infiltration capacities.

Report 5: Urban Catchment

The size of each catchment and subcatchment, along with their land use characteristics, must be described for the existing and future conditions. Typical urban land use distribution is shown in Figure 12. With this information, it is possible to calculate runoff coefficients and times of concentration. This is the minimum required information for rainfall-runoff computations using the rational method. For unit hydrograph analysis, additional information is required.

Figure 12 : Typical land use distribution for urban catchment



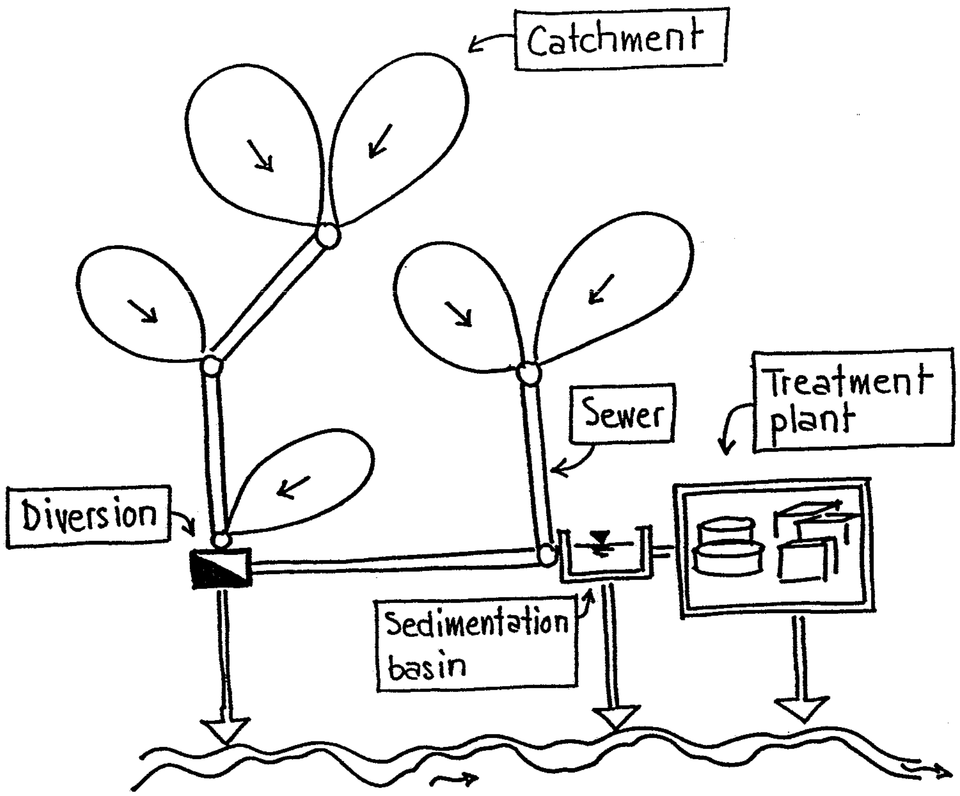
Report 6: Special Hazards

The probability of damage from industry, traffic, and dangerous liquid storage, as well as hazards to the sewer system, treatment plant, and receiving water must be discussed.

4.2 Conceptual Analysis of Existing System and Recommendations for Drainage System Components

A conceptual analysis of an existing system allows urban hydrologists to plan alterations with respect to the goals, main topics, tasks, and stated recommendations from VSA. Such analyses also allow for new alternatives that may be more appropriate for the particular community being dealt with. In many cases, simplified simulation models can be used to evaluate situations like the one illustrated in Figure 13.

Figure 13 :
Simplified simulation model for conceptual analysis of the drainage system



One such model is SASUM, which was specialy developped for the conceptual analysis of existing sewer and drainage systems. The purpose of SASUM is not to describe in detail how the system responds to a single design event. Instead , SASUM evaluates how the systems response to a series of historical rainfall events will impact on the receiving water. In other words, it does not adress specifically the hydraulics of a particular reach of pipe, but instead takes a simplified view of the components of the system so that the functioning of the entire system can be evaluated for many alternatives over many historical events [3].

The SASUM network model has sedimentation basins, overflow structures, diversions, pipes and catchments. The flow in the pipes is calculated with the kinematic wave and for the overland flow in the catchment a reservoir routing and a time-area relation are used. For the contunuous simulation, time series with about 1000 events are used. The time step of the historical rains is mostly 5 or 10 minutes. In chapter 5, an example of using SASUM is presented which illustrates that these computational methods are sufficient for deciding amongst alternatives. Complex models can subsequently be used for design phases.

Alternatives to take into consideration during an analysis are:

* a change from a combined sewer system to a separate sewer system (or vice-versa).

* an increase in infiltration which may reduce the amount of storm water collected by the drainage system.

* an addition of new sewer facilities, such as tanks, detention basins, screens, new parallel pipes, and changed sewer overflows.

At the conclusion of the conceptual analysis, all involved authorities must decide on a best solution for future drainage systems. At this point, concepts and major questions should be the only variable discussed; fine details of the project are to be held over until the critical analysis has been thoroughly discussed and decided upon. Finally, to aid in the complete resolution of a new system, proposed alternative concepts must be documented so all decisions are comprehensible.

4.3 Design of Drainage System Concepts

After all concepts and alternatives have been evaluated, and one has been committed to the chosen alternative, is finally evaluated in great detail. Typical project specification tasks include:

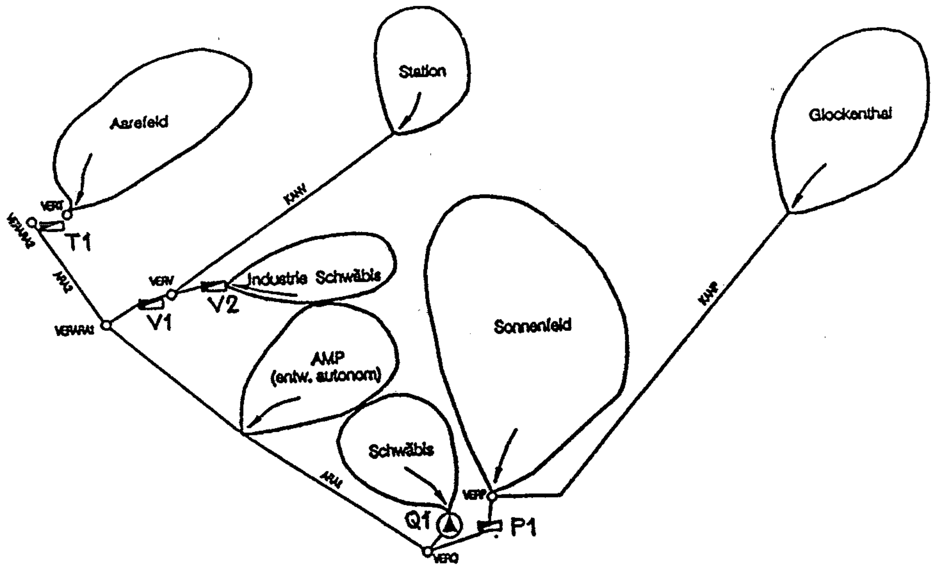
* hydraulic design of sewers
* determination of storm water infiltration
* design of facilities for treatment of overflow from sewer outlets
* description of methods for reduction of unpolluted water in the system.

Cost for new facilities must be estimated, and all design information necessary for plan execution and implementation must be presented during this final step to ensure a smooth transition from one system type to another.

5. Example of Conceptual Analysis for Steffisburg, Switzerland

Steffisburg lies between Bern and Interlaken, near the town of Thun. The city has grown quickly in the last 20 years to its current population of about 10,000 people. Future growth of Steffisburg is evident from the large number of homes currently in planning or under construction. Many of these new homes, along with almost all of the existing homes, will have their sewer facilities connected to an outdated combined sewer system that is 50 years old in some parts. This combined sewer system is having difficulty matching the city's rapid growth. Several precautions are being made, however, to prevent a potentially dangerous problem from occurring due to this sewer system. According to the New Guidelines for Urban Hydrology, for example, all data concerning existing sewers, land use of catchments and subcatchments, reception of surface and ground water, rates and origins of unpolluted water, and possibilities of storm water infiltration must be collected and published in reports. These reports, in turn, provide vital information for the safe use and regulation of the combined sewer system.

Figure 14 : SASUM network model used for the conceptual analysis



This chapter deals with the planning involved in accommodating the new homes to the old system, and includes a conceptual analysis of the existing systems and a description of recommendations for drainage. SASUM computer models aided in the analysis of Steffisburg's system, and provided valuable information about combined sewer overflows and the analysis of such a problem. The goal of these models was not the calculation of a design storm; rather the accurate measurement of the annual rain water overflow, total hours of overflow per year, and the volume of rainwater stored in the sewer system. By creating a simplified model of Steffisburg's system, SASUM aided hydrologists in meeting these goals, as well as the calculation of rainfall runoff with respect to time by analyzing approximately one thousand historical rain events in the area. Figure 14 shows the SASUM network model used in this scenario. This model includes seven catchments, five main sewers and overflows, and one pumping station. To solve Steffisburg's potential problems, the computer model evaluated the current combined sewer system, as well as seven possible variations. These include [4]:

1. Modify all combined sewer overflow structures to meet Swiss Federal requirements.

2. Modify all combined sewer overflow structures so that all flow less that 2 times the dry weather flow is treated.

3. Combined sewer overflow structure at V2 is eliminated.

4. Capacity of pump at Q1 is increased to 100 liters per second.

5. Add 20 m^3 storage facility at Q1

6. Add combined sewer overflow structure with a screen at P1 with an overflow elevation of 1.25m.

7. Add combined sewer overflow structure with a screen at P1 with an overflow elevation of .80m.

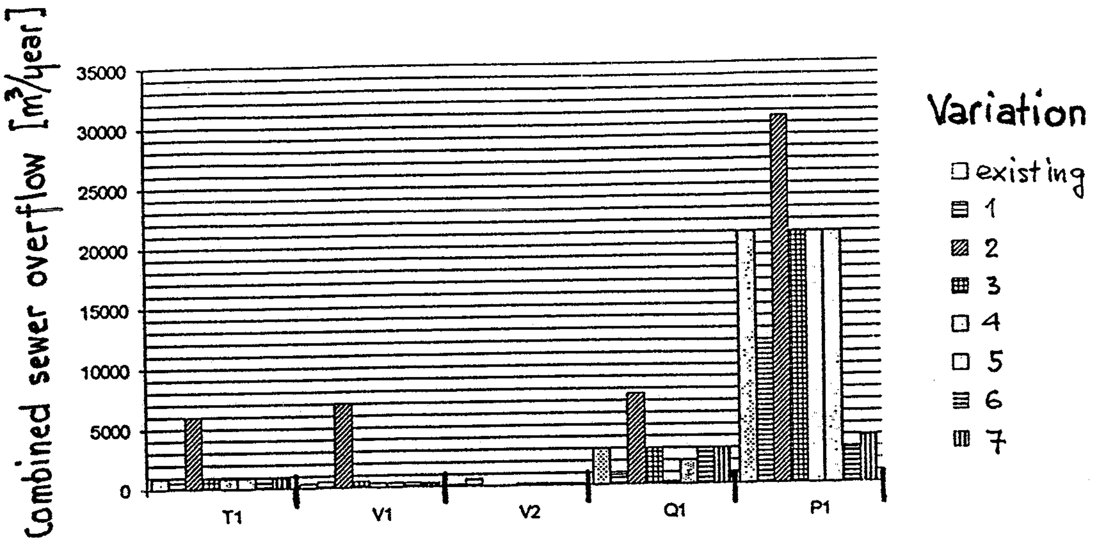


Figure 15 :
Impact of seven alternative drainage modifications

Figure 15 shows the impact on combined sewer overflow, relative to existing conditions, of the seven alternatives. As is evident from these results, modifications are necessary for the overflows Q1 and P1. Modifications made on other overflow buildings, however, show no significant effect on the annual wastewater overflow to the river. Therefore, the conclusions from the conceptual analysis are that a larger pump is required at Q1, and a new overflow structure, with a screen, is required at P1, shown in figure 16. An important point brought out by this example is that the conceptual level of analysis is fully adequate in predicting the necessary modifications. Even if the results shown in figure 15 are in error by 100% the same two alternatives would have been chosen. This is frequently the case.

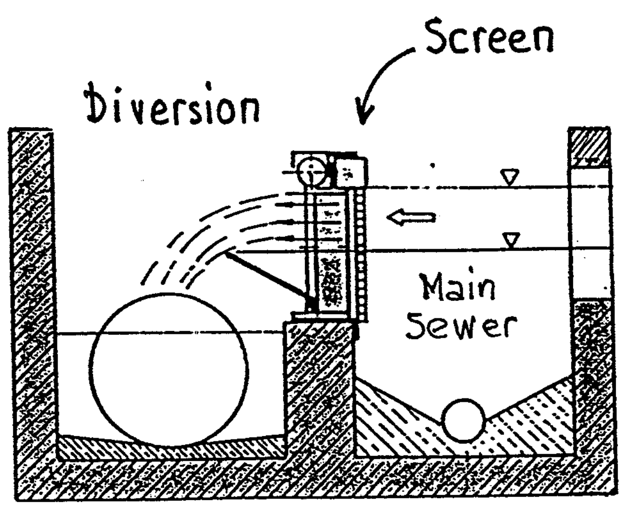


Figure 16 :
New overflow structure with screen required at P1

6. Conclusions

The new guidelines have been used since 1989 with an accompanying manual for engineers to help them better understand the new approach. This new approach represents a radical change in the Swiss way of thinking in terms of planning, execution, and cooperation between several agencies. Although initially more expensive, the total costs are estimated at 1% of the replacement cost associated with the older system; and the new method promises to recover costs soon after implementation. The VSA continually includes new ideas and experiences into the planning process, and encourages the integrated planning process to bring about maximum benefits for the environment.

7. References

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Problems Encountered and Solved in the Applications of SWMM

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ABSTRACT: A number of problems were encountered in using the Runoff and EXTRAN Blocks of the U.S. Environmental Protection Agency sponsored "Storm Water Management Model (SWMM)" in the analysis and design of storm water management systems. Because of the complexity and size of the source code of this model, code modifications to solve these problems were found to be tedious and too time-consuming to meet project schedules. It was found that these application obstacles could be overcome by adjusting the model set-up. However, these adjustments must be done in a manner that a reasonable representation of the system hydraulics at the design condition is still preserved. Although this approach has its limitations, reasonable and sufficient outputs were developed from SWMM for use in assessing the design adequacy of the storm water management systems. A list of problems encountered are presented together with suggested solutions. In addition, useful tips on what to watch out for in the outputs are also given.

1 INTRODUCTION

The Hydraulics/Hydrology (H/H) Group at Bechtel has been using the Runoff and EXTRAN Blocks of the U.S. Environment Protection Agency (EPA) sponsored "Storm Water Management Model (SWMM)" in many projects involving the design of storm water management systems. While SWMM is a powerful analytical tool with many capabilities, a number of problems were encountered during the applications of this model. As common to the applications of many of the existing numerical models, quite often, even experienced engineers are frustrated by application difficulties. The objective of this paper is to share this experience with fellow SWMM and other numerical model users; i) to summarize the problems encountered in using the Runoff and EXTRAN Blocks, ii) to present and discuss the solutions developed to overcome these application obstacles and iii) to reinforce the notion that SWMM is a powerful and useful tool for use in analyzing complex drainage systems .

The solutions, discussed in this paper, are pragmatic and practical. They relied solely on adjusting the model set-up. Although these adjustments in the model set-up did not result in an exact representation of the systems on hand, reasonable representations of the system hydraulics at the design condition were preserved. Sufficient outputs from SWMM were developed for use in the assessment of the adequacy of the systems under the design condition. Since many problems are related to the outputs from SWMM, useful tips in what to watch out for in the outputs are also given.

This approach was used because code modfications to fix these application problems were found to be too time-consuming to meet project completion schedule due to the complexity of the model. In addition, it would also require substantial funding which was not a priority in the current tight-budget business environment The suggested solutions should be considered as stop-gap measures enabling the engineers to develop useful information from SWMM to do their jobs until SWMM is further improved by U.S. EPA. In this paper, the problems and solutions discussed apply to both Versions 4.04 and 4.05 of SWMM.

In the past, there were regular SWMM

workshops organized and funded by the U.S. EPA to facilitate the exchange of application experience among the SWMM users and updating of the model by the designated EPA SWMM contactor. Because of funding cutback, these avenues only exist in a very skeletal form. The enactment of the Storm Water Regulations by U.S. EPA in 1990 will, hopefully, revitalize the usage of and ultimately program maintenance funding for SWMM in the United States. It is in this spirit that this paper is prepared: to renew and promote interest and technical exchange among SWMM users on SWMM applications.

2 AN OVERVIEW OF SWMM

SWMM is a comprehensive computer model for analysis of the quantity and quality of urban runoff. Simulations may be performed on catchments having storm sewers, combined sewers and natural drainage channels to predict flows, stages and pollutant concentrations anywhere in the system. All aspects of the urban hydrologic and quality cycles can be simulated, including surface runoff, dynamic transport through the drainage network, storage and treatment.

It consists of three basic computational blocks; namely,

- Runoff Block: The Runoff Block generates surface and subsurface runoff based on rainfall hyetographs and/or snowmelt runoff, antecedent conditions, land use, and topography.

- Transport and EXTRAN Block: The Transport and the EXTRAN Blocks are used to route flows through the drainage system. Dry-weather flow and infiltration into the sewer system may be optionally added into the system. The Transport Block is a kinematic wave routing model used for non-pressurized flow conditions while the EXTRAN Block is a dynamic routing model used for both pressurized or non-pressurized flow conditions.

  Special elements such as weirs, orifice and pumping stations may be included with EXTRAN Block. Reverse flow conditions and backwater effects can also be simulated. However, pollutant routing can only be performed in the Transport Block.

- Storage/Treatment Block: This Block characterizes the effects of control devices upon flow rates and quality.

3 A BRIEF DESCRIPTION OF SYSTEMS ANALYZED

More than ten storm water management systems were analyzed using SWMM by Bechtel's Hydraulics/Hydrology Group. They were primarily drainage systems located in existing refineries. These analyses were performed in response to recent U.S. EPA requirements related to National Pollution Discharge Elimination System (NPDES) permit in which the owners of the refineries are required to develop and submit a Best Management Practice (BMP) program to assure the compliance of the permit requirements under the current regulatory conditions. The systems analyzed vary in size and in complexity; from a small simple gravity flow storm sewer network terminating at a pump sump before discharging to a treatment faciltiy to a large complicated network having multi-basin systems linked by force mains with in- and off-line pumps, weirs with flap gates and temporary storage basins. Topographic conditions involved also vary greatly; from flat coastal plain to hilly terrains situated on escapements along the coast. It is not uncommon for a typical system to contain over 100 subcatchments, over 200 pipe segments which were interconnected at over 100 inlets/junctions and/or 5 or more pump stations. The analyses were complicated by the fact that many of these drainage systems do not have sufficient capacities to accommodate the design strom mandated by the regulatory agencies because they were designed and constructed prior to the imposition of the much more strigent regulatory requirements applicable at the present.

4 PROBLEMS ENCOUNTERED AND SOLVED

As stated previously, a number of problems were encountered in using both the Runoff and EXTRAN Blocks in the design of storm water management systems. In general, they can all be considered as "programming bugs". Some are related to model set-up, input limitations or inconsistencies between the simulation results presented in the SWMM printouts. Many are simply "applications difficulties" which have not been previously identified or well-documented. The following is a list of the problems encountered and solutions suggested.

4.1 Model Set-up and Simulation Problems

1) Number of Subcatchments Permissible at An Inlet

a) Problem Definition:

According to the user's manual for the Runoff Block, a single inlet may have up to a maximum of 5 subcatchments. However, when more than 4 subcatchments were assigned to the same inlet, an unreasonably large flow balance error would result.

b) Suggested Solution:

Limit the number of subcatchments to an inlet to a maximum of four and use dummy junctions/inlets for the rest of the catchments to connect them back to the inlet in question.

2) Flooding at Junctions

a) Problem Definition:

In EXTRAN, the flooding volume which overflows at a junction is lost from the system entirely if no special provision was made to recapture this overflow. This could lead to unacceptably large flow balance error in the flooded junction, thus resulting in incorrect simulation results. In reality, the overflow may eventually find its way back to the system at a catch basin downstream of the overflowing junction. In addition, if the area at the overflowing junction is depressed, the overflow may eventually re-enter the system through the very same junction where overflow occurs when the flow in the system subsides. (Note that the SWMM Manual also suggests that special treatment is required under these conditions.)

b) Suggested Solution:

Examine the topography of the area carefully to determine the drainage characteristics near the flooded junction. If the overflow is draining to a downstream catch basin, use a dummy conduit or an open channel connecting the junction to this downstream catch basin. The hydraulic characteristics of the dummy conduit or open channel should represent realistic field conditions leading to the downstream receiving junctions.

For flows re-entering the same flooded junction, one of the ways to capture the overflowed flood volume is to simulate that junction as a storage junction as suggested in the manual. The following is a suggested model set-up for using this approach: Below the ground level, the storage junction should have the same size as the actual junction. The cross-sectional area of the storage junction should expand rapidly above ground level to avoid an unrealistic buildup of head at this junction duringthe overflow period. The flood volume which flows into the storage junction can be calculated from the maximum water levels printed in the Junction Statistics Summary Table. It should be noted that only 20 storage junctions are allowed in EXTRAN, and the program will stop without explanation or warning once the limit of 20 is exceeded. This requirement must be observed in the development of the modified model set-up.

3) Weir/Flap Gate Option

a) Problem Definition:

The option of modeling a weir with a flap gate does not seem to perform properly. Significant reversed flows were encountered in the line immediately upstream of the flap gate during the period when the line downstream of the flap gate was receiving storm runoff pumped from other locations in the system.

b) Suggested Solution:

For the case in question, it appeared that the presence of the flap gate was the culprit. No reversed flows were encountered when the weir/flap gate combination was replaced by an equivalent conduit. The length of this equivalent conduit was chosen arbitrarily, but should be more than 50 ft, and the Manning's "n" for the equivalent conduit was assigned a value such that the head loss for the weir/flap gate combination at the time of the peak flow was preserved. The equivalent conduit was assigned an adverse slope with an outlet (conduit invert) elevation which is the same as the crest elevation of the weir. This modification in the model set-up preserved the hydraulic condition around the time of the peak flow when surge occurs, but it distorted the hydraulics at low flow conditions.

4) In-Line Pump Option

a) Problem Definition:

The use of the in-line pump option can sometimes cause severe instabilities in the solution. The causes or solutions for the unstable behavior seem to vary with the model set-up. For example:

i) The instability was observed when there is more than one conduit feeding into the in-line pump station, even though the User Manual implies that multiple pipes are permissible.

ii) An adverse slope or a relatively steep gradient in the conduit immediately upstream of the pump station could also cause instability problems.

b) Suggested Solutions:

i) This problem can be remedied by connecting these upstream conduits to a single dummy junction which then discharges to the pump sump via a single pipe.

ii) This could simply be a mathematical problem characteristic to many numerical solution schemes. A trial-and-error approach should be used by varying the slope or increasing the Manning's "n" value of the upstream conduit slightly until the instability goes away. Once this is achieved, check the slope and Manning's "n" to assure that with these adjustments, the hydraulics of the system is still reasonably preserved.

As suggested in the SWMM Manual, in some cases, very steep short pipes may be omitted from the simulation or aggregated with other pipes because they have negligible effects on routing since water is transported through them almost "instantaneously".

In addition, the SWMM manual suggests that a reduction in the time steps may help to eliminate the instability problem. We found it not always to be the case. As a matter of fact, based on our application experience, a reduction in time steps, in many cases, actually caused more severe instability in the system.

4.2 Output Problems

1) Flooding at Inlet Junctions

a) Problem Definition:

When flooding occurs at a junction which also serves as an "inlet" point to the storm sewer network, double accounting of the flow volumes has been observed occasionally. This problem leads to an erroneous increase in both the overall inflow and outflow volumes. In the Junction Inflow/Outflow Summary, these "problem" inlet junction appear twice in the list of junction inflows (they show up for the second time at the end of the normal listing). The inflow volumes at these inlet junctions, as shown in the Junction Inflow Summary, do not agree with those computed in the Runoff Block. It should be noted that not all the flooded inlet junctions are double counted. So far, no apparent trend is found as of when or which of these inlet junctions will be affected. The cause of the problem is not known and the impact on the EXTRAN results is not fully understood. (Version 4.2 has been "improved" to the point that it can detect this problem; i.e., a warning is printed. But the problem still exists in the algorithm.)

b) Suggested Solution:

Further research on the problem is needed. However, we have had success with the following modified model set-up in solving this problem: raising the ground elevation of the "problem" inlet junction high enough that flooding would not occur, inserting a new inlet junction downstream with the actual ground elevation, and connecting the two with a no-loss, large-diameter, minimum-length pipe. The end result of this modified model set-up is to create two junctions; an inlet junction to receive the runoff from a subcatchment and the storage junction where flooding may occur. It appears that SWMM cannot handle the case in which a junction is receiving storm runoff while, at the same time, is also overflowing.

2) Conduit Statistic Summary Table

a) Problem Definition:

The flow statistics presented in the Conduit Statistics Summary for the Runoff Block may differ when the order of the inlets specified for the printout is changed from one run to the other. This problem shows up randomly. However, it does not appear to affect the values of the inlet flow hydrographs which are eventually transferred to the subsequent blocks for computations via the interfaced files. Furthermore, the values indicated in the flow balance summary of the Runoff Block are consistent regardless of the order of the inlets requested for printout.

b) Suggested Solution:

The flow balance summary table for the Runoff Block should be checked for every run. Ignore the flow statistics given in the Conduit Statstics Summary and rely on the flow balance summary if inconsistencies between the two summaries are encountered.

3) Inflow/Outflow Summary and Junction Summary Tables

a) Problem Definition:

In a number of EXTRAN applications, inconsistences were found in the results presented in the Junction Inflow/Outflow Summary and those in the Junction Summary Statistics Table. In the

Junction Summary Statistics Table, some of the junctions were shown to have flooding (having positive flooding durations), but no corresponding flood volumes were reported for these junctions in the Junction Inflow/Outflow Summary. Since the flow balance calculations in EXTRAN are based on the values reported in the Junction Inflow/Outflow Summary Statistics, the aforementioned problem could lead to a fictitious increase in the flow balance error and subsequently errors in the output files.

b) Suggested Solution:

A special setup is required to capture the overflows at the junctions which have significant flooding as indicated in the Junction Summary Statistics Table, but are not reported in the Junction Inflow/Outflow Summary. The following modified model set-up is suggested: introduce at this junction, where flooding is indicated, a dummy conduit (with an adverse slope) which leads to a dummy junction with an invert elevation the same as (or slightly lower than) the ground elevation at the flooded junction. The size and the length of the dummy conduit are normally taken as the diameter and the height of the junction box, respectively. The dummy junction is then assigned as a free outfall. The total outflow at this artificial outfall, as reported in the Junction Summary statistics Table, will provide a good approximation of the overflow volume at the junction in question. However, unrealistically high hydraulic grade line at this flooded node may occur. This makes the nodal summary table unreliable. (Since the pipes would soon flow full, this pressure "spike" is very short-lived and does not affect the overall hydraulic characteristics of the system.)

4) Inlet Hydrograph Plots

a) Problem Definition:

In the Runoff Block, the plots of the computed inlet hydrographs were found to be inconsistent with the printout statistics when different simulation durations were specified. For example: For a 24-hour storm, the initial simulation indicated that the peak runoff rate for the system was 250 cfs occurring at about 8 hours when the model was allowed to go through the entire storm duration. However, if SWMM is rerun using a simulation period of 12 hours in order to save computer time, a much higher peak flow rate, equal to 400 cfs, was shown in the hydrograph plot while the flow statistics presented in the table still indicated a 250 cfs peak flow rate at about the 8th hour.

b) Suggested Solution:

Do not rely totally on the results presented in the hydrograph plots. The hydrograph plots should be checked carefully against the values given in the table of flow statistics to ensure that the peak runoff rates shown in the plots are consistent with the values in the table. Ignore the plot if there is a difference. In addition, if EXTRAN Block is also used, make sure that the simulation duration specified is sufficiently long that the residual flows in the system is minimal at the end of the simulation period.

5) Outflow Hydrograph Plots

a) Problem Definition:

Occasionally, the outflow hydrograph plot for a conduit was found not to be consistent with the values preented in the Conduit Summary Table for the period near the outflow peak. Although the peak flow was shown in the plot, zero flows were indicated straddling this peak for a fairly long duration. It seems to make no difference where in the sequence of plotting requests this particular outflow hydrograph was made, the plot will still be wrong.

b) Suggested Solution:

Ignore hydrograph plots with unreasonable shapes. Re-run SWMM to obtain time-history print-out and export the outflow hydrograph ordinates into a spreadsheet for graphic presentation.

6) Request for A Large Number of Output Data

a) Problem Definition:

Occasionally, a request of the printout of a large number of computed flow data (product of the number of printout time steps and the number of conduits) will cause the computer to hang up. This situation happened during a simulation of a 4-day storm with a 5-second time step, and the printout of the flow results of several conduits at a 2-minute interval was requested.

b) Suggested Solution:

Request the printout of only one single conduit at a time when a long duration storm is specified.

5. CONCLUDING REMARKS

A number of problems encountered in the application of SWMM have been presented together with suggested solutions which are pragmatic and practical, but not necessarily unique. They are not intended as the ultimate solutions to the application problems encountered which have their origins in the source code of the model. They should be considered as stop-gap measures which can be used until SWMM is further improved and more fully debugged. These suggested solutions, however, enable the engineers to derive sufficient and reasonable outputs from SWMM so that an assessment of the design adequacy of the storm water management system can be made. Despite these problems, SWMM remains a very useful and powerful model for the analysis and design of storm water management systems. As symtomatic with many comprehensive numerical models, application problems will always be encountered. There is no substitute for experience of the engineers who are using these models. The key words in any engineering application of a comprehensive numerical model are always: "know your system, know your model, and have application experience, have application experience and have application experience."

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MODEL CHOICE IN URBAN FLOOD FREQUENCY ANALYSIS

by

David F. Kibler[1]

INTRODUCTION

In modeling small urban drainage systems, the engineer has a great deal of latitude in both the choice of modelling approach as well as the scale at which he represents the system. This paper addresses the question of model choice by comparing various model applications to a 95-acre gaged catchment, known as the Lake Hills watershed, in Bellevue, Washington. The array of methods and models available for estimating design floods in the small urban catchment is significant, perhaps even overwhelming. As shown in Table 1, it includes: physically based continuous models such as EPA SWMM and USGS DR3M; single-event models such as the Penn State Runoff Model. In addition, there are microcomputer models based on the Rational formula, SCS TR-55 tabular hydrograph, and selected unit hydrograph procedures. Also, there are several regional flood frequency methods available for the small ungaged urban site. The main objective here was to compare alternative procedures for establishing flood frequency curves in a small urban watershed, recognizing that in the Lake Hills system, the author had access to two years of rainfall-runoff data collected by the USGS. A secondary issue, which became critical to the primary objective, was the best method of distributing hourly rainfall amounts in 5-minute intervals so as to utilize long-term hourly rainfall records in simulating the small urban catchment. Finally the author examines the effects of using various numerical solutions to the kinematic overland flow equation. This paper is adapted from an earlier paper by Kibler, Krallis and Jennings (1987).

CHARACTERISTICS OF THE LAKE HILLS WATERSHED

The Lake Hills watershed is a 95.3 acre (excluding roof tops draining to dry wells) residential area with single family homes built in the late 1950's. As indicated in Figure 1, it is located in the city of Bellevue, Washington, which lies along the eastern shore of Lake Washington just west of Seattle. The Lake Hills catchment is fairly steep with slopes of 14 percent. It is approximately 28 percent impervious, with Alderwood soils throughout the basin. The outfall of the separate storm drain system discharges to an open channel that flows into Kelsey Creek, which in turn discharges into Mercer Slough and then into Lake Washington. There is no detention storage in the present system, although surcharge and inadvertent ponding have been observed.

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Table 1. Common Urban Hydrology Models

Model Name	Agency Name	Precip. Excess Method	Hydrograph Synthesis Method	Channel or Sewer Routing Method	Detention Basin Option	Runoff Quality
TR 20	USDA Soil Conservation Service, Washington, DC	Curve number and SCS soil-cover complex	SCS dimensionless unit hydrograph (UH)	Attenuated kinematic wave	Storage-indication routing	No
HEC-1	Hydrologic Engrg. Center, US Army Corps. of Engrs. Davis, CA 95616	SCS curve number; Holtan eq.; Variable loss rate	SCS UH Snyder UH Clark UH Kinematic overland flow	Muskingum; Muskingum-Cunge; Kinematic	Storage-indication routing	No
HYMO	USDA/ARS; also Ontario Provincial Government, Ottawa	SCS curve number	SCS UH	Convex routing method	Storage-indication routing	No
ILLUDAS	Illinois State Water Survey, Champaign, IL 61820	Holtan eq.; SCS curve number	Time-area UH	Kinematic by Manning eq.	Storage-indication routing	Yes
SWMM	US EPA, Center for Exposure Assessment Athens, GA 30613	Horton; modified Horton; SCS curve number	Kinematic overland flow	Kinematic by Manning eq.; also by complete dynamic eqs. in EXTRAN	Storage-indication routing	Yes
PSRM/QUAL	Penn State U. Dept. Civil Engrg. University Park, PA 16801	Horton; SCS curve number	Kinematic overland flow	Muskingum	Storage-indication routing	Yes
DR3M	US Geological Survey Urban Studies Program Austin, TX	Phillips eq.	Kinematic overland flow	Kinematic by Manning eq.	Storage-indication routing	Yes
KINEROS	USDA Agricultural Research Service, Washington, D.C.	Smith and Parlange	Kinematic overland flow	Kinematic by Manning, Chezy or Darcy-Weisbach eq.	Storage-indication routing	Sediment only

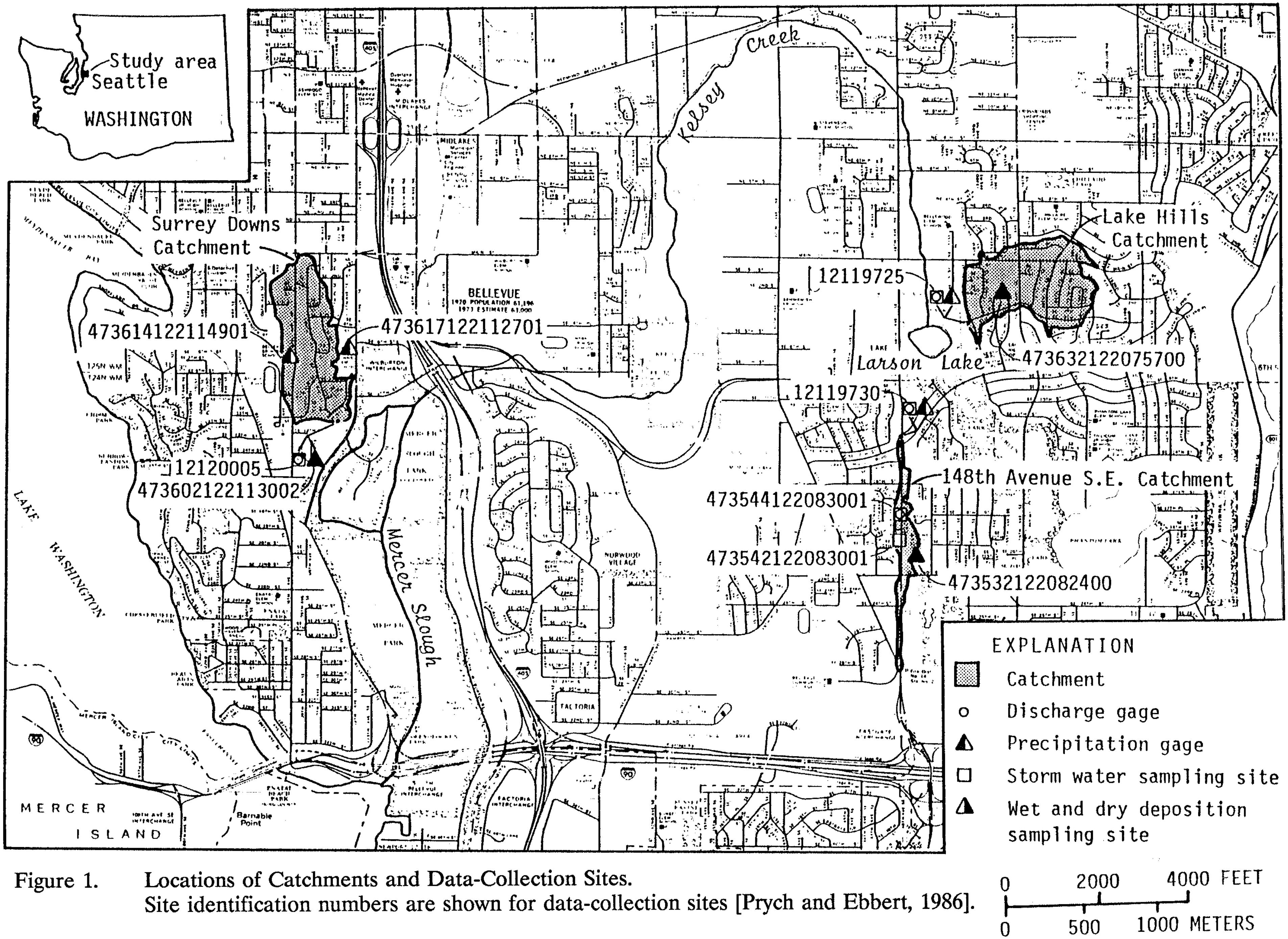


Figure 1. Locations of Catchments and Data-Collection Sites.
Site identification numbers are shown for data-collection sites [Prych and Ebbert, 1986].

The Lake Hills watershed, shown in detail in Figure 2, is one of three Bellevue sites monitored from late 1979 to early 1982 by the US Geological Survey, in cooperation with the City of Bellevue, as part of the US EPA Nationwide Runoff Program [see Prych and Ebbert, 1986]. As a result of the USGS monitoring program, two years of continuous rainfall and runoff date recorded at 5-minute intervals are available. This database is invaluable from the standpoint of model calibration. It is not usable of course to establish flood frequencies directly and forces the drainage engineer to make a choice of models.

MODELS AND METHODS APPLICABLE TO LAKE HILLS

As a first step in evaluating the possible choices of methods/models applicable to the Lake Hills catchment, the author assembled flood peak computations from 13 different procedures. In many cases these differ only in the choice of rainfall distribution which often is a governing factor in the runoff computation. This is especially true for the Lake Hills drainage system which has a 15-minute time of concentration and therefore is very sensitive to resolution in the rainfall hyetograph. Table 2 summarizes the six different rainfall distributions applied to the Lake Hills system.

Table 2. Summary of Rainfall Distributions Applied to Lake Hills Catchment (depth in inches) [Kibler et al., 1987]

Time min.	NOAA[a] Atlas	LH[b] No. 1	LH[c] No. 2	LH[d] No. 3	Yarnell[e] 5-min.	Yarnell[e] 10-min.
5	0.030	0.050	0.086	0.085	0.010	0.009
10	0.030	0.050	0.081	0.082	0.018	0.016
15	0.040	0.070	0.084	0.081	0.031	0.027
20	0.070	0.060	0.082	0.080	0.058	0.050
25	0.080	0.140	0.077	0.079	0.128	0.106
30	0.160	0.110	0.087	0.082	0.377	0.292
35	0.290	0.120	0.082	0.081	0.207	0.292
40	0.120	0.120	0.081	0.083	0.084	0.106
45	0.070	0.110	0.085	0.083	0.042	0.050
50	0.040	0.070	0.084	0.090	0.023	0.027
55	0.040	0.060	0.087	0.085	0.014	0.016
60	0.030	0.040	0.084	0.082	0.008	0.009
TOTALS	1.000	1.000	1.000	0.993	1.000	1.000

[a] From NOAA Vol. XI, 1973.

[b] From 15 Lake Hills storms. Distribution is by percent of total depth over most intense 60 minutes.

[c] From 2 years of Lake Hills data. Distribution is by count of occurrence vs. non-occurrence of rain in each 5-minute interval over all storms.

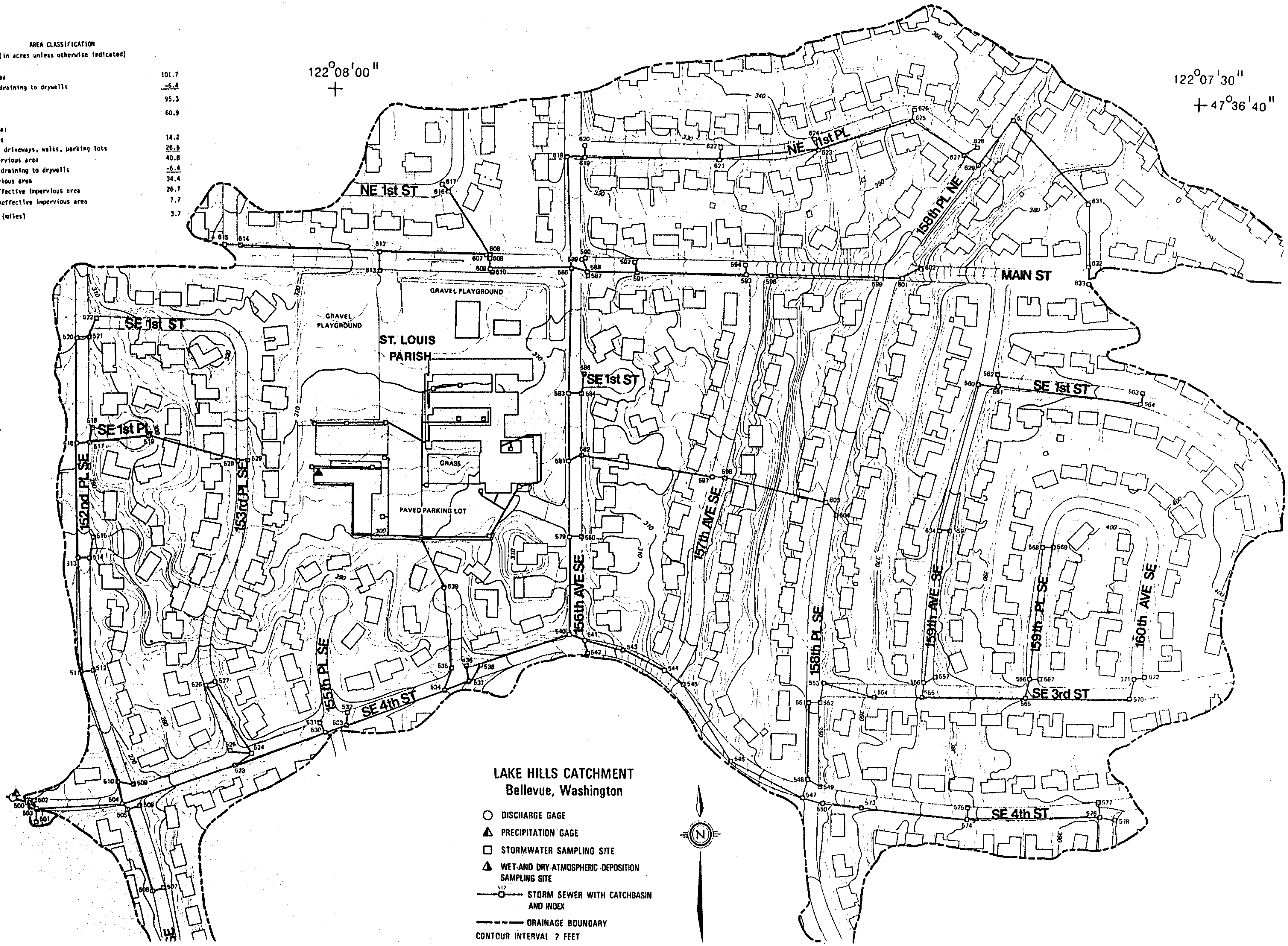
[d] From 2 years of continuous Lake Hills data. Distribution is by depth accumulation in each 5-minute interval over all storms.

[e] Yarnell 5 and 10-minute distributions use the standard form: $i = a/(t+15)^b$, where $a = 84 + 20e^{-p}$, $b = 0.2316 \ln (a/p)$, p = 60-minute rainfall depth for specified return period.

Figure 2. The Lake Hills Watershed [Prych and Ebbert, 1986].

AREA CLASSIFICATION
(in acres unless otherwise indicated)

Gross total area	101.7
Roofs draining to drywells	-6.4
Net total area	95.3
Pervious area	60.9
Impervious area:	
Streets	14.2
Roofs, driveways, walks, parking lots	26.6
Total impervious area	40.8
Roofs draining to drywells	-6.4
Net impervious area	34.4
Net effective impervious area	26.7
Net ineffective impervious area	7.7
Street length (miles)	3.7



As indicated in Table 2, the Yarnell distributions place between 29 and 37 percent of the total 60-minute rain in the most intense 5-10 minute period and therefore tend to represent sharp short duration events. The LH2 and LH3 distributions, on the other hand, provide a rather uniform distribution with less than 9 percent of the total 60-minute depth falling in any 5-minute period. The NOAA Atlas [1973] and LH1 distributions represent intermediate intensities and have been selected for use in the runoff peak computations which follow. They are shown schematically in Figure 3.

The runoff peak methods are summarized in Table 3. They range in complexity from Rational formula to discrete storm modelling by the Penn State Runoff Model (PSRM). It is noted that work on continuous modelling by DR3M is incomplete as of this writing and is not reported here.

Table 3. Summary of Flood Peak Methods in Lake Hills Catchment [Kibler et al., 1987]

Method No. and Name	Rainfall Distribution	Key Parameters and References
1) Rational	NOAA Atlas Vol. XI	A=95.2 acres; c=0.35; T_c=15 min
2) Rational	LH1	Same as (1)
3) Rational	Yarnell 5-min.	Same as (1)
4) USGS 7-parameter urban flood regr.	N/A	A=0.149; SL=70; R12=0.23; ST=0; BDF=10; P=45 RQ_T from Cummans et al., [1975]. See Sauer et al. [1983].
5) USGS 3-parameter urban flood regr.	N/A	Same as (4)
6) USGS 7-parameter urban flood regr.	N/A	Same as (4), except RQ_T is from area ratio applied to Mercer Creek.
7) USGS 3-parameter urban flood regr.	N/A	Same as (6)
8) SCS TR-55	24-hour Type IA	CN=80, T_c=.25 hour See SCS TR-55 [1986]
9) SCS TR-20	24-hour Type IA	CN=80
10) PSRM	NOAA Atlas Vol. XI	14 sub-areas; PSRM calibrated on 15 events. See G. Aron [1987] for PSRM ref.
11) PSRM	LH1	Same as (10)
12) USGS regression developed from Bellevue sites	MAXR15 from NOAA Atlas Vol. XI	MAXR15=max 15-min. rainfall rate, in/hr. Reference is Prych and Ebbert [1986].
13) USGS regression developed from Bellevue sites	MAXR15 from LH1	Same as (12)

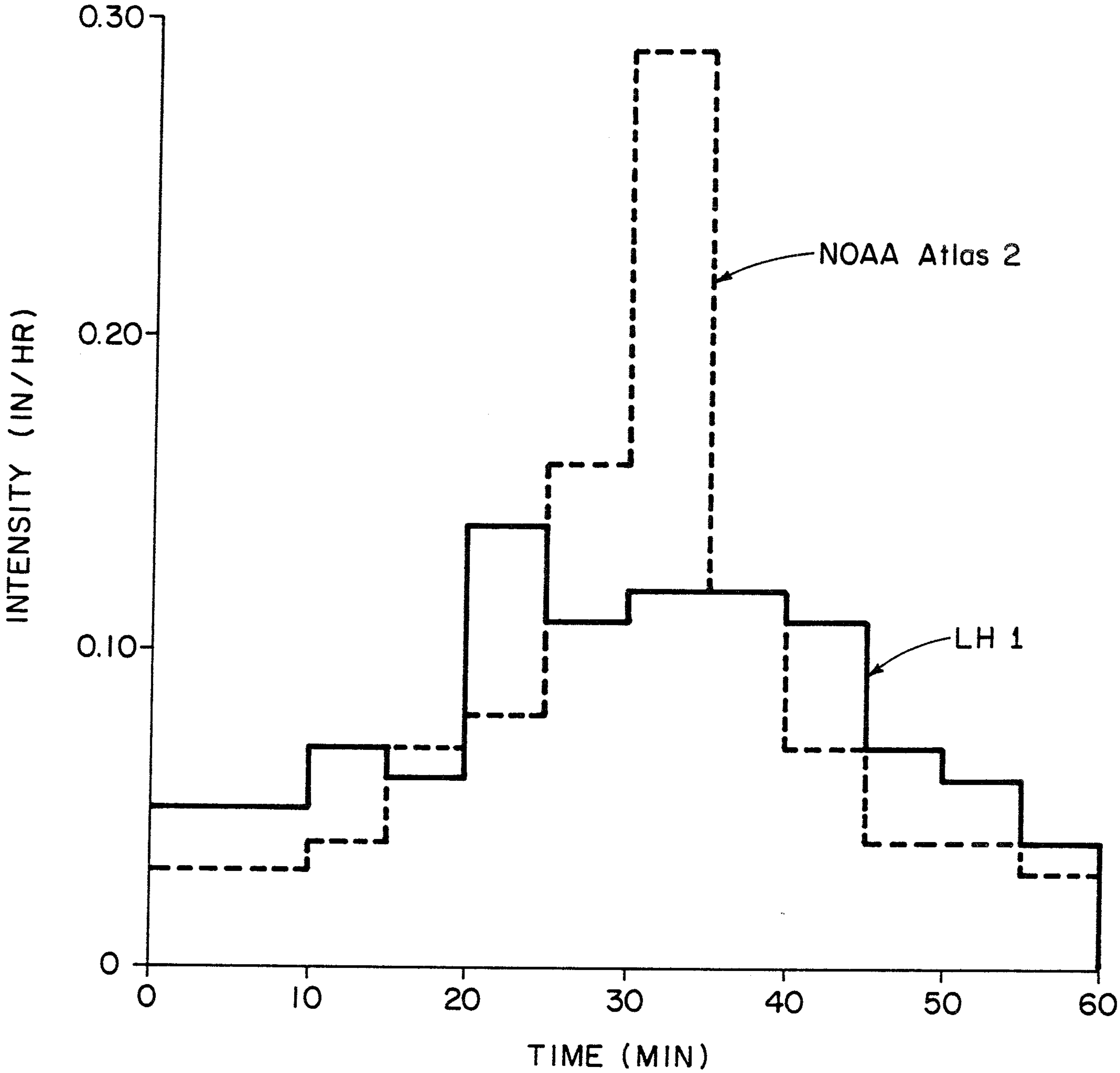


Figure 3. Lake Hills No. 1 (LH1) and NOAA Atlas 2 60 Minute Rainfall Distributions.

CALIBRATION OF PSRM AND TR-20

Prior to running PSRM and TR-20 on the six synthetic design storms, it was necessary to calibrate the two rainfall runoff models against actual storm events measured on the Lake Hills catchment. The results of PSRM calibration are shown in Figures 4 and 5. For the storm of 8/17/80 only hourly rainfall was available. The plot of Figure 6 illustrates the impact on the Lake Hills outflow hydrograph of employing both the NOAA Atlas 2 and LH1 rainfall distribution. It is clear in Figure 6 that the NOAA Atlas distribution tends toward over-estimation of the discharge peak, while the LH1 distribution tends toward under-estimation. This pattern shows up very clearly in the summary plot in Figure 5.

SUMMARY OF FLOOD PEAK RESULTS

The results of all flood peak computations for the Lake Hills catchment are shown in Table 4. The range in discharge peak for any given return period is substantial, but not unexpected. Several interesting patterns emerge from Table 4. First, the results indicate a rough agreement between runoff methods when the same rainfall distribution is involved. For example, methods (1), (10), and (12) are similar as are methods (2), (11), and (13) indicating a very strong dependence on the rainfall distribution method employed. The NOAA Atlas distributions produce higher flood peaks than LH1 as expected, but less than the Yarnell 5-minute distributions. Interestingly, the USGS nationwide regression equations in methods (4) through (7) produce lower flood peaks than the USGS regression equations developed from the Bellevue site data in methods (12) and (13). For reasons related to the runoff curve number and initial abstraction, the two SCS methods (8) and (9) produce flood peaks that are low at the 2- and 5-year levels, but close to the USGS regression results at return periods greater than 10 years. The PSRM results are quite similar to those produced by the Rational formula and the USGS Bellevue regression equations.

Table 4. Summary of the Lake Hills Flood Peak Computations [Kibler et al., 1987]

Method No. and Name	Q2 cfs	Q3 cfs	Q10 cfs	Q25 cfs	Q50 cfs	Q100 cfs
1) Rational, NOAA	33	42	49	58	65	72
2) Rational, LH1	22	28	33	39	44	48
3) Rational, Y5	50	58	65	70	77	83
4) USGS-7	13	15	20	21	24	26
5) USGS-3	13	15	19	20	23	24
6) USGS-7 with area ratio	15	20	24	27	31	33
7) USGS-3 with area ratio	18	23	25	28	31	34
8) TR-55	7	14	19	28	34	39
9) TR-20	7	14	19	28	34	40
10) PSRM, NOAA	26	35	42	50	59	67
11) PSRM, LH1	17	23	28	35	40	46
12) USGS Eqn, NOAA	27	37	45	55	65	74
13) USGS Eqn, LH1	17	23	28	34	40	46

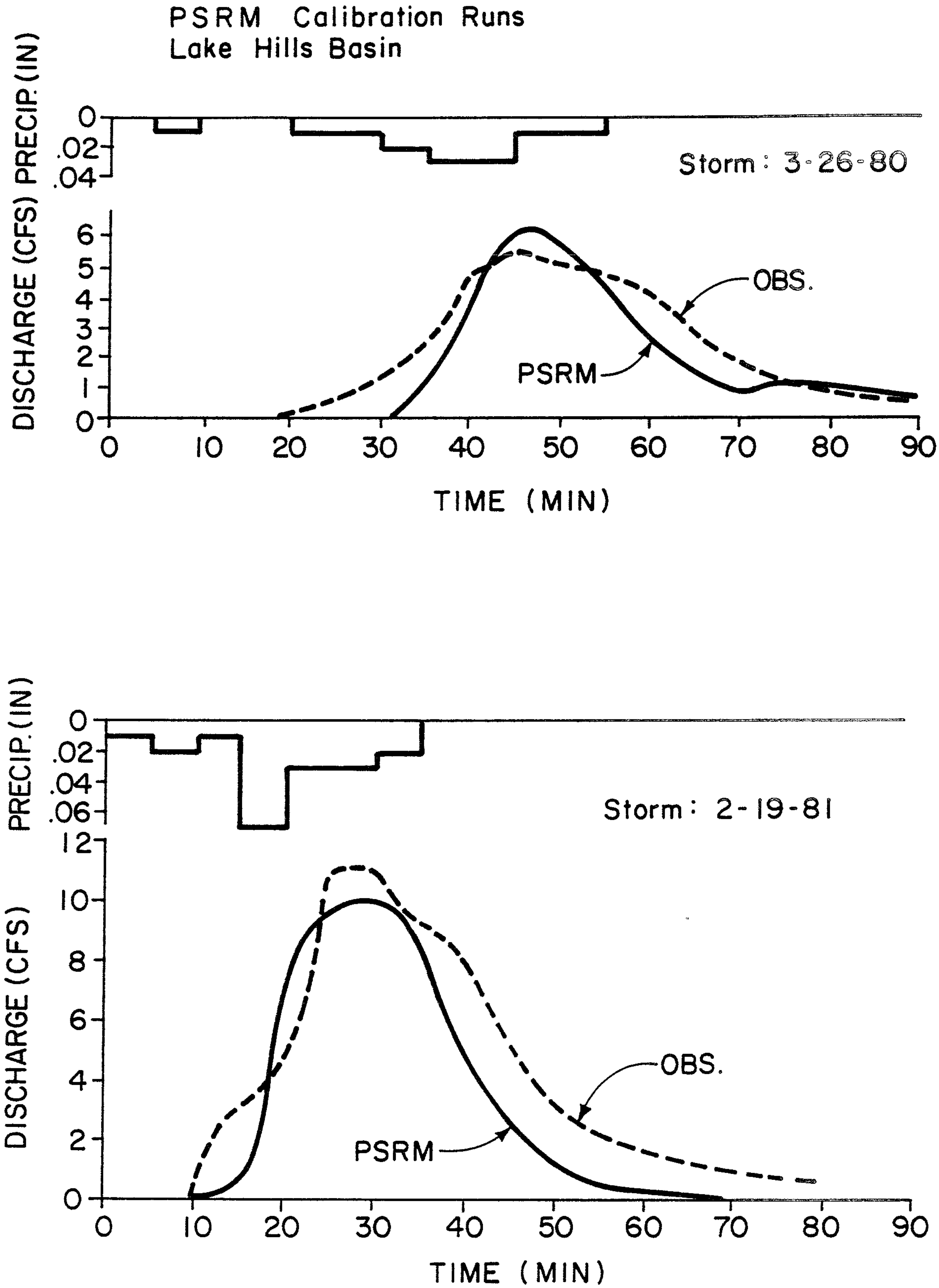


Figure 4. PSRM Calibration Runs for Lake Hills Basin.

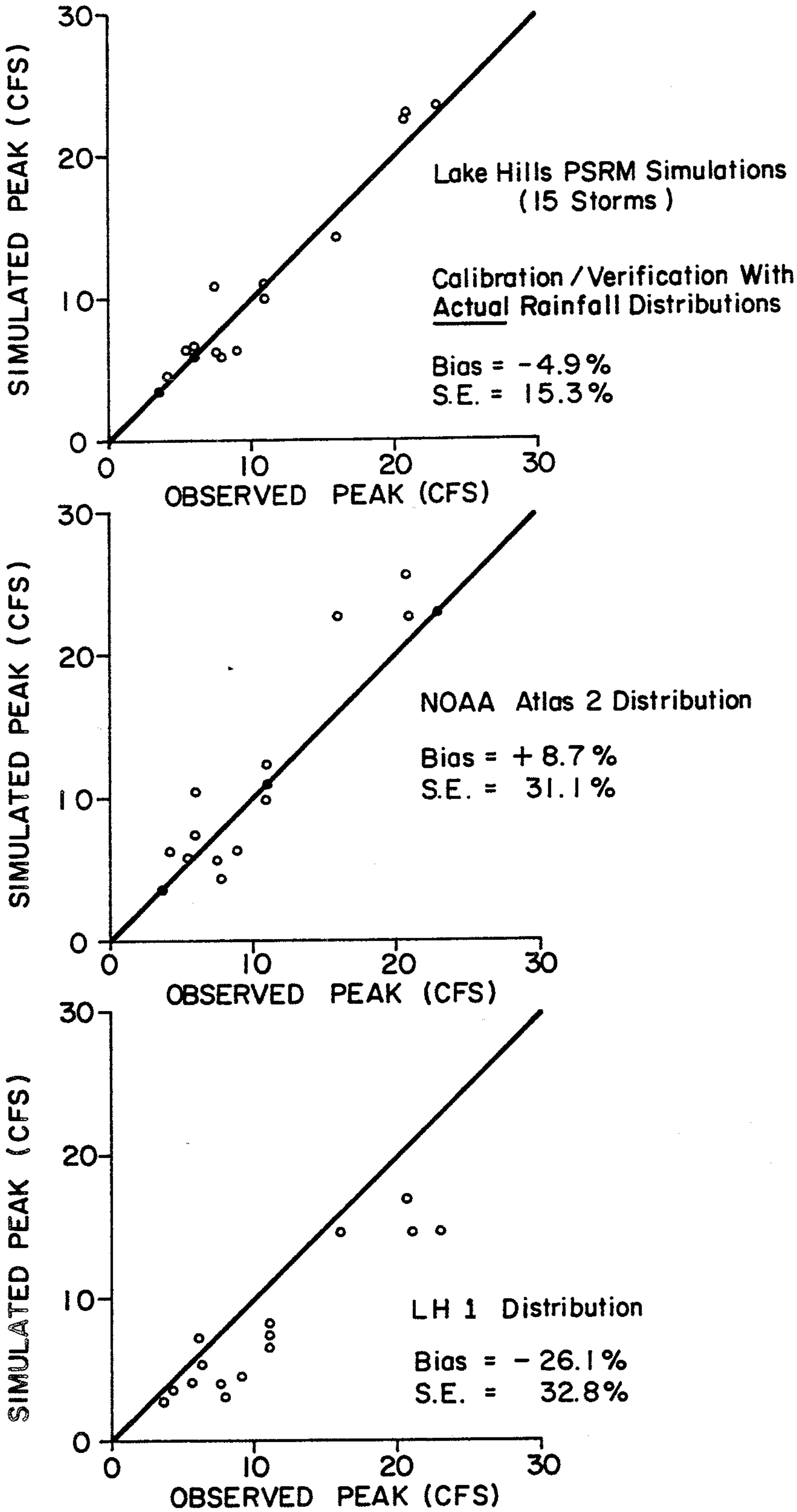


Figure 5. PSRM Calibration/Verification Results and Simulated Peaks Generated by PSRM Using NOAA Atlas 2 and LH1 Rainfall Distributions.

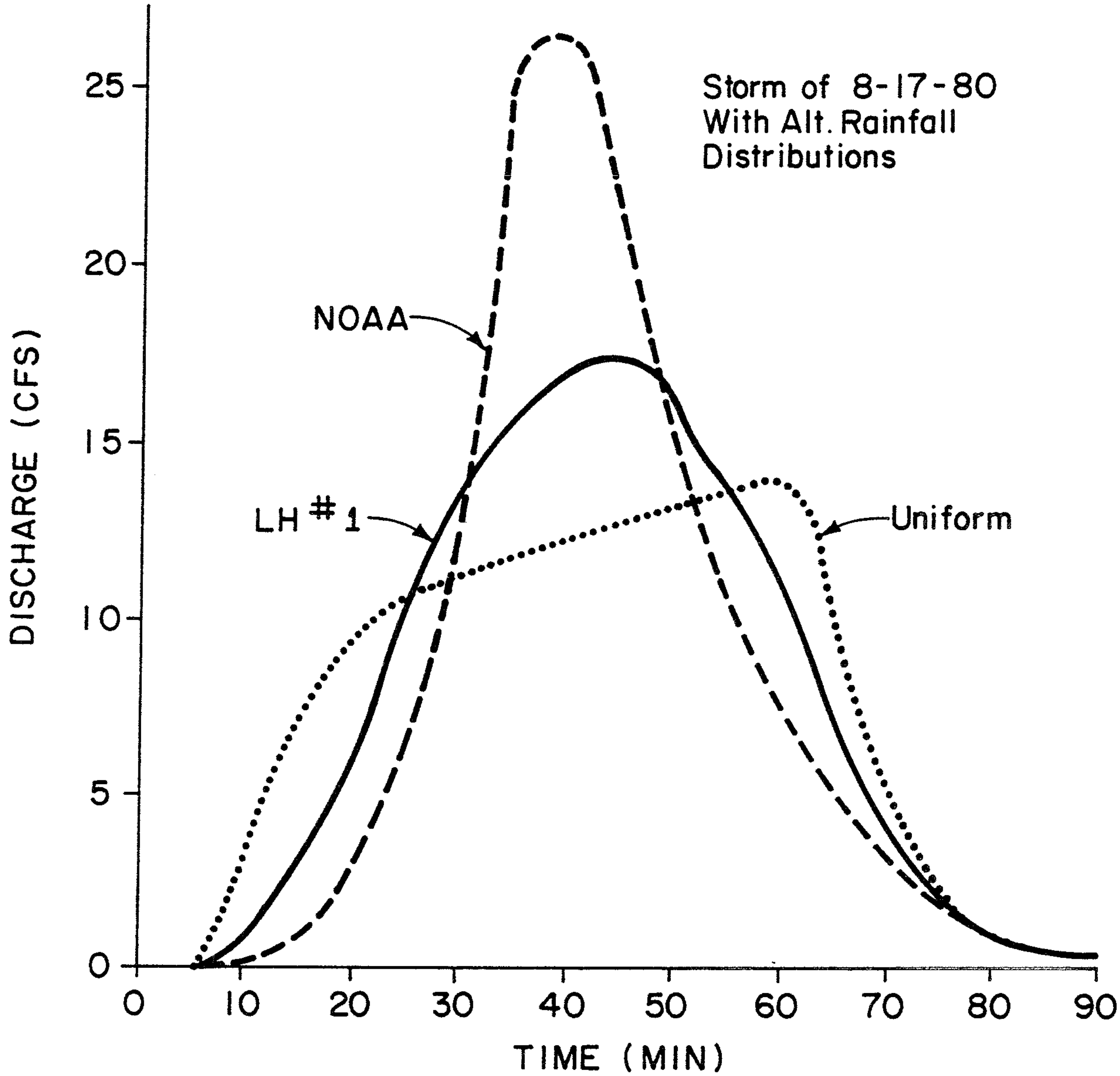


Figure 6. PSRM Application to Storm of 8-17-80 Assuming NOAA Atlas 2 and LH1 Rainfall Distributions.

In the absence of long-term flow records at the Lake Hills site, no firm conclusion on choice of method can be made from the results presented here. It is very clear, however, that selection of rainfall distribution is critical to this choice and may even be the governing factor. What remains to be done is to apply the continuous distributed USGS DR3M model to synthesize a long-term runoff trace so that a bench-mark frequency analysis can be made. Since the nearest available long-term rainfall record comes from Seattle in hourly intervals, the approach to segmental rainfall distribution remains crucial to the analysis of urban floods on small responsive systems such as the Lake Hills watershed.

NUMERICAL SOLUTION SCHEME FOR KINEMATIC OVERLAND ROUTING

Because the kinematic overland flow equations play such a prominent role in the urban runoff models in Table 1, the author has investigated three numerical schemes for solving the one-dimensional kinematic overland flow equations. They are compared by application to a basic building block in urban hydrology -- the asphalt surface. The three methods of solution are: characteristic method; Newton-Raphson as used in EPA SWMM; and a fixed step version of the explicit forward-in-time finite difference scheme used in HEC-1 [1987]. In this instance, the asphalt surface is 400 feet long, slopes at 0.0025 feet/foot, and has a Manning n of 0.025. The surface is assumed to be totally impervious and is a modified version of the problem developed by Overland and Meadows [1976]. The rainfall distribution and the characteristic solution method are shown in Figure 7. Note that the characteristic method applied to this simple surface provides a quasi-analytic solution which can be used as a bench-mark for comparing the other methods.

The outflow hydrograph obtained by tracking characteristics across the 400-foot surface is shown in Figure 7. This outflow hydrograph is also shown in Figure 8, together with the outflow hydrographs generated by applying the Newton scheme in EPA SWMM and the FD method in HEC-1. There is a significant loss in the peak flow region caused by the very long distance step of 400 feet in the latter two methods. This is improved dramatically in the case of the HEC-1 solution by shortening the distance step to 50 feet, and in fact the 1987 and 1990 versions of HEC-1 operate with a variable distance step to converge on the characteristic solution. However, the distance step in the overland flow calculations in EPA SWMM is set equal to some overland flow length and there is no internal sub-interval. Consequently, a large narrow subarea having a long overland flow path can have a reduced outflow hydrograph when computed by this procedure. Corrections for this sluggish response in the SWMM often take the form of artificially low Manning's roughness, or shortened overland flow distances.

EFFECT OF FRICTION RELATION

The choice of friction relation can also have a significant impact on the runoff hydrograph. Figure 8 also shows the laminar outflow hydrograph obtained by characteristic method for the 400-foot asphalt surface described earlier. Clearly, there is a difference and while it might be difficult to make a strong case for laminar flow under the influence of pelting rain on an asphalt surface, it is certainly conceivable that laminar conditions prevail over much of the early rising and late falling sides of the outflow hydrograph. The choice of a single fixed Manning n value for the entire hydrograph is also a compromise which needs to be investigated.

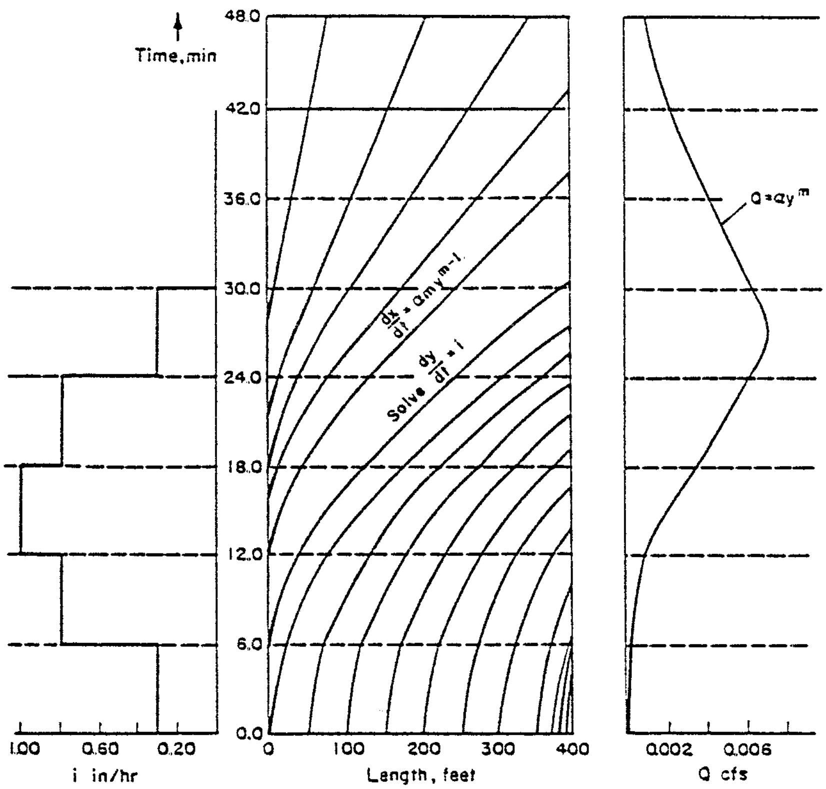


Figure 7. Characteristic Solution for Outflow Hydrograph [Kibler, 1991].

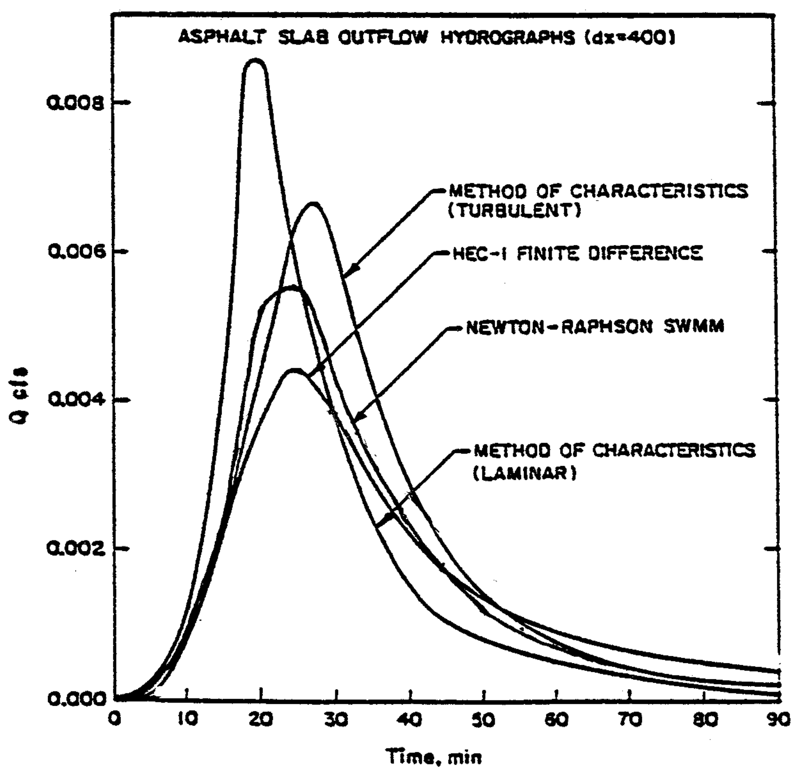


Figure 8. Outflow Hydrographs by Different Methods [Kibler, 1991].

SUMMARY

It is clear that these basic assumptions of rainfall distributions and numerical solution method play a major role in determining the urban runoff hydrograph generated by commonly used models. In our efforts to develop comprehensive stormwater models, we as engineers fail to distinguish between the effects of these fundamental model assumptions on one hand and the effects of such factors as impervious fraction and land use on the other. Consequently, the true hydrologic impacts of physical watershed attributes are usually masked by numerical-hydraulic artifacts of a particular runoff model. This would seem to have major significance in view of the wide-spread use of hydrologic models in stormwater management decisions involving restricted land-use and design of facilities for preserving pre-development flow peaks. Clearly, a set of model bench-marks for the urban drainage field is needed so that these hydrologic-physical effects can be separated from the artificial impacts of model choice.

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TOWARDS INTEGRATED URBAN WATER SYSTEM MANAGEMENT

by

James P. Heaney[1]

INTRODUCTION

The catalyst for this workshop is a new Corps of Engineers initiative on Urban Hydrology Methods/Models. The scope of this activity is to look at (1) the surface system; (2) the subsurface storm drainage system; and (3) the interaction between these two systems. The primary mandate of the Corps of Engineers has been to evaluate flooding and navigation. Other traditional purposes such as hydropower generation, water supply, recreation, and in-stream flow needs may be evaluated depending upon which legislative mandate is appropriate. In more recent years, the Corps of Engineers have become involved in wetland systems as part of the Section 404 permitting process and in evaluating proposed environmental restoration projects such as the Kissimmee River in Florida.

This paper presents my personal view of alternative future directions for the field of urban hydrology, with emphasis on future research. Three alternative futures, based on available research funding, are explored:

1. Continue the status quo in urban hydrology, i.e., very low level of research funding. This status has existed for the past decade.

2. Significant expansion of agency supported research within the missions of the individual agencies.

3. Creation of a new interdisciplinary funding agency with authority to support research initiatives in urban environmental infrastructure systems.

At present, we are locked in alternative future 1 which offers little hope for innovations in urban hydrology in particular much less showing how taking a more holistic approach can lead to major improvements in our ability to analyze and manage increasingly complex urban environmental infrastructure systems.

A review of the field of "urban hydrology" during the past 30 years is presented in order to provide an historical context from which we can evaluate the relative desirability of the three alternative futures described above. The paper concludes with recommended action items for moving us out of our current dormant state.

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URBAN WATER RESOURCES RESEARCH DURING THE 1960's and 70's

The mid 1960's were a period of great change in attitudes in the water resource field. The 1964 Water Resources Research Act established the Office of Water Resources Research (OWRR) with a mission of promoting interdisciplinary research because the individual federal agencies were only looking at their mandated piece of the total water system. Also, the 1965 Water Resources Planning Act established river basin commissions to better integrate water resources planning across federal agencies. Thus, recognition of the lack of an integrated planning and research agenda led to the creation of university based programs to fill this need. However, Federal agency programs remained divided by type of water function and whether they looked at water quality or quantity. Similarly, university research groups were divided into water resources (hydrology and hydraulics) and sanitary engineering.

Because of the vision of our professional leaders, young researchers in the 1960's were blessed with a relative abundance of support for working on a wide variety of urban water resources and environmental engineering problems which did not have to be restricted in scope to those problems falling within the purview of the Federal agencies. Our leaders in the 1960's also formed an **ASCE Urban Hydrology Research Council** to take a systems view of urban water problems. However, once the program got underway, they realized that "Hydrology" was too restrictive a word and they renamed it to be the **ASCE Urban Water Resources Research Council.** By replacing "Hydrology" with "Water Resources" they were able to include systems analysts, social and behavioral sciences, and environmental scientists who incorporated water quality considerations. Great strides were made in urban water and environmental management during the 1960's and 1970's because of generous federal support for research, a national mood to look at revitalizing our cities and restoring the environment, and the concomitant emergence of the systems approach and essential computer hardware and software.

The leadership in urban water resources during the early years can be traced to the ASCE Urban Water Resources Research Council headed by M.B. McPherson who was a mentor to many of us. He was ably guided by a steering committee of prominent water resources engineers:

W. C. Ackerman (Illinois State Water Survey)
J.C. Geyer (Johns Hopkins University)
C.F. Izzard (Bureau of Public Roads)
S.W. Jens (Consulting Engineer)
D.E. Jones, Jr. (Federal Housing Administration)

With funding from OWRR and NSF, this group sponsored research conferences and numerous research projects dealing with a wide variety of urban water resources issues. The early results are published in McPherson et al. (1968). As you will see from the discussion below, this group and the research team did some innovative thinking with regard to urban water management which is still applicable today.

Water within the Urban System. Jones (1971) outlines the systems approach to urban water resources as shown in Figure 1. He makes several key points:

1. The urban water system is a subsystem of the total urban infrastructure system which needs to be evaluated in an integrated manner.

2. Suboptimization can easily result if we only look at each individual subsystem.

3. Information driven approaches are essential to avoid making unwarranted generalizations based upon simulation and/or optimization models.

The 1968 Urban Water Research study correctly pointed out the need for an integrated look at the problem (McPherson et al. 1968):

"A single aspect research approach is totally inadequate and, indeed, is entirely inappropriate, for resolving multi-aspect problems. The former simplistic approach of regarding a unit of water as a fixed entity, such as storm water, must be abandoned for that same unit at a different point in time will be categorized as water supply, recreation, esthetics, etc., perhaps several times before leaving a given metropolis".

The ASCE research group defined urban water resources to consist of:

a. Urban water uses:
  Water supply (domestic, commercial, agricultural and for fire protection;
  Conveyance of wastes (from buildings and industries);
  Dilution of combined and storm sewerage system effluents and treatment plant effluents (by receiving bodies of water);
  Water-oriented recreation (and fish management);
  Esthetics (such as landscaped creeks and ponds in parks and parkways);
  Transportation (commercial and recreational); and
  Power generation.

b. Protection of urban areas from flooding:
  Removal of surface water at the source
  Conveyance of upstream surface water through the area.
  Barricading banks, detaining or expressing flow natural streams to mitigate spillover in occupied zones of flood plain;
  Flood proofing of structures.

c. Manipulation of urban water:
  Groundwater recharge;
  Recycling of water.

d. Pollution abatement in urban areas;
  Conveyance of sanitary sewage and industrial wastes in separate sewerage systems;
  Interception of sanitary sewage and industrial wastes;
  Interception and treatment of storm sewer discharges or combined sewer overflows;
  Reinforcing waste assimilative capacity of receiving water bodies;
  Treatment of sanitary wastes at point of origin.

THE SYSTEMS APPROACH
TO URBAN WATER RESOURCES

1. THE URBAN COMPLEX IS THE BASIC SYSTEM:

   * The urban complex is people and serves people.

2. THE URBAN WATER RESOURCE IS A SUBSYSTEM IN THE BASIC URBAN SYSTEM:

   * To address the urban water resource as an independent system, even for convenience, may lead to dangerously narrow conclusions.

3. TRADITIONAL THINKING OF WATER SUPPLY, DISTRIBUTION, SEWAGE, FLOOD CONTROL AND RECREATION AS SUB-ORDERS MAY BE INAPPROPRIATE:

   * These are interdependent service functions.

   * Perhaps the following breakdown might prove better:

     - The complete water cycle
     - The environment, including people.
     - The ecology, including people (if separable from environment).
     - Public and private economics.
     - Management.

4. GENERALIZATIONS AT THE SUB-SUB-SUBSYSTEMS LEVEL COULD DEFEAT THE OBJECTIVES OF THE SYSTEMS APPROACH:

   * The progress of science is measured by development of details. Research contributions typically come from multiple minute steps--not from giant strides forward.

   * Rewarding concepts, innovations and improvements will originate essentially at the sub-sub-subsystem level.

5. TEMPTATIONS TO GENERALIZE, TO INTERRELATE ONLY WITHIN THE FINITE CAPABILITY OF A MACHINE, AND TO IGNORE "INTANGIBLE" RELATIONSHIPS LACKING HARD DATA, MUST BE AVOIDED:

   * Neither a model nor a machine can think.

   * Man cannot excuse his failure to think.

Figure 1. Early view of the systems approach to urban water management (Jones 1971)

e. Interfacial public services;
    - Snowstorm and rainstorm traffic routing;
    - Street cleaning scheduling;
    - Snow removal strategies;
    - Lawn irrigation conservation; and
    - Air pollution control.

The review of the integrated approach to urban water systems which was in vogue in the late 1960's and 1970's indicates that these researchers had scoped the problem out very well.

Systems Approach to Urban Water Management. McPherson (1973) argued that developing an urban water budget was an essential first step in using a systems approach as shown in Figure 2. Concurrently, researchers at Resources for the Future were stressing the use of a materials balance approach for inventorying and evaluating the generation and disposal of "residuals" or the quality constituents associated with transport in the air or water as shown in Figure 3 (Kneese, Ayres, and d'Arge. 1970). A more recent summary of the residuals management approach and a comprehensive catalog of models is presented in Basta and Bower (1982).

EPA Urban Water Research Program. The 1970 decade was the period of most rapid advances in urban water resources research particularly in the area of urban stormwater quantity and quality management (Heaney 1986). The U.S. Environmental Protection Agency, through its Storm and Combined Sewer Program, headed by Richard Field, sponsored $60-70 million in research. This research program included major technology transfer activities and demonstration projects. Virtually, all of the current innovations in the field can be traced to this program. Concurrent with the Storm and Combined Sewer Research Program, EPA sponsored $250 million in areawide wastewater management studies in virtually all metropolitan areas in the United States. These so-called 208 studies provided the first major testing ground for the use of urban stormwater models such as SWMM and STORM. A major contribution of these studies was that they consistently showed that control of wet-weather pollution should be initiated before committing large amounts of funding for tertiary treatment of sanitary sewage, which was the recommended approach in the early 1970's. A strong criticism of the 208 studies was that they did not collect adequate data to characterize the nature of urban runoff and to evaluate the expected effectiveness of various controls. In order to address this need, EPA initiated a $30 million Nationwide Urban Runoff Program (NURP) which was directed by Dennis Athayde. NURP funded 30 studies across the country during 1979-1983 with heavy emphasis on data collection (U.S. Environmental Protection Agency 1983). The results of NURP provided the first national database on urban runoff quality. Also, the vexing question of receiving water impacts was addressed. These data-intensive studies have had a lasting impact on the field.

The $100 million of EPA sponsored research was designed to provide more cost-effective decision making for an expected national investment which is now estimated to be about $40 billion. Thus, the percentage investment in R&D for this activity is about 0.25 %. As a general guideline, a reasonable investment in R&D for newer technologies is 1 to 5% Kim et al. 1993). Thus, the R&D investment in urban stormwater quality management was on the low side even during this period.

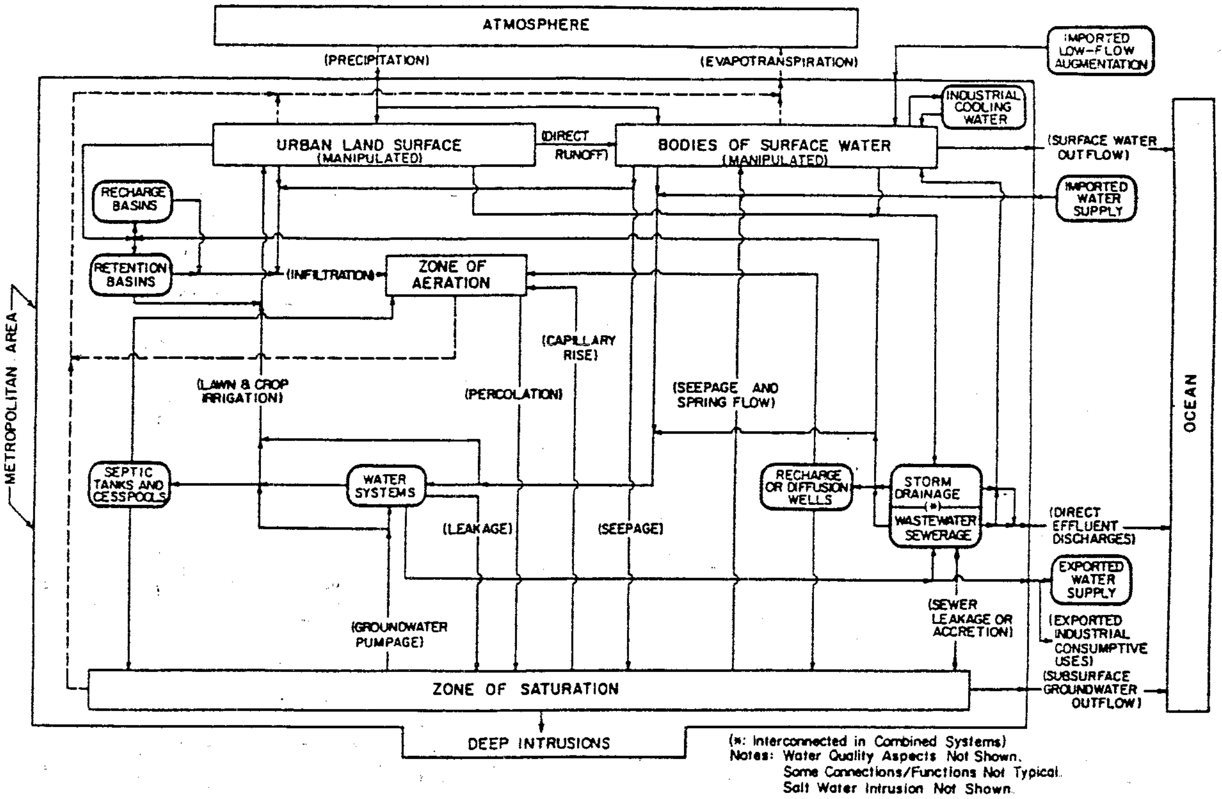


Figure 2. Water budget for urban system (McPherson 1973)

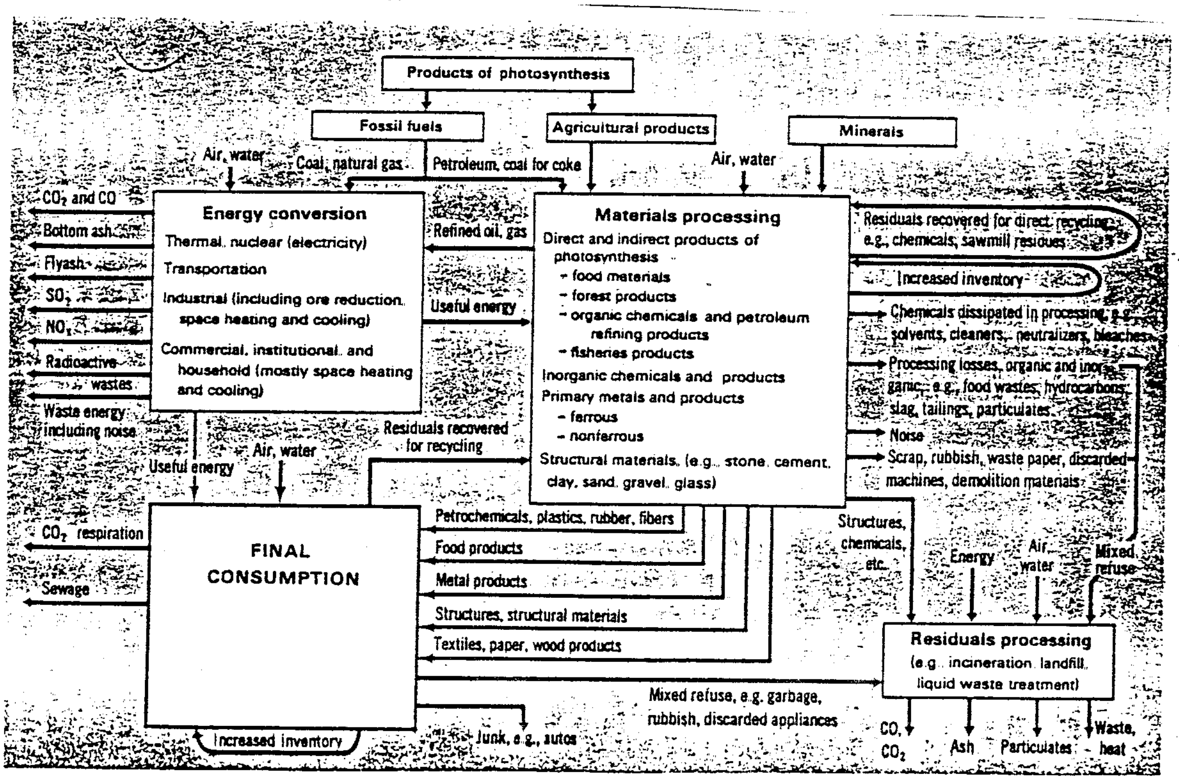


Figure 3. Schematic description of materials flows (Kneese et al. 1970).

Corps of Engineers Urban Water Management Activities. HEC was founded in 1964 through the efforts of A. L. Cochran, then Chief of the Hydraulics and Hydrology Branch, Office of the Chief of Engineers, U.S. Army Corps of Engineers, Washington, D.C. and Leo R. Beard, the Corps' leading expert in statistical hydrology who was in charge of the Reservoir Regulation Section in the Sacramento District (Feldman 1981). The purpose of HEC was to provide the essential link between the basic research of academics and the practical needs of the Corps field offices.

The closest we have come to an integrated look at urban water systems was the Corps of Engineers Urban Studies Program which existed in the 1970's at the same that EPA's 208 Areawide Planning Program was active. The scope of the Corps' Urban Studies Program and methodology are presented in the Federal Register (1974). The purpose of the Corps' Urban Studies Program was to provide an "integrated approach to water resources management." In accordance with standard Corps of Engineers operating policies, one of the proposed plans would be the one that maximized national economic development (NED) and one plan which emphasizes environmental quality (EQ). The following functional components were included in this program:

1. Urban Flood Control;
2. Flood Plain Management;
3. Municipal and Industrial Water Supply;
4. Wastewater Management;
5. Bank and Channel Stabilization;
6. Lake, Estuarine, and Ocean Restoration and Protection;
7. Recreation Management at Corps' Civil Works Projects;
8. Regional Harbors and Waterways.

These studies were conducted in numerous cities around the United States. As expected, there were concerns regarding the extent to which the Corps should deal with water quality issues that could alternatively be handled by the U.S. Environmental Protection Agency.

Urban Models in General. Similar integrated systems thinking prevailed in other areas of urban infrastructure, especially transportation and land use planning. Large models were developed and tested in numerous cities. However, Lee (1973) wrote an influential paper in the Journal of the American Institute of Planners titled "Requiem for Large-Scale Models" which was highly critical of the potential value of such models. Few urban models were developed after the early 1970's but we are now witnessing a renaissance in the development and use of these models. A recent symposium discussed the change in attitudes since Lee's pessimistic 1973 paper appeared. Wegener (1994) summarizes the state of the art:

"Twenty years after Lee's Requiem for Large-Scale Models', the urban modeling field is again full of life. There exist a dozen or so operational urban models of varying degrees of comprehensiveness and sophistication, which have been and are being applied to real-life metropolitan regions for purposes of research and/or policy analysis."

The resurgence of interest in urban planning models in the 1990's is partially due to the renewed recognition of the need to link transportation-land use models to urban environmental systems models.

URBAN WATER RESOURCES RESEARCH IN THE 1980'S

In stark contrast to the excitement and innovations of the 1960's and 1970's, drastic cuts were made in the early 1980's in funding of all aspects of urban water systems. The EPA Storm and Combined Sewer Research program was gutted. The EPA NURP Program ended in 1982. The Corps of Engineers Urban Studies initiative ended. The Office of Water Research and Technology (OWRT), the successor to OWRR, was eliminated in 1982 and the remnants of the program were transferred to the U.S. Geological Survey where the small remaining program has been under constant threat of elimination during the past decade. Thus, we have lost, not only more than a decade of research results, but also we have not educated the next generations of promising young researchers who have an appreciation of urban water resources systems.

SHOULD WE REVITALIZE URBAN WATER RESEARCH ACTIVITIES?

One could argue that it is unnecessary for the United States to be the leaders in urban water resources research and that it is acceptable to let others take the lead. This attitude can be countered by inspection of current EPA guidelines for urban stormwater management (U.S. Environmental Protection Agency 1993). While EPA no longer supports significant research in urban stormwater quality systems, it is enforcing regulations that require cities to solve these problems with only local resources. The recent National Conference on Combined Sewer Overflows held in Louisville in July 1994 showed the results of 14 years of no research (Water Environment Federation 1994). Virtually no U.S. innovations have appeared during this time. Cities with CSO problems face major problems in how to intelligently solve these problems without strong technical leadership at the federal or state levels. For example, deep tunnels are a potentially effective but very expensive control option with very long-term implications. Are they a good idea? What has been the experience to date? It appears that some cities have unwittingly installed such deep tunnel systems without thorough technical guidance on how well they work. This meeting of over 300 people was almost totally devoid of representation of professors and graduate students since they had no research results to report.

Recent Initiatives to Improve Research Funding

The only new research initiative in hydrology within the Federal agencies is the new NSF Hydrologic Sciences program. However, this program is focused on basic research and will not be of direct relevance to urban hydrologic systems.

Professional organizations have attempted to partially fill the vacuum left by lack of federal leadership. The American Water Works Association and the Water Environment Federation have established research foundations, e.g., Water Environment Research Foundation (1993). The total funding for these programs at present is less than $10 million per year. Because this research is directly supported by local contributions, it tends to be very applied and focused. We cannot expect these programs to fill this void. It is simply unreasonable to expect to obtain general research funding from the 100's of urban water related agencies around the United States. That is why it is properly a Federal role. McPherson (1973) realized this problem in the early years of the Urban Water Resources Research Council.

McPherson (1971) summarized 1967 statistics on the number of independent or quasi-independent governmental units in 56 Standard Metropolitan Statistical Areas (SMSA's). A typical larger SMSA would have over 200 local governmental units of which over 44 would deal with some aspect of urban water resources. Thus, a major effort would be needed to organize an integrated urban water management system. Some consolidation of local water agencies did occur in the past twenty years as cities formed regional or metropolitan water supply, wastewater, and/or stormwater districts. Even with this consolidation, it is still unrealistic to expect these local entities to provide the essential leadership, especially in the area of research.

At the Federal level, responsibility for water management is spread over many agencies (38 in 1968) with no single agency having the ability to look at the total problem. The only group at present who periodically takes an integrated look at water resources issues is the Water Science and Technology Board (WSTB) of the National Research Council (NRC) which was formed in the early 1980's in anticipation of the leadership vacuum caused by severe cutbacks by the Federal agencies. However, NRC and WSTB only sponsor overview studies and do not conduct research.

Urban Water and Infrastructure. The Federal Government is reassessing its role in supporting the development of infrastructure for the 21st Century, e.g., see Kim et al. (1993). A key recommendation of these large studies is to develop and support a major research and technology transfer initiative as shown in Figure 4. The R&D model incorporates a consortium of academia, industry, and government labs with continuing dialogue and feedback between the users and the researchers. Technology transfer would be an integral part of the process. Urban infrastructure initiatives have been on the horizon for over a decade, e.g., National Research Council 1984, Grigg 1985, Civil Engineering Research Foundation 1993, U.S. Army Corps of Engineers 1993). However, no significant new research programs have become operational.

FUTURE DIRECTIONS OF "URBAN HYDROLOGY"

As I mentioned in the introduction to this paper, alternative futures in urban environmental infrastructure systems can be classified into three options:

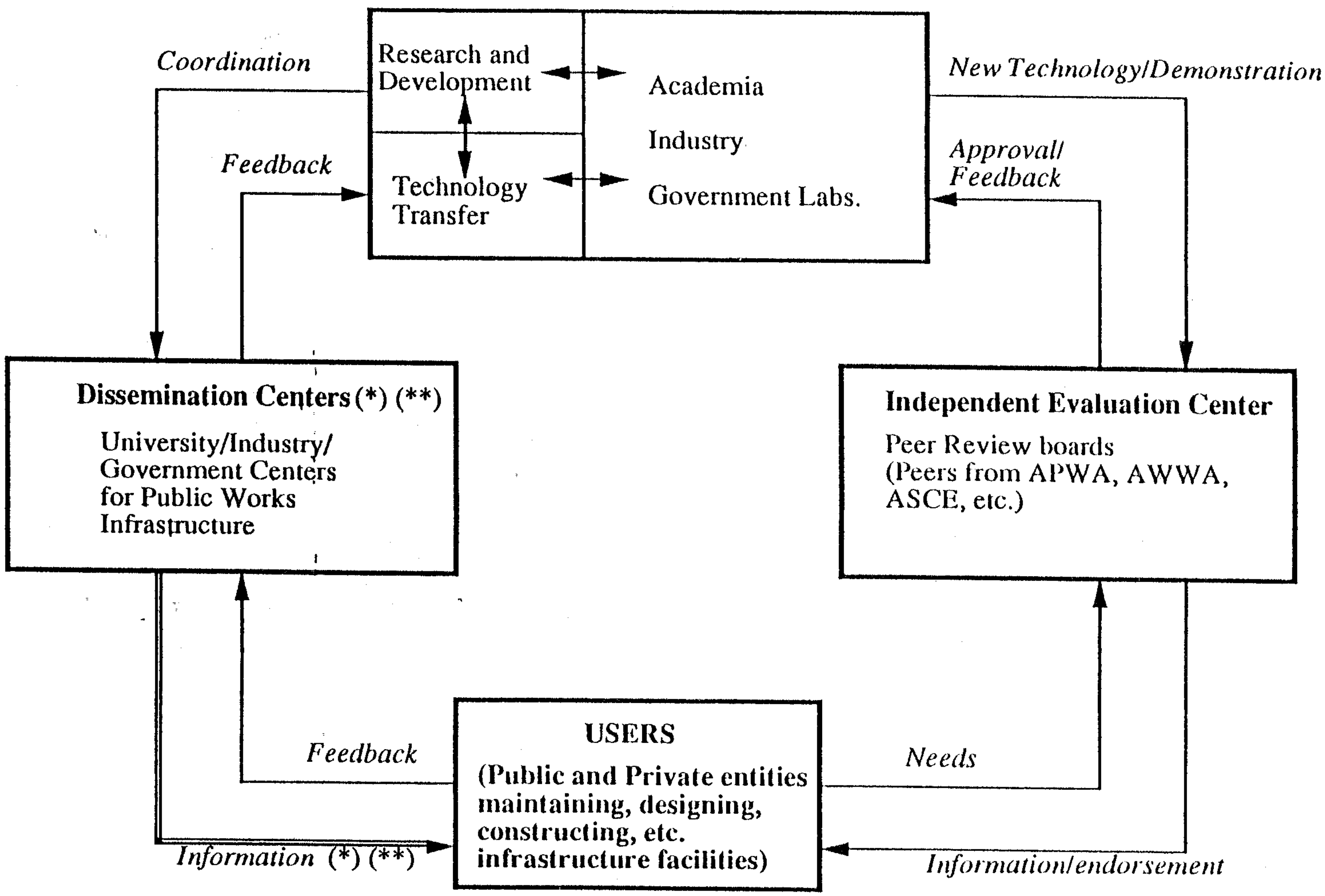
1. Continuation of the status quo of little or no research in urban and environmental systems. Users can adapt innovations from Europe, Japan, and elsewhere as they become available.

2. Federal mission oriented agencies greatly increase their funding of research within the scope of their missions. This alternative future would at least support much of the essential applied research needs in areas with specific gaps in our knowledge.

3. Create new funding source(s) that provide the support to evaluate urban environmental infrastructure systems in a holistic, integrated and sustainable manner. The model for such a system existed in the 1960s and 1970s so this precedent can be used as a guideline for this new initiative.

We would probably agree that option 3 is preferable to 1 or 2 but the major question is how do we go from option 1 continuing to be reality to move towards options 2 or 3? Some ongoing activities and possible initiatives are

National Catalyst for Public Works Infrastructure R&D



(*) Recommended immediate action.
(**) In the long term these dissemination centers will become technology transfer centers, providing not only information on new technologies, but implementation assistance, support, and training.

Figure 4. Strategy for Infrastructure R&D (Kim et al. 1993)

listed below:

1. Working through the ASCE Urban Water Resources Research Council, initiate discussions with appropriate funding agencies to revitalize research support. This activity is ongoing and I would be happy to involve any interested parties.

2. Encourage the Corps of Engineers to take the lead role in this area. With the combined research and development activities at HEC, IWR, and WES, its leadership in the Federal Infrastructure Initiatives Program (U.S. Army Corps of Engineers 1993), and its past Urban Studies Program, the Corps appears to have the vision to promote and successfully implement such a program.

3. Encourage the expansion of existing NSF programs in hydrology and hazards management to incorporate urban water engineering research.

4. Promote the creation of a new federal infrastructure agency which can more effectively coordinate and manage these broader initiatives as recommended by Andrew Lemer (1992), director of the Building Research Board of the National Research Council.

RESEARCH NEEDS

Because of the dormant state of the field during the past decade, few research needs have been fulfilled. Heaney (1986) prepared a list of research needs in urban stormwater pollution. Grigg (1985) prepared a general list of research needs for infrastructure systems. Smolen et al. (1990) prepared a list of research needs in nonpoint impact assessment based on a review of the literature and a judgment on how the Water Environment Research Foundation could best invest its very limited resources. The interested reader is referred to these papers and reports for more detailed information. Lists of the recommendations of Heaney (1986) and Smolen et al. (1990) are included as an appendix to this paper. Watershed planning and management has reemerged as an issue of the 1990's (Heaney 1993). Research support is needed to devise new methods for evaluating watershed-scale systems. Sustainable development is a new area of concern for urban water systems. We need to rethink our approach so that infrastructure decisions do provide long-term benefits to posterity.

SUMMARY AND CONCLUSIONS

The purpose of this paper was to present my ideas on the essential need for an integrated approach to management of urban water systems as a component of urban infrastructure systems. The Hydrologic Engineering Center (HEC) has been a leader in the field of urban water resources for the past 30 years. The role and mission of the Corps of Engineers have changed over this period. Thus, new models are needed for us to rekindle the innovative spirit of the early years when we were able to take a systems view of urban water problems.

In my opinion, the field of urban hydrology is in a state of dormancy due to lack of research support during the past decade and that, absent significant improvements in research support, we will see few, if any, innovations.

Our early leaders of the 1960's and 1970's provided a relatively clear guideline on what needs to be done. During those years, support was available from mission oriented agencies and OWRR. Thus, we had a nice mix of solicited and unsolicited research. The field of urban water resources flourished with this support and major advances were made. However, with the virtual cessation of research support in the early 1980's, we entered a period of inactivity which remains with us to date.

A new agenda is needed in order to make significant future progress in urban water resources. Suggestions include major increases in research support from existing federal agencies to creating a new agency to permit us to take a holistic look at urban water systems. In order to build off of existing thrusts in the agencies, this expanded program may be cast in various ways such as hazards management, civil infrastructure systems, and/or watershed systems.

I hope that the prestigious group of professionals at this workshop can guide us on this important task.

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APPENDIX

A. Research needs cited by Heaney (1986). See Tables 3-6.
B. Research needs cited by Smolen et al. (1990). See Tables 7-1 & 7-2.

TABLE 3.—Selected Research Needs to Characterize Urban Runoff Quantity and Quality

Item (1)	Topic (2)	Reference(s) (3)
Runoff quantity	Relationship of runoff and basin structure	3
	Linkage of surface and subsurface phenomena	41
	Use of dense rain gage networks to describe storm patterns	25
	Integrate soils/land use information systems with hydrologic models to refine spatial analysis capabilities	29
	Flow meters to measure widely varying flows	11
Runoff quality	Flow-weighted composite sampling devices for widely varying flows	11
	Influence of soils, land use, location, and season on runoff quality. Analysis of 1983 data base	42 5 7
	Time series analysis of combined sewer influent data to separate dry- and wet-weather components	32

TABLE 4.—Selected Research Needs to Characterize Receiving Water Impacts

Item (1)	Topic (2)	Reference(s) (3)
Quantity	Hydrograph separation techniques to estimate the origin of flow in rivers	
	Criteria for defining receiving waters	20
Quality	Quantity benefits of stormwater quality control	15
	Bioassay procedures for short-term intermittent exposures to urban runoff	
	Bioassay procedures for long-term exposures to heavy metal accumulation in benthos	41
	Mass balance analysis of urban runoff input and receiving water quality response in water column and sediment	20
	Criteria for wet-weather water quality standards	41
	Behavior of urban runoff in the mixing zones of rivers, lakes, and estuaries	37
	Methods for estimating the probability distributions of sewage, urban runoff, and upstream flows	43

TABLE 5.—Selected Research Needs to Control Urban Runoff Quality

Item (1)	Topic (2)	Reference(s) (3)
On-site control	Glen Ellyn, IL studies indicate that source control is not very promising. Sources are either too diverse or not amenable to control	38, 41
Topic 1	Effectiveness of swales in reducing the quantity and improving the quality of runoff	36
Collection systems	Variety of options have been evaluated, e.g., remote flow monitoring and control, sewer flushing, polymer injection, improved regulators, reducing infiltration inflow	11, 41
Topic 1	Instrumentation and monitoring systems to evaluate hydraulic performance of sewer systems	
Topic 2	Cost-effective ways to rehabilitate and replace sewers	
Topic 3	Laboratory studies of deposition and scour in sewers	
Topic 4	Use of simulation models in storm sewer design	
Topic 5	Calibration of transport models using recently developed US EPA data base	41
Downstream controls	Effectiveness of storage in conjunction with numerous treatment devices has been evaluated	13, 14, 41
Topic 1	Laboratory studies of high-rate, physical, chemical, and/or biological treatment systems for widely variable flows: stormwater alone and combined with domestic wastes	
Topic 2	Effectiveness of high-rate disinfection with widely variable flows	
Topic 3	Design of storage/release systems to maximize pollutant removal	16
Topic 4	Evaluation of groundwater injection systems	
Topic 5	Engineering considerations in using wetland treatment systems	6
Topic 6	Application rates for land disposal of urban runoff	28
Topic 7	Reuse of urban runoff as cooling water	
Topic 8	Probabilistic performance criteria for wet-weather controls	
Topic 9	Treatment of heavy metals in urban runoff	
Topic 10	Sludge/solids disposal aspects of control options	24
Topic 11	Develop process control systems to optimize performance of control units	

TABLE 6.—Selected Research Needs to Develop Decision-Making Models

Item (1)	Topic (2)	Reference(s) (3)
Simulation models	Develop accurate and robust parameter estimation procedures for calibrating available simulation models	41
	Incorporate sludge and solids handling into the simulation models	11
Economic optimization	Interface simulation models with economic optimization models	9, 34
Statistical analysis	Use systems-type statistical models as preliminary screening procedures for simulation models	10
	Perform detailed statistical analysis of NURP, USGS, and other data bases to develop sound parameter estimates for all phases of urban stormwater management	1
Data base	Combine Univ. of Florida, NURP, and USGS data bases on stormwater quality to provide current summary of available data	1
Finance	Develop methods for equitably assigning costs of multi-purpose, multi-group stormwater management programs	18
Socio-political aspects	Apply recently developed methodologies for evaluating public response to alternative control programs	
Computer-aided engineering	Develop computer-aided engineering methods to integrate analysis and design	

Table 7–1
HIGHEST PRIORITY RECOMMENDED NPS RESEARCH PROJECTS

- Identifying a Representative and Cost–Effective Sampling Protocol for NPS
- Determining the Effect of Time–Scale of Discharge on Acute and Chronic Toxicity in Different Ecosystems
- Establishing a Framework for Model Selection, Application, Calibration, and Validation
- Developing a Program to Identify and Evaluate Use–Attainability Methodologies
- Identifying Processes that Define Sediment Toxicity Interactions in Different Ecosystems
- Documenting Nonstructural Controls to Reducing NPS Pollution
- Describing Riparian Buffer Zone Function
- Understanding Impacts of Snowmelt NPS on Receiving Water
- Determining Decomposition Rates for Various Polyaromatic Hydrocarbons (PAHs) in Both Aerobic and Anaerobic Conditions
- Preparing Effective and Implementable NPS Regulations

Table 7-2
LOWER PRIORITY RECOMMENDED NPS RESEARCH PROJECTS

- Matching NPS Control Design Criteria to Objectives
- Establishing the Relationship Between Land Use and Toxicity of NPS Pollutants
- Identifying the Best Available Technology that Has Been Implemented for Urban Stormwater Control
- Preparing Holistic Ecosystem Approaches Case Studies
- Evaluating and Improving Process Models for Pollutant Kinetics to Define the Relationships Between Pollutants on the Landscape and Runoff (Nitrogen and Phosphorus)
- Developing a Holistic Receiving Water Body Methodology
- Developing a Watershed Assessment Methodology
- Researching and Developing a Model that Jointly Estimates Surface and Groundwater Quality Loadings
- Establishing a Nongovernmental National Water Quality Information Exchange Repository
- Determining Regional Atmospheric Pollutant Loading Concerns from NPS
- Developing Mathematical Models for Urban Stormwater System Design to Control Runoff Quality
- Designing a Demonstration-Scale "Constructed Wetlands Treatment System" for Mine Drainage for Hard Rock Mining
- Determining the Feasibility of Identifying Representative Watersheds or Waterbodies to Define "Natural Conditions"
- Comparing Alternative Modeling Approaches for Rural and Urban Areas
- Identifying the Synergistic Effects of Combinations of Pollutants

UPDATING, MAINTAINING AND IMPROVING AVAILABLE MODELS

by

Wayne C. Huber[1]

INTRODUCTION

Purpose. Several HEC, EPA and other models are widely used by the engineering profession for analysis of problems in urban hydrology. Because of the non-proprietary nature of these federal agency models, they are subject to evaluation of methods, code and user-interface to a degree beyond that of most other models. This feedback is a direct source of information for improvements, updates and general maintenance of such models. In addition, the model developers and third-party organizations often provide model enhancements that may be included in new releases. And the model developer provides vision and direction by organizing all such input and establishing policy regarding code, interface, user definition and long term support. These and other factors are reviewed in the context of several widely-used hydrologic models.

Key Issues: What options exist for agencies and others for updating old models and developing new ones? What experience is available to help answer such questions? What has been learned from over 25 years of hydrologic model development that might aid future model development? How can model developers take advantage of the "information age"?

AGENCIES AND MODELS

Anyone can write (and many have written) a rainfall-runoff model. At one extreme, the Rational Method for prediction of peak flows is a "model" that has been available for 150 years and has been computerized by many groups in recent decades. But organized computer code for rainfall-runoff modeling dates from the early 1960s, with one of the most influential models, the Stanford Watershed Model (SWM), first appearing in 1966 (Crawford and Linsley, 1966). The SWM seldom is encountered today in its original form or even in one of its many immediate derivatives (e.g., Kentucky model, Georgia model, Sacramento model), but does, in fact, enjoy great popularity in the form of the federally-supported Hydrologic Simulation Program - Fortran (HSPF) model (Johanson et al., 1984). The Environmental Protection Agency, Center for Exposure Assessment Modeling (CEAM) at Athens, Georgia incorporated SWM hydrology into its nonpoint source assessment models in the 1970s, culminating in HSPF, currently at version 10 and actively employed in watershed studies throughout the U.S. The Stanford Watershed Model thus represents a good example of a model with origins at a university for which the source code was readily available and non-proprietary and from which several derivative models were successfully developed, one of which, HSPF, endures strongly today because of subsequent

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federal support. Thus, major contributions to longevity include: non-proprietary nature, available source code, good documentation, and federal (or other stable agency) support.

The EPA CEAM has promoted model development through extra-mural funding, e.g., for HSPF, WASP, SWMM and other models. Another agency approach is to retain in-house expertise in hydrologic modeling; the Corps of Engineers, Hydrologic Engineering Center (HEC) is the premier example of such an arrangement. Organized in 1964 to support Corps modeling efforts, the HEC also provides hydrologic modeling support to other federal agencies and the general public. Two HEC products are arguably the best known of all hydrologic models: HEC-1 for watershed rainfall-runoff analysis, and HEC-2 for steady-state backwater analysis. These models and others from the HEC inventory have been primarily developed in-house, with occasional significant contributions from outside groups and individuals. The HEC thus exemplifies internal agency model development and is perhaps unique in its isolated location and mission apart from large Corps offices. Other federal agencies that have promoted significant hydrologic model development internally include the Soil Conservation Service (SCS), the U.S. Geological Survey (USGS), the National Weather Service (NWS), the Tennessee Valley Authority (TVA), and the Corps of Engineers, Waterways Experiment Station (WES), although the last agency deals primarily with receiving water models. Agencies for which primary model development has been extra-mural include the EPA, the Federal Highway Administration (FHWA), and the Federal Emergency Management Agency (FEMA). Of course, there are exceptions for all groups listed above.

The primary point here is that federal (or other stable) agency support is essential for the kind of feedback and maintenance that leads to successful and useful models in the engineering work place. In a review of nonpoint source models coupling hydrology and water quality, virtually all operational models (see below) enjoy federal agency support (Donigian and Huber, 1991). Although there are numerous proprietary hydrologic models marketed by various software houses, there are relatively few examples of hydrologic models originating from such firms that have withstood the test of time for wide application by the general profession. Indeed, most offerings from such groups consist of value-added versions of federal software from the HEC (e.g., HEC-1, HEC-2), SCS (e.g., TR20, TR55) and NWS (e.g., DWOPER, DAMBRK).

Two exceptions outside the U.S. are the Hydraulics Research Ltd. models from Wallingford in the United Kingdom (e.g., WALRUS, SPIDA, MOSQITO), and the Danish Hydraulic Institute (e.g., MOUSE, MIKE 11). The Hydraulics Research Ltd. models resulted from a privatization of modeling and other hydraulic activities formerly performed by a federal agency, whereas the DHI models originated with university research followed by a consortium with private industry for further development and marketing. Because the code for models from both groups is highly proprietary, there is little or no user feedback for enhancement of numerical and other routines, which is not to say that the models from these groups are not very good. They are, but at a loss of efficiency due to exclusive reliance on internally-produced code modifications. Models such as MOUSE and WALRUS are marketed at prices on the order of $15,000, because the groups must be self-sufficient. Model multiplicity and federal origin result in prices for most U.S. hydrologic models on the order of hundreds rather than thousands of dollars. And some agencies provide their models absolutely free, made even easier via the Internet and computerized documentation. Nonetheless, the role of private enterprise in hydrologic model development is an open question. For instance, it seems to be the only option by which the SWMM model has attained a true graphical user interface (GUI).

WHAT ARE ACCEPTABLE MODELS?

What constitutes an acceptable model from an agency point of view? Regulatory agencies rely heavily on models supported in some way by a federal or state agency. Certainly in rainfall-runoff modeling in the U.S., models from the HEC and SCS dominate the practice, with models from EPA and other groups somewhat less prevalent. Models from these groups are "operational" in the sense of documentation, support and experience. They have an established track record and promote user confidence. There is usually confidence in the model developers and continuous improvement in code, with consequent reduction in endemic model errors.

Regulatory agencies also desire consistency. One of the reasons for the overwhelming popularity of SCS methods is their ability to produce the same result when applied by different engineers. There is relatively little room for judgment in selecting curve numbers. Designs are consistent from application to application, making the review process easier. On the other hand, it is difficult to calibrate the SCS loss model and unit hydrograph method. Thus, the SCS method is often applied to design situations where calibration is not possible or necessary, whereas somewhat more physically based models are applied to situations in which calibration is required, such as to existing urban drainage systems subject to remediation, for which some historic data are available.

In principle, any model can be made just as consistent as the SCS method by appropriate guidelines for parameter selection. Manuals for this purpose are often prepared by regulatory agencies. The better the model algorithms reflect the true physics of the rainfall-runoff process, the easier is this task, at least in the absence of calibration data. ("Black box," input-output models can often be calibrated very precisely, but are not as useful for prediction when physical characteristics of a basin are changed.) It is often easier to develop a physical basis for a model of urban hydrology than for the non-urban situation because parameters such as imperviousness, pipe dimensions and roughness are easier to determine. Hence, there are additional reasons to lean toward physically-based models in urban hydrology.

With the availability of long-term meteorological data bases, "acceptable" models should also be able to perform continuous simulation, that is, simulation for the period of record of the meteorological data base. The HEC STORM model (Roesner et al., 1974) was developed for just this purpose in urban hydrology (simulating runoff and quality using 62 years of hourly rainfall data in San Francisco), following the historic lead of the Stanford Watershed Model, the first (and enduring, through HSPF) continuous model. Other continuous models include SWMM, DR3M-QUAL, and the HEC IFH (Interior Flood Hydrology). Among its advantages, continuous simulation avoids vexing questions of initial and antecedent conditions, permits frequency and risk analysis, and obviates the need for single (often synthetic) design storms.

It does, however, require an extensive meteorological data base (now conveniently available on CD-ROMs), more computer time, and more training in interpretation of the results. Some regulatory agencies are moving toward this scheme. For example, King County, Washington provides eight "representaive" years of precipitation and generalized runoff simulation data for application of HSPF to urban hydrologic studies.

IN WITH THE OLD OR OUT WITH THE NEW?

New models continue to be developed for application to urban and multitudinous other hydrologic problems. This is very healthy, maintaining diversity and diffusing new ideas and concepts into the profession. Often, a new model (e.g., developed by a graduate student) will contain a few real advances bonded to many existing procedures. If the advances are coupled to an existing model it is easier to incorporate them into the "official" agency version of the model. For this purpose, it is easiest to work with existing models that are non-proprietary and for which the underlying computer code is open and viewable.

Should an old model be updated or a new one developed? A common example in urban hydrology deals with flow routing through the drainage system, for which alternative approaches range from simple hydrologic and reservoir routing techniques to solution of the complete Saint Venant equations. Most new work focuses on new numerical approaches toward the latter end, or simpler non-inertia (diffusion) and kinematic wave/Muskingum-Cunge methods. Is it better to update a model such as SWMM Extran (Roesner et al., 1988), or should the modeler "start over" with something new, such as the HEC UNET (HEC, 1993). As always, there are advantages and disadvantages to each approach.

An older model often has many special cases and conditions built in. SWMM Extran, for example, spends much of its computational time in examining special cases of sub- vs. super-critical flow, regular vs. adverse slopes, free-surface vs. surcharged flow conditions, and ordinary channel/pipe flow vs. flow through weirs, orifices, tide gates and pumps. It is not enough to originate a better numerical scheme for solving the Saint Venant equations; it also must be put into practice for real systems under varied hydraulic conditions. For this reason, there is truly only a small number of widely-used hydraulic models that solve the complete Saint Venant equations (e.g., DWOPER, Extran, UNET, MOUSE, Wallingford).

The advantages of a completely new model include better numerical techniques and less extraneous computer code left over from prior efforts in an old model and difficult to extract without damaging the overall program in the process. In addition, a new model may be written in a structured way, in the current language/technique of choice (e.g., object oriented, the "new paradigm"), with full access to state-of-the-art graphical user interfaces (GUIs), statistics, data bases, and decision support systems.

It is much more difficult to enhance "legacy software," such as the SWMM Fortran code, to take advantage of new technology. The usual form is that of a shell, written in a language such as C++, Visual Basic, etc. for the purposes of the GUI, that accesses the underlying model "engine" left in its original form. This is in lieu of completely re-writing the original code in a new language. The HEC has wrestled with this problem in its NexGen effort (Feldman, 1994).

In general, development of a shell for an older model leads more rapidly to an end product with a more credible model engine, while re-writing from scratch takes longer but may result in a more sophisticated end product that is fully integrated with GUI technology such that it performs more efficiently and more reliably in the long run. In some cases, a new language may not be suitable for the engineering calculations incorporated in the underlying model. Thus, a coupling of the geographical information system (GIS) ArcInfo with SWMM uses the Arc Macro Language (AML) for development of the GUI, while retaining the SWMM Fortran for the model engine (Curtis and Huber, 1993). Other examples of SWMM shells include EPA Windows SWMM, XP-SWMM, MTVE, and PCSWMM4. A convenient shell for HSPF is the USGS ANNIE program.

The importance of GUIs and useful shells cannot be over-emphasized. Much simplistic or at least dated hydrologic technology is still widely used because it is incorporated into attractive and very user friendly GUIs. It is regretable that the current (May 1994) EPA CEAM SWMM release, version 4.30, still does not have a method for hydrograph plots beyond the line-printer option from 1970. A principal reason for this is past CEAM policy against incorporating proprietary commercial graphical software into products it distributes. However, it is possible to provide plotting routines using a variety of languages that would not lead to licensing problems. It is no wonder that simpler less sophisticated models are often used instead of SWMM. Shells may also provide access to digital maps (including AutoCad files), radar images and ancillary statistical and plotting packages.

INTERNAL VS. EXTERNAL MODEL DEVELOPMENT

A special problem for hydrologic model development and support groups such as the HEC that consist of motivated, experienced hydrologists, is whether and how to incorporate models written by outsiders into the menu of modeling options supported by the group. Internal model development means complete knowledge and control of the model coding and algorithms, good quality control, consistent format, and full intellectual rights. Acquisition of code from others means less reinvention of wheels, and probable cost savings and development time savings. It also means reliance on outsiders for support, at least until agency personnel are trained in all aspects of the model. If an outside model is adopted for use and distribution by an agency, an arrangement must also be made for coordinated maintenance, updating and improvements, unless parallel model releases are acceptable. Nonetheless, the experience with algorithms in existing code may make adoption of an existing model worthwhile.

As an example, SWMM's code has been in the public domain since its inception in 1970. As a result, various groups have made changes to the code, and different versions exist within the marketplace, e.g., Sacramento SWMM. The Extran Block in particular has been used as the basis or core of half a dozen European hydraulic routing models. The code on such derivatives is sometimes proprietary, leading to uncertainty as to exactly what SWMM methods are being used. Sometimes code changes in the "official" release are not included in older derivatives, leading to problems with the derivative that no longer exist in the official version, e.g., Missouri River District COE SWMM. On the other hand, some groups are careful to respect the official code even in the face of the need for improvements, e.g., PCSWMM4. Unlike the EPA CEAM, the HEC has largely controlled this problem for its public domain programs through insistence that code changes to the model engine be performed by the HEC itself. Open code for the models still leads to more direct feedback on what exact changes need to be made, however.

INTERNET AND THE "INFORMATION HIGHWAY"

The revolution in computer technology has exceeded the ability of most models to take advantage of it. Data bases exist on CD-ROMs of meteorologic and hydrologic time series as well as soils characteristics, digital maps and other information useful to modeling. Models themselves can be accessed quickly and remotely via the Internet and the "ftp" (file transfer protocol) procedure. For instance, all EPA CEAM models can be obtained via anonymous ftp at the address: ftp.epa.gov The distributed files often include documentation in the form of word processing files printable by the user. (Another ftp source at Oregon State University for a current version of SWMM is: engr.orst.edu, subdirectory: /pub/swmm/pc)

Data themselves are becoming available via the Internet. For instance, the National Geophysical Data Center (NGDC) in Boulder, Colorado has made several of its data bases available on the Internet at the address: ftp.ngdc.noaa.gov Unfortunately, the NOAA National Climatic Data Center (NCDC) in Asheville, North Carolina, repository for all meteorological data, still requires a conventional request, although they now offer several options for data transmittal, a vast improvement over the 9-track magnetic tapes that once were the only option. Fortunately, the NCDC data are also available commercially on CD-ROMs, as are USGS streamflow records.

Perhaps of the most use is the ability on the Internet to communicate with peers regarding modeling enhancements and general exchange of information on a professional level. Model code, input/output and problem statements flow across oceans in an instant (often to the dismay of the model developer who has to address them). But the time saving is monumental, and the volume of information that can be sent via the Internet far exceeds that of a fax (although figures and sketches sometimes present a problem).

Discussion groups exist for some models as well. The EPA CEAM has long sponsored a bulletin board system (BBS), accessible via modem and telephone (706/546-3402). However useful, this particular BBS requires a long distance telephone call that is not toll-free and thus is not used as much as it might be. A better, although currently under-utilized option exists on an Internet "list server" at the University of Guelph. Sending the Internet message SUBSCRIBE SWMM-USERS <your name> to listserv@uoguelph.ca results in a person's Internet address being added to the list of recipients of any message sent to the address by another enrolled participant, thus facilitating rapid exchange of SWMM information. To be really useful, a larger enrollment is desirable than the current (September 1994) total of approximately 50. Discussion groups or "forums" exist about thousands of topics on several commercial and other networks, although to the writer's knowledge, there are none about urban hydrologic modeling.

Another useful alternative is newsletters. The HEC publishes a quarterly newsletter describing advances in its modeling efforts. The EPA CEAM formerly published a similar newsleter. The effort for SWMM has been taken over by William James at the University of Guelph with a quarterly "SWMM News and Notes." But newsletters never have the immediacy of Internet conversations.

Finally, conferences and other formal mechanisms for technology exchange can be fruitful. Stormwater modeling has been supported informally since 1974 by "Stormwater Modelers User's Group Meetings," held semi-annually on the average by a variety of interested sponsors. Proceedings have been published by the EPA and elsewhere (James, 1993). These informal meetings often provide very explicit feedback on the capabilities of a wide variety of models and other stormwater-related analysis.

CONCLUSIONS AND RECOMMENDATIONS

Modeling agencies should take advantage of all available resources for model enhancement and new model development and not be strictly "inward looking." For some agencies, such as the EPA CEAM, this is their only budgetary option, sometimes leading to inconsistent coding practices, incompatible software (in the sense of one program feeding another), and a wide variety of languages and GUI capabilities. On the other hand, CEAM offers a wonderfully diverse array of water quantity and quality models. Other agencies, such as the HEC, can have the best of both worlds through careful acquisition of outside software coupled with consistent in-house development.

Federal or other stable agency support is essential for the long-term viability of models. Whether supported internally by an agency or via extra-mural contracts, model maintenance is rarely successful without such an agency effort.

Non-proprietary code and models leads to enormous volumes of user feedback, sometimes directed very specifically to an identified problem. Public domain software also attracts a larger user base, thus leading to greater model experience and feedback. Proprietary software sometimes has extensive internal support and good user support, but does not enjoy the advantage of such intense scrutiny as does non-proprietary software.

New models must do more than provide a superior numerical scheme, especially for urban hydrology. They must also provide for special hydraulic conditions in the drainage network and be applicable to "real" problems. For this reason, existing models that include such features should be examined carefully for adoption by an agency. Alternatively, existing non-proprietary models may offer code or algorithms for insertion into new models.

It behooves model developers to utilize modern methods for information exchange, such as the Internet. Few other computational innovations have led to such easy and rapid transfer of data and ideas.

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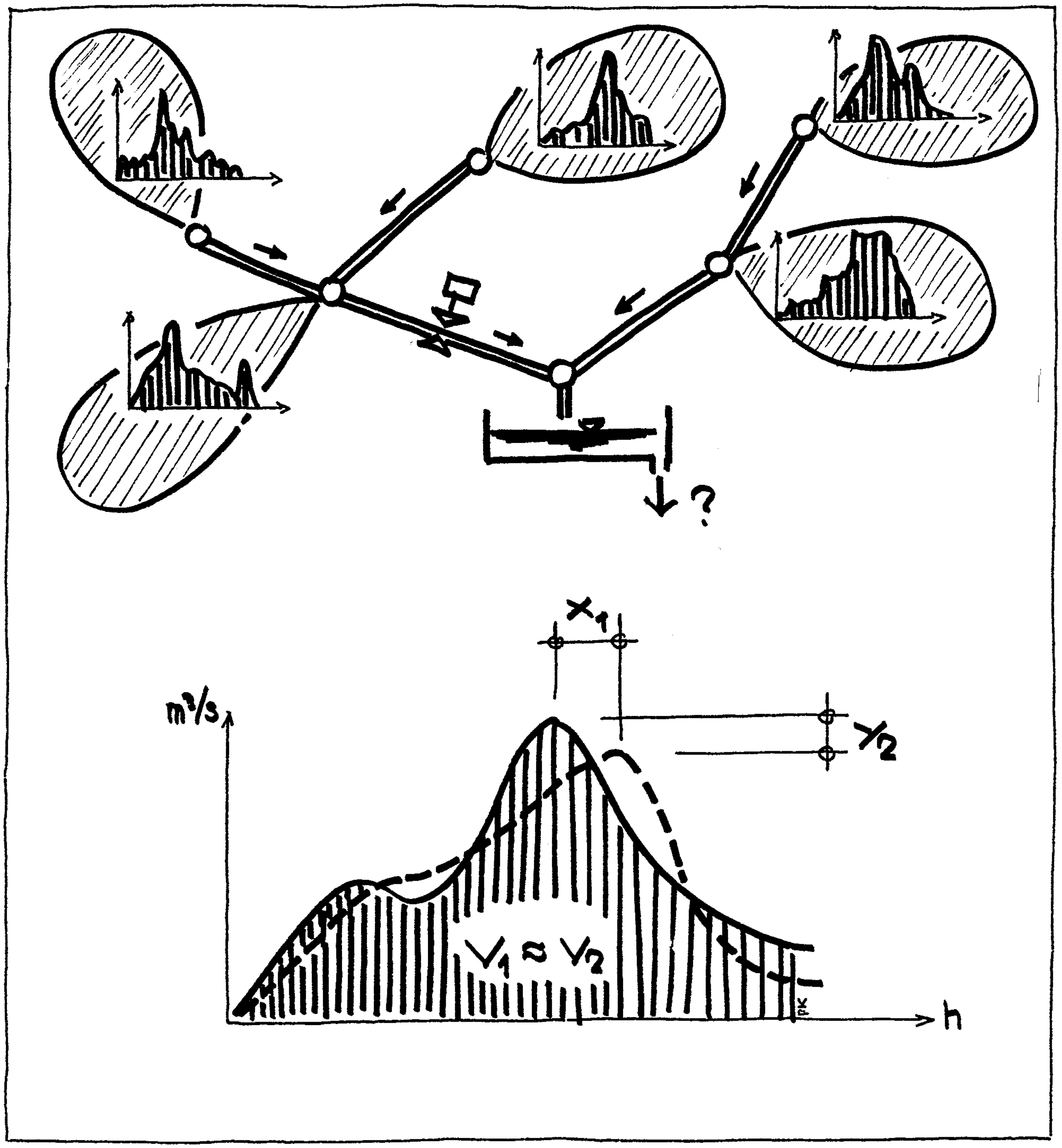
Session 4:

Hydraulics



SUMMARY OF SESSION 4: **Hydraulics**

Overview

Five papers were presented during the hydraulics session related to urban hydraulics modeling. Several case studies were described which involved the use of a rainfall-runoff model coupled with some unsteady flow model. Choosing the appropriate hydraulic model, and exchanging information between models were common issues. Another common issue was the characterization of flow which does not travel a regular dendritic stream system.

Paper Presentations

Paper 16 Clyde Hammond, Hydraulic Engineer, Charleston District, presented a paper titled "Urban Storm Water Modeling of Charleston, South Carolina." The subject study is a reconnaissance level effort. Therefore, a representative subbasin was analyzed. The models used by Clyde included HEC-1 for rainfall-runoff analysis and EXTRAN for closed conduit routing. Hydrologic information such as flow paths and land use were derived with the aid of GIS. Part of his modeling effort addressed flow which leaves a manhole, travels some overland path into another manhole or ponds until the water level falls. He described how he accomplished this in EXTRAN. His recommendations for new modeling capabilities include integrated capability between SWMM and GIS, and more friendly input and output.

Paper 17. Nancy Powell, Chief, Hydrologic Engineering Section, New Orleans District, was not able to attend the workshop but submitted a paper titled "Jefferson Parish Urban Flood Control - Feasibility Study - Risk Analysis Plan." Her paper describes a study in which HEC-1 was used for rainfall runoff modeling with resultant hydrographs routed using UNET. A version of UNET was utilized which was specially modified to handle the pumping system. Her study involved risk analysis to address uncertainty in several areas. The stage-storage curves and interconnection of storage areas to canals and other storage areas proved to be significant sources of uncertainty.

Paper 18. Tom Fogarty, Chief, Hydrology and Hydraulics Branch, Chicago District, presented a paper titled "Chicagoland Underflow Plan - Urban Hydrologic and Hydraulic Modeling." This ongoing project attempts to solve both overbank flooding and sewer backup. Tom described this study by way of profiling the different models used for the feasibility, design, and feature level stages. At the feasibility level, watercourse models were developed using HEC-1 and HEC-2, and pilot areas were evaluated at the sewer level using SWMM. The results from the pilot areas were translated to non-pilot areas using regression methods. At the design level, HSPF was used, along with SCALP, TNET (a modified version of UNET for tunnels), and UNET. A modification was made to HSPF to access HEC-DSS files. WHAMO and Song's models were used for transient analysis. One of Tom's conclusions is that for complex studies, a variety of models is needed - some with enhancements. He also stressed the importance of communication between models using HEC-DSS.

Paper 19. Mike Schmidt, Camp Dresser and McKee, reviewed issues from several recent studies he was involved in. He started off with a discussion of issues in urban stormwater modelling, such as choosing level of detail, continuous versus design storm modeling, and technology limitations. He then presented some of his experience with several approaches to modeling. Some of the topics covered included cumulative effects of encroachment in floodplains, volumetric flooding analysis, and dynamic versus steady state analysis of typical urban hydraulic phenomenon. Particular attention was paid to modeling of surcharged culverts and storm sewers.

Paper 20. Bob Barkau, a private consulting engineer in St. Louis, presented a paper titled "Simulation of Open Channel and Pressure Flow using the One-Dimensional Open Channel Flow Equations." Bob gave a brief review of the open channel flow equations and described how they can be used for closed conduit flows by use of the Preismann slot. To illustrate the method, he discussed an application in Chicago, as well as for a network of sewers. He concludes that UNET can be successfully used for subcritical flow in simple sewer networks, and that the program needs work on some features.

URBAN STORM WATER MODELING OF CHARLESTON, SOUTH CAROLINA

by

Clyde A. Hammond[1]

INTRODUCTION

This paper describes the efforts currently ongoing in an urban flood damage reduction study for the City of Charleston, South Carolina. The purpose of this paper is to both summarize the efforts of this particular study as well as to highlight areas of particular interest and importance in urban hydrology as they relate to Corps of Engineers water resources planning studies.

STUDY AREA

Storm water drainage is a major problem within the City of Charleston. Frequent surface flooding occurs throughout the city as a result of moderate to heavy rainfall events. Damages and consequences resulting from the flooding include the disruption of vital services, loss of mobility and income, property damage and loss, and a threat to the health and safety of the population. Causes of the flooding can be traced to a number of factors, including the capacity of the existing storm sewer system, tidal backwater effects, and the natural terrain of the area. The downtown area of the City of Charleston lies upon a peninsula bounded by the Ashley and Cooper Rivers, which combine to form Charleston Harbor, a tidal estuary (see Figure 1). The area can be characterized as low-lying with slight undulations and gentle slopes. Ground surface elevations within the peninsular city range from 0 to 15 feet referenced to the National Geodetic Vertical Datum of 1929 (NGVD), while surface slopes are generally less than 1 percent. The peninsular city area is heavily developed, with approximately 90% of the land area presently developed. Peninsular development consists of single and multi-family residential dwellings (65%), commercial and institutional areas (25%), and industrial areas (10%). Due to the low ground elevations of the area, much of the city is subject to flooding from high tide levels, either due to astronomical tides or a combination of astronomical tides and storm surges. The National Oceanic and Atmospheric Administration (NOAA) has maintained a tide gage at the U.S. Custom House in Charleston Harbor since 1902. Based upon this long-term gage record, the normal tide range in this area is 5.3 feet, while the average spring tide range is 6.1 feet. The average spring high tide elevation is 4.36 feet NGVD, while the highest elevation recorded during the period of record

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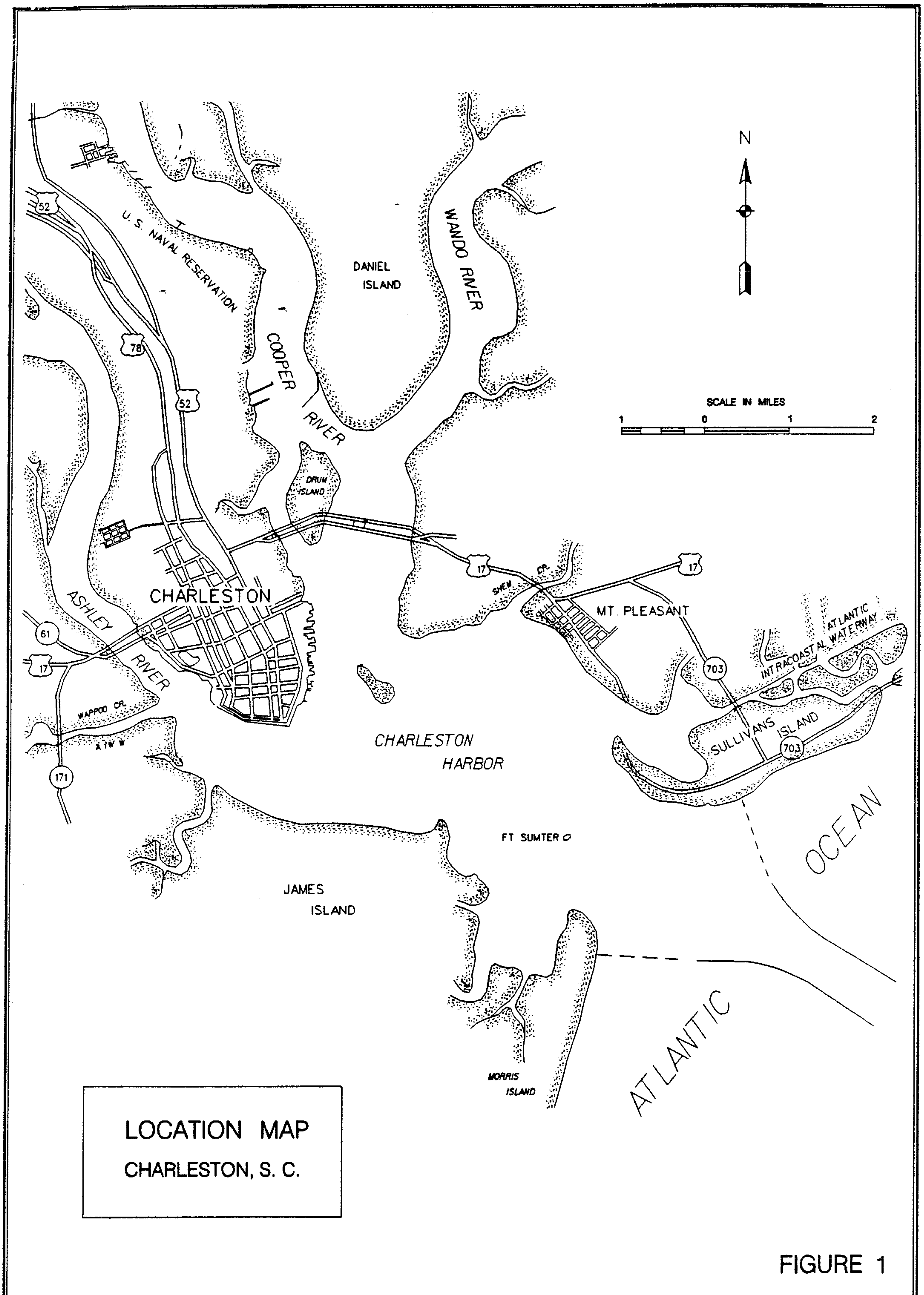


LOCATION MAP
CHARLESTON, S. C.

FIGURE 1

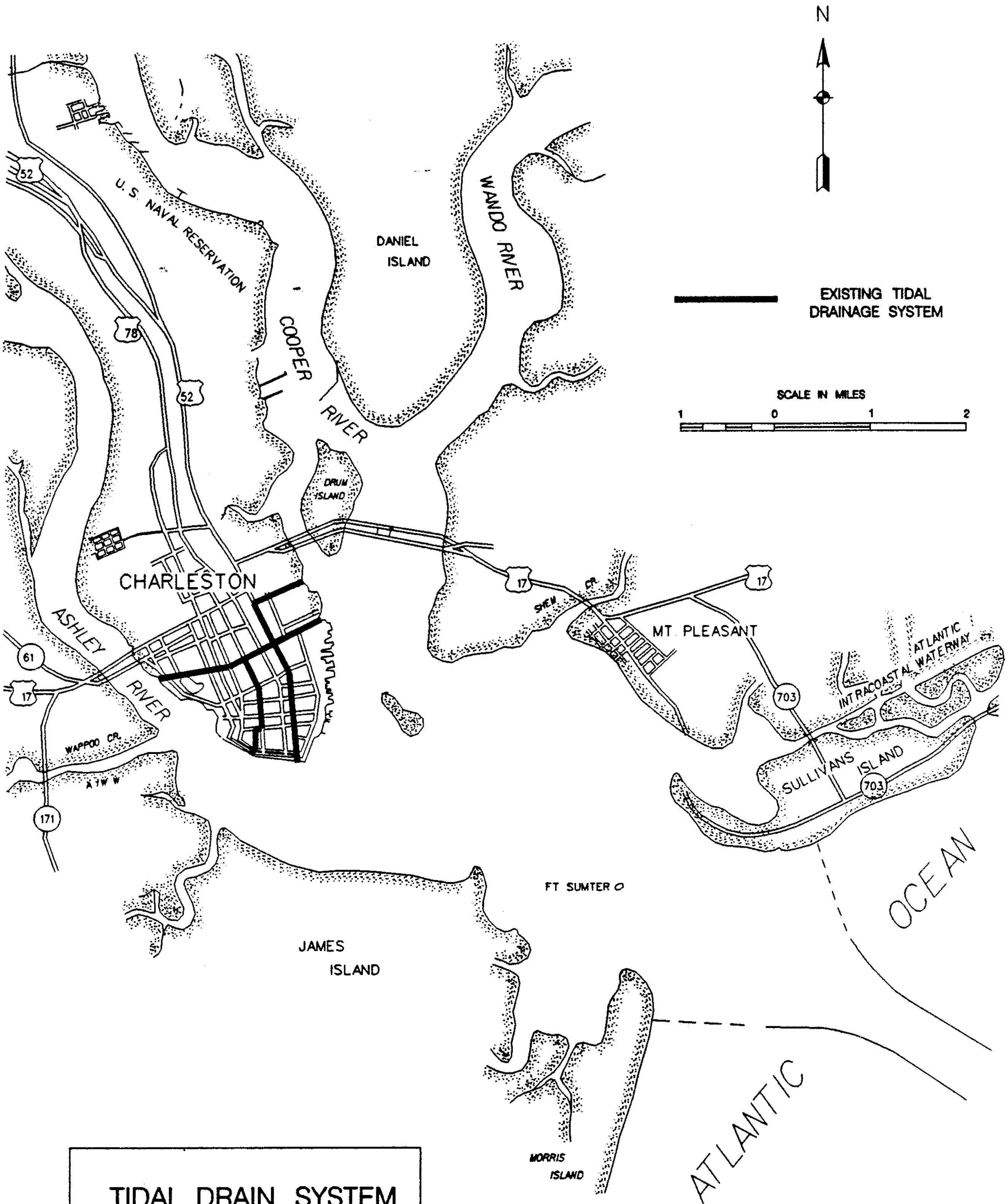
of the gage is 10.5 feet NGVD, which occurred during Hurricane Hugo in 1989. In addition to direct flooding from extreme tide levels, high tide levels severely influence the carrying capacity of the existing storm water drainage facilities for the peninsular city. The existing storm sewer system consists mainly of vitrified clay pipe and brick arches. The main component of the existing system is a brick arch tidal drain system which was constructed in the 1850's as a combined sanitary and storm sewer (see Figure 2). These arches, which are approximately 3 to 4 feet wide and 7 feet high, are interconnected with outfalls to the Ashley and Cooper Rivers. Originally this system of archways was controlled by a tide gate at each discharge point. Each day at low tide one gate was opened, which allowed the system to flush. This operation kept the system relatively free of siltation. Over the years the tide gate system was abandoned, resulting in siltation within the system. The discharge points are now uncontrolled and are open to water level fluctuations due to tidal exchange. Much of the carrying capacity of these arches is not available if rainfall events occur in conjunction with periods of high tides. Rainfall events which occur in coincidence with extreme high tide levels such as those resulting from spring tides or storm surges result in severe flooding of the peninsular areas of the city due to the limited flow capacity during these periods.

PREVIOUS STUDY

In 1984, a master drainage plan was completed for the City of Charleston by the A/E consulting firm of Davis & Floyd (Davis & Floyd, 1984). This was a reconnaissance level study which inventoried and evaluated the capacity of the existing storm water drainage facilities. Peak runoff capacities of components of the existing system were computed using the rational method to develop peak discharges and Manning's equation for computation of the hydraulic grade line. This study also provided recommendations for improvements to the system. These recommendations were based upon design criteria of a 0.1 exceedance probability rainfall event occurring during a spring high tide in the peninsular area of the city. Proposed upgrades included the addition of 9 storm water pumping stations throughout downtown Charleston, as well as upgrades and replacements for feeder and collector elements within the storm drainage pipe network.

CORPS STUDY

In 1993, Congress authorized the Charleston District Corps of Engineers to investigate solutions to flooding within the City of Charleston through a General Investigations study. The primary goal of this reconnaissance study was to determine whether Federal interest exists in a storm damage reduction project for the City of Charleston. Although urban storm water problems normally do not meet the requirements for Corps of Engineers involvement, it was decided, during early coordination meetings, that there may be Federal interest in the Charleston urban flood problem since backwater from tidal flooding is a pertinent contributing factor. The process of determining Federal interest requires that certain steps be taken in the study process. These



TIDAL DRAIN SYSTEM

CHARLESTON, S. C.

FIGURE 2

tasks are fairly well defined, and with some slight variations, are applicable to all Corps of Engineers flood reduction planning studies. Summarized (and somewhat simplified) these tasks are:

(1) Estimate the existing flood elevations and flood durations for the study area along with their associated exceedance probabilities.
(2) Determine flood damages resulting from various flood levels and durations. Using this data in combination with the results from (1) , predict flood damages occurring under the existing conditions.
(3) Formulate plan(s) of improvement to minimize the impacts of flooding and predict the resulting flood levels, durations, and exceedance probabilities with these improvements in place.
(4) Predict residual flood damages occurring with the proposed improvements in place. Determine flood reduction benefits resulting from drainage improvements.
(5) Develop a cost estimate for the drainage improvements.
(6) Determine the benefit to cost ratio for the proposed improvements. Determine whether Federal interest exists.

These requirements dictate that the final product of the hydrologist's work are elevation/duration/exceedance probability relationships for flood levels throughout the study area. These relationships need to be developed for both the existing condition and the improved condition. In developing a numerical model of an urban area, some important mechanisms which must be accounted for include the rainfall-runoff process, overland flow routing, and pipe network routing. For this particular study area, several other factors complicate the modeling process. These include the backwater from estuary tide levels, flooding of the system due to surcharging of inlets which exceeds the ground elevation, and the routing and storage of these surcharge flood flows. Physical features to be modeled for this study include flow through arch, box, and circular conduits, flow though open channels, pump stations, and depression storage areas. Due to these requirements and the desired final product, the Storm Water Management Model (SWMM) of the Environmental Protection Agency (USEPA, 1994) was selected as the primary analysis tool for the simulation. The EXTRAN block of SWMM was selected to perform all hydrograph routings and storage computations. The Corps of Engineers HEC-1 rainfall-runoff model (HEC, 1990) was selected to compute the flood hydrographs for input to SWMM.

STUDY METHODOLOGY

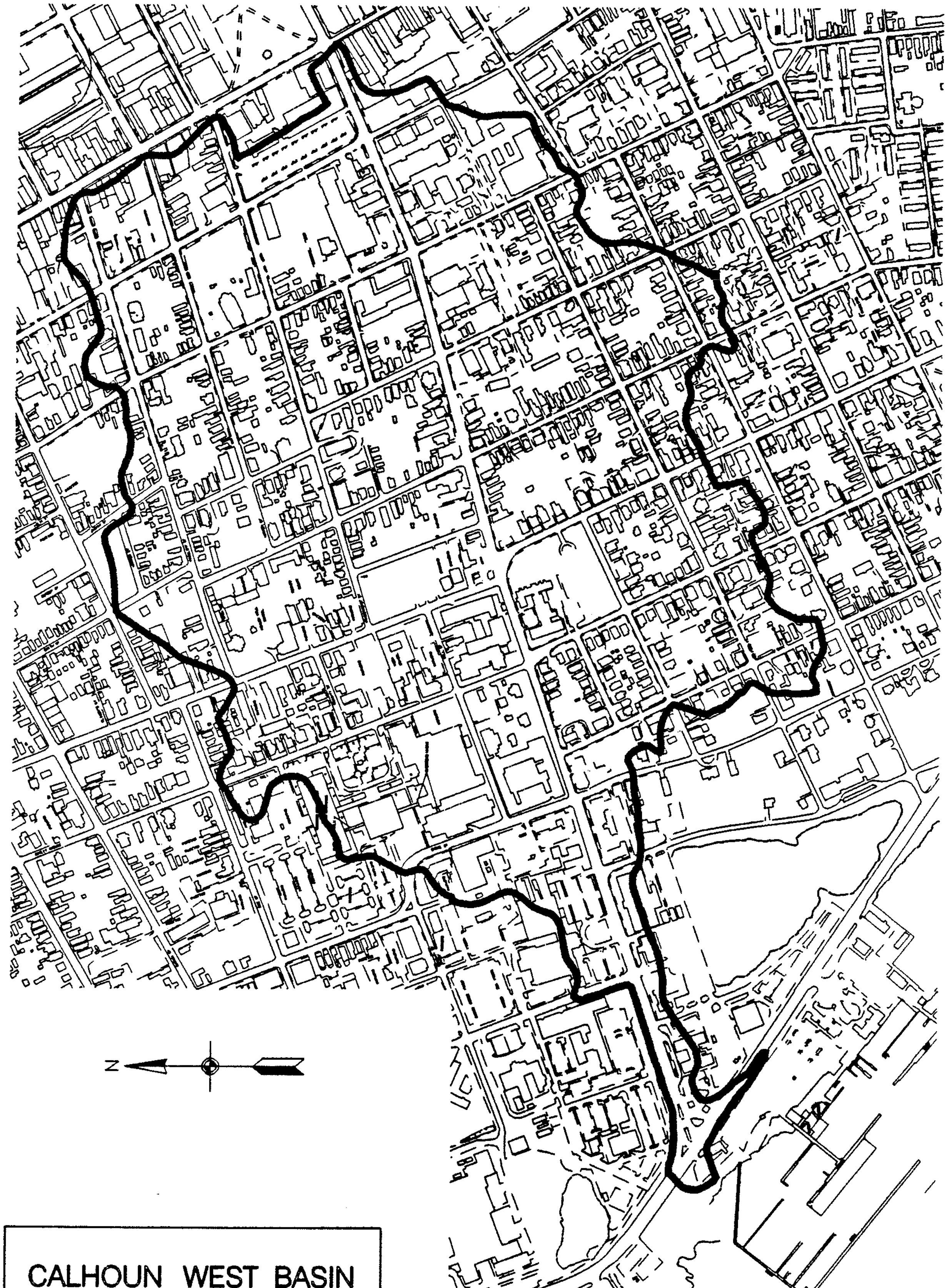
As a reconnaissance level study, existing available data was utilized wherever applicable. The master drainage plan of Charleston (Davis & Floyd, 1984) identified 52 subwatersheds covering 2430 acres within the peninsular area of the city. The plan also identified nine proposed storm water pump stations which would serve 1514 acres of the peninsular area. In order to keep the reconnaissance study to a manageable scope based upon time and funding constraints, it was decided that one "representative" basin in the downtown area would be analyzed to determine whether federal interest exists in a storm damage reduction project. After consultation among the study team members and the study sponsor, the Calhoun West basin was selected for study during the reconnaissance phase. The Calhoun West basin encompasses over 200 acres within the central and west central regions of the peninsular (see Figure 3). This basin was selected primarily due to existing flood problems, flood damage potential, and the sponsor's priorities. Included in this basin are three major hospitals, a university, an elementary school, numerous commercial and residential structures, and a large portion of one of the main east-west thoroughfares of the city, Calhoun Street.

Hydrology

The Calhoun West basin encompasses 212.6 acres within peninsular Charleston. The basin is almost fully developed, with the primary land uses being residential, institutional, and commercial. For simulation purposes the basin was divided into 30 subbasins (see Figure 4). The subbasins were selected primarily based upon existing and anticipated future drainage features. While SWMM's RUNOFF block has the capability to compute inflow hydrographs, it was decided instead to use the Corps HEC-1 model for this task. The HEC-1 model allows more flexibility in defining the site conditions and it was felt that this model could be utilized to achieve more accurate results. The HEC-1 results would then be input to the SWMM EXTRAN block for routing. The kinematic wave catchment analysis was utilized to model the runoff process, while the loss rates were estimated using the Soil Conservation Service (SCS) curve number methodology. All physical parameters including slopes, overland flow paths and lengths, and land use were taken from topographic/planimetric maps and site investigations of the area. Synthetic rainfall data for the Charleston area was used to define the precipitation events.

Hydraulics

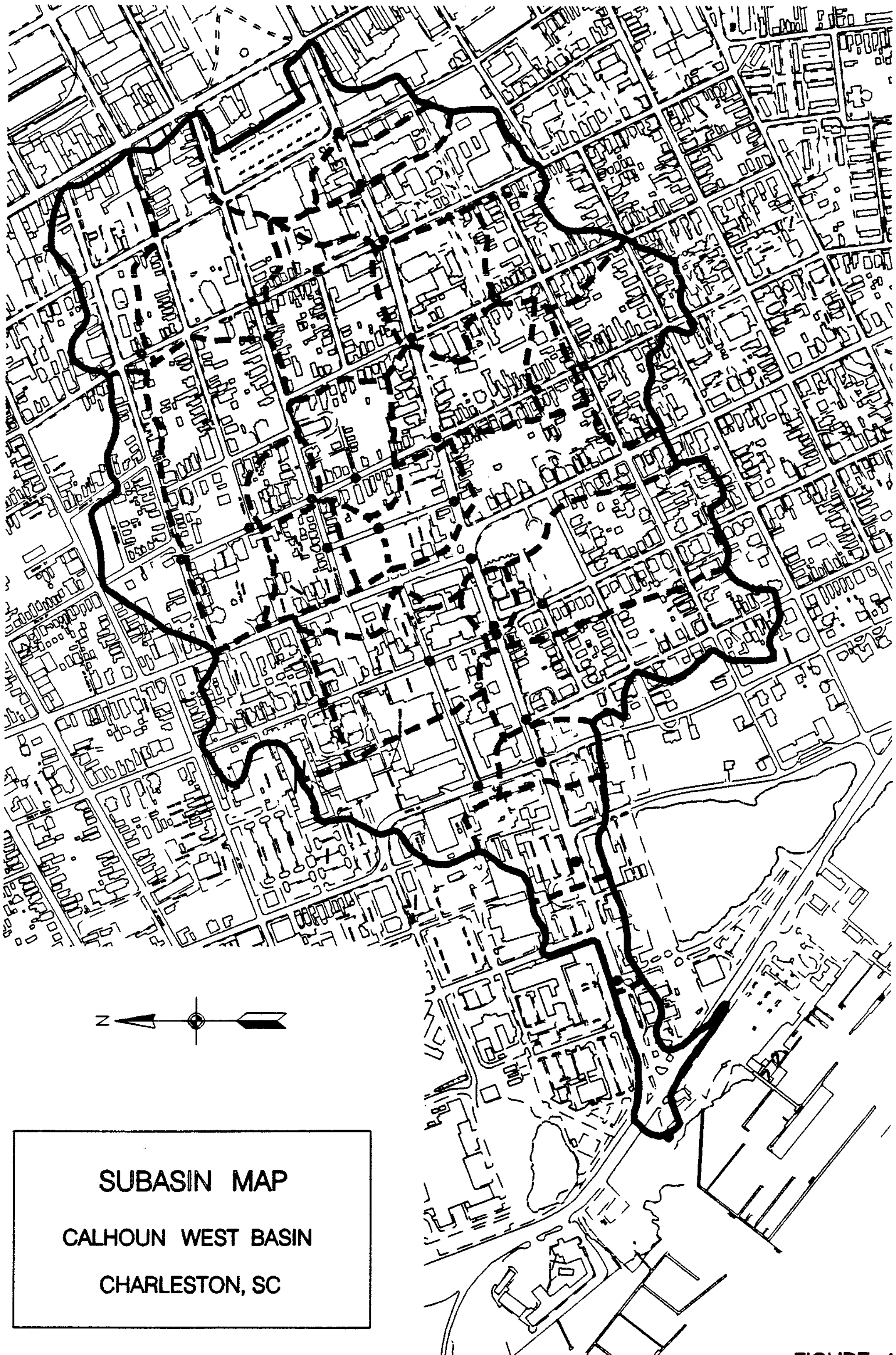
The SWMM EXTRAN block is a link-node model, with links (such as conduits or open channels) which connect nodes (such as manholes, inlets, and outfalls). Input hydrographs from the HEC-1 output were read into SWMM and assigned to the proper nodes. The physical properties of the existing system were taken from the previously completed master drainage plan for the city (Davis & Floyd, 1984). Data taken from this report included conduit types, dimensions, lengths, and slopes for all significant pipes in the system (see Figure 5). This data was utilized to define the existing storm sewer network in the model. Equivalent pipes were used wherever possible to reduce computational requirements, provide fairly consistent pipe lengths, and promote numerical stability in the model. Detailed maps of the area showing all topographic and planimetric features were digitized into three-dimensional CADD drawing files. In addition, the existing storm sewer system was digitized and overlaid onto the planimetric map files. The topographic information (contours and spot elevations) was used to create a digital terrain model of the Calhoun West basin using a surface modeling software package. In order to model surface flows resulting from the flooding of nodes, surface flow paths also had to be defined in the model.



CALHOUN WEST BASIN

CHARLESTON, SC

FIGURE 3

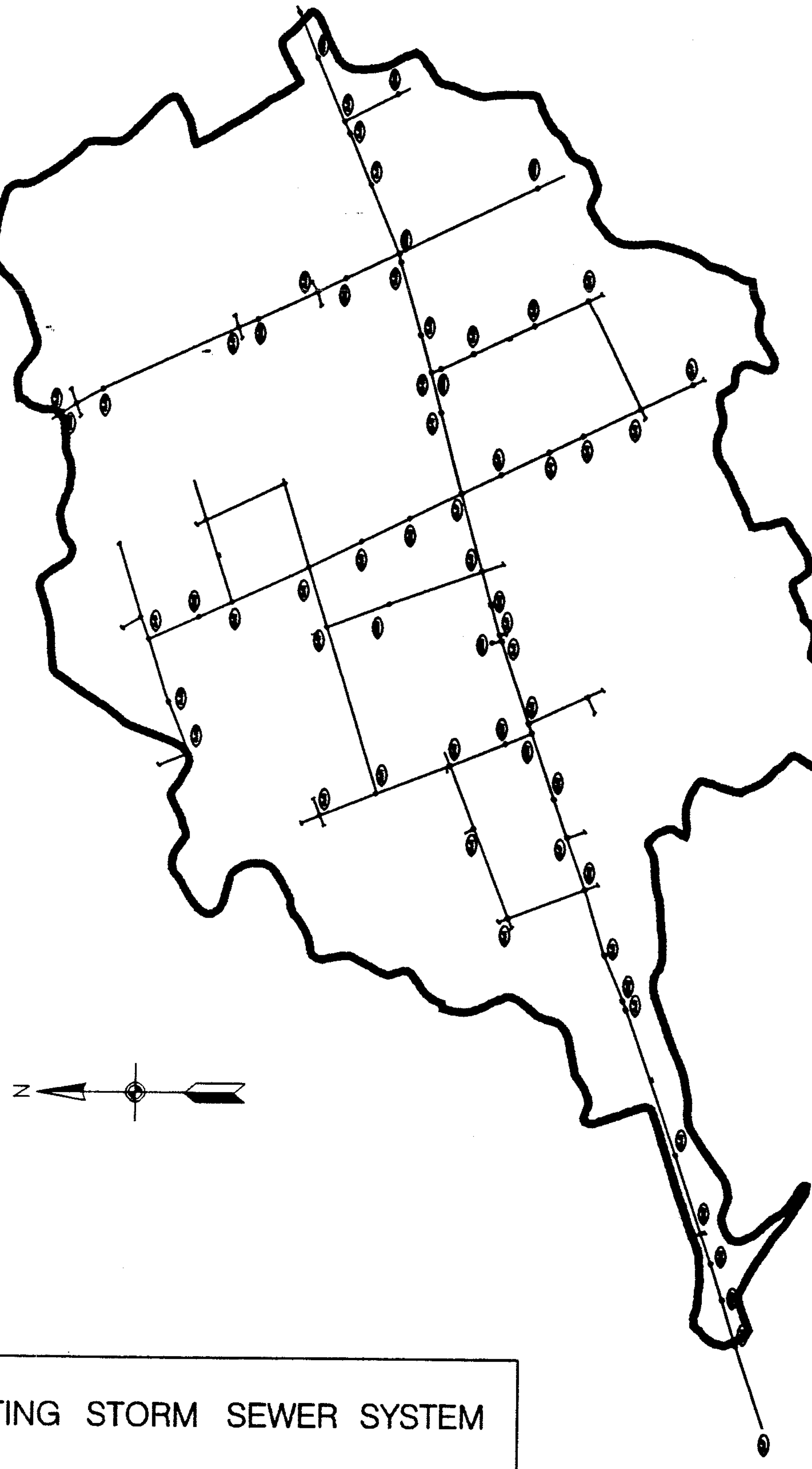


SUBASIN MAP

CALHOUN WEST BASIN

CHARLESTON, SC

FIGURE 4



EXISTING STORM SEWER SYSTEM

CALHOUN WEST BASIN

CHARLESTON, SC

FIGURE 5

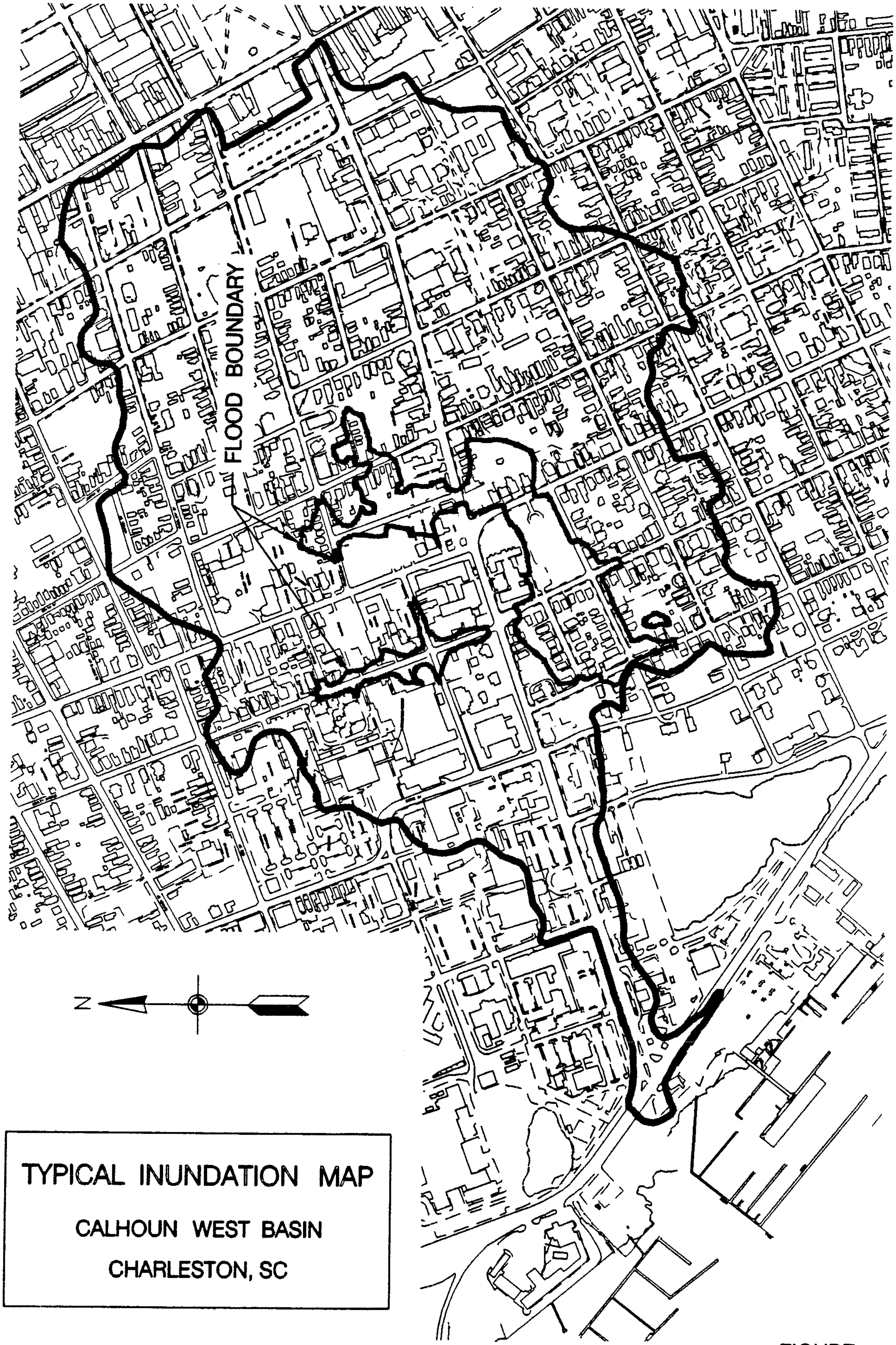
It has been observed in the field that when flooding of manholes and inlets occurs, the excess water either ponds in the immediate area if surface depression storage exists, or flows overland until it reaches either another inlet at which it can reenter the drainage system or until it reaches a depression storage area. Typically, the majority of this overland flow from surcharging takes a flow path along the street and curb, which tends to form a sort of asphalt/concrete-lined, shallow, rectangular channel. These "channels" offer minimum frictional resistance to flow. For this model, the overland flow paths were defined as open channel link elements. Manholes were represented as equivalent pipes in the model to connect the underground conduit portions of the storm sewer network with the open channels occurring at the ground surface elevations of the junctions. The open channel surface flows were interconnected to adjacent manholes and eventually to depression storage areas in the model. For this model, three major surface storage areas were identified. These were defined in the model as variable-area storage junctions and were assigned area-capacity curves. Excess flow can enter the storage nodes through the surficial open channels when flooding of junctions occurs and can subsequently exit the storage nodes and return to the underground portion of the storm sewer network as flood flows recede and the hydraulic grade line drops throughout the system.

RESULTS (so far)

Presently, this study is still ongoing. To date an HEC-1 model of the basin has been completed and a SWMM model of the existing storm sewer system has been developed. Currently these models are being calibrated to recent flood events which have occurred in the area. Initially some stability problems were encountered with the SWMM model of the basin, and it has been found that very small time steps (less than 5 seconds) were necessary to maintain continuity and stability in the model. The variable surface area storage junction option of SWMM was used to define three storage areas within the basin which were used to model surface flooding and depression storage. The model results will yield peak flood elevations for these areas. These flood elevations will be overlaid on the digital terrain model produced from topographic data to produce digital maps showing the inundated areas for various flood events (see Figure 6). Once existing conditions have been established, improvements to the system will be simulated with the model by the incorporation of additional components to the network. Anticipated proposals for improvements include the addition of a storm water pump station to minimize backwater effects during periods of high tides, the addition of either a tunnel or large conduits to convey flow from the existing system to the pump station, and the addition of extra conveyance in feeder lines where the capacity of the existing lines has been exceeded.

CONCLUSION

SWMM has been a valuable analysis tool for the Charleston District's Storm Damage Reduction Study for the City of Charleston. The model can handle those aspects of urban studies unique to Corps of Engineers projects; that is, the ability to analyze system performance not only



TYPICAL INUNDATION MAP

CALHOUN WEST BASIN

CHARLESTON, SC

FIGURE 6

under the design conditions, but also when the design conditions are exceeded. Corps projects are typically justified based upon surface flooding damages; therefore, it is very important to accurately quantify system surcharges and track total volumes. SWMM has the capability to compute and route entire hydrographs through a system undergoing surcharging and flooding. Thus, SWMM is very applicable to Corps of Engineers studies of urban areas and definitely should be considered for use by Corps District offices when they face complex urban hydrology problems.

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JEFFERSON PARISH URBAN FLOOD CONTROL FEASIBILITY STUDY RISK ANALYSIS PLAN

by

Nancy J. Powell[1] and Charles Shadie[2]

INTRODUCTION

Purpose. This paper presents a risk analysis plan for the Eastbank Jefferson Parish Urban Flood Control Feasibility Study. The feasibility study is currently ongoing; verification of hydrologic and hydraulic models and the development of existing condition frequency information are underway.

Key Issues. The Eastbank of Jefferson Parish is a complex urban area, with minimal hydrologic information, which requires sophisticated modeling techniques. This is the first study of its type for which the New Orleans District will perform risk and uncertainty analysis using guidance contained in EC 1105-2-205.

BACKGROUND

Study Area. The study area includes the Eastbank of Jefferson Parish, bounded on the east by Orleans Parish, on the west by St. Charles Parish, on the north by Lake Pontchartrain, and on the south by the Mississippi River. The study area is protected by levee systems from river and hurricane flooding.

The Eastbank Jefferson Parish drainage system is an intertwined network of subsurface culverts, ditches, canals, and pumping stations. The drainage system consists of a network of major north-south canals connected by lateral east-west canals. Storage areas exist between natural ground contours and drainage canals. Natural ground elevations vary from +12 feet NGVD to -6 feet NGVD. Drainage is generally from high contours near the Mississippi River northward to Lake Pontchartrain. Because most of the natural ground elevations of the study area are lower than water levels in the lake and river, all water must be pumped. Five pumping stations, located at the downstream end of major canals, pump water into Lake Pontchartrain. The pumping stations contain several variable speed pumps. The combined nominal pumping capacity of the five stations is over 17,000 cfs. These stations drain 44.2 square miles of Eastbank Jefferson Parish, or 92 percent of the parish. The remaining portion of the parish drains to a pumping station at the Jefferson-Orleans Parish line and is shared with drainage areas within Orleans Parish.

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Jefferson Parish operates its pumping stations to maintain specific water surface elevations in the major outfall canals. Once these elevations are exceeded, the pumps are engaged to discharge the excess. The lateral canals equalize flow between the major outfall canals. This allows runoff to flow in different or opposite directions depending on rainfall patterns and available capacities of the pumping stations.

Figure 1 shows the major canals, laterals, and pumping stations in the study area.

Existing Hydrologic Data. Two National Weather Service rainfall stations, New Orleans' Audubon Park and Moisant Airport, are in the vicinity. Some information from local rainfall stations is also available. At Audubon Park, a maximum 24 hour rainfall of 12.6 inches occurred in June 1991. At Moisant Airport, a maximum 24 hour rainfall of 12.66 inches occurred in November 1989.

Virtually no discharge gage records exist within the study area. Stage data are limited to high water readings following major events.

Floods and Storms of Record. There have been several floods in the study area caused by runoff from heavy rainfall. Flood insurance claims paid for six major rainfall events between 1978 and 1989 amounted to $227 million. Table 1 summarizes recent flood events.

Table 1
Recent Flood Events in Jefferson Parish

Year	Date	Audubon Rainfall, inches	Moisant Rainfall, inches
1978	3 - 4 May	10.60	6.80
1980	2 - 3 Apr	6.72	4.85
1980	12 - 13 Apr	8.55	8.55
1982	24 - 25 Apr	6.80	4.21
1983	6 - 7 Apr	8.29	7.81
1985	25 - 31 Oct	10.30	9.72
1988	30 Mar - 3 Apr	13.36	10.37
1989	6 - 8 Nov	9.33	14.37
1991	9 - 11 Jun	13.10	2.33

Urban Flood Control Improvements. Jefferson Parish officials requested that the Federal government participate in projects that would alleviate rainfall flooding in the parish. Local interests have made improvements but have been unable to keep pace with the increasing severity of rainfall flooding. A reconnaissance study, completed in July 1992, focused on the development of economically feasible drainage improvement plans that can accommodate a 10-year rainfall event with no residual flooding. The study provided sufficient analysis to indicate a flood control plan that is both economically feasible and environmentally acceptable. This plan is shown on Figure 2. The first cost for the plan is $95 million.

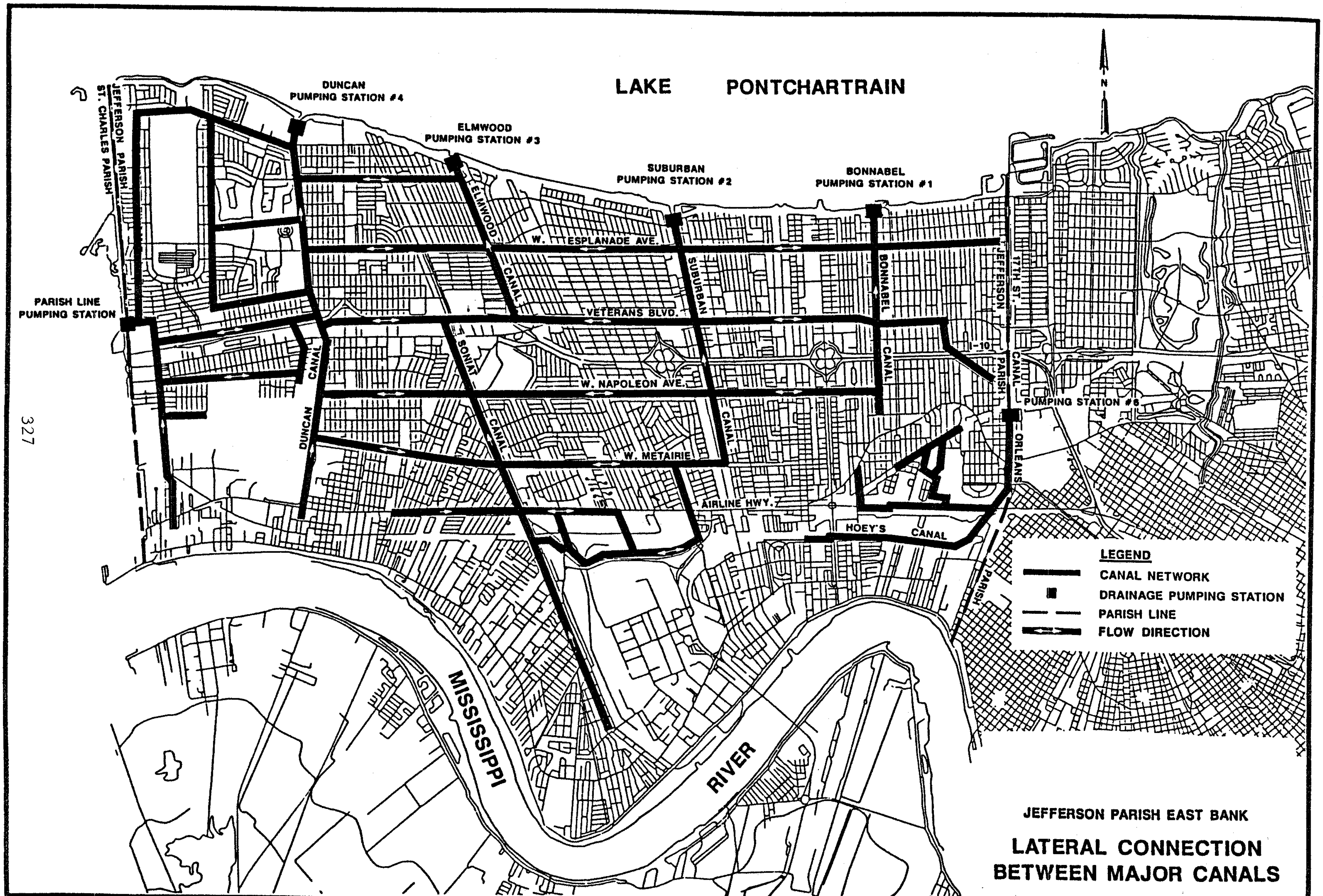
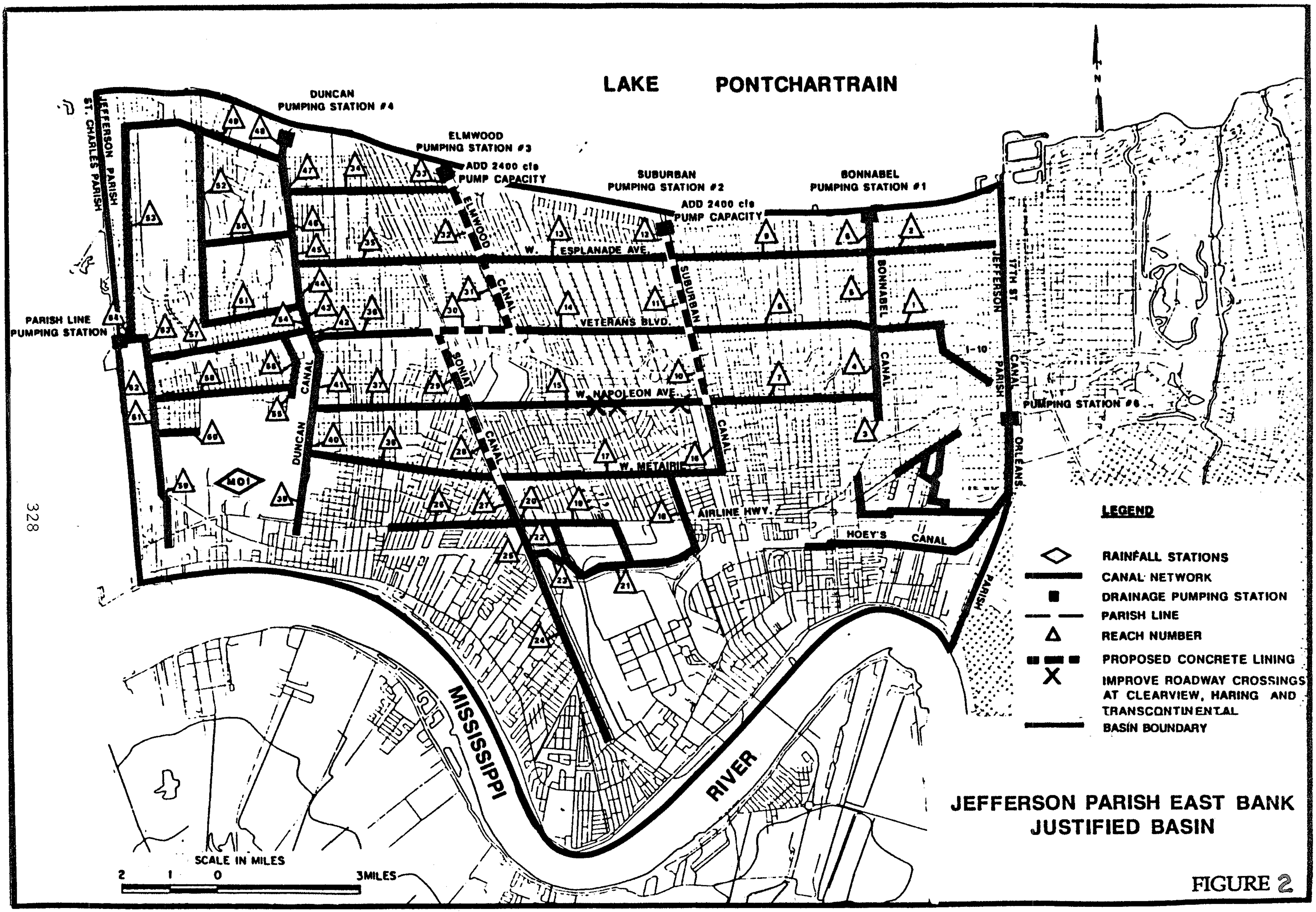

LAKE PONTCHARTRAIN
DUNCAN PUMPING STATION #4
ELMWOOD PUMPING STATION #3
SUBURBAN PUMPING STATION #2
BONNABEL PUMPING STATION #1
PUMPING STATION #6
PARISH LINE PUMPING STATION
JEFFERSON PARISH
ST. CHARLES PARISH
W. ESPLANADE AVE.
VETERANS BLVD.
W. NAPOLEON AVE.
W. METAIRIE
AIRLINE HWY.
ELMWOOD CANAL
SUBURBAN CANAL
BONNABEL CANAL
DUNCAN CANAL
SONIAT CANAL
HOEY'S CANAL
17TH ST. CANAL
JEFFERSON PARISH
ORLEANS PARISH
I-10
MISSISSIPPI RIVER
N
LEGEND
CANAL NETWORK
DRAINAGE PUMPING STATION
PARISH LINE
FLOW DIRECTION
JEFFERSON PARISH EAST BANK
LATERAL CONNECTION BETWEEN MAJOR CANALS

FIGURE 1



JEFFERSON PARISH EAST BANK JUSTIFIED BASIN

FIGURE 2

The plan presented in the reconnaissance study is a starting point for the feasibility phase. Modifications to the plan and alternatives suggested by the local sponsors will be analyzed during the feasibility study.

STUDY MODELING

Hydrologic Modeling. An HEC1 model has been provided to the New Orleans District by parish consultants. For the feasibility study, rainfall information for the study area is taken from National Weather Service Technical Paper Number 40, "Rainfall Frequency Atlas of the United States" and National Weather Service Hydro-35 publication, "Five to 60-Minute Precipitation Frequency for the Eastern and Central United States." Duration-frequency information from these two publications is used to define each frequency rainfall event using a triangular precipitation distribution. Loss rates are computed using the SCS method. Over 100 subbasins are modeled using HEC1. Runoff in each subbasin is routed to the canals using the kinematic wave option.

Hydrographs have been also developed for the 1988 flood event from observed rainfall information. The hydraulic model, described below, is calibrated to the 1988 event.

Hydraulic Modeling. A UNET hydraulic model of the study area has also been provided to the New Orleans District by the parish consultants. UNET is a one-dimensional unsteady flow model that can analyze a full network. Computed hydrographs from the HEC1 model are read into the UNET model at various locations along canal reaches. As canals overflow their banks, UNET computes flow into storage areas and allows storm water to reenter the areas adjacent to the canals. Over 60 reaches and 114 storage areas comprise the UNET geometry for Eastbank Jefferson Parish.

The UNET model furnished to the district is a specially modified version of the standard UNET computer model to handle the parish pumping system.

As the Jefferson Parish drainage system is self contained, inflow to the system is from rainfall only. Outflow boundary conditions do exist, however. Stages in Lake Pontchartrain can vary over 5 feet and have an impact on the capacity of the pumping stations. For a stage of 4 feet NGVD in Lake Pontchartrain, the capacity of the five pumping stations is reduced 9 to 24 percent. Review of Lake Pontchartrain water levels and Jefferson Parish flood information over the last 10 to 15 years suggests a high incidence of elevated lake stages when interior events occur. Pumping curves have been modified to reflect a reduced capacity. During a flood event, the differential head between inside stages and Lake Pontchartrain stages varies, also affecting the capacity of the pumps. This variation in differential head is not taken into account in the pump curves.

The parish consultants provided a UNET model that is calibrated to the 1988 flood event. Water level and rainfall information were used in the calibration effort. The New Orleans District has taken the model, modified the pump curves as described above, and is in the process of running various frequency events. Stability problems have arisen in the model regarding the pump on and off elevations. The time step used in the model was not small enough, causing the canals to go dry and the pumps to turn off. Cyclical flow and water level hydrographs are created because the pumps are turned on and off numerous times during the flood event. Reducing the time step significantly is not a viable option, however, because the length of time to model an event becomes unworkable. The New Orleans District plans on working with Mr. Bob Barkau, who developed UNET, to correct the stability problems. If not corrected, these stability problems may have an impact on the district's risk and uncertainty assessment by limiting its ability to run sensitivity analyses.

Stage Frequency. From the reconnaissance study, the existing condition stages within the study area for the 10-year event varied from -5.8 feet NGVD to 1.8 feet NGVD. The 100-year stages varied from -4.5 feet NGVD to 3.0 feet NGVD. The difference between the 10- and 100-year events for each economic reach was between 0.5 and 1.5 feet. It is anticipated that similar results will be achieved from the feasibility analysis.

RISK ANALYSIS

Areas of Uncertainty. Several areas of uncertainty have already been identified during the reconnaissance phase and the feasibility phase to date. They are:

stage-storage curves for the storage areas. Because of the flatness of the terrain, the larger the error in the survey information used to develop the storage curves, the greater the error in the curves. Subsidence is an issue in the study area. The datums for benchmarks used in the surveys may have been determined using different epoch of levels. This creates a situation where surveys will show two areas having similar elevations where in fact the elevations can vary by 0.5 feet or more. The storage areas often represent the residential and commercial areas of the watershed. The error in surveys can also create errors in slab elevations, affecting the economic side of the risk analysis.

operation of pumping stations. Changing differential heads occur during a flood event, affecting the pump capacities. This change in pump capacities is not captured in the UNET model.

design criteria such as Manning's n values and range of rainfall events.

runoff simulation to canals; specifically, the subsurface system, overbank flow, and interconnected storage areas. The contribution of the subsurface system, particularly to storage, may not be captured. The lack of hydrologic information from which to calibrate the model makes it difficult to determine if the interconnectivity of the storage areas and canals is modeled correctly.

The stage-storage curves and the interconnection of storage areas to canals and to other storage areas appear to have the most impact on the stage frequency information. Therefore, it is important that the uncertainty in these areas be captured in the risk and uncertainty analysis.

Methodology. EC 1105-2-205 requires that all flood damage reduction studies adopt a risk-based analysis framework. This framework is defined as an approach to evaluation and decision making that takes into account risk and uncertainty. Risk and uncertainty exist because of measurement errors and the inherent variability of factors and parameters that make up the economic and engineering analyses. Generally, in a hydrologic and hydraulic analysis, the parameters are discharge and stage. In a risk analysis, the variability of discharge is captured in confidence limits for the discharge-frequency curve. Variability in stage is identified in the standard deviation of the stage-discharge relationship. A Monte Carlo simulation is performed where a discharge is sampled from a cumulative frequency curve, a distribution of discharge uncertainty developed and sampled, and an error associated with the stage-discharge relationship sampled to determine a stage with error for a particular frequency. This stage is then used with stage-damage and other economic information in the rest of the risk analysis.

HEC has recommended to the New Orleans District that risk analysis for the Eastbank Jefferson Parish study be performed using the procedures outlined in a draft ETL, "Uncertainty Estimates for Non-analytic Frequency Curves." This ETL presents methodologies for computing the uncertainty about non-analytic frequency curves. Frequency curves are generally estimated from a graphical fit of observations and the uncertainty calculated from order statistics. Where observations are not available and synthetic rainfall-runoff analyses are performed, equivalent record length is used. A computer program, "LIMIT," has been developed to calculate uncertainty about a non-analytic frequency curve.

The frequency analysis for the Eastbank Jefferson Parish study is based on hypothetical events. Rainfall information is from TP40 and Hydro-35. HEC1 and UNET are used for the rainfall-runoff analysis and development of stage frequency information. Calibration is to one event with limited rainfall and high water mark information for that event. An analysis using equivalent record length is appropriate. Table A-1 from EC 1105-2-205 provides guidelines for equivalent record length determination. The analysis setting for Jefferson Parish is similar to the "handbook/textbook model coefficients only" setting from Table A-1. Therefore, an equivalent record length of 10 to 15 years is appropriate for the risk analysis.

Risk Analysis Plan. For each hydraulic reach or damage reference point, stage frequency curves with confidence limits will be developed using HEC's recommendation described above for existing conditions and for each alternative condition that appears to have a reasonable possibility of being economically feasible. If, during an initial or preliminary screening, an alternative proves to be economically infeasible, no risk analysis will be conducted. Some sensitivity analyses will be performed to ensure that the analysis captures the uncertainty identified in the stage-storage information, the interconnectivity of the canals and storage areas, and the operation of the pumping stations.

CONCLUSION

A risk analysis plan has been developed for the Eastbank Jefferson Parish Urban Flood Control Feasibility Study using guidance from EC 1105-2-205 and a draft ETL on uncertainty estimates for non-analytic frequency curves. This plan can be applied to future studies in the New Orleans District, such as Morganza to the Gulf, that use similar methodologies to compute frequency information.

CHICAGOLAND UNDERFLOW PLAN
URBAN HYDROLOGIC AND HYDRAULIC MODELING

by

Thomas Fogarty[1]
and
Lynette Moughton[2]

INTRODUCTION

This paper describes the hydrologic and hydraulic methodologies and modeling used during the feasibility and design level and proposed for the feature level of the Chicagoland Underflow Plan Study (CUP). The portion of the study area considered here is the combined sewer area tributary to the Mainstream System of the Tunnel and Reservoir Plan (TARP). The TARP plan is being constructed to reduce basement flooding, raw sewage bypasses to local watercourses and backflows to Lake Michigan. The Mainstream System, plus the Little Calumet, Des Plaines and O'Hare Systems, constitute the 353 square-mile combined sewer service area (see figure 1) under the jurisdiction of the Metropolitan Sanitary District of Greater Chicago (MWRDGC).

DESCRIPTION OF PROBLEM

Within the study area the flooding problem takes two forms, overbank flooding and sewer backup. The sewer backup flooding problem in the CUP area is attributable to either inadequate sewer capacity or to sewer outfall submergence.

The major watercourses in the study area consist of the North Shore Channel, the Chicago River and the Chicago Sanitary and Ship Canal in the Mainstream System; the Des Plaines River and Salt Creek in the Des Plaines System; and the Calumet River, the Little Calumet River and the Calumet-Sag Channel in the Calumet System. The drainage pattern in the study area is mainly from north to south through these watercourses and into the Illinois River Basin.

Within the Chicago area there are four controlling works which are used in setting the elevations in the watercourses. The three controlling works along the Lake Michigan shoreline (on the Chicago River at Wilmette, downtown on the Sanitary and Ship Canal, and at the O'Brien Lock and Dam on the Calumet-Sag Channel) are used to divert water

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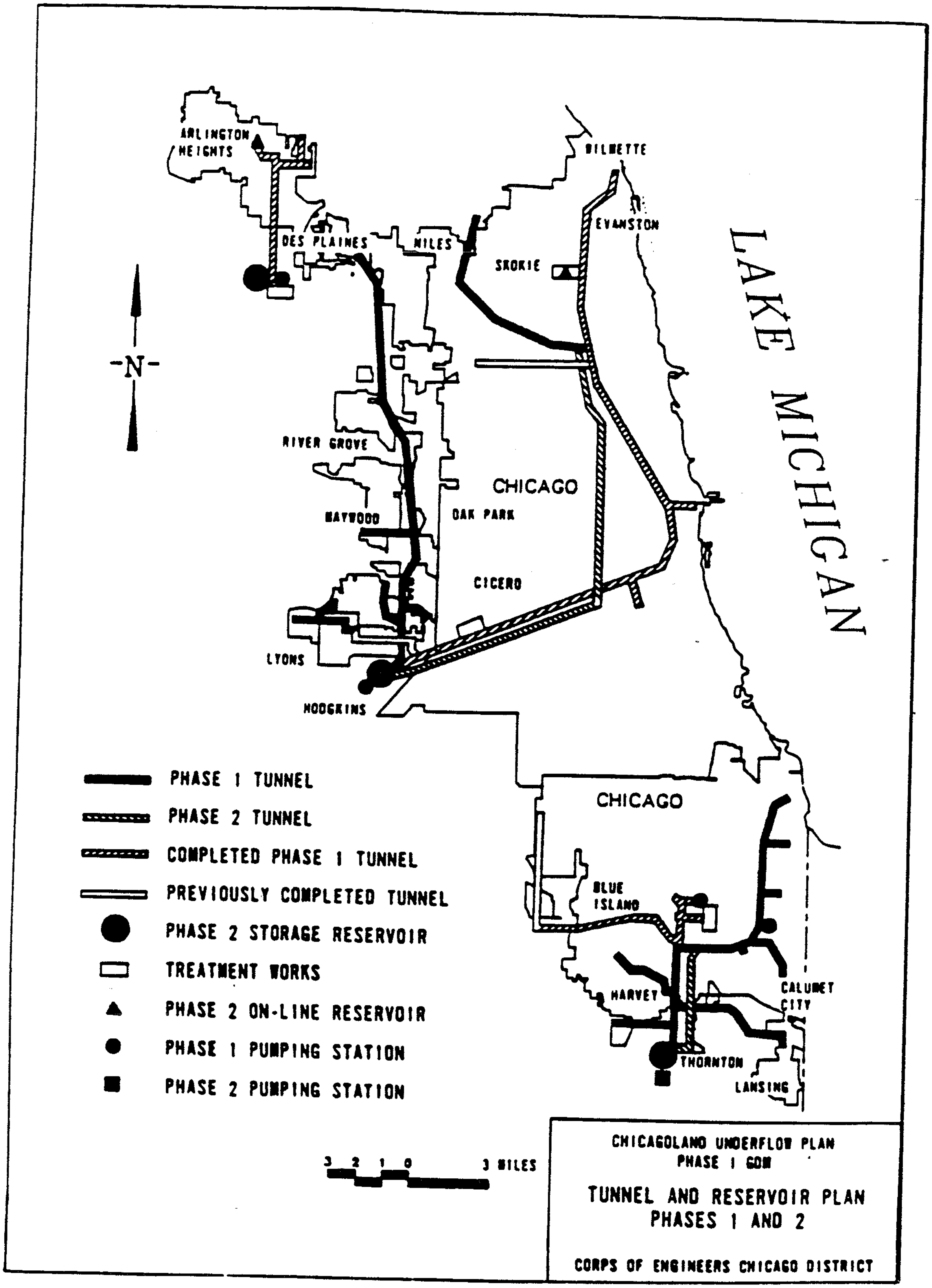

ARLINGTON HEIGHTS
WILMETTE
EVANSTON
DES PLAINES
NILES
SKOKIE
LAKE MICHIGAN
-N-
RIVER GROVE
CHICAGO
MAYWOOD
OAK PARK
CICERO
LYONS
HODGKINS
PHASE 1 TUNNEL
PHASE 2 TUNNEL
COMPLETED PHASE 1 TUNNEL
PREVIOUSLY COMPLETED TUNNEL
PHASE 2 STORAGE RESERVOIR
TREATMENT WORKS
PHASE 2 ON-LINE RESERVOIR
PHASE 1 PUMPING STATION
PHASE 2 PUMPING STATION
CHICAGO
BLUE ISLAND
HARVEY
CALUMET CITY
THORNTON
LANSING
3 2 1 0 3 MILES
CHICAGOLAND UNDERFLOW PLAN
PHASE I GDM
TUNNEL AND RESERVOIR PLAN
PHASES 1 AND 2
CORPS OF ENGINEERS CHICAGO DISTRICT

Figure 1

to and from Lake Michigan. During severe rainfall events, storm runoff is allowed to backflow into the lake to relieve high water levels in the canal system. The Lockport Controlling Works and Powerhouse, downstream of the study area on the Sanitary and Ship Canal, is used to draw down the waterway system to increase the discharge capacity of the canal.

The features of a typical combined sewer system within the study area are illustrated in figure 2. This type of system transports both sanitary waste water and storm water runoff in a single sewer pipe. Sanitary water, foundation drainage, and roof runoff from individual houses are carried by house drains to lateral sewers located in the streets. Stormwater runoff enters the lateral sewers through catch basin drains. Under normal dry weather conditions, sewer flows move from lateral sewers through submains and main sewers into interceptor sewers which convey the flow to a waste treatment plant. When the capacity of an interceptor is exceeded by storm flow, the untreated excess runoff overflows directly into a local watercourse.

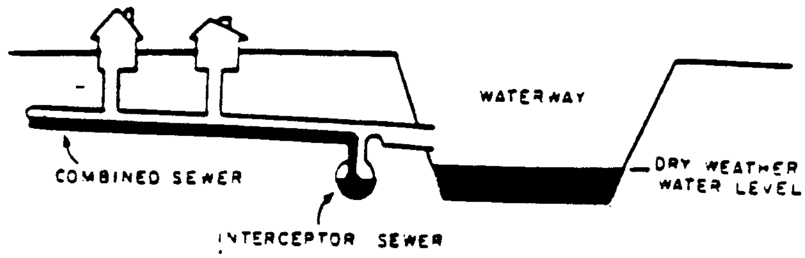
LEVEL OF DETAIL AND MODELING OBJECTIVES

The draft design memorandum for the CUP study will be completed during September 1994. Several of the feature memorandums, including tunnel design, groundwater protection and aeration will be initiated at the beginning of fiscal year 1995. The hydrologic and hydraulic methodologies and modeling used during each study phase have been consistent with the required level of detail. The techniques have also been consistent with satisfying the general objectives, as listed below, for each study phase.

<u>Feasibility Level</u>: The major objectives of the feasibility stage are to first determine if a feasible plan exists, to next determine which feasible plan satisfies the national economic development criteria, and to finally optimize the features of the selected plan. To meet these objectives the emphasis on the hydrologic and hydraulic modeling during the feasibility phase was on benefit computations and project optimization.

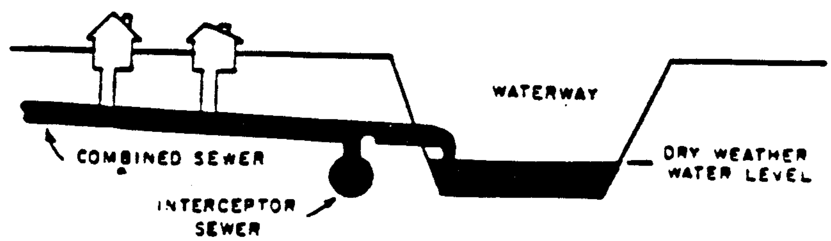
<u>Design Level</u>: The objectives of the design stage are to initially insure that the selected project is technically feasible, and then to determine an accurate cost estimate for the project cooperation agreement. To meet these needs the hydrologic and hydraulic modeling during the design phase was focused on the workability of the project as a whole, and on the design of specific hydraulic components.

<u>Feature Level</u>: Feature Design Memoranda are used to support the plans and specifications for complex portions of projects. Detailed operating plans are also frequently developed during this phase. As such, the hydrologic and hydraulic modeling for this phase of the CUP Study will be focused on, in even greater detail, the workability of specific hydraulic components.



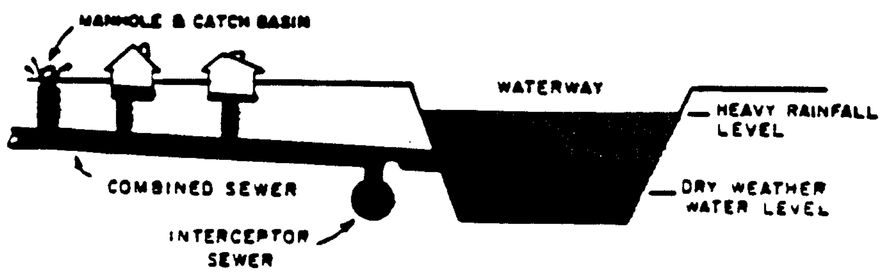
OPERATION OF EXISTING OUTFALL DRY WEATHER CONDITION

Under dry weather conditions, the combined sewer system carries sanitary sewage to treatment plants via interceptor sewers. The system has sufficient capacity to handle dry weather flow without backup into basements or discharge into streams.



OUTFALL IN OPERATION AFTER INTERCEPTOR CAPACITY IS EXCEEDED

At the beginning of a storm period, river levels are low. As rain continues, the sewer system fills up. To relieve pressure in the sewer system, a mixture of storm runoff and sanitary sewage is discharged, untreated, from sewer outfalls into streams.



OPERATION OF EXISTING OUTFALL-HEAVY RAIN CONDITION

During periods of continuing rainfall, river levels rise, submerging the relief outfalls. Pressure then builds up within the sewer system, causing storm water, mixed with raw sewage, to back up from the sewers into basements and streets.

CHICAGOLAND UNDERFLOW PLAN
PHASE I GDM

COMBINED SEWER OUTFALL SUBMERGENCE

CORPS OF ENGINEERS CHICAGO DISTRICT

Figure 2

FEASIBILITY LEVEL

The set of hydrologic and hydraulic models developed for the CUP feasibility study (USACE, Chicago District, 1986) consist of three loosely integrated segments (figure 3).

<u>Watercourse models</u>: The North Branch Chicago River, the Little Calumet River and Des Plaines River were modeled using HEC-1 and HEC-2. The results of these standard hydrologic and hydraulic models are used to compute the effects of the tunnel and reservoir projects on overbank flooding. As indicated below, the watercourse models also provide input into the regional models and pilot area models. Additional details of the watercourse modeling are provided in the feasibility report.

<u>Regional models</u>: These models are used to determine the flows from each of the CUP subareas. These flows are then routed through a synthetic sewer system to an unsteady state model of the canal system. The regional models have the capability of being used to size any downstream tunnel or reservoir project. The only input required from the other models are the downstream flows from the watercourses. An updated set of regional models is described in the next section on the design phase of the CUP Study.

<u>Pilot area models</u>: The pilot area models provide detailed hydraulic analyses of the pilot area sewer systems and the corresponding levels of flooding. This information is translated to non-pilot areas via a regression analysis. The regression analysis not only uses the results of the pilot area models as input, but it also uses the results of the regional models, economic data and basin characteristics as the parametric information to perform the translations. The pilot area models require stage hydrographs from the watercourse and regional models to establish downstream water surface elevations for the detailed hydraulic analyses. The use of the pilot area models in computing basement and street flooding volumes, used in the benefit-cost analysis, is outlined in the following section.

<u>Basement Flooding Computations</u>: The CUP study area is too large to be studied and analyzed in detail. Therefore the approach used to perform the economic analysis was to study the several areas (pilot areas) in detail, compute basement and street flooding volumes for these areas, and then generalize the results for the remaining subareas. Eight representative, or pilot, areas were selected that are hydraulically, physically and economically similar to many subareas within the study area. The complex hydraulic functioning of the pilot areas were studied in extensive detail using the Stormwater Management or SWMM model (US Environmental Protection Agency, 1981). A series of regression equations were then used to translate the flooding information obtained from the pilot area models to the entire 200 CUP subareas. Both the SWMM models and the regression equations were calibrated and verified using basement flooding information obtained from telephone and mail surveys.

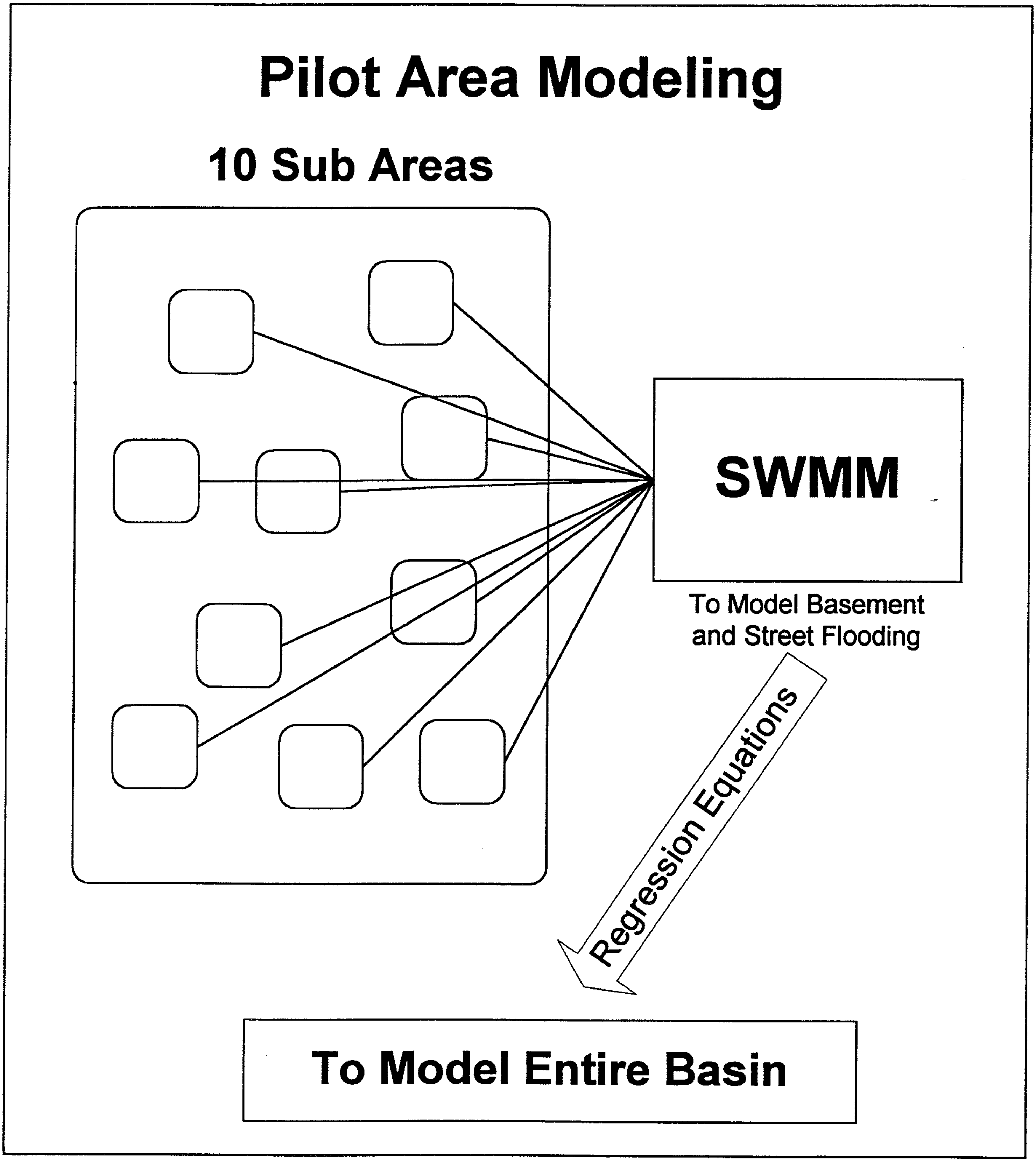

Pilot Area Modeling
10 Sub Areas
SWMM
To Model Basement
and Street Flooding
Regression Equations
To Model Entire Basin

Figure 3

SWMM: SWMM is a comprehensive unsteady flow model that simulates urban runoff quantity and quality for use within either storm or combined sewer systems. For the pilot area analysis the RUNOFF and EXTRAN blocks from the SWMM model were employed. The RUNOFF block generates surface runoff based on rainfall hyetographs, antecedent conditions, land use, and topography. The EXTRAN block routes the flows generated by the RUNOFF block through the modeled sewer system based on the full St. Venant equations and is able to simulate surcharge conditions. The SWMM models of the pilot areas were constructed using the actual physical dimensions of the watersheds and sewer networks. A stage hydrograph of the appropriate receiving water was used as the downstream control for each pilot area model. The downstream stage hydrographs that vary for with and without TARP conditions were obtained from the watercourse and regional models. The SWMM model of each pilot area is divided into smaller sub-basins with at least one "basement segment" of a one to two block subbasin. The basement segments were developed to obtain more detailed information regarding basement and street flooding, and each segment contains storage reservoirs to represent the basement and street storage volumes. To capture the full range of operation of the pilot areas, each pilot area model was run for 2, 6 and 12-hour duration events for 1, 3 and 6-month and 1, 2, 5, 10, 25, 50 and 100-year recurrence interval events.

Regression analysis: The results are generalized via a process of transferring the basement surcharge flooding volumes developed for the pilot areas to the remaining areas within the CUP study area. From these transferred volumes, damage levels are then computed. The procedure used to transfer the surcharges consists of applying two set of regression equations. In the first set of equations the basement flooding volumes, at given frequencies and durations, are used as dependent variables and rainfall depth is used as the independent variable. In the second set of equations the independent variables are descriptive parameters of each subarea (i.e. land use, sewer and basement characteristics) and the dependent variables are the regression coefficients from the first set of equations. This procedure is analogous to the standard COE procedure for determining flood flows on ungaged watersheds (Beard, 1962).

Calibration: Basement flooding is a phenomena that is not directly measurable. To characterize the basement flooding problem in the CUP study area the District undertook two major sampling based data collection studies. The initial study was a mailed survey in which several thousand residence were queried concerning their basement flooding and basement use. The second survey was designed to collect storm specific data for select drainage areas. When a significant storm event occurred telephone calls were made to randomly preselected households and residents were queried relative to their flood depth, incidence and damage experience. Both sets of survey data were then used to calibrate and verify the SWMM pilot area models and the regression analyses.

DESIGN LEVEL

The hydrologic and hydraulic models utilized for the CUP design analysis (USACE, Chicago District, 1994) consists of a set of regional continuous simulation models used to evaluate the functioning of the TARP project for the canal system, the tunnels and the reservoir, and two transient analysis models used to provide preliminary design data for a series of gates, valves and surge control features:

Continuous Period Models: These models (see figure 4) consist of a hydrology model (HSPF), a sewer routing model (SCALP), an unsteady model of the TARP tunnels and reservoirs (TNET), and an unsteady canal system model (UNET).

HSPF: The Hydrological Simulation Program - FORTRAN, (Bicknell, 1992) is a sophisticated model which is capable of analyzing water quality parameters and chemicals, sediment transport, and hydrology and hydraulics. The District uses HSPF for hydrology only. The District has just completed an interagency agreement with the US Geological Survey (USGS) to convert the HSPF program to use the Corps of Engineers time series database, DSS. The USGS contracted with Aqua Terra to produce the product, HSPF-DSS. HSPF begins with segments of land divided into pervious and impervious land segments. These segments of land have areally uniform properties. If the land segment is pervious, it is further divided, from the ground surface downward, into the snow zone, surface zone, upper zone, lower zone, and ground-water zone. The land segments are connected to stream (sewer) reaches. This simulates the hydrologic cycle over and under the land surface. Snow conditions are also considered. There are 11 precipitation gages covering the CUP basin whcih are split into Theissen polygonal areas. Each area has its percentage of pervious and impervious land surface area. The output from HSPF are three unit runoff parameters: pervious surface runoff, pervious subsurface runoff, and impervious surface runoff for each subbasin.

SCALP: The output from HSPF is the input to SCALP, the Special Contributing Area Loading Program (Donald Hey and Associates, undated). The original version of SCALP has been revised by the Chicago District to interface with HSPF and to use the DSS database. The pervious and impervious surface runoff is assumed to reach the sewers through inlets at street level. The pervious subsurface runoff is assumed to infiltrate into the sewers. The amount of water in the sewers at any one time is determined based on land use characteristics, population figures, and seasonal, weekly, and hourly fluctuations. The sewer system is modeled as a main, a submain, and a lateral. The routing method is kinematic wave or storage routing. Flow in the sewers is carried to either the sewage treatment plant (STP) or overflows into CUP tunnels or adjoining rivers or streams. This is determined by a split factor. Any amount of flow above the value of the split factor is considered overflow and is sent to the CUP tunnels. The flow below the split value is considered interceptor flow and is sent to the STP.

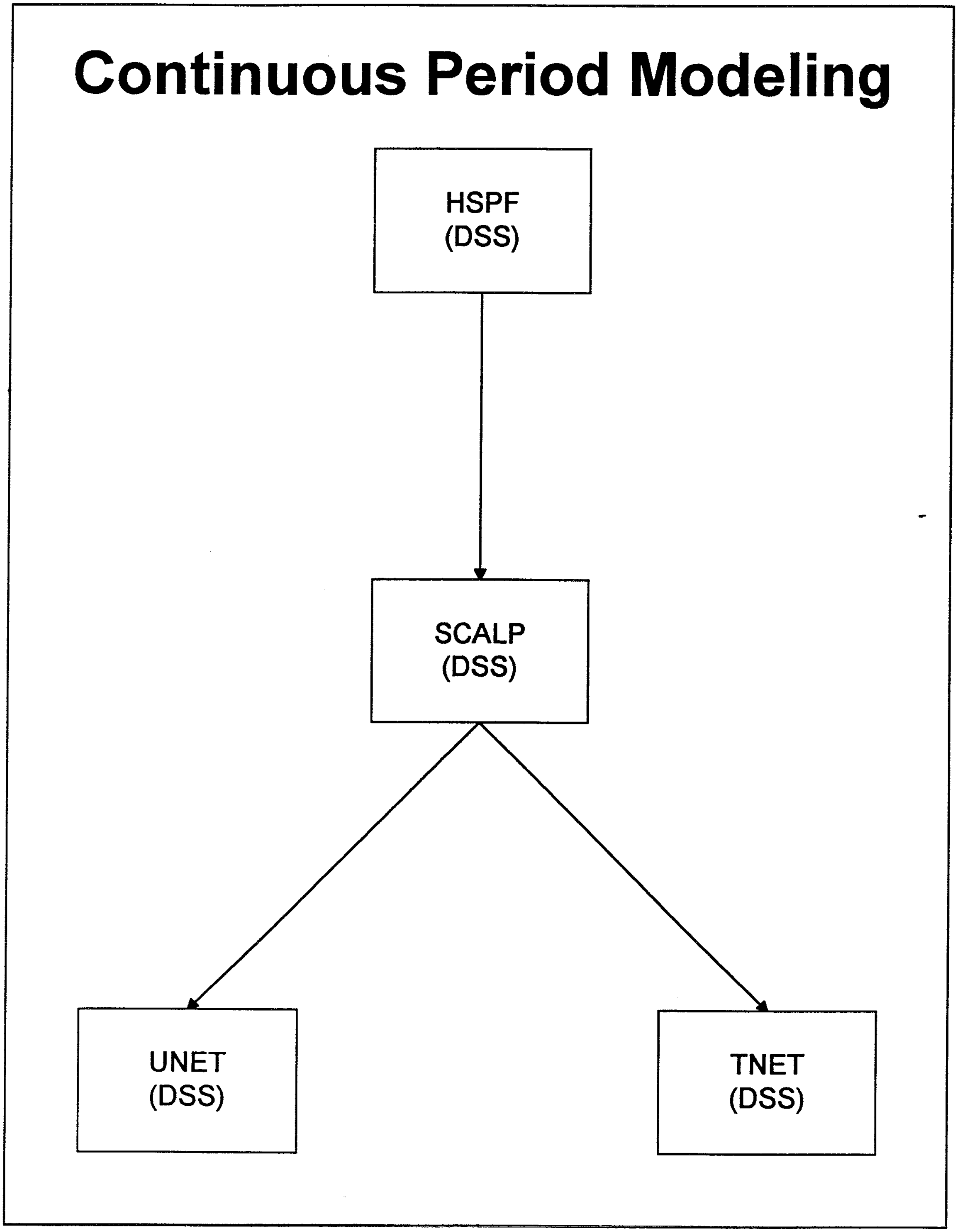

Continuous Period Modeling
HSPF
(DSS)
SCALP
(DSS)
UNET
(DSS)
TNET
(DSS)

Figure 4

In the mainstream and Des Plaines systems of the CUP basin, there are approximately 72 subbasins or special contributing areas (SCAs). These subbasins are related to the Theissen polygonal areas by percentage of area contained within these Theissen polygons. The output from SCALP is an overflow hydrograph for each subbasin and a total STP hydrograph for the entire basin. In the mainstream CUP basin, there are two separate treatment plants, the Northside and Stickney or West Southwest. Therefore, the outputs are split between the two treatment plants, depending on the subbasin's proximity to either of them.

TNET: The Tunnel Network model is an extension of the Unsteady Network flow model UNET (Barkau, 1992), modified for the TARP tunnels and reservoirs. The input to TNET is the SCALP overflows, which TNET splits to the various drop shafts, and the STP flows, which TNET uses to determine when it can pump from the tunnel to the STP or from the reservoir to the STP. TNET incorporates the CUP tunnel system operation, including drop shaft gate operation, STP operation, target elevations for pump operation, pumping rates and target direction for the pumping (to STP or reservoir), main gate operation, and reservoir filling and emptying. TNET is a one-dimensional model, and assumes open channel flow equations apply for all conditions in the tunnel. A Priesmann's slot is used to simulate pressure flow conditions when the tunnel is pressurized. TNET uses implicit finite difference schemes to solve the continuity equation and the momentum equation for unsteady flow. A pilot channel is added in the tunnel for low flow conditions. TNET was used to analyze the entire 40-year period of record for the basin, namely, January 1949 to December 1988. This is its primary strength. TNET uses flow hydrograph input from a DSS database at the point where drop shafts occur in the model. The downstream boundary condition is a storage area simulated with a storage-elevation curve (i.e. the McCook Reservoir). Storage areas at the upstream end of each reach in the model simulate the capacity of drop shafts to act as surge shafts and discharge flow from the tunnel to the canal when the drop shaft gates are not closed. All drop shafts in each reach are simulated as one storage area for this purpose. These storage areas also have storage-elevation curves. The model is currently providing excellent total long-term system operational results.

UNET: As stated above, UNET is an unsteady network flow model. The equation solver and fluid mechanics involved in UNET are the same as those for TNET. UNET uses stream cross sections rather than the circular tunnel cross sections in TNET. Data from TNET are passed on to a UNET model of the canal system which analyzes canal water levels and flows. In addition, modifications to UNET allow for controlled backflow operations to Lake Michigan if canal levels exceed predetermined target control levels. This model is used extensively throughout the Corps of Engineers.

Transient Analysis Models: The transient analysis models used in detail for the CUP design report are the WHAMO and Song models. Additional work has also been done at this study phase using TNET, however the majority of transient work using this model will be done at the feature level and is discussed below.

WHAMO: This model is a general computer program designed to simulate Water Hammer and Mass Oscillation in hydropower and pumping facilities (Camp Dresser & McKee, 1984). The program determines time varying flow and head in a network which may include pipes, valves, pumps, turbines, pump-turbines, surge tanks and junctions arranged in different configurations. The Chicago District has used this model under the direction of the North Pacific Division, Hydroelectric Design Center, to analyze transients in the CUP tunnel system. An extensive series of model runs were done for the distribution system (i.e. side tunnels, valves and gates connecting the main tunnel and reservoir to the pumping station and STP). These analyses resulted in design loadings for valves and gates in that section of the tunnels. A limited number of runs were also done for the mainstream section of the tunnel system to determine preliminary main gate closure speeds. WHAMO assumes a fully pressurized tunnel at all times, and that a steady state condition exists before the transient condition moves through the tunnel. For example, if a pump were to be shut off or turned on, a transient would be set up in a steady state system. Water hammer is observed from the model in the first few time steps (on the order of 10 minutes each, maximum), then a surge (mass oscillation) moves through the system. Such transients are generated due to any variation in the operation of a turbomachine or valve within the network, or due to changes in the head or discharge at boundaries of the network. WHAMO begins with time zero and proceeds with the computation from time step to time step. A complete solution of the system is computed at each time step before proceeding to the next time step. Between each pair of linked nodes in the network, two governing equations can be written in terms of the heads and flows at the nodes. These equations are based on momentum or energy conservation and continuity principles. If the equations are differential equations, a finite difference approximation is formulated, and if non-linear, a linear approximation is formulated. At each time step these equations are determined anew, depending on the properties, operating node (where applicable), and past history of each element. An implicit finite difference technique is used to solve the equations. The WHAMO model of the mainstream CUP tunnel system has a reservoir on the downstream end, but it has a fixed elevation. If set at the bottom of the reservoir, the reservoir is essentially infinite in size. Multiple runs at different reservoir elevations must be done to simulate filling and its effects on the tunnel system. Transient analyses require extremely short time steps to allow calculation of the water hammer effects, so only small windows of time can be simulated. A full period-of-record run would not be possible. However, given the nature of the tunnel distribution system, the WHAMO model proved to be an excellent model for analyzing transients.

Song's model: This is a transient analysis model that uses the equations of conservation of momentum and continuity (Song, 1992). The model solves these equations using implicit finite difference and the method of characteristics. Open channel flows and pipe flows are analyzed separately. In a mixed flow condition, there is an additional boundary condition at the interface of the pipe flow and open channel flow. Dr. Song's model also can consider, to a limited extent, the compressibility of water due to air entrainment in the fluid. This could give results of spill which are greater than those obtained from TNET or WHAMO. The model has been used since the late 1980s to analyze the transient phenomena in the CUP tunnel system.

FEATURE LEVEL

The hydrologic and hydraulic models proposed for use in the feature phase of the CUP study will consist of continuous simulation models, transient analysis models, and a three-dimensional hydrodynamic model.

Continuous Period Models: These are the same models as used for the design phase (i.e. HSPF, SCALP, TNET, and UNET). In the feature phase these models will be used to give continuous simulation results for the design of the various features of the project (for example, the groundwater protection and aeration systems). Additionally, these models will be used to give long term system operational information that will be used in the development of the operating plan.

Transient Analysis Models: In addition to the transient analysis models used in the design phase, the WHAMO and Song models, the results from two additional models will be used in the feature phase (see figure 5). The transient version of TNET, along with an operational model developed by Keifer will be utilized in finalizing the design of the gates, valves and surge control features.

TNET: This unsteady state tunnel network model is being modified and tested to analyze transient phenomena. Of interest are pressures on the main gate due to surging and whether geysering or air blow-off would occur at the drop shafts. In the modifications a mixed (water and air) flow medium may be considered by modifying the bulk modulus of elasticity of the water. Additionally, TNET is a continuous simulation model, and as such it will be able to analyze many scenarios which could lead to problematic transient conditions.

Keifer's model: This model consists of a computer program called KIMRES, developed for MWRDGC (Keifer, 1993). KIMRES is similar to Song's model, but it makes simplifications in the computations. KIMRES has the capability to close drop shaft gates in simulation of MWRDGC's operating plan. Initially, all gates are fully open when the CUP tunnels are only partially full. In a major storm

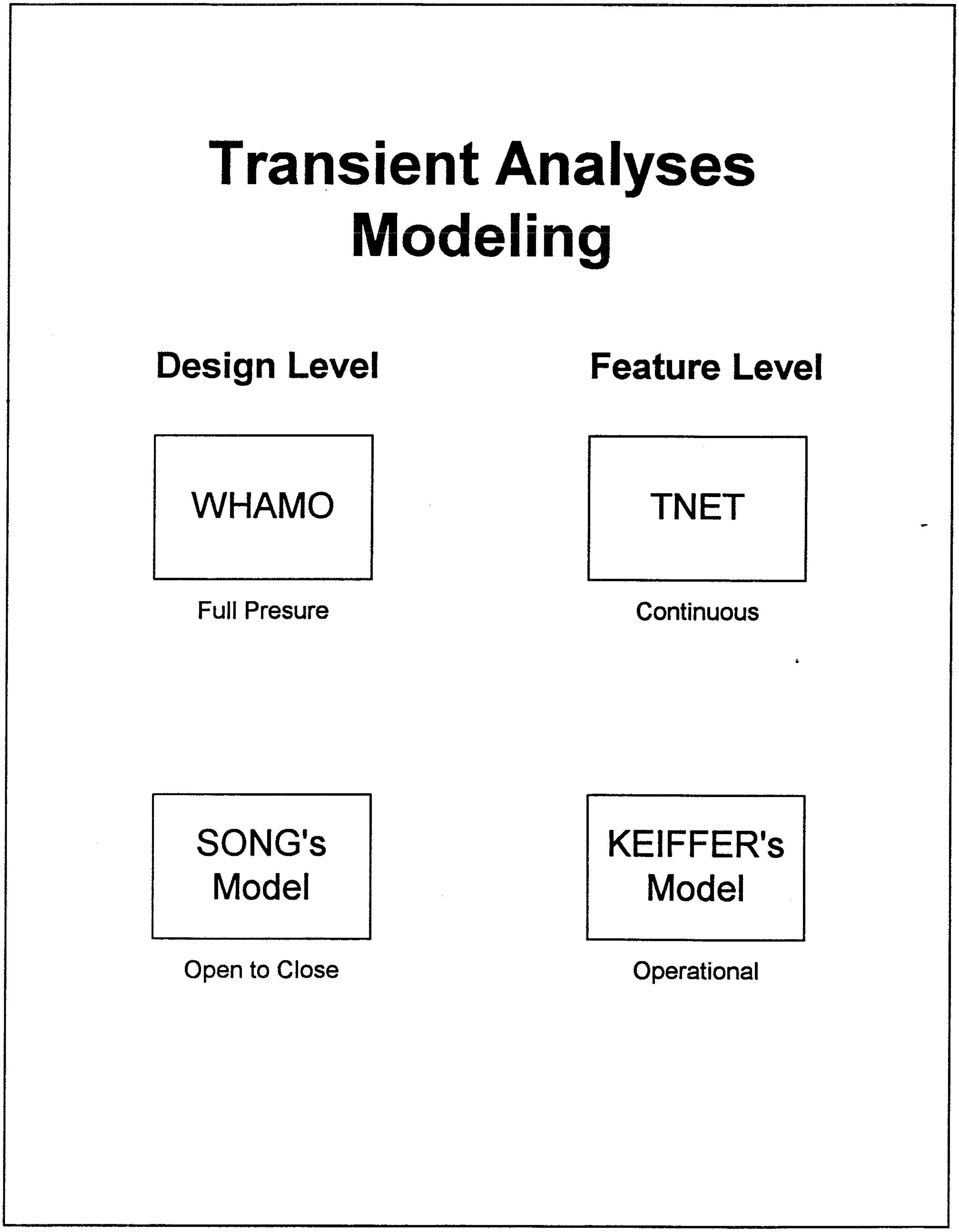

Transient Analyses Modeling
Design Level
Feature Level
WHAMO
Full Presure
TNET
Continuous
SONG's Model
Open to Close
KEIFFER's Model
Operational

Figure 5

covering the mainstream tunnel system watershed, the midsection of the tunnel length fills first. Keifer's initial plan is to control the flow from the large sewer system in the central area until the majority of tunnel is filled. This may control excessive surging at upstream shafts and maximizes upstream inflows into the tunnel system. Controlling the elevation in the waterway so as to reduce backflows at Wilmette (at the upstream end of the system) requires shutting drop shaft gates downstream, while upstream gates would remain open to discharge either to the tunnel or the waterway, whichever is lower. The KIMRES program also computes the waterway levels at the sewer system outfall and the maximum volume and duration of backflow to Lake Michigan, if any, during the storm being run.

3-D Hydrodynamic Model: In support of the McCook Reservoir project, WES has developed a three-dimensional hydrodynamic model (Bernard, 1994). The WES 3-D model utilizes incompressible flow in reservoirs and other deep-water situations where vertical motion and vertical acceleration are both important. Optional features include turbulence modeling, temperature driven buoyancy and passive transport. To run the 3-D Model, the reservoir is divided into composite (multiblock) grids for computational purposes. The 3-D Model assumes that all solid boundaries and free surfaces are impermeable and thermally insulated, so that heat and other passive constituents can enter or leave the flow field only through the inflow/outflow boundaries. The hydrodynamic 3-D Model's main uses are to compute the effects of various aeration systems on the motion of water within the reservoir and to identify areas within the reservoir where minimal or no mixing occurs. The model may also be used to evaluate flows and velocities in the region of the tunnel outlet works (a four-barreled manifold) into the reservoir. The basic hydrodynamic procedures for the WES 3-D Model are complete. Additional work that is currently being undertaken includes adding procedures to the model that will compute the effects of oxygen transfer and kinetic degradation. Options to field test the model and the proposed diffuser system are also being expolored

SUMMARY

The Chicago District has developed a series of methodologies that can be used to perform urban hydrology and hydraulics for a combined sewer system with deep tunnels and terminal reservoirs. The pilot area methodology and models can be used to determine flood damages reduction benefits for the feasibility level. In the design phase the continuous simulation models can be used to evaluate various design features and to develop an operating plan for the TARP system. The transient models and 3-D hydrodynamic model can be used to finalize the design in the feature phase of the study. The District has been able to finalize these methodologies by expanding the use of many of the models through enhancements and utilization of the DSS database. Throughout the study the District has maintained a great deal of flexibility in model selection and utilization.

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IMPROVING UPON CONVENTIONAL STORM WATER DETENTION ORDINANCES

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ABSTRACT

Ordinances designed to control post-development stormwater impacts are common among many municipalities. The requirement of detention storage to maintain the post-development peak flow of a given return period to no greater than the corresponding pre-development peak flow is often used as a means to achieve this objective. In many instances, however, this requirement is insufficient for maintaining downstream pre-development flood elevations under post-development conditions due to the increased discharge volume over the period of time critical to peak flood stage conditions from the post-development hydrograph.

An alternative (and more effective) method of maintaining pre-development downstream flood elevations under post-development conditions is to require the volume from the controlled post-development hydrograph over the period of time critical to peak flood elevations in downstream reaches to be no greater than the volume from the pre-development hydrograph over the same critical time period. The critical time period varies from basin to basin. The determination of the time period requires a comprehensive, basin-wide study.

This paper contains two examples of detention facilities designed to meet a volumetric discharge criterion. These two facilities are compared to similar facilities designed under a conventional peak discharge criterion. Performance of the volumetric and peak discharge criteria on an example basin is also presented.

INTRODUCTION

Urbanization of watersheds results in increased stormwater flows, primarily because of the increased levels of impervious area. Increased stormwater flows result in increased flood damages and erosion problems (1). One measure that is typically taken to control these adverse impacts is to require (e.g., through an ordinance) that the post-development peak flow be no greater than the pre-development peak flow for a given storm event (2). In many cases, this criterion is best achieved by use of detention storage. The peak flow criterion is a presumptive criterion in that it is presumed that if peak flows are controlled, then peak stages will be controlled. However, many communities have experienced flooding problems due to development, even with this requirement in place. Modeling studies have also verified the inadequacy of this requirement at controlling downstream impacts due to hydrograph timing, duration, increased volume, and dynamic channel routing effects.

Examination of several hydrographs from a large basin may illustrate the inadequacy of controlling peak flows in order to control peak stages. Figure 1 shows three hydrographs from a basin that is approximately 20 square miles. The two tallest hydrographs are the flow and stage hydrographs near the mouth of the basin from a 100-year synthetic rainfall event (essentially, a 24-hour SCS Type III distribution). The shortest hydrograph is a flow hydrograph from a 200 acre sub-basin within the basin. Two features are important to note on Figure 1. First, there is an extended period of time during which the stage hydrograph at the downstream end of the basin is at peak or near-peak conditions. This period of time plus the period of time where the stage hydrograph is rising sharply will be

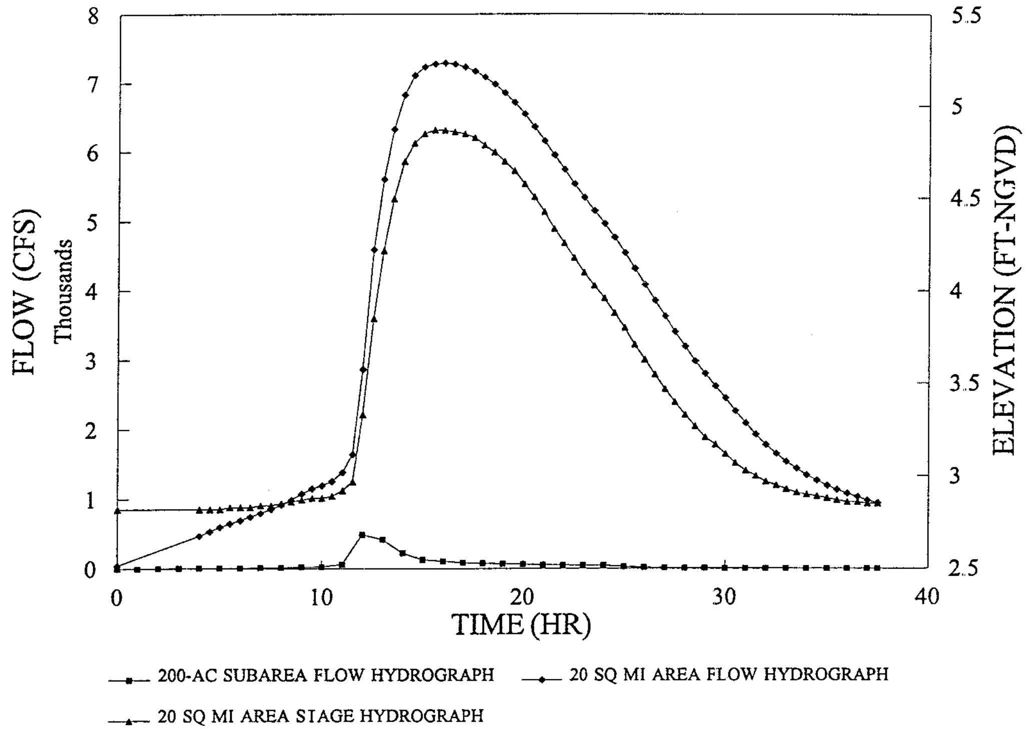

Figure 1. Response to synthetic 24-hour, 100-year rain event.

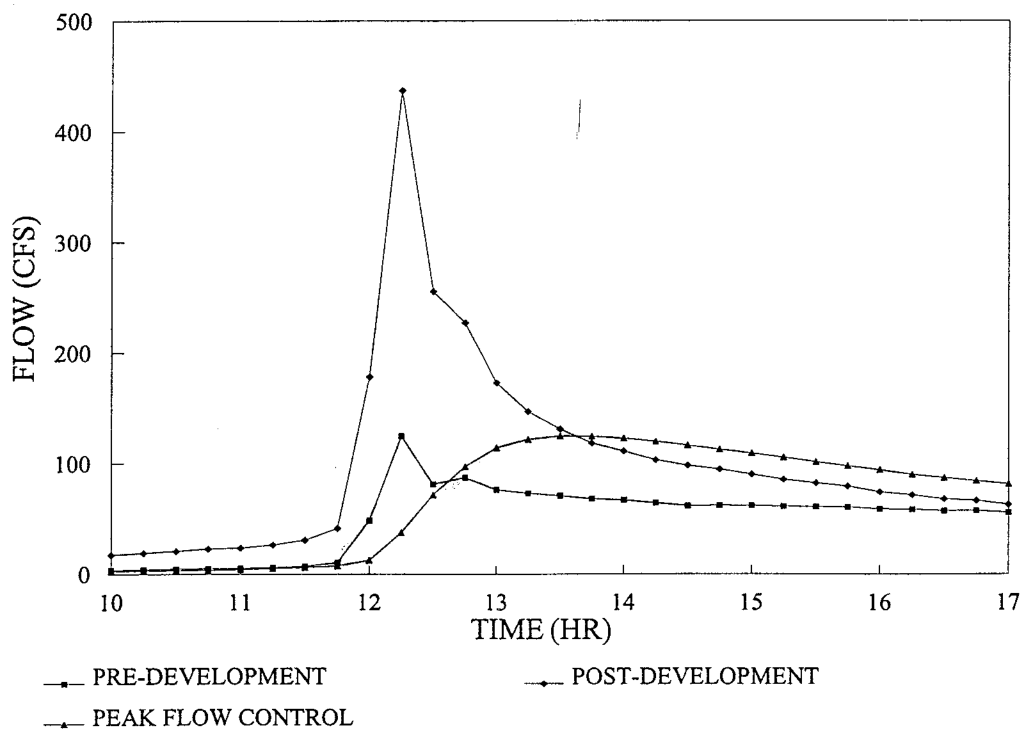

Figure 2. Comparison of hydrographs over critical time period.

referred to as the "critical time period". Second, the total flow volume discharged over the critical time period from the 200-acre sub-basin will have a greater effect on the downstream stage hydrograph than will the peak flow from the 200-acre sub-basin.

Three hydrographs from the 200-acre sub-basin are shown in Figure 2 for the critical time period, which, for this example, is defined as hour 10 to hour 17 of the 24-hour storm event. The tallest hydrograph is the post-development hydrograph with no detention. The earlier peaking of the two other hydrographs is the pre-development hydrograph. The third hydrograph is the post-development hydrograph from a peak flow control detention pond. Basing the effectiveness of the detention pond on the volume of flow discharged over the critical time period, the hydrographs in Figure 2 illustrate that the detention pond will provide some reduction in downstream peak stage over providing no controls, but that some increases in downstream peak flood stages over pre-development conditions will occur because a greater volume is discharged over the critical time period.

STUDY SITES

Calculations for the design of two detention facilities designed to meet the volumetric discharge criterion are given in this paper. The purpose of presenting the calculations is to show that designing a detention pond under the volumetric discharge criterion is not significantly more difficult than designing a detention pond under the peak discharge criterion. Characteristics of the two areas for which these two facilities are designed are shown in Table 1. The two areas differ only in size--one is 200 acres and the other is 10 acres.

Performance of the volumetric discharge criterion and the peak flow criterion in an example basin is also presented in this paper. The example basin is West Branch, which is shown in Figure 3 (3). The West Branch basin was subdivided into four subareas. Characteristics of these four subareas are shown in Table 2. In order to evaluate the performance of the criteria on West Branch, it was necessary to predict peak flood stages along the primary stormwater management system (PSWMS). The junctions in Figure 4 show the locations where peak flood stages were predicted along the PSWMS.

Table 1 - Example Design: Area Characteristics

Size	10 acres and 200 acres
Predominant Pre-Development Land Use	95% forest/open
Pre-Development Imperviousness	6%
Predominant Post-Development Land Use	86% medium density residential
Post-Development Imperviousness	27%
Soils	Hydrologic Class C

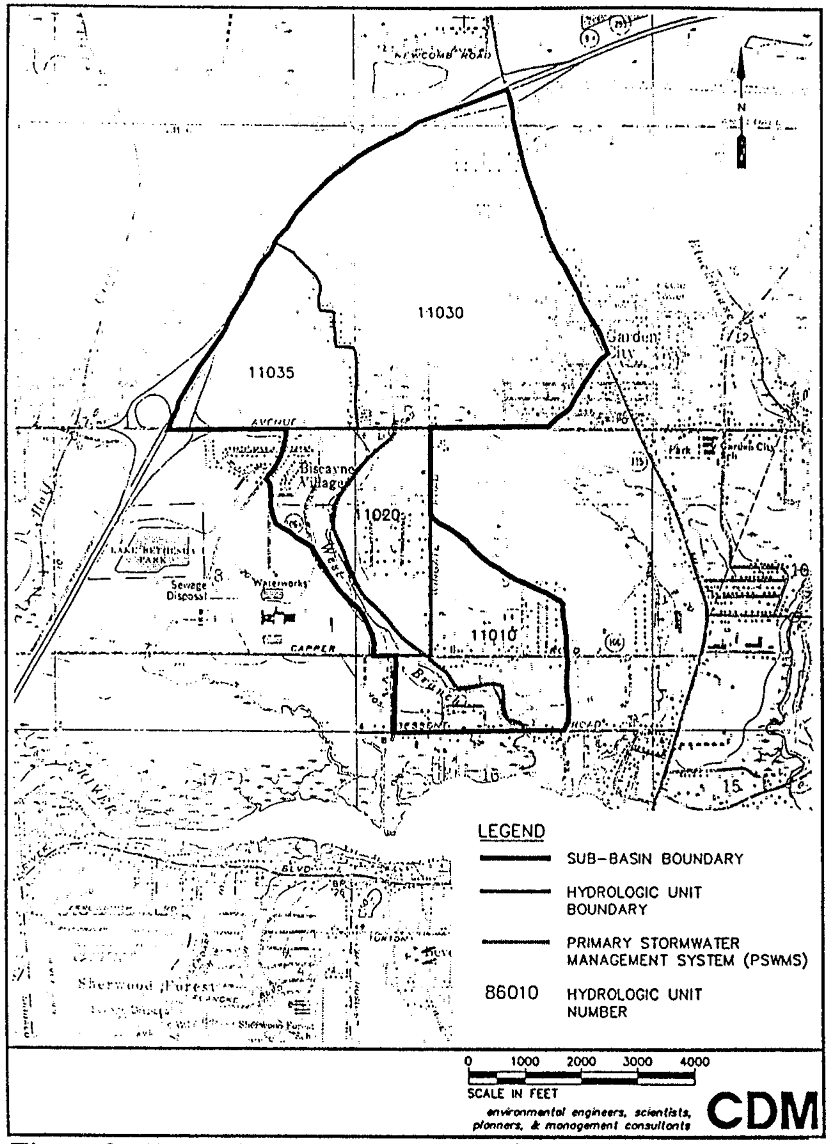

Figure 3. Example basin: West Branch.

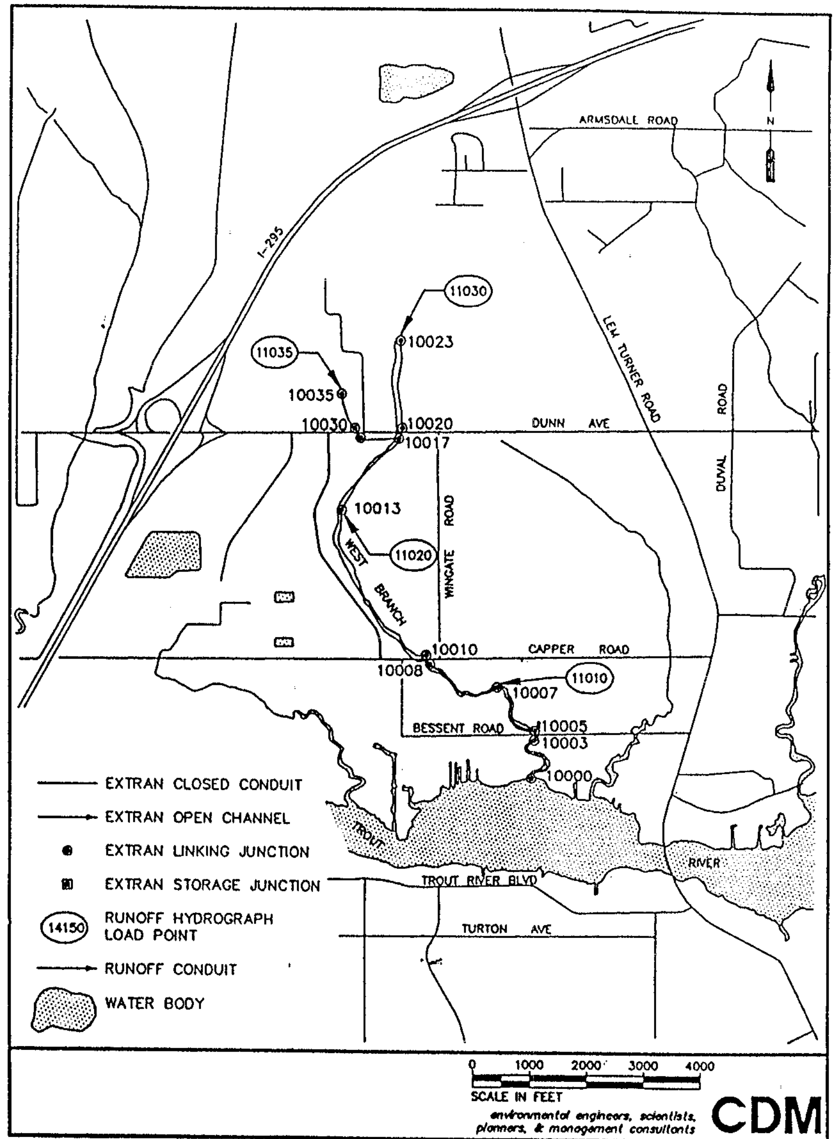

Figure 4. West Branch model schematic.

Table 2 - Chracteristics of the West Branch Subareas

Subarea	Area (acres)	Overland Slope (ft/ft)	Soils	Pre-Dev. Imp	Post-Dev. Imp
11010	173	0.065	C and D	9%	29%
11020	190	0.0100	C and D	8%	28%
11030	503	0.0028	D	21%	41%
11035	150	0.0037	D	6%	56%

METHODOLOGY

In order to properly evaluate the performance of the two criteria, the use of a model with dynamic flow routing capabilities was necessary. A modified version of the EXTRAN block of the Environmental Protection Agency (EPA) Stormwater Management Model (SWMM) was used for the hydraulic simulation portion of this study (4). A modified version of the RUNOFF block of SWMM was used for the hydrologic portion of this study (5). The most significant modification of RUNOFF for this study was the addition of a lake routing routine. The routine is based on the storage-indication method.

One important aspect to any detention ordinance is implementability. For a detention ordinance to implementable, the the detention ponds must not be too difficult to design or review. The following steps explain how detention ponds are designed for two areas under the volumetric discharge control:

- Establish hydrologic parameters
- Calculate runoff hydrographs under pre-post-development conditions
- Calculate the difference between the pre-and post-development runoff volumes over the critical time period (hour 10 to hour 17 for this example), and multiply this difference by 1.5 as a first guess of the detention pond size
- Determine the allowable depth in the pond (1 meter for this example) 3ft
- Vary the pond volume and outlet size until the peak pond depth is equal to the allowable depth and the volume restriction is satisfied. A V-notch weir is recommended for combined volume/peak and water quality control structures.

RESULTS

Table 3 summarizes the pre- and post-development hydrographs over the critical time period for the two example areas. As shown in Table 3, the goal for the 10-acre area is to design a pond that discharges no more than 4.1 acre-feet over the critical time period under post-development conditions. Likewise, the goal for the 200-acre area is to design a pond that discharges no more than 51.6 acre-feet over the critical time period under post-development conditions. Table 4 shows the iterations that were required to design the two example ponds. Table 5 shows a comparison of the two example ponds using the volumetric discharge criterion as a basis for design and using the peak discharge criterion as a basis for design. Figure 5 illustrates the pre-development, post-development with no control, the post-development with peak flow control, and the post-development with volumetric discharge control hydrographs over the critical time period.

Table 3 - 100-Year Flows For Critical Time Period

	10-acre		200-acre	
	Pre	Post	Pre	Post
Total Volume (ac-ft)	4.1	5.2	51.6	88.3
Peak Flow (cfs)	21.8	49.2	184.0	586.0

Table 4 - Iterations For Pond Sizes and Outlet Capacities

Iteration	Area	Volume at 3 ft (ac/ft)	V-notch Weir Angle (degrees)	Depth (ft)	Critical Outflow Volume (ac/ft)	Peak Outflow (cfs)	Comments
1	10-ac	1.64	15.0	3.32	4.7	22.8	inc vol, dec weir
2	10-ac	2.56	13.0	3.03	4.2	16.0	inc vol, dec weir
3	10-ac	2.58	12.9	3.00	4.1	15.8	okay
1	200-ac	55.0	100.0	2.92	49.3	148.0	dec vol, dec weir
2	200-ac	48.5	95.0	3.02	53.1	159.0	inc vol, inc weir
3	200-ac	50.3	96.0	3.00	51.8	155.0	inc vol, dec weir
4	200-ac	50.5	95.5	3.00	51.6	154.0	okay

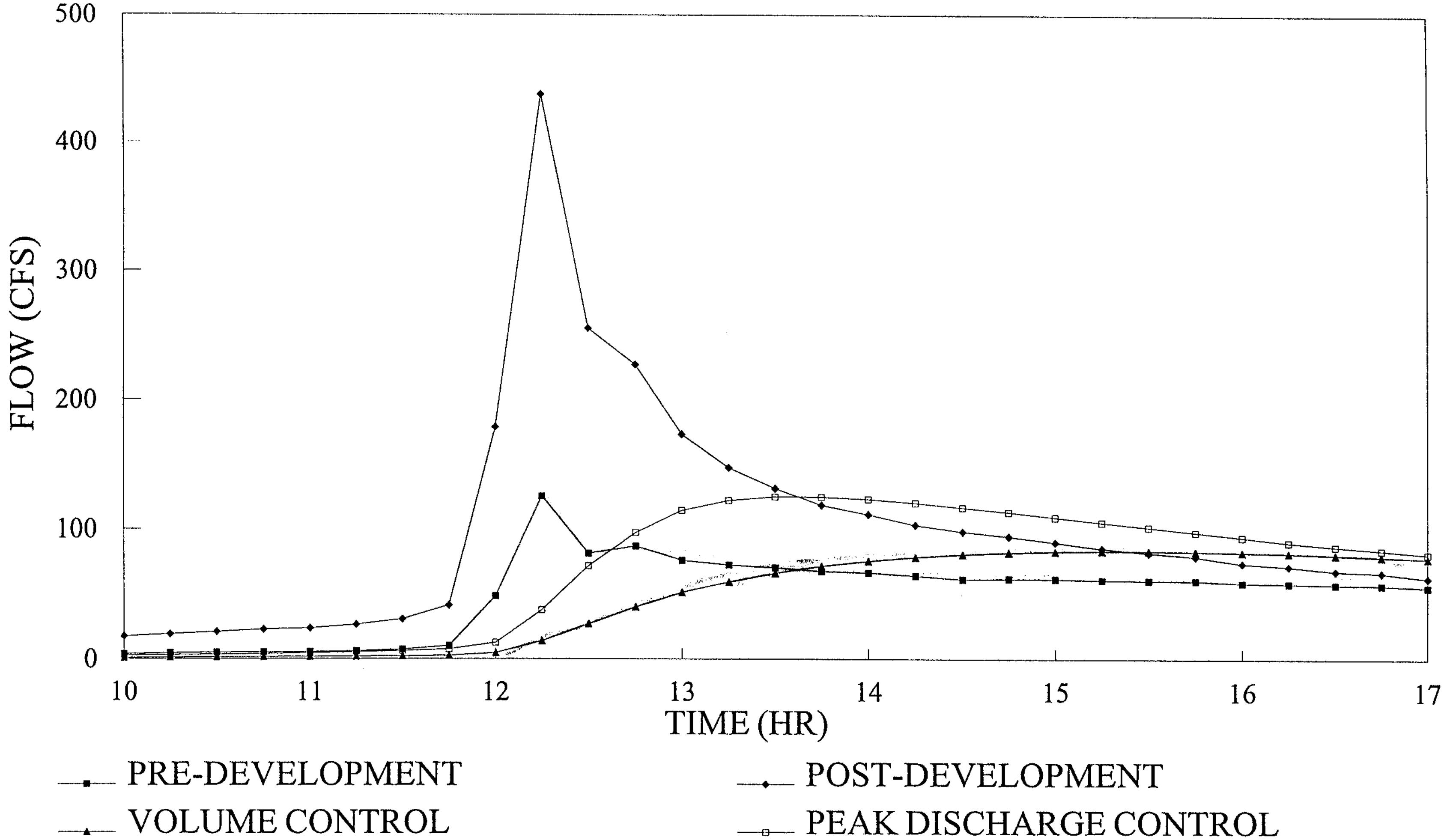

Figure 5. Comparison of hydrographs under various controls.

Table 5 - Pond Design Under Volumetric and Peak Flow Criteria

Area	Pond Volume (ac-ft))	Volumetric Control: Pond Surface Area (ac)	Volumetric Control: V-notch Weir Angle (degrees)	Volumetric Control: Percent of Land Area	Peak Flow Control: Pond Volume (ac-ft)	Peak Flow Control: Pond Surface Area (ac)	Peak Flow Control: V-notch Weir Angle (degrees)	Peak Flow Control: Percent of Land Area
10-ac	2.58	0.97	45.5	9.7%	2.09	0.79	18.3	7.9%
200-ac	50.5	17.3	152.0	8.7%	37.2	12.80	115.0	6.4%

As stated previously, the two detention criteria were applied to the West Branch basin in order to examine their performance at minimizing increases in downstream flood stages. A summary of peak flood stages under pre-development, post-development with no control, the post-development with peak flow control, and post-development with volumetric discharge control conditions is given in Table 6. Note that the junctions numbers in Table 6 correspond to the junction numbers in Figure 4.

Table 6 - West Branch: 100-Year Flood Level Comparison

Node	Undeveloped (ft)	Developed With No Controls (ft)	Developed With Volumetric Controls (ft)	Developed With Peak Controls (ft)
10000	2.9	2.9 (0.0)	2.9 (0.0)	2.9 (0.0)
10003	3.0	3.2 (0.2)	3.0 (0.0)	3.1 (0.1)
10005	3.3	3.8 (0.5)	3.3 (0.0)	3.5 (0.2)
10007	4.6	5.5 (0.9)	4.6 (0.0)	5.0 (0.4)
10008	5.3	6.4 (1.1)	5.3 (0.0)	5.8 (0.5)
10010	5.4	6.8 (1.4)	5.4 (0.0)	6.0 (0.6)
10013	10.2	11.3 (1.0)	10.2 (0.0)	10.7 (0.5)
10017	11.3	11.9 (0.7)	11.2 (0.0)	11.5 (0.3)
10020	12.3	14.5 (2.2)	12.1(-0.1)	13.2 (0.9)
10023	15.0	15.6 (0.5)	14.5 (-0.6)	15.1 (0.1)
10030	13.3	17.0 (3.7)	13.3 (0.0)	13.4 (0.1)
10035	15.1	17.5 (2.4)	15.0(-0.1)	15.1 (0.0)

( ) - change from undeveloped

DISCUSSION

The design of the detention ponds for the two example areas illustrates two points. First, the design process under the volumetric discharge criterion is not significantly more complicated than the design process under the peak flow criterion. Second, the size of the ponds and the amount of area required under the volumetric discharge criterion is reasonable when compared to the size and amount of area required under the peak flow criterion.

The West Branch basin example illustrates that the volumetric discharge criterion is more effective at controlling the downstream impacts of development, which is part of the intent of most detention ordinances. Conventional methods of controlling downstream flooding that are based on controlling peak flows may not provide the desired level of control.

The bleed-down times for the detention ponds designed under the volumetric discharge criterion are on the order of a half-day to one day longer than those designed under the peak discharge criterion. This extended detention time coincides well with the longer detention times required for water quality control. The benefit of this is that the bleed-down volume required for water quality control can be incorporated into the flood control volume.

The effectiveness of this volumetric discharge criterion is limited to basins where the use of a 24-hour design storm is appropriate. For large riverine systems, this volumetric detention criterion, like most any detention criterion, will likely be ineffective at controlling peak stages where the contributory drainage area is very large.

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Simulation of Open Channel and Pressure Flow using the One-Dimension Open Channel Flow Equations

by Robert L. Barkau, Ph.D., P.E.[1]

The unsteady flow equations for one-dimensional open channel flow can be "tricked" into simulating pressure flow in conduits through the use of the Preismann slot (Cunge, 1982). Figure 1 shows a cross-section of a circular conduit with a narrow slot. The width of the slot is set such that the inherent celerity of the open channel flow equations equals the celerity of a pressure wave, about 4,900 feet per second. The Preismann slot allows the open channel flow equations to simulate both open channel and pressure flow in conduit.

Equations of the Open Channel and Water Hammer Flow

The equations of unsteady, one-dimensional, open channel flow are

Continuity-

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{1}$$

Momentum-

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2 / A)}{\partial x} + gA\left[\frac{\partial h}{\partial x} - S_o + S_f\right] = 0 \tag{2}$$

where: x = distance along the channel.
t = time.
Q = flow.
A = cross-sectional area.
h = depth.
g = acceleration of gravity
S_o = bed slope.
S_f = friction slope.

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The water hammer equations are (Wylie, 1978),

Continuity -

$$\frac{\partial H}{\partial t}+V\frac{\partial H}{\partial x}-V\sin\alpha+\frac{a^2}{g}\frac{\partial V}{\partial x}=0 \tag{3}$$

Momentum -

$$\frac{\partial V}{\partial t}+V\frac{\partial V}{\partial x}+g\frac{\partial H}{\partial x}+S_f=0 \tag{4}$$

where: H = the elevation of the hydraulic gradeline.
V = the velocity.
a = the celerity of a pressure wave.
α = the angle of the pipe from the horizontal.

By manipulating equations 1 and 2, the following equations are obtained:

Continuity-

$$\frac{\partial z}{\partial t}+V\frac{\partial z}{\partial x}-V\sin\alpha+\frac{c^2}{g}\frac{\partial V}{\partial x}=0$$

Momentum-

$$\frac{\partial V}{\partial t}+V\frac{\partial V}{\partial x}+g\frac{\partial z}{\partial x}+S=0$$

where: z = the elevation of the hydraulic grade line.
$c=\sqrt{\frac{gA}{B}}$ = the celerity of a gravity for a non-rectangular channel.
B = topwidth.

The open channel and the water hammer equations are identical if the wave celerities are equivalent, $c = a$. The wave celerities for pressure flow through a pipe with rigid walls is given by

$$a = \sqrt{\frac{gK}{\gamma}}$$

where: K = bulk modulus of water.
γ = unit weight of water.

Hence, the wave celerities are equal if the topwidth of the slot is equal to

$$B = \frac{A\gamma}{K}$$

Demonstration of Open Channel and Pressure Flow

The function of the open channel and pressure flow routing function of UNET are demonstrated on the Mainstem Tunnel of the Chicago TARP tunnels. The segment of the tunnel that was modeled extends from Willamette, which is north of the downtown area, to the main pumping station near the Stickney Water Treatment Plant. The total length of this segment is about 148,000 feet and the diameter of the circular cross-section varies from 22 feet at the upstream end to 33 feet at the downstream end. For the test the branch tunnels were ignored.

The first test simulates open channel and pressure flow. The inflow hydrograph at Willamette is a triangular hydrograph with a crest of 3,500 cfs. The downstream stage is held constant, which initially pressurizes the lower 60,000 feet of the tunnel. The upper portion has a free water surface. Figure 2 shows the routing of the triangular hydrograph at three cross-sections. The first hydrograph is at the upstream end and is the inflow hydrograph. The second hydrograph, which is 100,000 feet downstream, is lagged and attenuated by the open channel flow routing. The third hydrograph is 27,000 feet further downstream. This hydrograph exactly coincides with the second hydrograph because the tunnel is pressurized through this region and the wave celerity is 4,900 feet per second.

The second test simulates the effect of a pressurized tunnel. The downstream elevation is held at a constant 500 feet; therefore, the entire length of the tunnel is pressurized. The triangular hydrograph from the earlier test is once again input. As shown in Figure 3, the routed hydrograph 127,000 feet downstream exactly coincides with the input hydrograph. Once again, the wave celerity of 4,900 feet per second is demonstrated.

The coincidence brings up another point; the linearized, implicit finite difference scheme is not dissipating the hydrographs in spite of the fact that the implicit weighting factor (theta) has a value of 1.0 and the time step is 5 minutes.

The Sewer Network

To demonstrate the ability of the model to simulate the components in a sewer network, the network of sewers and open channels in Figure 4 was developed. The network consists to two systems - a subsurface network of circular conduits and a surface network of open channels open channels. The subsurface network consists of 11 reaches. The reaches join at storage cells which mimic manholes. Below the ground, the cells are cylindrical with a diameter of 6 feet. Above the ground, the cell enlarges in surface area to 1/3 acre in three feet, which simulates a ponding area above a storm drain. The subsurface network drains into the detention basin at cell 10 which overflows into an underground cave which is given by cell 11.

The surface model simulates the roads above a portion of the subsurface network. The surface model consists of four reaches and one cell. The reaches are shown as dotted lines on Figure 4. The cell, No. 12, is a the junction of reaches 12, 13, 14. The entire surface network drains toward cell 12 which drains into cell 5 entering the subsurface network. The surface network is also connected to the subsurface network at cell 1, 2, and 4. These connections simulate the overflow from the subsurface network onto the roads above.

Triangular hydrographs with crests of 20 cfs were input at cells 1, 2, 3, 4, 5, 6, 7, 8, and 9. The computation interval was 1 minute. Figures 5, 6, 7, 8, and 9 show the flow and stage hydrographs in reaches 2, 3, 4, and 5. The conduits were initially in open channel flow. As the event progressed the conduits pressurize and later return to open channel flow. Cell 4 overflow returns to the subsurface model. Figure 10 and 11 show the overflow hydrographs. Figure 12 shows the stage hydrograph for cell 3 and Figure 13 shows the stage hydrograph for cell 12 which is at the intersection of reaches 12, 13, and 14.

Conclusions

The open channel flow equations can simulate the routing of gravity and pressure waves - the latter through the use of the Preismann slot. Test on the Chicago Tunnel system show that the routing of pressure wave over long distances is nearly instantaneous and that the numerical scheme used to solve the unsteady flow equations does not dissipate the waves even at longer time steps. The simple sewer network also demonstrates that the UNET program can be used to simulate simple sewer network, as long as the flow is subcritical. The programs needs additional work on manholes, supercritical flow reaches, low flow, and dry nodes.

References

Barkau, Robert L., 1992. "UNET: Unsteady Flow through a Full Network of Open Channels." Program Documentation, St. Louis, Missouri.

Cunge, J. A., 1982. *Practical Aspects of Computational River Hydraulics.* Pitman Publishing Co., London.

Wylie, B. A., 1978. *Hydraulic Transients.* J. W. Wylie publishing Co., New York.

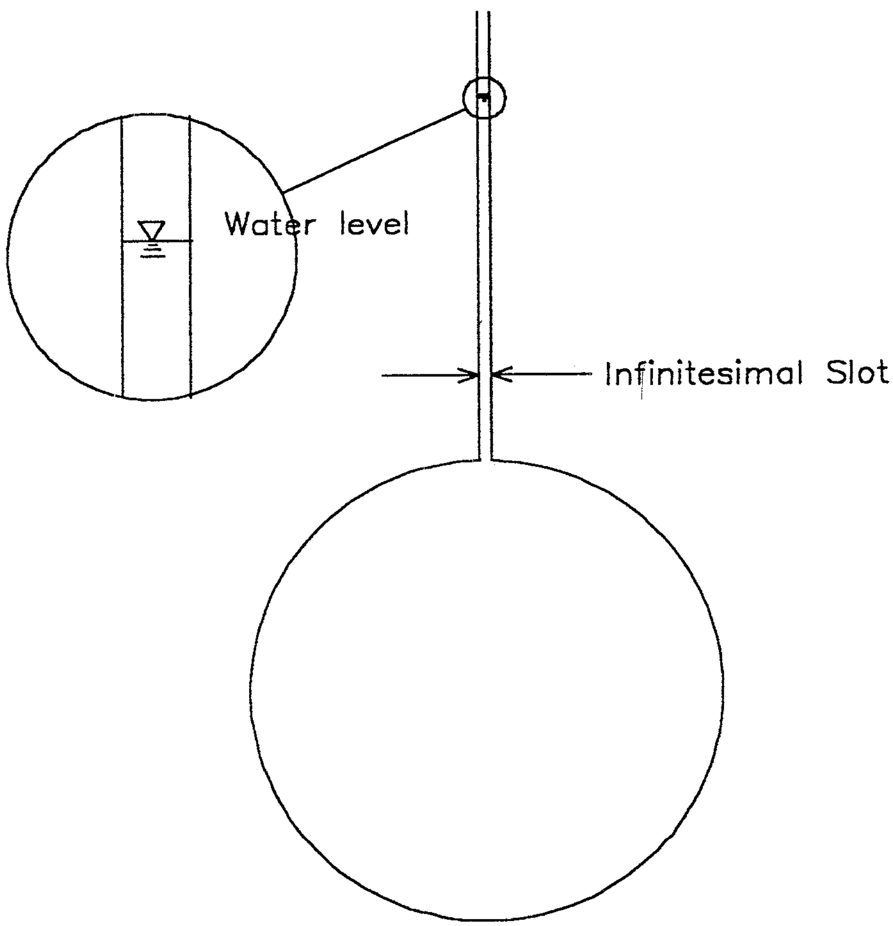

Figure 1. Circular conduit with Preismann slot.

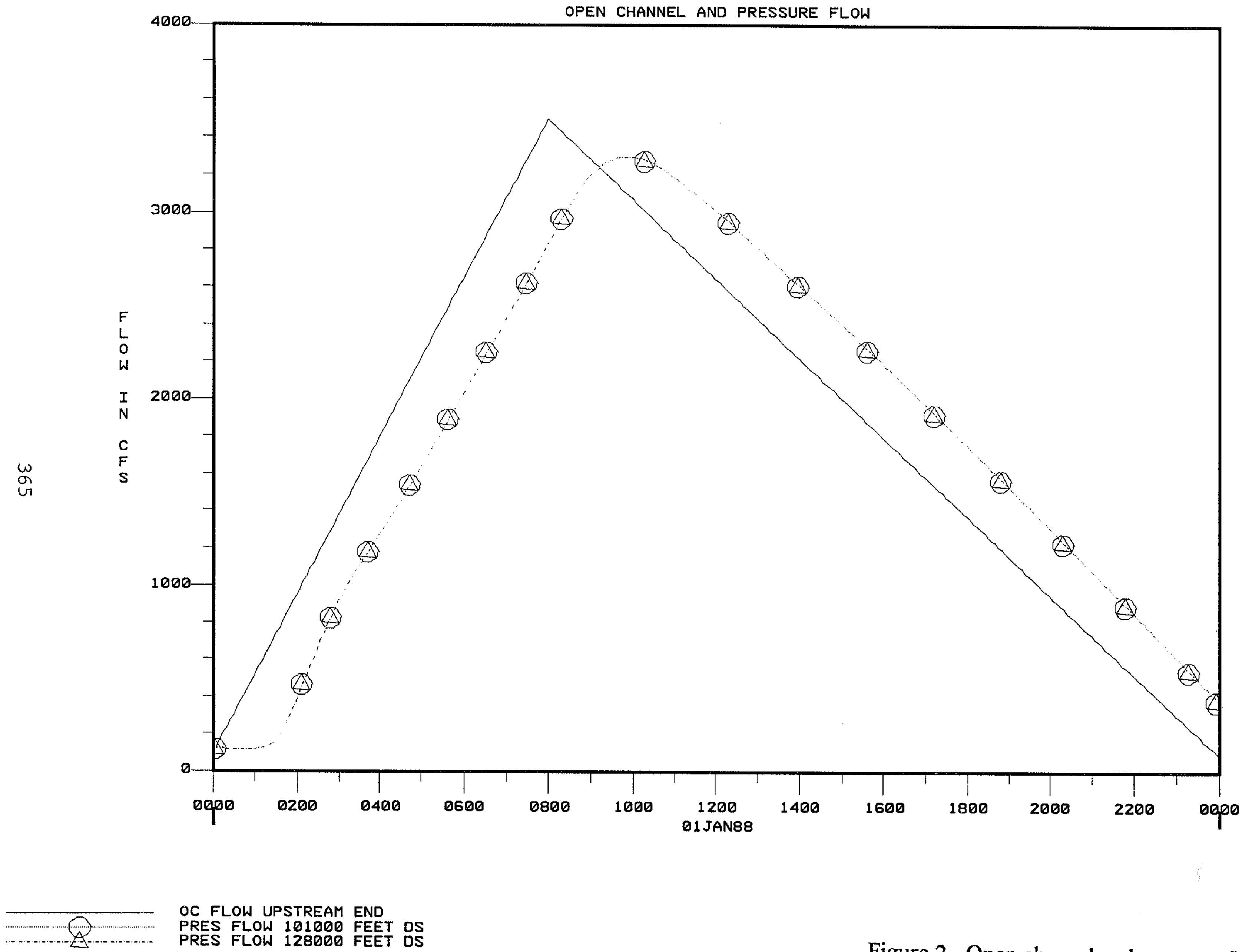

Figure 2. Open channel and pressure flow.

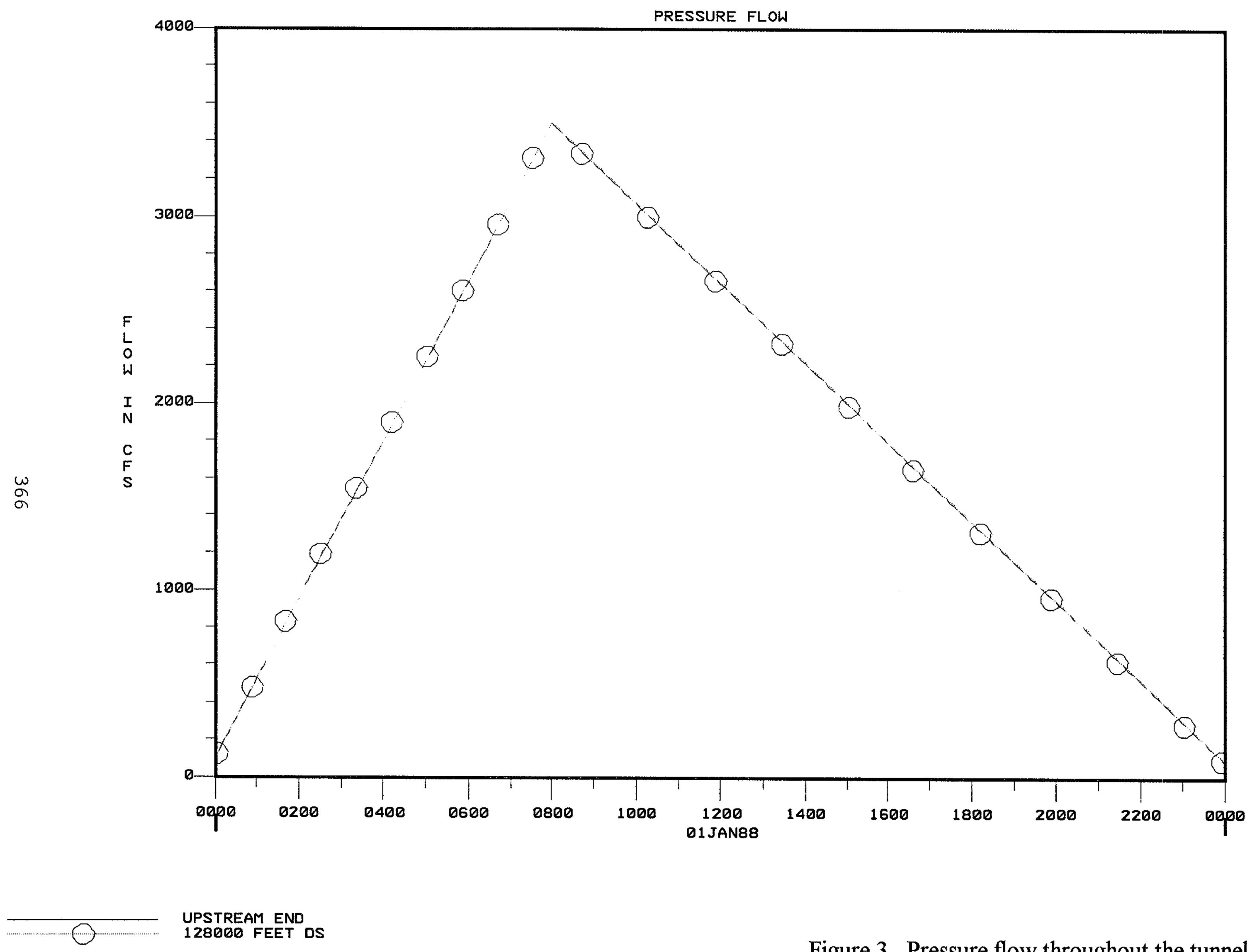

Figure 3. Pressure flow throughout the tunnel.

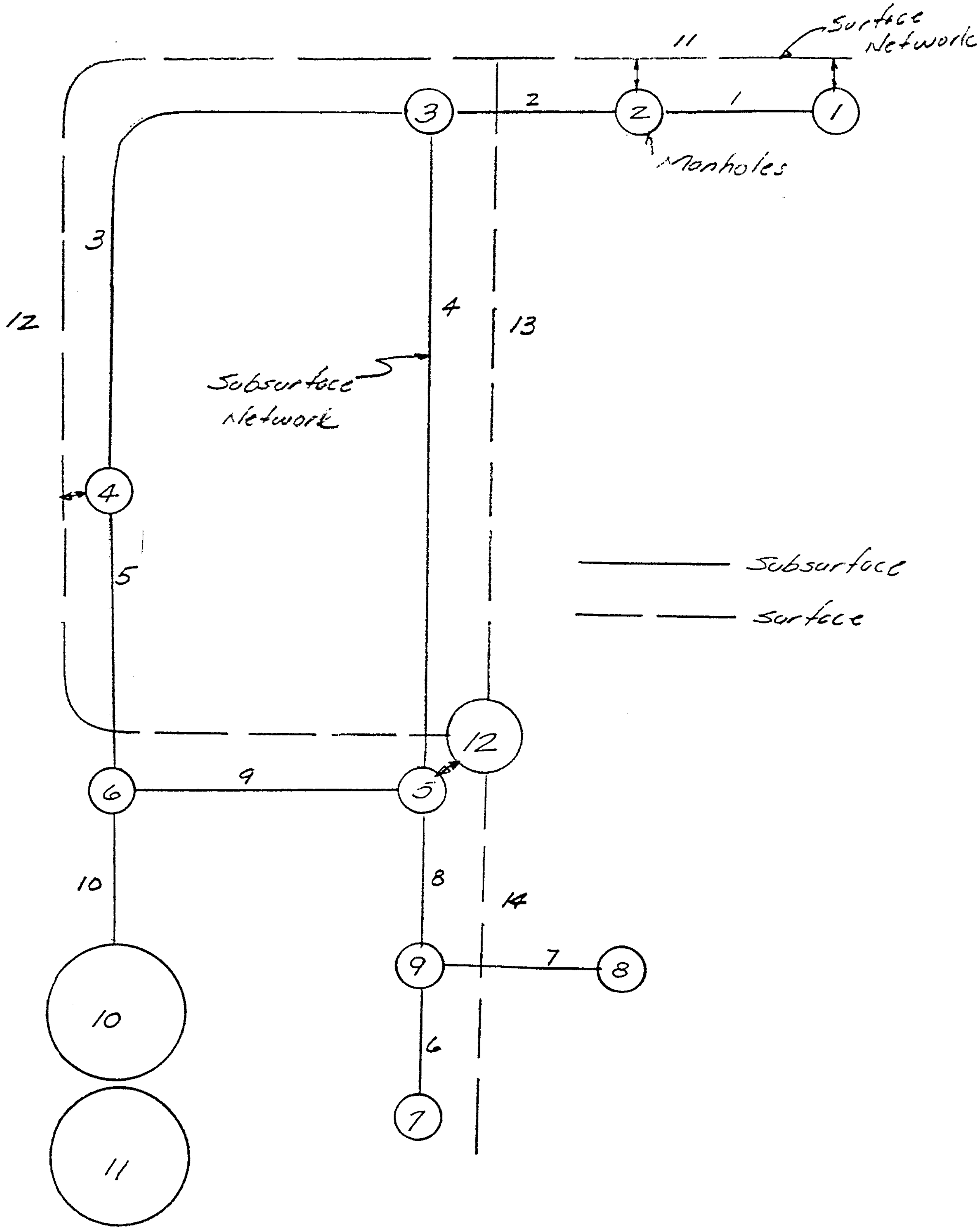

Figure 4. Sewer network problem.

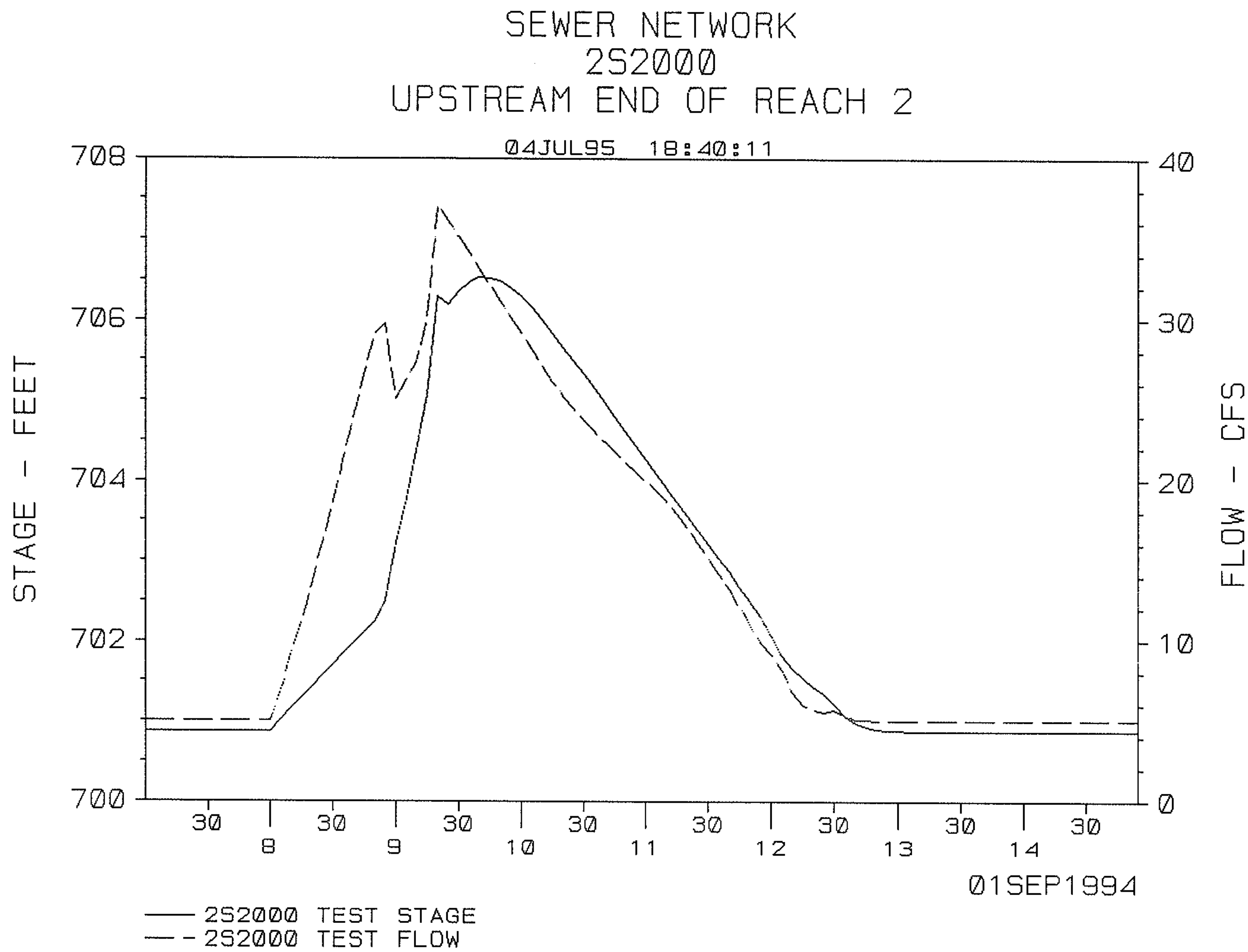

Figure 5. Stage and flow hydrographs within sewer at the upstream end of Reach 2.

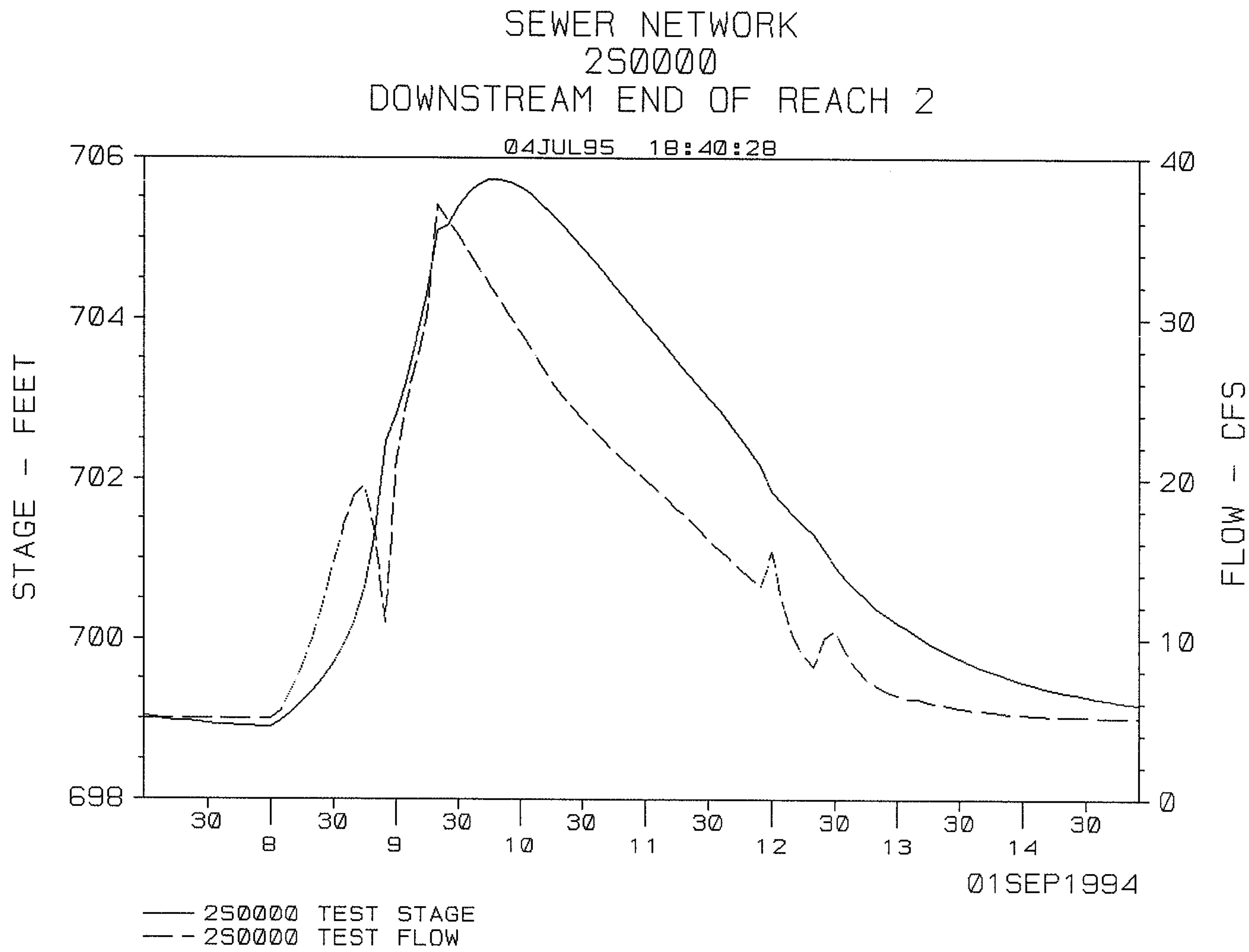

Figure 6. Stage and flow hydrographs within sewer at the downstream end of Reach 2.

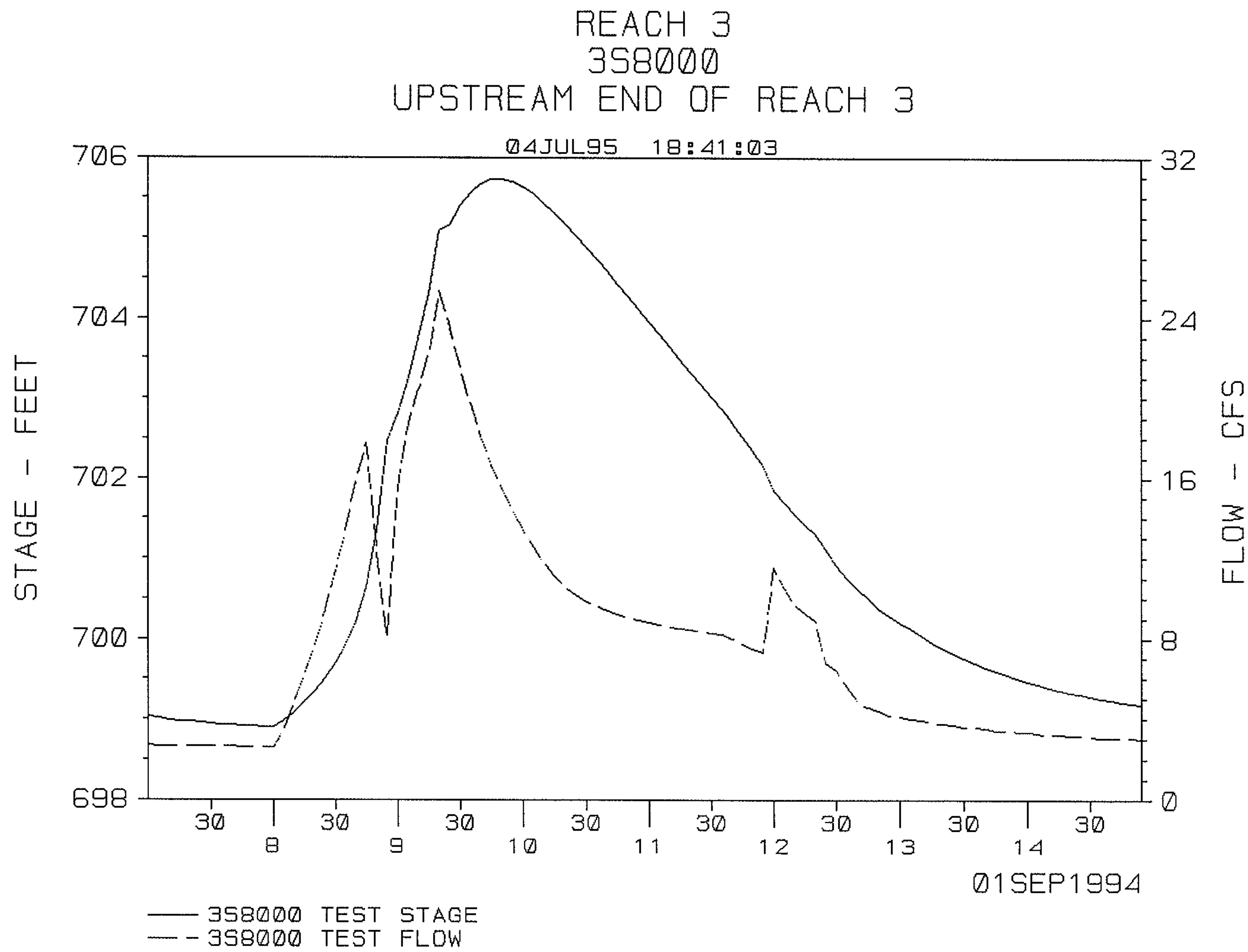

Figure 7. Stage and flow hydrographs within sewer at the upstream end of Reach 3.

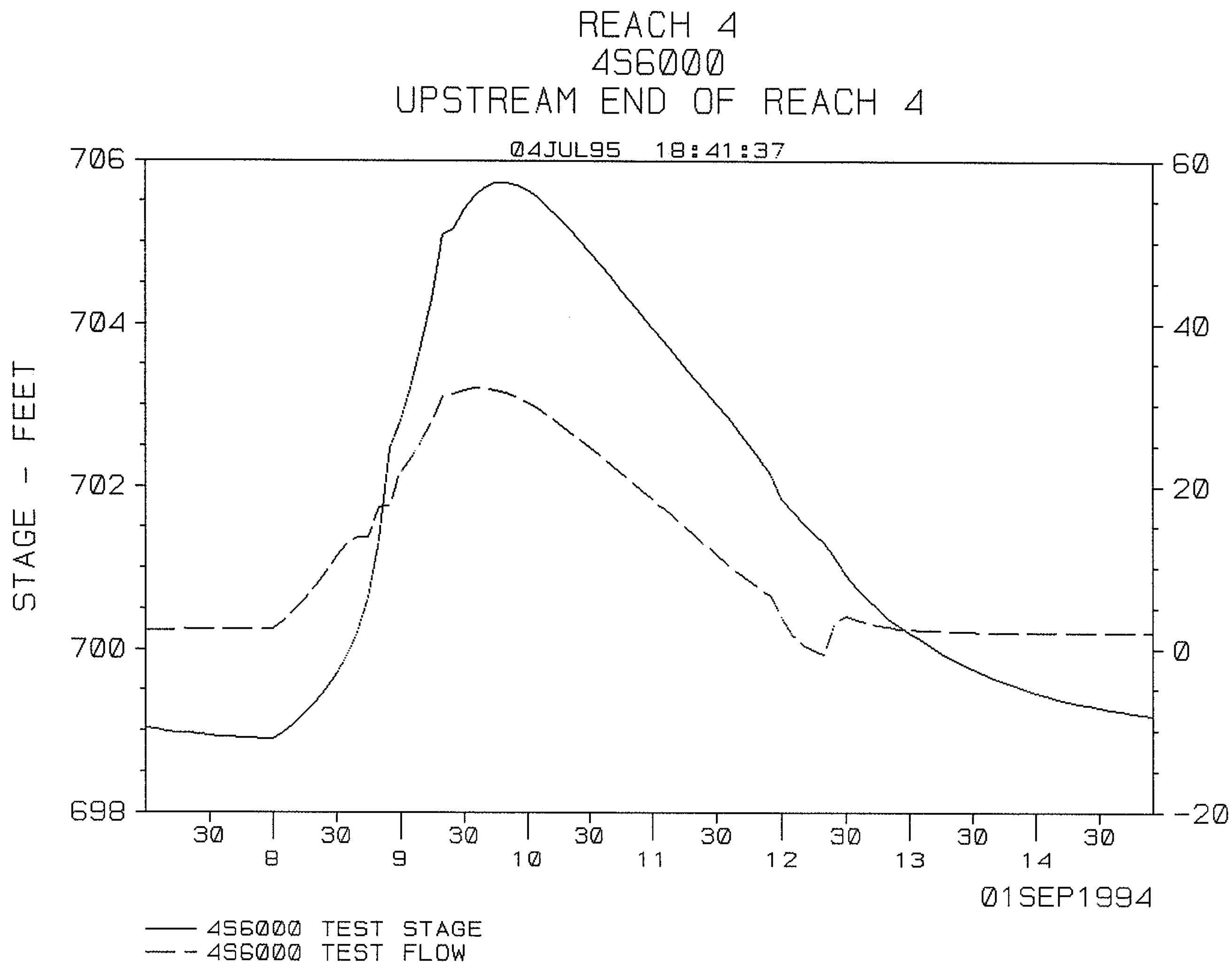

Figure 8. Stage and flow hydrographs within sewer at the upstream end of Reach 4.

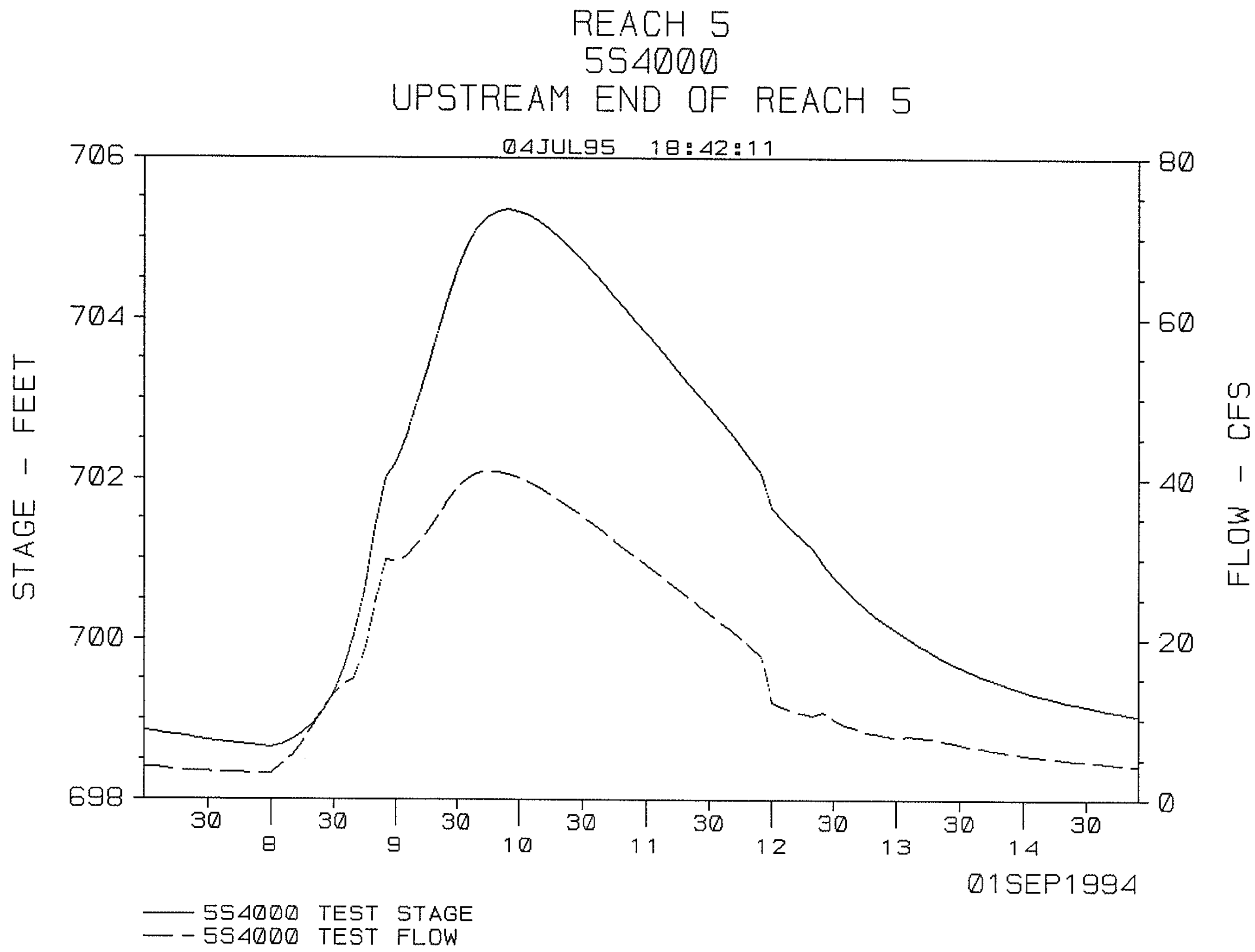

Figure 9. Stage and flow hydrographs within sewer at the upstream end of Reach 5.

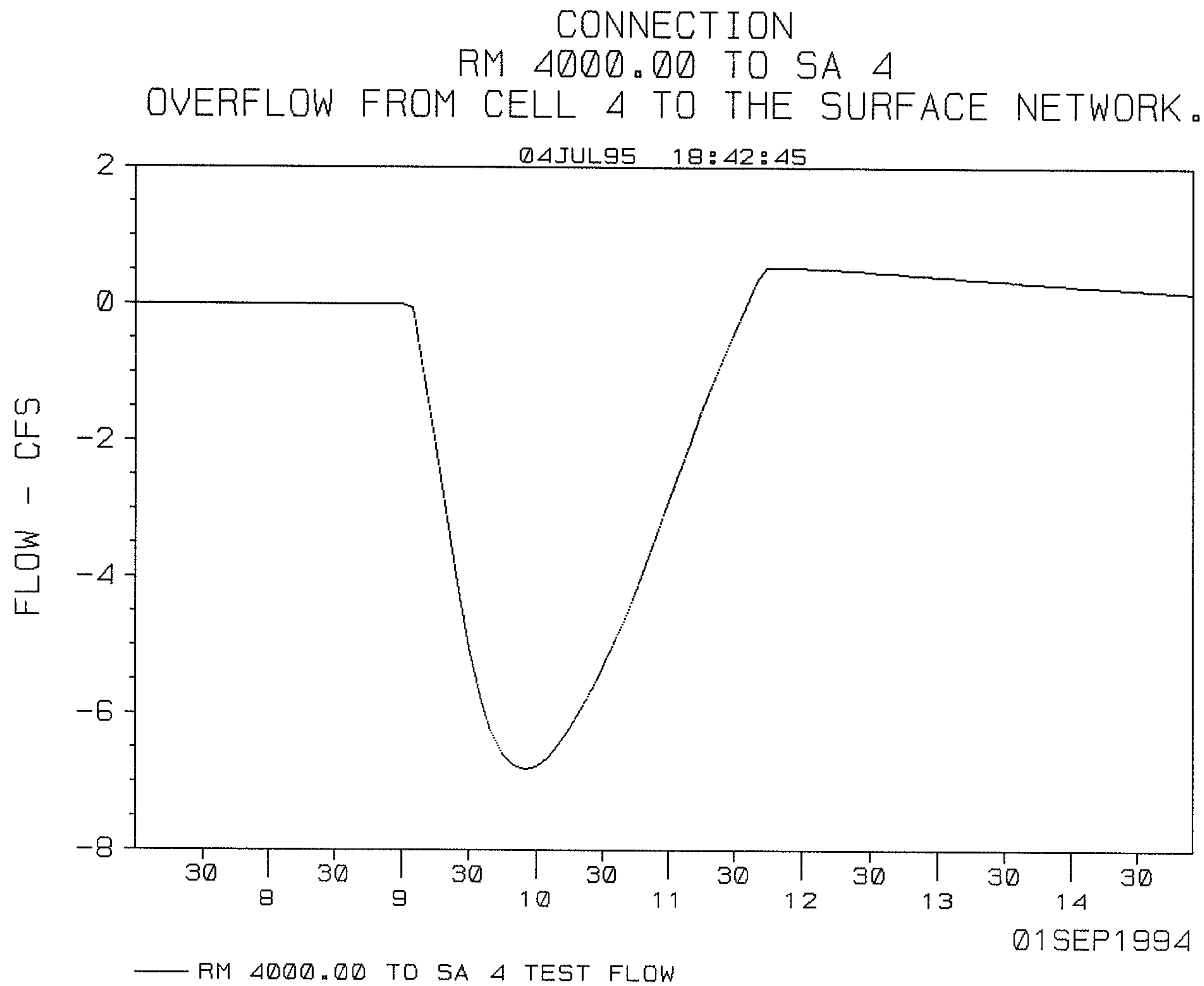

Figure 10. Overflow from cell 4 to the surface network.

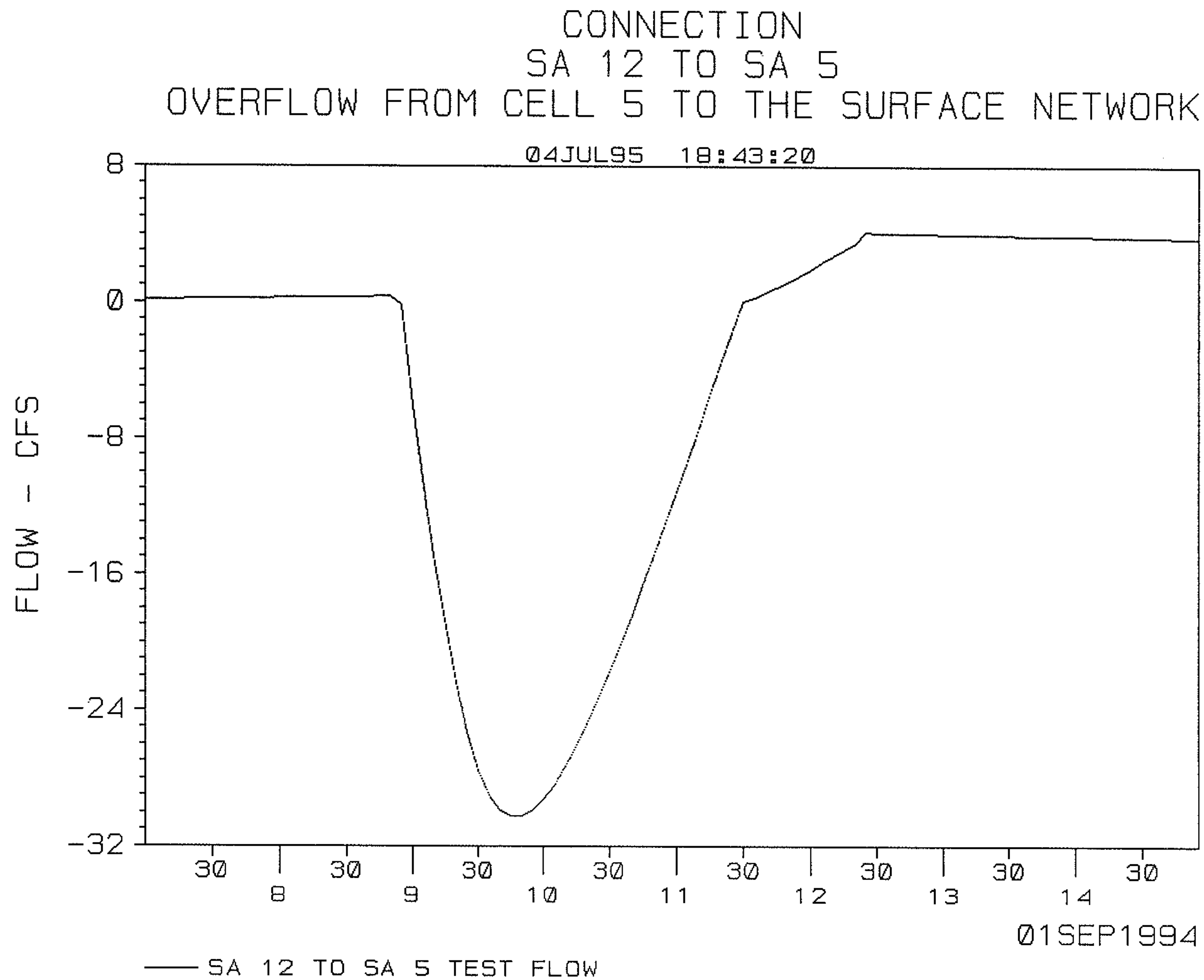

Figure 11. Overflow from cell 5 to the surface network.

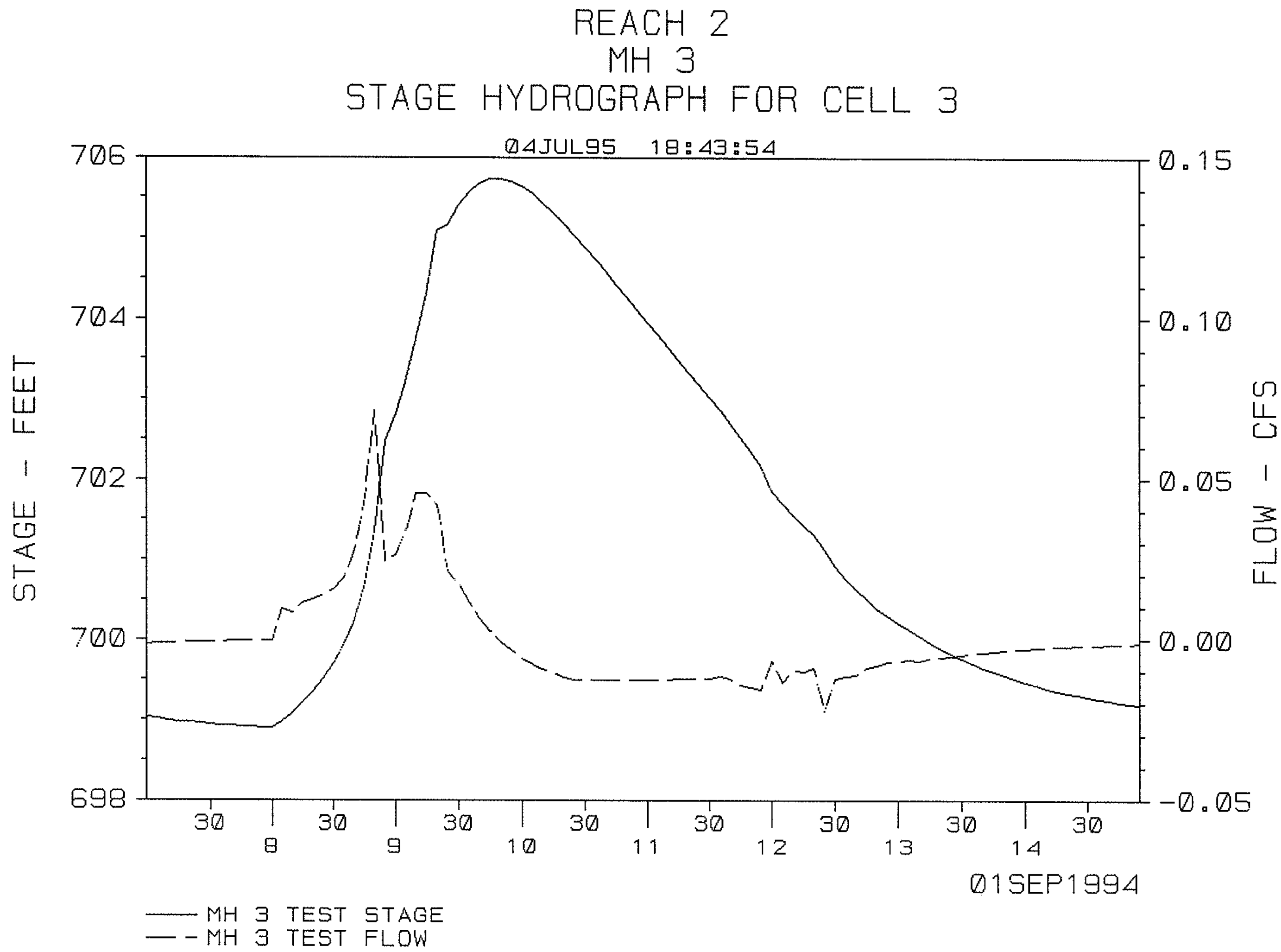

Figure 12. Stage hydrograph for cell 3.

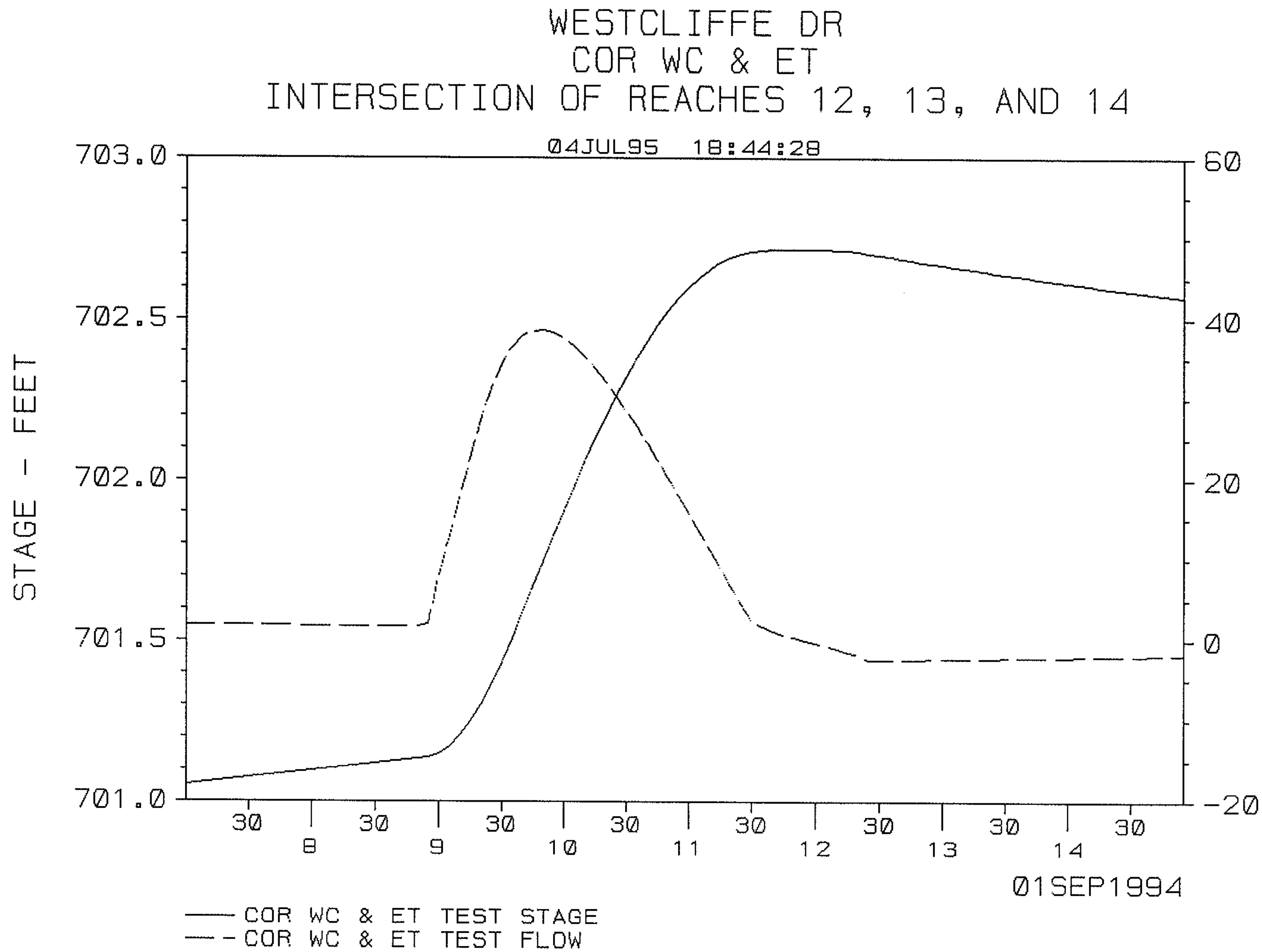

Figure 13. Stage hydrograph for cell 12 at the intersection of reaches 12, 13, and 14.

AGENDA

Urban Hydrology and Hydraulics Workshop; 13-15 September 1994

Tuesday, September 13

8:00 - 8:15 Welcome/Introductions (Darryl Davis, Director, HEC)
8:15 - 8:45 Historical Perspective (Arlen Feldman, Chief, Research Division, HEC)
8:45 - 9:15 Workshop Overview (Troy Nicolini, HEC)

Session 1: Corps Problems in Urban Hydrology and Hydraulics

9:35 - 11:45 **Panel Discussion**

A. Jacksonville District Experiences in Urban Modeling (Mike Choate, Jacksonville District)
B. Pacific Ocean Division Experiences in Urban Modeling (Steve Yamamoto, Pacific Ocean Division)
C. Urban Hydrology Problems of the Southwest (Joe Evelyn, Los Angeles District)
D. Omaha District Experience with the Storm Water Management Model (SWMM) (Bill Doan, Omaha District)
E. Fort Worth District Urban Hydrology (Paul Rodman, Fort Worth District)
F. Urbanization Problems and Design Case Studies in Hydrologic Engineering (Joe Weber, Seattle District)

11:45 - 12:45 Lunch

Session 2: Hydrology

12:45 - 1:35 Implications of Variations in Small Watershed Modeling to Engineering Design (Richard McCuen, University of Maryland)

1:35 - 2:20 Role of Hydrologic Modeling within an Interdisciplinary Approach to Urban Watershed Planning and Management - Experiences from King County (David Hartley, King County Surface Water Management)

2:20 - 2:35 Break

2:35 - 3:20 Lag Equation with Physically Measurable Parameters, and Verification of Parameters by Implementing a Stream Gaging Program (Pete Hall, Sacramento ounty, California)

3:20 - 4:05 Role of Synthetic Rainfall in Urban Hydrology (Joe DeVries, University of California, Davis)

4:05 - 5:00 **Open Discussion**

5:30 - 8:00 **Evening Dinner Session**: Continuous Simulation Versus Design Storms (William James, University of Guelph, Ontario)

Wednesday, 14 September

Session 3: Modeling Approaches

8:00 - 8:45	Hydrologic Analysis of Interior Drainage of West Columbus, Ohio (Jerry Webb, Huntington District)
8:45 - 9:30	Napa River Interior Flood Hydrology (IFH) Study (Harry Dotson, HEC)
9:30 - 9:50	Break
9:50 - 10:35	Regional Consistency and Modeling without Calibration Data (Ben Urbonas, Denver Urban Drainage District, Colorado)
10:35 - 11:20	Urban Hydrology in Switzerland (Peter Kaufmann, Burgdorf Engineering School, Bern, Switzerland)
11:20 - 12:00	**Open Discussion**
12:00 - 1:00	Lunch
1:00 - 1:45	Storm Water Management Models; Identification of Problems and Discussion of Solution Methods (J.S. Wang, Bechtel, San Francisco, California)
1:45 - 2:30	Scale and Model Choice in Urban Drainage Analysis (David Kibler, Virginia Tech, Blacksburg)
2:30 - 2:50	Break
2:50 - 3:35	Towards Integrated Urban Water System Management (Jim Heaney, University of Colorado, Boulder)
3:35 - 4:20	Options for Updating/Maintaining/Improving Available Models (Wayne Huber, Oregon State University, Corvallis)
4:20 - 5:00	**Open Discussion**

Thursday, 15 September

Session 4: Hydraulics

8:00 - 8:45	Urban Storm Water Modeling of Charleston, SC (Clyde Hammond, Charleston District)
8:45 - 9:30	UNET Application and Modeling Considerations for Risk Analysis (Nancy Powell, New Orleans District)
9:30 - 9:50	Break
9:50 - 10:35	Use of HSPF Continuous Simulation and TNET for a Complex Reservoir System with Deep Underground Tunnels (Tom Fogarty, Chicago District)
10:35 - 11:20	Hydraulic Modeling of Surcharged Flows (Mike Schmidt, Camp, Dresser, & McKee, Jacksonville, Florida)
11:20 - 12:05	Current and Future Capabilities of UNET (Bob Barkau, St. Louis, Missouri)
12:05 - 1:05	Lunch
1:05 - 2:05	**Open Discussion**

Session 5: Agency Perspectives

2:10 - 2:55	Current USGS Urban Studies, Methods and Data Considerations (Allan Lumb, USGS, Reston, Virginia)
2:55 - 3:30	Corps Headquarters View on the Future of Urban Hydrology and Hydraulics Modeling (Ming Tseng, Headquarters, Washington, D.C.)
3:30 - 3:45	**Open Discussion and Closing**

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