US Army Corps
of Engineers
Hydrologic Engineering Center

Proceedings of a Seminar on

Real-Time Water Control Management

17 - 19 November 1975
Davis, CA

SP-7

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)*	2. REPORT TYPE	3. DATES COVERED *(From - To)*
17 - 19 November 1975	Seminar Proceedings	

4. TITLE AND SUBTITLE
Real-Time Water Control Management

5a. CONTRACT NUMBER

5b. GRANT NUMBER

5c. PROGRAM ELEMENT NUMBER

6. AUTHOR(S)
CEIWR-HEC

5d. PROJECT NUMBER

5e. TASK NUMBER

5F. WORK UNIT NUMBER

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
US Army Corps of Engineers
Institute for Water Resources
Hydrologic Engineering Center (HEC)
609 Second Street
Davis, CA 95616-4687

8. PERFORMING ORGANIZATION REPORT NUMBER
SP-7

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)

10. SPONSOR/ MONITOR'S ACRONYM(S)

11. SPONSOR/ MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION / AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES
Collection of papers presented at a seminar held at the Hydrologic Engineering Center, Davis, CA on 17 - 19 November 1975

14. ABSTRACT
A seminar on Real-Time Water Control was held on 17 - 19 November 1975 at the Hydrologic Engineering Center (HEC) in Davis, California. The purpose of the seminar was to provide a forum for hydraulic and hydrologic engineers of the Corps of Engineers to assess present methods of water resource project operation and to share information on promising new techniques. Topics addressed included data management on a real-time basis, techniques for streamflow forecasting, techniques for making regulation decisions, and current and anticipated applications of computer technology.

15. SUBJECT TERMS
real-time, water control management, hydraulic, hydrologic, engineers, project operation, data management, streamflow forecasting, regulation, computer technology, meteor burst, stage data, storage, data collection, river lock, dams, reservoir, empirical model, evaporation, hydropower, LANDSAT, flood control, satellite

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT	b. ABSTRACT	c. THIS PAGE			19b. TELEPHONE NUMBER
U	U	U	UU	352	

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Attendees:
Corps of Engineers

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SP-7

FOREWORD

A three day seminar on Real-Time Water Control Management was held at The Hydrologic Engineering Center on 17-19 November 1975. The purpose of the seminar was to provide a forum for hydraulic and hydrologic engineers of the Corps of Engineers to assess present methods of water resource project operation and to share information on promising new techniques. Topics addressed during the seminar include data management on a real-time basis, techniques for streamflow forecasting, techniques for making regulation decisions, and current and anticipated applications of computer technology. Seventeen papers presented during the seminar are contained herein, along with a Seminar Summary.

The views and conclusions expressed in these proceedings are those of the authors and are not intended to modify or replace official guidance or directives such as engineer regulations, manuals, circulars, or technical letters issued by the Office of the Chief of Engineers.

CONTENTS

SEMINAR SUMMARY

Vernon K. Hagen
Chief, Hydrologic Engineering Section
Office, Chief of Engineers

REAL-TIME COMPUTERS IN THE REAL-WORLD
MANAGEMENT OF THE COLUMBIA RIVER - PART I 1/

BY

David M. Rockwood 2/

INTRODUCTION

The stated purpose of this presentation is to summarize our experiences in NPD in the use of computers and data systems in real-time management and control of the Columbia River. This could lead to a dull recital of the history of our developments made over the past 20 years in the form of computer programs, integration of data systems, statistics regarding their use, in managing our reservoir systems, and technical descriptions of the methods used. I am going to discard these explanations, however, in the hope that I can relate something of a personal nature that will have more meaning in establishing better communication and understanding of the fundamental problems that we have encountered in this type of work. My remarks in this particular discussion, therefore, will be subjective and non-technical in nature. Please forgive me for the use of the personal pronouns, which we normally avoid in our technical discussions with the hope I can better relate to you some of my personal feelings and thereby emphasize what I feel to be the important aspects. Gordon Green, who will follow me, will summarize some of the specific problems of water control in NPD, but I am looking forward to hosting a day of technical explanations on next Friday in NPD, when it will be our pleasure to spend a better part of a day in technical explanations prepared by our Water Control Branch staff, in order to provide fuller explanations and demonstration of the actual operation of systems that we use.

Basic Goals for Computers in Real-Time Management. Let me start off with a discussion of basic attitudes that may prevail in your minds regarding computers and real-time data systems. The days have long since passed when, in people's minds, computers possess exotic and all powerful capabilities. Computers have passed through many stages of general acceptace or rejections, e.g., try COEMIS on for size. A basic "good" or "bad" attitude towards these systems may

1/ Presented at "Seminar on Real-Time Operation of Water Resource Projects" on 17 November 1975 at HEC, Davis, California

2/ Chief, Water Control Branch, NPDEN-WC, Portland, Oregon

warp a person's attitude to taking a pre-judged negative rather than positive position. I hope that participants in this seminar will take a balanced approach in the value of real-time computer analysis for water control systems, recognizing both the "good" and the "bad". Let me state that I come here in general strong support of these systems. From our continued use of our systems, we have experienced the many problems and frustrations that are encountered in this type of work. Nevertheless, the challenge of continuous representation of hydrologic and water control systems, by a flexible, numerical objective near real-time systems, has been met, and this provides the satisfaction of accomplishment in the real-world that puts me into full support of the concept. It is a system which has been "Put to the Test", under many years of operating experience, thereby providing an important link in assuring, in actual operation, the reservoir system benefits for which the projects were planned and designed. It assures the highest degree of professional accomplishment in application of modern technology to water management problems insofar as practical. The overall good of this effort is to provide us with the capability for analyzing changed conditions, either with regard to hydrology or project operation, that we are constantly faced with in the real-world context, and thereby base operating decisions on the best technical data available. Let me be the first to point out that this system does <u>not</u> operate the entire system of projects in an "automatic" mode (although many of our projects encompass some major accomplishments in automatic control and dispatch systems), nor does it necessarily "optimize" the monetary return on multi-purpose investments because of the many practical "soft" and "hard" operating constraints that must be dealt with and, in many cases, have overriding importance in the real-world context. To me, these ideals of completely automatic and optimized solutions are "pie-in-the-sky" objectives that will never be obtained in the foreseeable future, in a real-world, real-time situation. While we will advance, step-by-step, to achieve a higher degree of perfection in our technical knowledge, we will probably never replace the judgmental factors required in day-to-day operation, nor, for that matter, the judgment of experienced engineers in assessing results of computer evaluations. To sum up, our position as engineers should be that of a <u>master</u> rather than a <u>slave</u> to computers. Let's make full use of their capabilities in a practical sense, but let's be benevolent of their shortcomings when something "goes wrong", (as it most assurredly will and always by our misdoings in one sense or another), and <u>never</u> expect them to do our thinking for us, or for them to accept the responsibility for the results, good or bad.

WATER MANAGEMENT AS A MAJOR FUNCTIONAL EFFORT BY CORPS OF ENGINEERS

I wish to lend emphasis to the need for use of advanced computer techniques in water control management, by pointing out what I believe to be a major shift of emphasis that should be taken by the Corps of Engineers role in management of water resource developments. Many of us have spent our career with the Corps' Civil Works programs, with regard to the planning, design, and construction of major water control facilities. Because of the tremendous engineering effort in these undertakings, the emphasis of the Corps' effort has been focused largely on these traditional engineering disciplines. The management of the completed projects has, up to now, held a relatively minor role in Corps of Engineers activities. The planning and design documents contained the general features of reservoir regulation procedures, and reservoir regulation manuals contained specific operational criteria. Under actual operational conditions, however, these procedures do not cover many of the changed conditions that usually develop. Furthermore, many of the procedures were derived years ago and did not consider the ability to maintain continuity of hydrologic and operating parameters on a day-to-day or hour-to-hour basis. Also, new demands are constantly being thrown upon us to regulate for unforeseen constraints -- environmental, energy, public use, etc., -- which were not foreseen when the projects were formulated but certainly cannot be ignored in actual operation. So what is now needed is the capability to adjust the regulation to meet these changed conditions on a logical, objective basis. Our responsibility to the public for water control management of river systems can be best met by having a <u>demonstrated</u> technical capability in use of these advanced techniques, and aggressively pursuing them in day-to-day management and control. In my opinion, in the long run as the major structual efforts are being completed, the water resource management problems, particularly those related to the management of multi-purpose reservoirs and water control systems, should become a leading, if not the dominant, role of the Corps in relation to its other activities. On the other hand, a laizzez-faire attitude may well result in a gradually diminishing position of the Corps in management of water resource developments to the point where we would become ineffective and our work taken over by others.

<u>Columbia River Developments</u>. By way of illustration of the above-stated position, I wish to point to our experience in the Pacific Northwest. Over the past 40 years, the Columbia has become

a nearly fully developed river system. I will not take the time now to describe it, but only to say that some 77 major projects in the Pacific Northwest, constructed and operated by various U. S. or Canadian governmental agencies, together with public, or private utilities, are integrated hydraulically or electrically and thereby require system coordination. New projects are now "scarce as hen's teeth," partly because the system is so highly developed, and also because of major environmental objections to remaining major potential developments. We still have a major construction program in the further development of hydro-power resources at existing projects that will last for ten years or more. Major system planning efforts are now concentrated on restudy of the existing system, for consideration of increased hydro-power capability, for incorporating changed conditions (primarily environmental) in order to obtain a more balanced multi-purpose plan of operation, and for recommending modifications to existing projects or possibly additional new projects, to achieve these goals. Our work in Water Control is now largely on analyzing the existing system, either in the sense of carrying out our responsibilities for day-to-day operation, or for reanalyzing the existing system for planning additional facilities at existing projects, or adjusting operating plans. These efforts all go hand-in-hand, and there is significant feed-back to planners from day-to-day operation. The "guts" of our effort is in the form of computer evaluation of systems operations (hydrologic, reservoir, and hydro-power, in various combinations). Because of the centralized nature of the Columbia System operation, we have concentrated our efforts in the Division Office, where operational responsibility for Columbia System Reservoir Regulation has been in the Water Control Branch since the early 1950's. A large part of the effort in utilization of computers for day-to-day operation began in the mid-1950's, and we have been working on it ever since. The design and utilization of computer programs, therefore, has been developed largely to achieve real-time operation, but their use also feeds back into system planning and design studies. In the next section, I will elaborate on the importance of this basic approach. In any case, through these continued efforts, the Corps has maintained a progressive stature among the various operating groups, and we have held a leading role in system integration.
In summary, I feel that our work in computer system utilization is one of the major factors that has been helpful in enhancing our functional role in water control management.

PHILOSOPHY OF DESIGN OF COMPUTER MODELS FOR REAL-TIME OPERATIONAL USE

In this section I will attempt to highlight certain aspects of design of operational models, which I feel to be of overriding importance in the overall philosophy of design for practical application in a real-time situation. I realize that at this time, you have not had the opportunity to review the "innerworkings" of our models. In fact, I would say that the only practical way to really become familiar with them and understand the engineering and data processing techniques is by actual use in a "hands on" situation, through application to your own set of conditions. My objective here is to point out what we have found to be important from our continued development and use of models (primarily the SSARR model, together with the SSARR package and related data systems), gained from our experience of almost 20 years in this activity.

Probably the most important single aspect of design is to achieve a proper balance between theory and practice, in order to achieve hydrologically sound yet operationally practical solutions for day-to-day use. Closely related to this, and of nearly equal importance, is the concept of continued development and refinement, from experience gained under actual operational use in a real-time mode. Let me reflect on this a bit. First of all, I often use the analogy between development and refinement of aircraft, and the development of these rather complex modelling systems. The general principles of aircraft design were well formulated back in the 30's, and the designers proceeded, step-by-step, to develop and produce a cascade of model improvements, as, for example, the development from the DC-3 to the DC-10. But such improvement came about only through continued experience in operational use, along each step of the way. Secondly, I feel that such experience (in the case of operational hydrologic modelling) can come only under a real-world operational situation, where you have to "put it on the line", insofar as use of the information is concerned. Again, by way of analogy, the methods must be developed and tested in an actual situation, just a medical technique is usually developed, tested and finally applied routinely in a hospital. I feel that much of our work is in the grey area between research and routine application, and we have been very fortunate in having the ability to derive these techniques in connection with the management and operation of one of the great river systems of the world.

Another major concept in the development of the SSARR model is that of designing a completely generalized model rather than one which is developed specifically for a particular hydrologic river or reservoir system. The many algorithms that are contained in the model structure are designed to operate on a set of specific characteristics supplied by the user which define the hydrologic, river, and reservoir conditions in the system. Thus, the model provides the means for simulating the hydrometeorological processes of runoff ranging from simple, single basin evaluations to highly complex hydrologic system and reservoir regulation analysis, under a variety of hydrologic conditions, including snowmelt as well as rainfall runoff. Continuous simulations of streamflow can be carried out conveniently for periods of time, ranging from a few days to as long as several years. When combined with other components of the SSARR package and related data systems, the model is capable of automatically simulating the operation of a system of reservoirs regulated for downstream control, based on system requirements for hydro-power, flood control, irrigation, navigation, or other water uses. Furthermore, the modelling system is highly generalized in relation to data input and methods of introducing functional relationships (for example, hydrologic relationships in the form of linear or non-linear mathematical equations, or single or multi-variable tables which represent linear or non-linear relationships between hydrologic variables. I won't attempt to go further into these descriptions at this time, but I wish to point out that, in my judgment, this type of flexibility is highly desirable in adapting a model to a particular set of circumstances.

Hydrologic Representation. The matter of hydrologic representation of runoff processes in a watershed model is highly subjective. In my experience, I have yet to find two hydrologists who look at watershed runoff processes in exactly the same light, and in some cases, I find their opinions as varied as night from day. Nevertheless, there are some underlying principles that in my opinion, must be preserved, in the formulation of a deterministic hydrologic watershed model. These include the logical accounting of each of the basic elements in the hydrologic cycle (rainfall, snowmelt, inter--ception, soil moisutre, interflow, groundwater recharge, evapotranspiration, and the various time delay processes), together with the ability to maintain continuity of each of the processes and to represent each by objective functions, which relates them to observed hydrometeorological parameters. The differences between model representations are in the various refinements that are incorporated to represent a particular process.

Here is one of the basic elements where proper judgment must be used in the degree of refinement that is warranted in a model used for hydrological forecasting. In the real-world situation, the limitations of availability of basic data -- even looking out well into the future -- are an overriding constraint which negate the value of an overly complex representation of the hydrologic processes. Indexes of each of the processes are necessary, and these indexes should be as simple as possible, in order to provide a workable solution for day-to-day application in a real-world situation. I sometimes feel that those who advocate a highly rigorous and therefore overly complex representation of the hydrologic processes, feel that they must do so because of their particular knowledge of the intricacies of scientific-hydrology. Having spent a considerable time in directing hydrologic research in the area of snowmelt runoff, I strongly support research effort to better understand and develop models for representing the hydrologic process by a rigorous method. Nevertheless, we must short cut these methods to some degree in a model to be used for practical application, particularly for real-time hydrology. Many overriding considerations (for example, quantifying true basin precipitation from rain-gage networks, the ability to handle data on a real-time basis, the effect of unforecastable meteorological elements, QPF, etc.) negate the value of overly complex hydrologic models used for this type of work. This will continue to be the case for decades to come, and as we gain more experience, we shall refine our models as conditions warrant.

The balance between an empirical and rigorous approach is a matter of judgment. The watershed model portion of the SSARR model is designed to give the hydrologist as much flexibility as possible in the way the model is applied to a particular case. The degree of empiricism and the complexity of the representation can be varied by the user to best fit a set of circumstances. Some hydrologists approach the modeling problem with the idea of developing a single representation which will solve optimally all his runoff evaluation problems simply by plugging data into the model. I look at the modeling problem quite differently. I believe the model should be as general and flexible as possible, so that it will be a convenient tool for the hydrologist to adjust or modify characteristics in order for him to provide, in his judgment, the best representation of the hydrologic processes. In this sense, then, the model becomes a tremendous hydrologic bookkeeper and data processor, which can become the slave of hydrologists to work with. Thereby, the hydrologist who uses it can become truly creative in developing a process which combines both the art and the science of hydrology for his best use in a particular case. Again, in this sense, the SSARR watershed model (which is really only a small part of the model as a whole) is not intended to be a single representation of the hydrologic processes, but rather a data processor which will provide the framework structure for a multiplicity of representations, depending upon the initiative of the user. In summary, I would not credit the model for success, or discredit it for failure, any more than I would credit or discredit the results of my work to the sliderule or desk calculator.

I would, however, desire to have the capability and flexibility in the model to conveniently represent, adjust, or modify the parameters and to be sure that it is usable in real-world application.

Design of Model With Respect to Computer Utilization. In some cases, models are developed by engineers whose competency is mainly in the field of hydrology, but by reason of training or experience, they may lack full knowledge of computer science technology. One of our objectives in NPD is to "marry" the competency of individuals working in both areas of expertise (hydrologic and computer science). The computer based analytical systems that we are now working with are extremely complex from a computer programming point of view. This comes about from our overall requirement for development of the systems, to achieve the generality and flexibility in application and operation referred to in the previous section. The complexity of the computer program is no way a detriment to the user, however, because this complexity is invisible to him, and he is simply utilizing the power of the system in a prescribed manner. For large-scale applications, I feel that the use of fully developed data processing techniques which require relatively large-scale processors is the only practical processing method. This is particularly true for interactive real-time application that requires repetitive solutions of complex system analysis. We have developed a scaled down version of the SSARR model, however, which can be processed on a small scale computer. The use of this version would be satisfactory for small scale applications, but would not provide all of the convenience, capability, and flexibility of the larger scale version. In summary, I do feel that the design of the computer program with regard to the file structure, data processing techniques, and efficient utilization of specific hardware is extremely important. Further explanation of the requirements for computer utilization will be made by Mr. Davis next Friday in Portland.

Summary of Design Objectives of the SSARR Package. The SSARR model and the augmented SSARR Package of models have been developed, through its evolution, with the above principles in mind. I would like to summarize the basic specific design objections other than hydrologic elements of the watershed model, as follows:

a. Ease of application in operational use.

b. Development of necessary characteristics by a systematic and simplified method.

c. Flexibility and ease in specifying characteristics are functions or tables.

d. Flexibility and ease in supplying input data, by use of specially designed formats and data processing techniques, and developing output which is easily interpreted.

e. Use of advanced data handling file systems for application of the model to complex multi-variable river systems involving large data files of input, including both time-variable, river and watershed characteristics, and reservoir regulation conditions.

f. A means of automatically adjusting the model on a day-to-day basis, to maintain continuity of all hydrologic, watershed, river and reservoir elements, as computed from observed conditions,

g. Incorporation of a generalized river and reservoir model, including routing with backwater effects, reservoir regulation optional operating modes, generalized continuity of routing waters in channels with provisions for overbank, side channels, irrigation diversions, etc., and a means for establishing necessary characteristics either from a theoretical basis or from empirical reconstruction.

h. The ability to simulate and specify the operation of reservoirs in a convenient and simple manner and to account for varying modes of operation on a continuous basis, together with the ability to operate the system automatically for system hydro-power, flood control, navigation, or irrigation requirements.

i. A generalized time routing procedure which is flexible and can operate as required in periods ranging from 0.1 hour to 24 hours, and also the ability to vary the routing period within the period of simulation.

j. The ability to add or alter the program code in a fairly simple manner, in order to incorporate changes in logic within the framework of the basic program.

k. The ease with which others can be trained in the use of the model.

Application to Planning Use. The models with which we deal also have wide application to planning and design studies. For example, the SSARR model and the SSARR Package have had extensive use by our District Offices for a variety of project studies. A major example of its use in the Division Office was in the derivation of the Columbia System standard project and probable maximum floods, numerous types of power and flood control operation studies also utilize our system of program. A description of this type of work, however, is beyond the scope of this paper.

UTILIZATION OF COMPUTERS FOR WATER MANAGEMENT PURPOSES IN NPD

General. Because of the shortness of time, I cannot describe in any detail the various phases in application of computers to our work in the Water Control Branch, except in summary form. Beginning in 1956, with the advent of the IBM 650 computer, our first production efforts in reservoir system operation studies for power planning and operations were conducted. Shortly thereafter, the original version of the SSARR model was developed, and it was applied to the real-time operation of the Columbia System on an experimental basis for the 1957 high water (flood control) period. Ever since those times, we have concentrated our efforts in water control analysis to expanding and refining these techniques, and applying them routinely to operational controls, developing additional programs and data systems to accomplish specific objectives. The basic models themselves have been completely redeveloped several times, and they, together with the data systems, are in a constant state of refinement. The use of the programs has expanded to any number of applications in the various district offices. The SSARR model has also been used constructively by some 23 agencies or organizations outside of NPD in the United States, and by 29 organizations in foreign countries, particularly in Canada, Southeast Asia, Indonesia, Brazil, Poland, and Yugoslavia. It was also tested on a variety of river basins on a world wide basis through the WMO Program for testing conceptual models for hydrological forecasting.

Basic Simulation Models. The following is a listing of our currently operating simulation models:

a. SSARR (Streamflow Synthesis and Reservoir Regulation) Package used operationally for short and long term reservoir regulation system analysis.

(1) Basic SSARR model, for current daily operation of reservoir system (Watershed, river system and reservoir regulation), utilized by NPD and Portland River Forecast Center, NWS.

(2) HYSYS Power System regulation model, for analyzing effects of power peaking on system regulation, for current reservoir operation, in conjunction with basic SSARR model.

(3) SYSREG regulation of reservoir system for downstream control, for current flood regulation, in conjunction with basic SSARR model.

(4) HYSSR Hydro-System Seasonal Regulation for system analysis of reservoir system operation on seasonal basis.

b. Auxiliary Water Control Programs for River and Reservoir Analysis.

(1) HLDPA Hourly Load Distribution and Pondage Analysis Program, for detailed planning and analysis of hydro-power peaking operation.

(2) Water Supply Forecast Model, developed by the Portland River Forecast Center, NWS, for operational seasonal runoff volume forecasting.

(3) WRE Reservoir Water TEmperature Simulation Model, for analyzing water quality conditions in reservoirs.

(4) Nitrogen Gas Model, for analyzing dissolved gas supersaturation in river and reservoir systems.

(5) Summary Hydrograph, for summarizing long period hydrologic data records.

Computer Utilization. The use of the NPD computer system by Water Control Branch presently requires the use of about 25 percent of the available computer processing time. (This general purpose computer serves four districts as well as the division office). The following is a summary of annual computer utilization by NPD Water Control Branch and the NWS Portland River Forecast Center, in terms of central processing time.

	CPU Units (Hours)
1971	552
1972	615
1973	588
1974	595
1975	729

It is estimated that about 40 percent of this utilization is in connection with current operations. The above totals represent some 18,000 to 20,000 job submittals per year.

Interagency Cooperation. A vital aspect in the development of the SSARR model, particularly as related to its application to streamflow forecasting, has been the long term supporting role by the Portland River Forecast Center of the National Weather Service, and in recent years by the Bonneville Power Administration. Gordon Green will later describe the Columbia River Forecasting Service. The Portland RFC has assisted materially in development of the model, and they routinely use the SSARR model for all their operational forecasts. The Bonneville Power Administration also has adopted the SSARR model for hydrological forecasting, and they are doing development work in refinement of methods of application. The Bureau of Reclamation Regional Office at Boise also uses the SSARR model for study purposes. This type of cooperation helps solve many inter-agency problems related to project operation, and it is hoped that even greater degree of team effort will result from these activities.

"Put to the Test." I don't have time here to discuss the actual application of these programs to day-to-day water management. For those who are interested, I am attaching to this paper a copy of an article from "Water Spectrum," for a broad overview of recent Columbia River System accomplishments. The thrust of the article is concerning the operation of the reservoir system but it also includes reference to our computer analysis.

Data Systems and Large Screen Display. My emphasis in this paper is on the development of the analytical tools rather than the data systems used in the Columbia System, but the two must go hand in hand for effective management. The development of the data systems, together with our new on-line Large Screen Display System, will be discussed in Portland on Friday. Gordon Green will also give a brief description of the presently operating Columbia Basin Teletype System. The integrated Columbia River Hydromet and Management System (CROHMS) is a subject unto itself, and I will defer this subject until later.

APPLICATION TO OTHER AREAS

There is no point to go to the effort of preparation for this seminar, and holding the seminar itself, without an objective or goal for some worthwhile accomplishment. I feel that if there is accomplishment to be achieved by this seminar, it is in the technical cross-fertilization for making full use of available techniques by other offices.

I hope that I have stimulated interest and provided some basic understanding of our work in system development. I strongly recommend that other Corps offices give serious consideration to use of this fully developed and tested system, and further that HEC support this effort through coordinated effort with our office for its application to other areas. I say this not from the viewpoint of presuming to have all the answers, which certainly we do not. But I do sincerely believe that the use of SSARR Package for operational water control management could easily save five to ten years of development time and countless thousands of dollars in software costs that would be required to develop a comparable system for real-time operation.

Attachment
"Put to the Test"

Reprinted from Vol. 6, No. 2, 1974 issue of Water Spectrum,
a Corps of Engineers quarterly.

PUT TO THE TEST

by David M. Rockwood
Chief, Water Control Branch
North Pacific Division

Effective reservoir control and water management tamed a major flood and overcame a widespread drought in the Columbia River Basin.

The coincidence of nature's whims with near completion of the Columbia River reservoir system put the design to the test in an unexpected "real world" context. Within a brief 2-year span nature unleashed two of its most devastating weapons—flooding and drought—on the Pacific Northwest.

A near-record flood runoff occurred in the Columbia in 1972. This was followed by a near-record low-water period in 1973 which, in turn, was followed by another high-runoff period this year.

Such extremes of wet and dry periods do not normally occur back-to-back in the usual cycle of Columbia River Basin events. Therefore, the past 2 years have been what constitutes an unusually premature, but nevertheless effective, test of the new system. It has operated essentially according to plan.

The 1972 flood could have been a major disaster of a magnitude ranking with the Tropical Storm Agnes flooding on the East Coast. However, use of the system's multipurpose reservoirs according to a preconceived plan tamed this flood to relatively nondamaging proportions.

Then the system was immediately called upon to conserve its water resource to alleviate the hydroenergy shortage that occurred during the summer and fall of 1973. Although the principal multipurpose values of the system were preserved, special energy conservation programs were put into effect during the period of shortages.

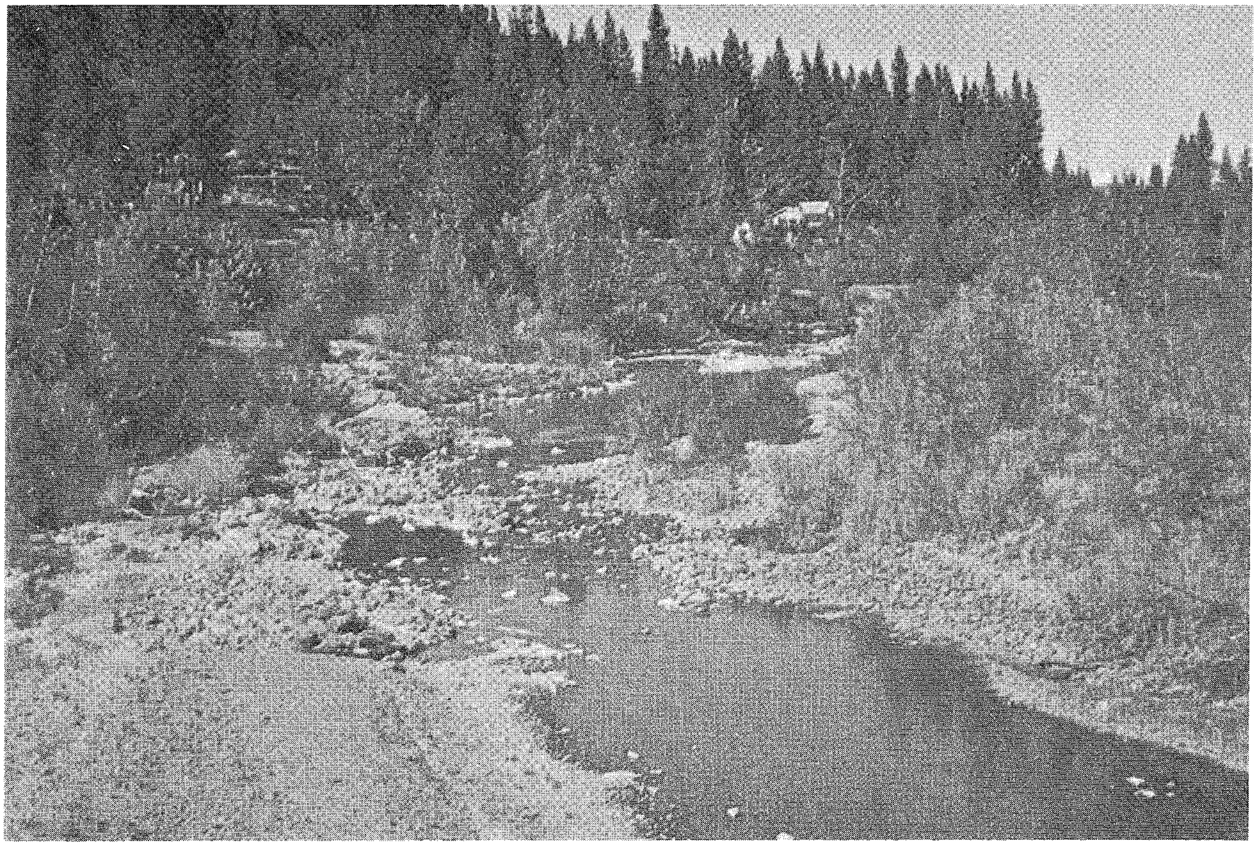
The basic problem in both situations stems from the fact that the Columbia River is the major snow-fed river in the country, outside of Alaska. The river's 259,000-square-mile basin drains the rugged mountain regions of Oregon, Washington, Idaho, the western part of Montana and some 39,500 square miles in British Columbia, Canada. Each spring the runoff from melting snow in the mountains gradually increases to an annual peak flow usually reached during the first half of June. Thereafter, streamflows recede and diminish down to base flow by early autumn. Occasionally, though, secondary winter peak discharges occur from unusually heavy rainfall—such as happened during the early months of 1974. By the time this runoff reaches the mouth of the Columbia it averages 180 million acre-feet annually.

The 1972 Flood

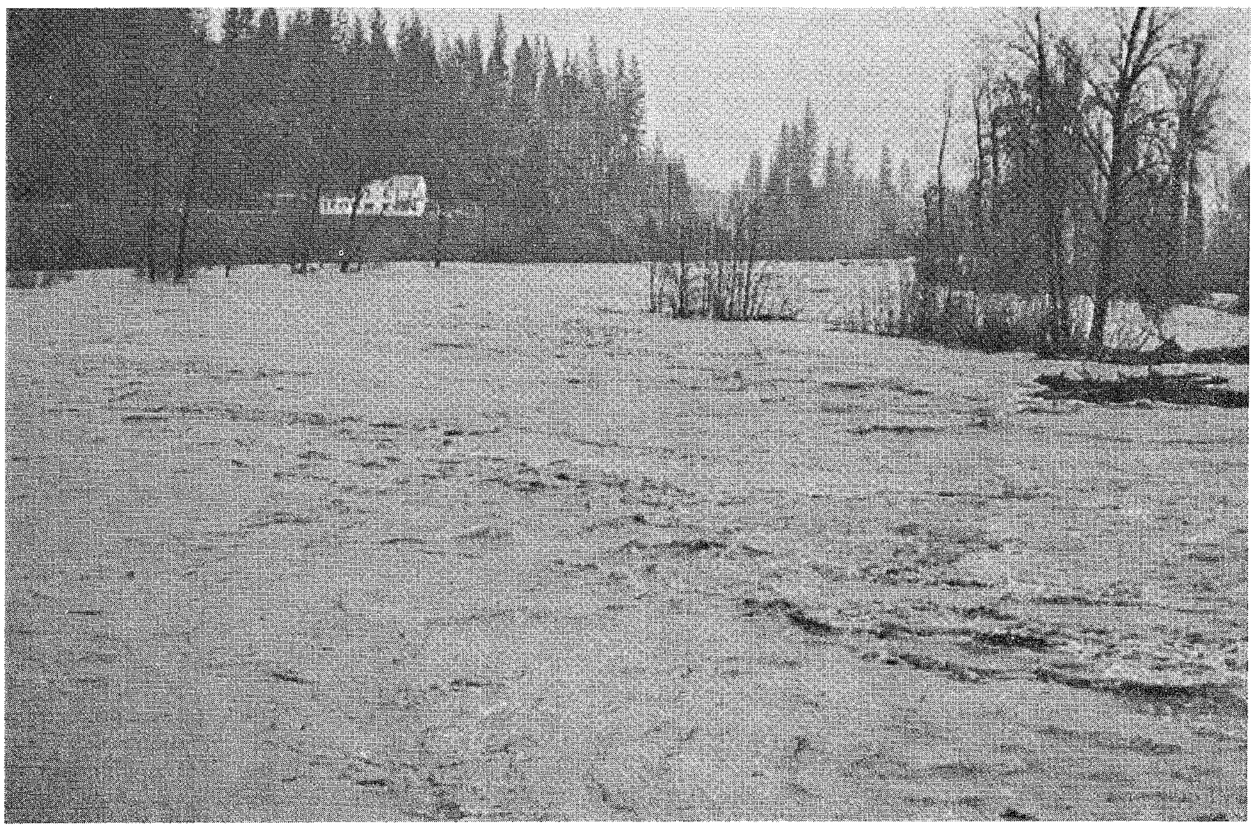
In the spring of 1972 the snow accumulation over the entire Columbia River Basin was so much above normal that very high runoff was expected with the melting of the snow in late spring and early summer. Forecasts of flood peaks and volumes indicated that amounts equal to or greater than those observed over the past 50 years would be experienced without the use of reservoir control. The key question then became: what degree of regulation could be counted on by the not quite completed reservoir system?

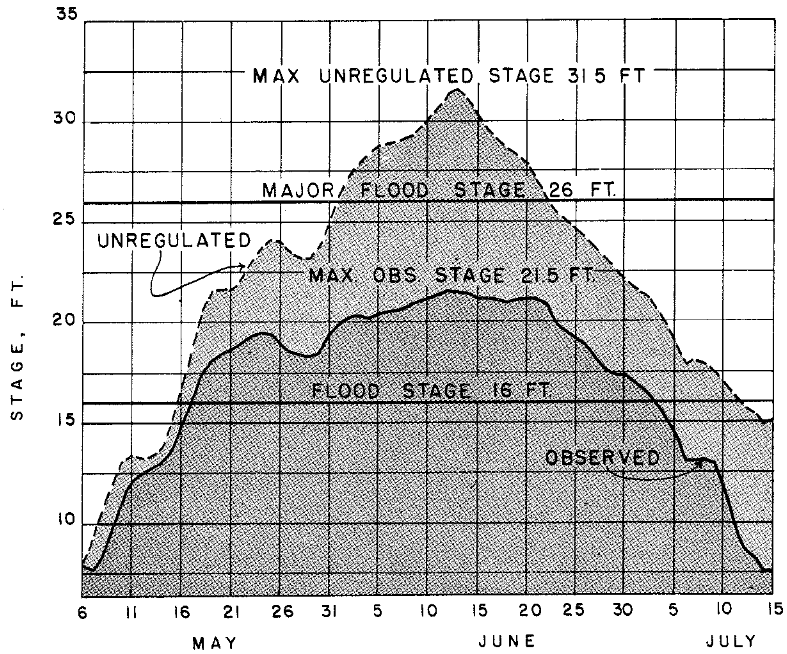
A fortunate circumstance encouraged optimism about the reservoir system's ability to control the coming flood to manageable levels. This was the fact that the new Dworshak and Libby Dam and Reservoir projects were being put into operation as part of a network of 77 major dams and hydroelectric projects in the Pacific Northwest, most of which were integrated into the Columbia River system either hydraulically or electrically.

Although Dworshak and Libby could only be counted on for partial use of their respective water storage capabilities at that time, they still added about 5 million



Only five months separate these views of southern Oregon's Applegate River taken from the same site August 1973 and January 1974.





Coordinated reservoir regulation of the Columbia's main stem limited the June 1972 high water to only 5.5 feet above flood stage at Vancouver, Wash., near the confluence of the Willamette and Columbia Rivers.

developed and has, for many years, routinely utilized a particular series of computer simulation models known as the Streamflow Synthesis and Reservoir Regulation (SSARR) Package. This effort has been enhanced by the Portland River Forecast Center of the National Weather Service under a joint agreement between the two agencies since 1962.

Recently, the Bonneville Power Administration joined this coordinated effort to provide forecasts for water management of the Columbia through the Columbia River Forecasting Service. The nerve center of the whole operation is the U.S. Custom House in Portland, Ore., headquarters of the Columbia River Forecasting Service and the Corps' Reservoir Control Center.

Effective control of a major flood, however, requires the cooperative efforts of many organizations. In addition to those already mentioned, the coordinated flood effort involved the Bureau of Reclamation, the British Columbia Hydro and Power Authority and many public and private power utility companies. There were also the State and Federal agencies responsible for streamflow and weather monitoring and analysis, the emergency organizations who may be called in to act if trouble should develop, and a host of organizations responsible for the other river and reservoir uses—irrigation, recreation, navigation and fish and wildlife management.

The 1973-74 Hydroelectric Shortage

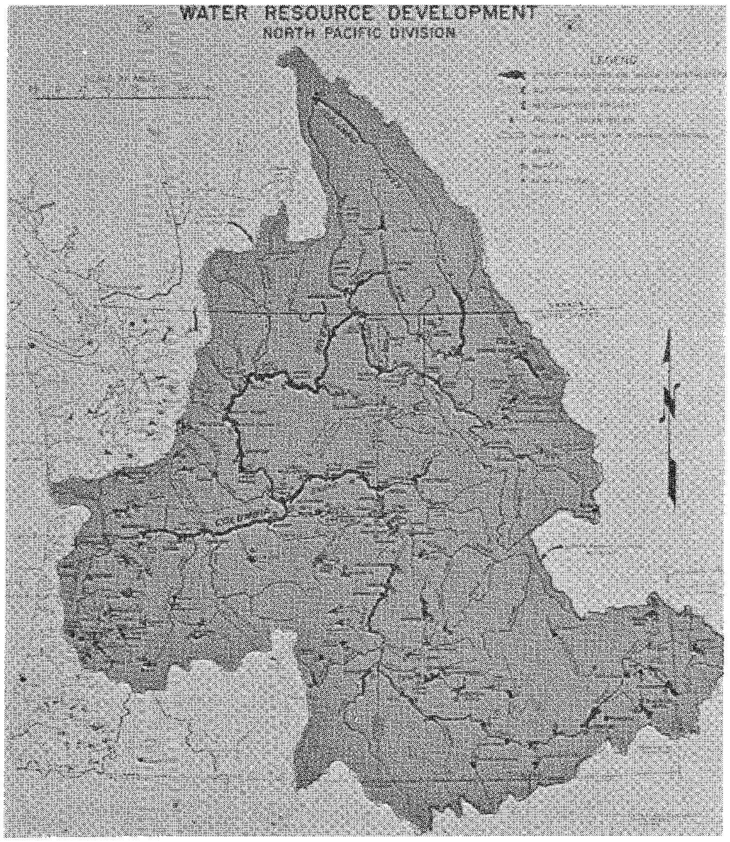
It was indeed ironic that the year of high floods in 1972 was followed by a year of drought in 1973. In terms of severity, the light snow accumulation in the spring of 1973 resulted in runoff volumes so low that only 2 of the 95 years during which records have been kept had lower runoffs.

The water supply from melting of the spring 1973 snowpack was insufficient to refill the reservoirs which had been evacuated to supply power demands in the preceding winter. At the end of the refill period, the reservoir system contained only 22 million acre-feet of water compared with the approximately 29 million acre-feet of total refill capacity available. Because of this deficiency, special measures were taken to reduce electrical energy consumption in the Pacific Northwest. These efforts were almost entirely centered on voluntary measures by the general public and commercial and industrial power users. Such measures reduced system power loads by perhaps as much as 5 percent.

Also, special water conservation measures were taken to provide more efficient use of the water available for power production through modification of normal reservoir regulation criteria. These included reduced use of water for fish passage facilities during the late fall when migration of anadromous fish had virtually ceased. There was also some relaxation of certain operating limits at some of the run-of-river projects. However, the major operating criteria for flood control, navigation and irrigation remained in effect. Only the effectiveness of recreational use was reduced at a few of the reservoirs in the system. The article in the last issue of WATER SPECTRUM written by Don Hodel, Administrator of the Bonneville Power Administration, describes in detail the power aspects encountered during the fall of 1973 drought.[1]

Fortunately, this particular low-water period has passed. Heavy rains and mountain snowfall during the months of November 1973 through March 1974 changed the hydrologic balance of the Columbia system from a water deficit to a surplus. In fact, severe flooding occurred in January in the coastal rivers and low-elevation tributaries of the interior. However, normal winter flood control operation of the system's reservoirs provided significant flood reductions in the Willamette and lower Columbia Rivers.

[1] "Northwest Power Crunch," Vol. 6, No. 1, 1974, pp. 1-7.



The Columbia River Basin's reservoir program is being developed jointly with Canada under the international Columbia River Treaty.

acre-feet of usable storage space to existing projects in Canada and the United States. This increased the total usable storage area by more than 20 percent, providing a total of about 24 million acre-feet. While this amount did not provide the capacity envisioned for the fully completed operational system, it was sufficient to prevent a major flood catastrophe.

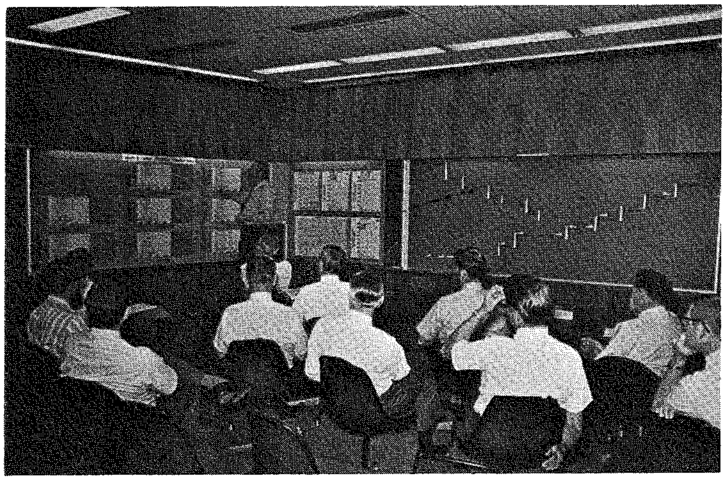
Thus, it was recognized in advance that the control system was at least developed to the level where it could be put to the test in a real-time operation. Preparations were made to provide the necessary reservoir space. Computer simulation techniques were put in readiness for making day-to-day reservoir control decisions based on hydrologic and reservoir regulation forecasts.

As it turned out, the flood did indeed develop as foreseen. The hydrometeorological conditions during the snowmelt period were particularly severe. The unregulated rivers and streams in the basin reached unusually high discharge levels over a prolonged period of time. Many anxious days were spent by the Water Control Branch of the North Pacific Division in analyzing and evaluating computer-derived forecasts for the next 30 or more days. The purpose of these forecasts was to account for the changing flood potentials as the snowpacks melted and the reservoir storage conditions changed, while also considering the amount of regulation needed to meet all the usual operational requirements—such as hydroelectric power generation, irrigation, navigation, water quality, fish and wildlife management, and recreation.

The results of the regulation did, in fact, meet all the requirements within the limited capability of each project in the system. The flood—which would have reached a crest discharge of 1,053,000 cubic feet per second (cfs) at The Dalles, Ore., without reservoir regulation (higher than the disastrous flood of 1948)—was controlled by regulating the Columbia River flow down to a peak discharge of only 620,000 cfs.

Controlling maximum flow produced a peak river stage of 21.5 feet at Vancouver, Wash., where the bankfull stage is 16.0 feet and major flood stage is 26.0 feet. Without the storage reservoirs, the crest stage would have been 31.5 feet. Similar reductions in flood stages were produced at many upstream locations, both on the main stem of the Columbia River and on those tributaries controlled by reservoir storage. For this flood alone, the damages prevented by reservoir storage amounted to $260 million.

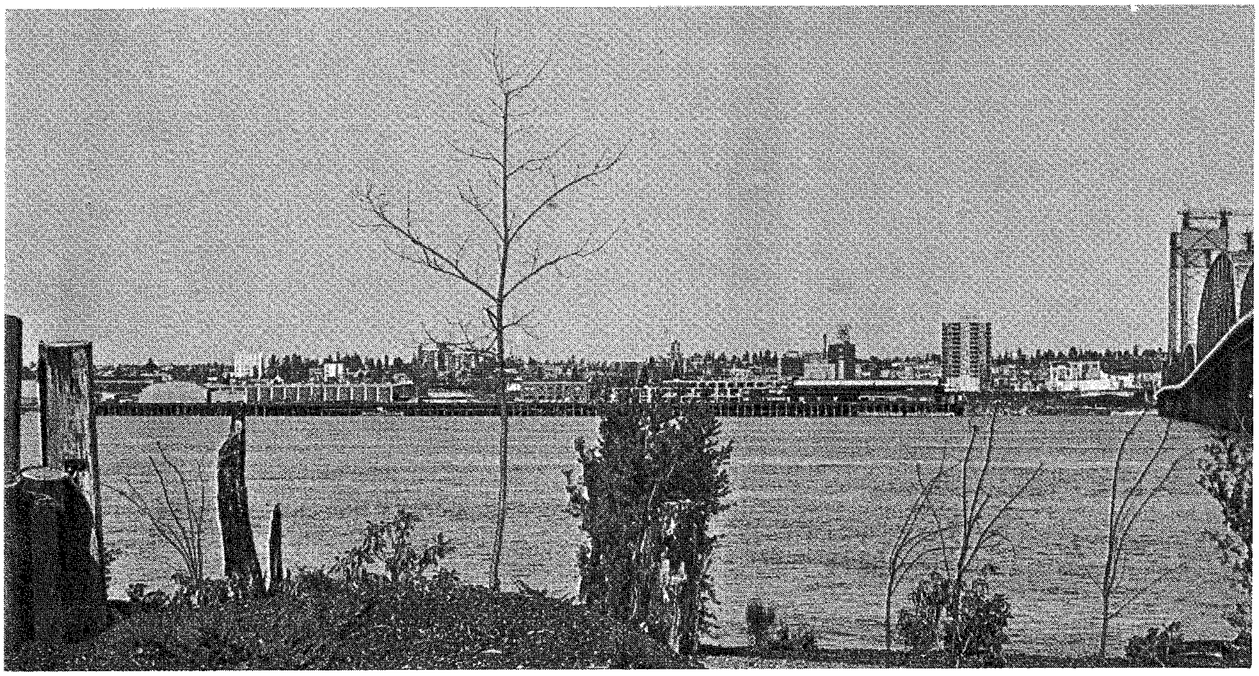
Much of the ability to achieve the reservoir regulation described above lies in the use of computerized streamflow simulation models. The North Pacific Division

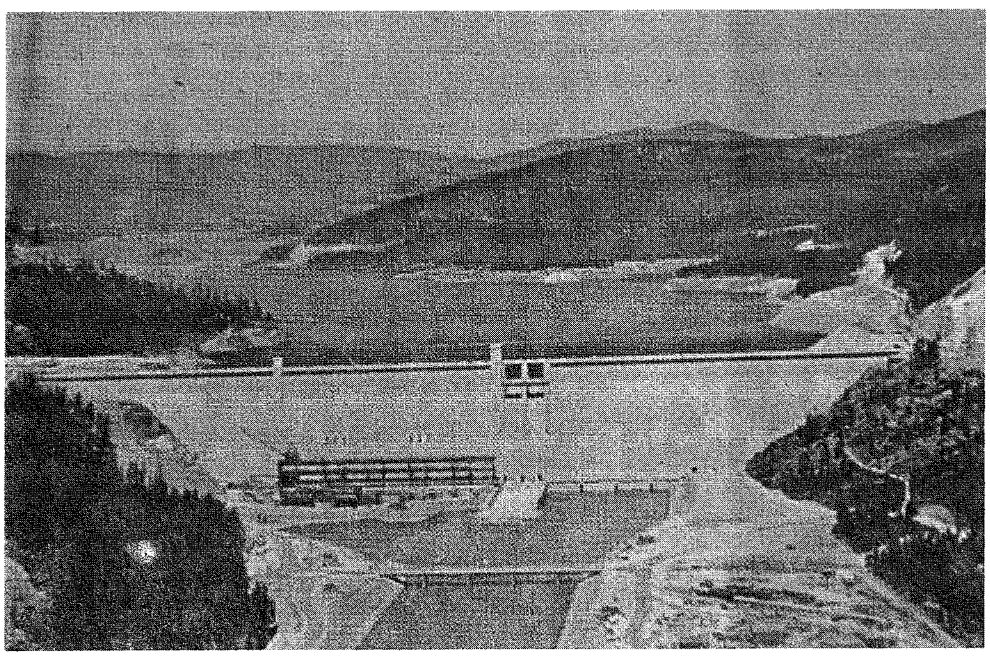


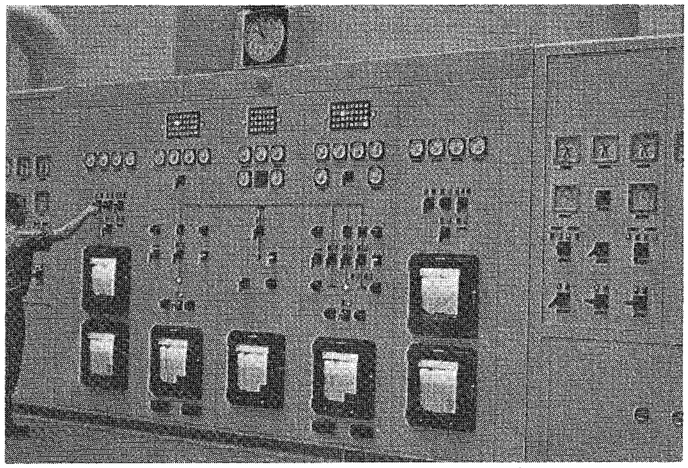
Here at Portland, Ore., is the nerve center of the Reservoir Control Center and the Columbia River Forecasting Service.

Center- An unregulated Columbia River could have severely damaged downtown Vancouver if the river had gone above major flood stage during the near-record 1972 runoff.

Bottom- Libby Dam's new reservoir was being integrated into the Columbia River reservoir control system along with Dworshak's as 1972's high water period approached.



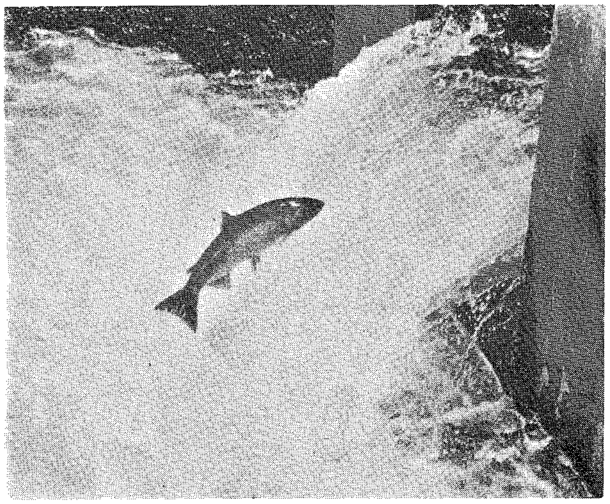




Right-Control-panel operator at McNary Dam coordinates his reservoir's activity with the control center at Portland.

Below-Fish ladder flows during the 1973 drought were reduced that fall, as soon as upstream fish migration stopped, to conserve the limited water supply.

Below right-Snowpacks melting in the late spring and early summer make the Columbia one of the major snow-fed rivers in the United States.



In retrospect, last year's low-water period provided a good example of the system's capability to meet its power commitment under adverse water conditions. The system has been designed to meet the most severe sequence of natural events that has ever been experienced in the area since records have been kept. However, when the reservoirs failed to refill, there was concern about the possibility of the most critical sequences of water conditions on record recurring in the wake of an already deficient water supply situation. Obviously, this would have involved a theoretical sequence of events which would have been more severe than any real occurrences of the past. Fortunately, the ever changing seasonal supply of moisture in the region conformed to historical statistical variations.

When we were operating the system in "real time," though, there was considerable pressure to relax certain aspects of the multipurpose reservoir regulation criteria. However, the recent occurrences illustrated that, although minor modifications were instituted, the major multipurpose aspects of the system were preserved.

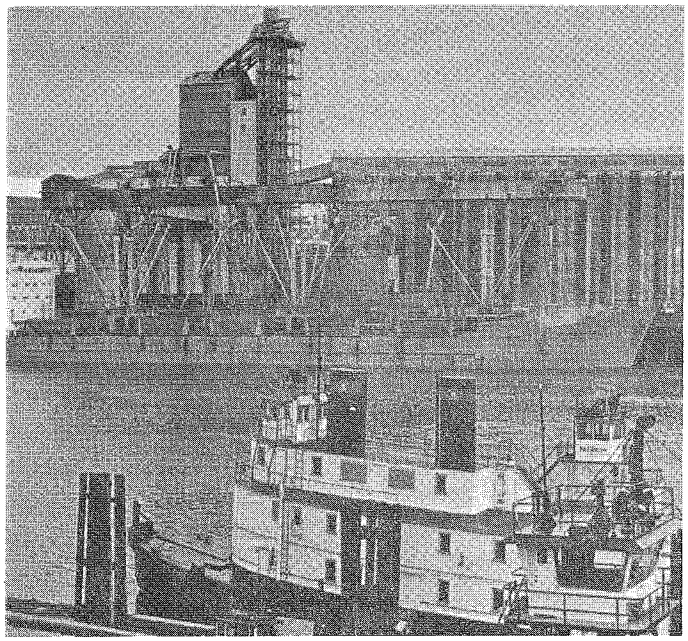
The Columbia is now a nearly fully developed system of reservoirs and hydroelectric projects. Since there is such a wide variety of ownerships and operating authorities for individual water control projects, the current system of coordination has come about by the voluntary actions of various interests.

The projects in operation in the Pacific Northwest generally consist of multipurpose reservoirs upstream and run-of-river navigation and hydroelectric power projects downstream. Of these major projects, 29 are Corps operated multipurpose projects, 20 are Bureau of Reclamation multipurpose projects, 23 are non-Federal hydroelectric projects and 5 are multipurpose or hydroelectric projects located in the Canadian part of the Columbia River Basin. The Corps has been instrumental in coordinating these projects into an integrated system.

The usable storage in reservoirs for downstream control of the main stem of the Columbia is about 40 million acre-feet. In addition, there are numerous small reservoirs constructed by local interests for irrigation use. The electrical transmission network constructed by the Bonneville Power Administration integrates the Northwest electrical generation facilities operated by the various government agencies and private utility companies. The total capital investment in these projects, including transmission facilities, is estimated to be about \$10.5 billion.

The values represented in the beneficial uses of water in this system are almost beyond comparison in terms of how well the water is utilized for all its multiple pur-

Paper 1



Upstream reservoirs on the Willamette River protect the heavy concentration of industries at Portland Harbor from severe flooding.

poses. This system has the most highly developed hydropower resource of any major river basin in the world. It has some 20.9 million kilowatts of installed hydroelectric capacity, and the total will increase to 27.8 million in the next few years. By comparison, the installed hydroelectric capacity in the Tennessee Valley Authority system is about 4.4 million kilowatts. If the Columbia system's 100 billion kilowatt-hours of hydropower were generated by oil-fired thermal power plants instead of hydropower, the electrical energy produced in the system this past year would have required the consumption of some 185 million barrels of oil.

Flood control benefits resulting from reservoir regulation have averaged well over $100 million annually for the past 10 years. The Bureau of Reclamation estimates that the value of crops produced on lands irrigated by the waters of the Columbia system averages more than $600 million annually. These crops, directly or indirectly, produce the agricultural products to feed approximately 5 million people.

Navigation through the up-river lock and dam system now carries in excess of 4 million tons annually, and is steadily increasing as the navigation system is being completed. Oceanborne commerce in the lower Columbia between the Portland-Vancouver area and the Pacific Ocean—approximately 105 miles—is over 20 million tons a year.

The annual commercial and sport fishery value of the anadromous fish runs harvested from those species of salmon and other fish originating in the Columbia system is estimated by the National Marine Fishery Service to be about $30 million. Water-based recreational use of Federal reservoirs in the Pacific Northwest was recorded at over 22 million visitors last year for purposes of sightseeing, fishing, camping, picnicking, swimming and boating.

A high degree of water quality enhancement of the Willamette, a tributary of the Columbia, has been achieved through low flow augmentation provided by the reservoir system. This effort has been coupled with State and locally sponsored water pollution control facilities for municipal and industrial wastes.

Future Outlook

Within the past 2 years the Columbia system has reached a plateau of reservoir development with the near completion of projects related to the Columbia River Treaty for joint development of Canadian and United States water resources. The improvements scheduled to be undertaken within the next 15 to 20 years are mainly concerned with the construction of additional hydropower facilities at existing plants. The exceptions are those local projects on tributary streams which have little or no effect upon the main system.

The additional units which will be constructed were authorized in connection with the Federal and utility-sponsored Hydro Thermal Power Program for the Pacific Northwest. Although not immediately installed, the provision for additional hydroelectric generating turbine units was constructed into each of the new dams with the future in mind. Projections of future loads versus available resources indicate that there will be, generally, a deficit in meeting normal energy and peak power demands for electricity for many years to come. Since this projection is based on critical water supplies, any unusually low streamflow conditions will put additional stresses on the power system. It is our principal objective to maintain this balance of supply and demand for all types of water use through multipurpose regulation of the system in accordance with the present authorizations, licenses and operating criteria imposed upon us.

The flood control system now in existence with the completion of the new Mica Reservoir project in Canada—together with all other projects now in operation in Canada and the United States—provides us sufficient assurance for the future. We now have the storage regulation capacity to control floods of a magnitude equal to the largest on record. Even the Standard Project Flood, a computer simulated set of flooding circumstances considerably more severe than any which have been experienced, would be controlled sufficiently to prevent a major disaster to the area.

Some may feel we have unnecessarily overbuilt our regional hydropower and reservoir capacity. Yet when potential disasters struck the Pacific Northwest in rapid succession within a 2-year period, our system was put to the test and passed with exceptionally high marks—both in flood control and water conservation. ■

Paper 1

REAL-TIME COMPUTERS IN THE REAL-WORLD MANAGEMENT OF THE COLUMBIA RIVER - PART II[1/]

By

Gordon G. Green[2/]

INTRODUCTION

This paper builds upon the information given in the previous discussion by Dave Rockwood. The objective of this paper is to describe the present methods used on a day-to-day basis to regulate reservoirs in the Columbia Basin System. Emphasis is on computer utilization, data management and communications. It also describes future improvements expected and planned.

This paper is presented primarily from the viewpoint of the Reservoir Control Center (RCC), as the major element to carry out the real-time management functions of the Water Control Branch. In the North Pacific Division, the RCC is designated as a section in the Water Control Branch of the Engineering Division. Other sections parallel to the RCC in the Branch include the Hydrologic Engineering Section, the Power Section and the Water Quality Section. As Dave mentioned in the preceding discussion, one of the major areas of work in the Water Control Branch is now in analyzing the existing river and reservoir system, either for carrying out its work in day-to-day operation, or for projecting system operation into the future for planning and design purposes. The amalgamation of the technical expertise contained in each of the sections in the Branch is the key to successful application of this technology and knowledge to day-to-day water regulation activities. This merging of expertise in analyzing hydrologic and hydro-power systems, together with the analysis of water quality effects, provides the interchange of technical information for two-directional feed-back, both to planning and operation. To be effective, the four sections must operate as a team effort at the Branch level. The hour-to-hour and day-to-day operating problems are handled by the RCC, and by means of our short daily briefings on current river and reservoir conditions, the various team elements are kept abreast and provide inputs, as required. The other members of the Water Control Branch team also provide the major effort on development of hydrologic techniques and system analysis.

1/ Presented at "Seminar on Real-Time Operation of Water Resource Projects" on 17 November 1975 at HEC, Davis, California.

2/ Chief, Reservoir Control Center, NPDEN-WC, Portland, Oregon

Other elements in NPD also support the current regulation activities. By far the largest support of these is that provided by the Automatic Data Processing Center, but also included is the support by the Operations Division, Hydroelectric Design Branch, and others in the NPD Office. Because of the highly centralized operation required for the Columbia River System, the District Office activities in reservoir regulation are largely in a supportive role. The Districts, however, supply very important contributions to the coordinated regulation of the system by providing current information on operating problems at the District and project levels. For this reason, the District offices are also "plugged in" to the daily river and reservoir briefings using telephone conference facilities.

On a broader scale, the regulation of the Columbia Reservoir System requires the coordinated effort and cooperation of a large number of agencies and entities. This characteristic of the Columbia system results in a highly complex organizational and management process that must be dealt with in a practical way through negotiation, adjustment, and tolerance. The regulation of the Columbia, being an international river, also involves coordination with our neighbors to the north in Canada. The cooperative effort of Federal operating agencies in the U. S. (the Corps of Engineers, Bonneville Power Administration and Bureau of Reclamation), supporting U. S. agencies (particularly the National Weather Service Portland River Forecast Center, the U. S. Geological Survey, Soil Conservation Service, and National Marine Fisheries Service) and numerous non-Federal public and private utilities, must all be combined to meet the goals of system control and management. The major role of the RCC is to provide leadership among these operating groups in integrating the day-to-day regulation in the interest of multi-purpose rather than single purpose operation.

Our efforts are often involved more in coordinating than controlling the river system, particularly to accommodate special operating requirements that are occurring constantly because of the interactions between the projects of various ownerships and operating authorities. With these ideas in mind, you can understand that the complexities of operation require computer analysis of the system as a whole in order to make the necessary judgments for operating alternatives and decisions.

CENTRALIZED REAL-TIME MANAGEMENT

There is a good reason for all of the following definitions to be applied to the letters "RCC":

a. Reservoir Control Center.
b. Regulation Compromise Center.
c. Rumor Control Center.
d. Reservoir Communications Center

We find also that "RCC" is confused with other centers, such as the National Weather Service Portland "River Forecast Center" (RFC) and the Bonneville Power Administration Dittmer "System Control Center." Centralized real-time management is logical for other agencies as well as the Corps. The Columbia Reservoir System is complex and vast, with over 77 major projects in the basin. Of these 77 major projects, 29 are Corps of Engineers multi-purpose projects, 20 are Bureau of Reclamation multi-purpose projects, 23 are non-Federal hydroelectric projects, and 5 are projects located in the Canadian portion of the Columbia Basin. The NPD Water Control Branch has been highly involved in coordinating these projects into an integrated system. The Reservoir Control Center is the real-time communications element in the Water Control Branch helping the NPD Office carry out its regulation responsibilities, including those at non-Corps of Engineers projects. The Columbia Reservoir System is dynamic, constantly changing in details. Occasionally, even frequently, there are basic changes such as the addition of Lower Granite Lock and Dam earlier this year, the initial generation from Libby last summer and the addition of the first 600 megawatt unit at Grand Coulee last month.

In total, the Division Office is responsible for regulating 25 of the 30 projects operated by the Corps in the Pacific Northwest. The five Corps projects not regulated by NPD are regulated by District offices because they have no hydropower facilities and are either not in the Columbia Basin or are regulated for local effects. In addition to scheduling the regulation of 25 Corps multi-purpose projects, the RCC coordinates the regulation of numerous non-Corps projects for special purposes on a seasonal basis when appropriate. Monitoring and coordinating individual projects and combined system operational effects in the highly developed Columbia Basin is a vast and complex task assigned to the RCC as part of the Water Control Branch specifically and to the NPD office in general.

"Real-time management" of the Columbia River is a relative term. In our sense of the word, current day-to-day management is the real-time real-world management as compared to long-range planning, theoretical operation or reconstitution studies. When project operating personnel, computer or electrical utility representatives talk of real-time operations, they are usually referring to fractions of a second. Others mean hourly schedules. The Reservoir Control Center is involved in day-to-day and hourly operation, as well as the seasonal regulation of the reservoir system.

It is interesting to compare the classic long-range predevelopment planning viewpoint with real-time reservoir management. While there are many similarities, in one basic way they are on opposite sides of the coin. Planning has as one of its primary objectives the determination of the best level for future development whereas real-time management looks for the best regulation of a given system. In the case of planning, a method of operation is assumed and the level of development is the unknown; whereas, in the case of real-time management, the level of development is given and the objective is to determine the best method of operation.

In the North Pacific Division, the Reservoir Control Center is in a good position within the organization. It is in the Engineering Division and in the same Branch where much of the long-range water resource system planning and analysis is done by the Hydrologic Engineering Section and the Power Section. While the Reservoir Control Center is involved in the day-to-day operation of the projects, it is appropriate that it be in the Engineering Division where there is considerable related activity in the other sections. In this way, we can better assure that the projects are regulated to accomplish the results and benefits for which they were originally intended.

The RCC can be compared to the quarterback on a professional football team. Routinely he calls signals and develops short-range policy. While the spotlight may be on one player, every player is important and it takes total team effort and support for success. The long-range strategy and plays have been developed and diagrammed by others (Water Control Branch and District Offices). Occasionally the coach (Branch Chief) sends in a special play while the team manager (Division Engineer) monitors overall results for the good of the owners (public).

REGULATION EXAMPLES AND OPERATIONAL PROBLEMS

During periods of high flow, the Reservoir Control Center sends operating instructions to numerous major storage reservoirs (both Corps and non-Corps), <u>scheduling daily and hourly</u> releases in order to keep flows downstream within bankfull and to meet other control objectives. During periods of normal flow (after consultations with BPA), the RCC <u>schedules hourly generation</u> at eleven Corps storage reservoir projects, while at nine Corps major run-of-river projects <u>operating limits are scheduled</u> (usually on a seasonal basis). At these nine major run-of-river projects, BPA schedules hourly generation within the forebay limits set by the RCC. Generation at these nine projects is instantaneously changed automatically by BPA's automatic generation control (AGC) equipment in order to maintain power system stability. Frequently the normal maximum or minimum forebay limits at these run-of-river projects are temporarily changed by RCC to accommodate construction activities, fishery research, streamflow measurements, special navigation requirements, recreation enhancement, extra power generation, etc. The RCC also routinely <u>schedules daily releases</u> for five Corps projects without hydropower facilities.

Not infrequently, power interests desire to operate a reservoir outside of its normal range either to meet short-term heavy loads or briefly store surplus water. The Reservoir Control Center in these cases must decide, after consultation with other elements in the Water Control Branch and District Offices, if these requested short-term changes can be accommodated without adversely affecting other project functions or activities. For example, navigation and irrigation pumping could be seriously affected by low reservoir stages. Also fishery or recreational problems can result from some of these requests. If the storage reservoirs are drawn too low for power or for flood control, this could jeopardize refill and adversely affect recreation and future power capability. At the present time in the North Pacific Division there is considerable construction activity at several of the mainstem projects. Powerhouses are being expanded and spillway deflectors are being constructed. Generating unit and navigation lock and channel maintenance also add complexity to the system. To further complicate the reservoir system operation there are numerous requests received to provide special regulation for low flow augmentation, non-project construction activities, movement of special equipment under bridges, movement of log rafts, search and rescue, special flow conditions for fish, boat races, ship launchings, agricultural activities, etc. All of this and other conflicts must be taken into account in scheduling project releases and reservoir elevations on a day-to-day and seasonal basis. Equitably satisfying the needs and interests of the people of the Columbia Basin and the Pacific Northwest through the coordinated operation of the total reservoir system is the daily challenge of the staff of the Reservoir Control Center.

REAL-TIME USE OF COMPUTERS

Computer applications are extensively used in the North Pacific Division by all sections of the Water Control Branch. Within the Reservoir Control Center the following six computer programs or group of programs are the most frequently utilized:

a. SSARR Package

(1) Streamflow Synthesis and Reservoir Regulation (SSARR)
(2) Hydro-Power System Regulation Analysis (HYSYS)
(3) System Reservoir Regulation (SYSREG)

b. Willamette Generation (WILGEN)

c. Columbia River Operational Hydromet Management System and the Columbia Basin Teletype Data Network (CROHMS-CBTT)

d. Graphic Display Programs (CRT-LSD)

As Dave Rockwood discussed in the preceding paper, the SSARR package has been developed for hydrologic and reservoir systems analysis required for day-to-day regulation. The package has been integrated to provide uninterrupted operation of the specialized functions of simulations for reservoir operation without intervention by the user. Each of the basic elements can be operated either interactively, or in batch mode. The package can operate as a complete system, or, for analysis which requires evaluation for one of the functions, that segment of the package may be operated independently. All of the elements of the package are designed primarily for "real-time" operation, but they may also be used in planning type studies.

The basic SSARR model consists of three elements: (1) a hydrologic watershed model, (2) a river system model, and (3) a reservoir regulation model. The first element is concerned primarily with sub-basin runoff. The second element, river system, routes streamflows either free-flowing or as constrained by backwater effects of reservoirs or tides. The third element, reservoir regulation, computes the flow through reservoirs on the basis of several alternative regulating criteria and determines regulated outflows which are routed to develop flows and river stages at all desired locations downstream. The SSARR model is utilized in NPD to forecast streamflow and storage reservoir conditions. This program computes and plots flows at specified control points for a selected number of days into the future on the basis of initial and anticipated hydrometeorological conditions and reservoir regulation plans. It is predominantly a river and reservoir systems analysis program incorporating hydrologic analysis routing procedures, snowmelt and precipitation data and reservoir regulation. This program is used by NPD daily during the winter and spring flood seasons and during the early part of the summer holding season. It is also used weekly by NPD during the fall and winter storage control (draw down) season to compute the results of probable streamflow and stipulated reservoir regulations.

The application of the SSARR model to river forecasting is accomplished routinely on a day-to-day basis by the Portland River Forecast Center, NWS, supported by both the Hydrologic Engineering Section and the RCC of the Corps.

The HYSYS Computer Program is designed to analyze hydro-power system operation on an hour-to-hour or day-to-day basis. As an adjunct of the SSARR package, the program is used to determine the hourly or daily project schedules for a system of hydropower and storage reservoirs based on predicted regulated inflows and system power loads. Detailed computations are performed to simulate channel flow and tailwater characteristics taking into account any backwater effects. The program results serve as an operating guide to the Reservoir Control Center and provides data within the time required to make necessary decisions. Emphasis is placed on rapid turn-around time yielding the maximum useful information while requiring as little manual input data as possible. A main feature of this program is the technique used to simulate the distribution of the system power load among hydropower projects. Requirements in sequence are evaluated by successive approximations and adjustments and then made to accomplish specified objectives. Generation priorities among projects are based on balancing reservoir pools within specified target elevations. These target levels may be violated if necessary to meet the system power load or to minimize spill. This application of a target technique provies the ability to select strategies that approximate optimum multiple-purpose operation policies and readily allows changes in system operating priorities. Normally the HYSYS program is used by the Reservoir Control Center daily except during flood season to simulate the hourly operation for a period of four days in the future. However, periods of longer duration and time increments up to 24 hours may be simulated.

The SYSREG Computer Program is designed primarily to regulate a system of reservoirs for downstream control. It performs trial simulations for a given time frame, using trial reservoir outflow or generation values. Trial values are finalized from adjustments based on maintaining balanced storages within target elevations if possible. The program then increments the next time period and repeats this process. By adjusting the project values rather than recomputing them, consistency is maintained from one period the the next. If the computed operation does not provide desired results, the engineer can intervene with the simulation and manually specify the project operation or respecify target ranges to accomplish the desired results. The SYSREG program will automatically vary project regulations to meet downstream control criteria if desired. This program is used by the Reservoir Control Center at the present time as an adjunct of the SSARR model, primarily for daily regulation of the Willamette Basin reservoirs to meet downstream flood control and low flow augmentation objectives.

In summary, the SSARR package of programs includes the basic SSARR model and the HYSYS and SYSREG Computer Programs. All three are used extensively in the daily operation function but in different modes at different times of the year. They may all be used interactively whereby the regulating engineer may control the processing as the simulation progresses and he may interject or modify input to satisfy various functions. The HYSYS program simulates the regulation based on system power demands, and SYSREG simulates the regulation based on downstream flood control or low flow requirements. All programs regulate within given constraints and regulation schedules can be conveniently supplied.

The WILGEN Computer Program is a relatively straightforward computation of generation schedules determined after basic decisions have been made, such as how much water is available. The program is used primarily to compute and format hourly generation schedules for the Willamette Basin projects.

The CROHMS-CBTT Computer Programs are an extensive group of data processing programs. They are used in the collection, verification, processing, tabulating, listing and plotting for a vast amount of project and hydromet data of numerous types. These are intensively used daily by the RCC and others.

The CRT-LSD group of programs are used in conjunction with the cathode ray type (CRT) terminals and with the large screen display (LSD) system used in the Reservoir Control Center. These are relatively new applications at NPD and are used daily at the present but will be used even more extensively after computer facilities have been upgraded.

An essential element in computer utilization for real-time management is the convenience with which input data can be supplied. Timely information is necessary. Accuracy is important but can be second to speed. User control and manipulation is vital. It should be understood that the NPD computer programs do not automatically optimize reservoir regulations and neither do they automatically send operating instructions to projects.

COLUMBIA RIVER MANAGEMENT OVERVIEW

Real-time management of the Columbia River can be conveniently discussed using the four following subject areas:

a. Seasonal hydrology
b. Types of reservoirs
c. Multi-purpose functions
d. Interagency coordination

Seasonal Hydrology. The Columbia River high flow period occurs during the spring due to snowmelt. A portion of these high flows are stored and then held in reservoirs over the summer, providing good conditions for recreation. The reservoirs are then drawn down in the fall and the winter, providing power generation and normally the elevations become low enough to store the next spring's high flow runoff. West of the Cascade Mountains the hydrology differs mainly in that winter is the flood season due primarily to rainstorms.

Types of Reservoirs. Many of the so-called "reservoir" projects are actually run-of-river projects, or as we call them, pondage projects. Some are true annual storage reservoirs which refill each year and some are cyclical reservoirs which will not refill every year if drawn all the way to the bottom. One of the basic differences in types of reservoirs are those projects with power generating facilities and those without power.

Multi-Purpose Functions. As in other parts of the country the seven main purposes for which we regulate reservoirs are as follows:

a. Flood control
b. Hydro-power
c. Navigation
d. Irrigation and water supply
e. Recreation
f. Fishery and wildlife
g. Water quality and low flow augmentation

Interagency Coordination. This is a major factor in real-time management of the Columbia River. With so many different operating agencies and interests in the Columbia Basin, coordination is essential. This is one of the primary functions of the Reservoir Control Center and an entire subject within itself. The three agencies, Bureau of Reclamation, Bonneville Power Administration and the Corps of Engineers, are operating agencies in the Federal Columbia River Power System. The Canadian projects operated by British Columbia Hydro and Power Authority are an essential part in the Columbia System. The RCC coordinates directly with B. C. Hydro. Also there are special international treaty committees established to coordinate regulations. The Columbia River Water Management Group is comprised of representatives of numerous Federal and State agencies involved in water resource management. This is one of the most important groups facilitating interagency coordination.

Of special interest and value to the RCC is the Columbia River Forecasting Service which NPD, the NWS Portland River Forecast Center (RFC) and BPA have joined to coordinate forecast activities. There are significant mutual benefits in such a relationship. Suffice it to say that the RCC works as closely with RFC and BPA as with other offices within the Corps.

It is interesting to note that BPA has two operating offices manned 24 hours a day, seven days a week. The power dispatchers talk in terms of megawatts, whereas the water schedulers talk in terms of c.f.s. RCC obviously deals mostly with the BPA Scheduling Section. Our contact with BPA is so frequent and so extensive that a thick Memorandum of Understanding between our two agencies has been developed to facilitate coordination.

The coordination among operating agencies in the Pacific Northwest would not be complete without mentioning the Northwest Power Pool and the Pacific Northwest Coordination Agreement. The operating agencies have signed an agreement which gives downstream projects rights to upstream storage in exchange for energy or dollars. Upstream storage reservoirs receive headwater benefits. In this way more firm power capability is provided for the good of the total system. There is interchange of energy between reservoir operators. Energy in lieu of water releases can be made when appropriate and under certain conditions. In some cases to prevent spilling water, generation can be provided to another reservoir operator so his reservoir can store water which then belongs to the generating party. The coordinated operation of the Columbia System is an intricate, complex network which operates to the advantage of the region. RCC representatives attend meetings of the Northwest Power Pool while the Power Section provides the NPD representative on the Coordination Contract Committee.

At the present time there has been in operation for several years a special coordinated operation of the mid-Columbia hydro-electric projects. This is referred to as the Mid-Columbia Hourly Coordination. In this operation, the downstream reservoirs are kept as full as possible, being primed by the large Grand Coulee Reservoir at the upstream end of this reach of the river. This reduces forebay fluctuation and permits higher heads to be maintained at the downstream projects which enables the system to produce more power.

SUMMARY OF PERTINENT RCC DAILY ACTIVITIES

At the end of each day, the RCC prepares a Daily Log and at the end of the week a Weekly Summary Log is prepared, covering pertinent reservoir regulation activities of interest. Both the Daily Log and the Weekly Summary Log are transmitted to all projects and the District Offices via the Columbia Basin Teletype (CBTT) to keep the projects and districts as fully informed as possible.

The RCC sends hourly generation schedules by the Columbia Basin Teletype to some Corps projects each afternoon for the following day. On Fridays, a schedule is transmitted to the projects for the entire weekend.

The RCC conducts a briefing for water management personnel in the Division Office, including Chiefs of the Water Control Branch, Engineering Division, Public Affairs Office, Operations Division and others. The briefing is open to everyone and is held at 1:30 each afternoon in the briefing room. The first portion of the briefing is conducted by a National Weather Service meteorologist and a hydrologist.

We have found in the RCC that extra hours are required during the workday and, therefore, one engineer is scheduled to stay on duty for an hour after normal office hours. In this way the engineer is able to contact the swing shift operators at the projects and at BPA, complete regulation instructions and write the Daily Log. Each engineer in the RCC is subject to being called in the evenings, at night or on weekends, when necessary. Weekend overtime work is scheduled during the winter and the spring flood season. This is a burden but there is also the excitement and responsibility attached to the job. The hydraulic engineers in the RCC are not just mechanics following inflexible rules. Considerable judgment is required since there is usually some flexibility in a large system such as the Columbia Basin.

COMMUNICATIONS AND DATA MANAGEMENT

The Columbia Basin Teletype Circuit (CBTT) is a primary means of communication and coordination among reservoir operators within the Pacific Northwest. The RCC manages this circuit on a cooperative basis. At the present time the circuit uses Model 35 teletypewriters leased from the telephone company in an 8A1 communications system which includes a line controller permitting only one station at a time to transmit, but any numer or all of the stations can receive simultaneously when appropriate. Non-Corps, non-Federal terminals on this circuit are billed separately each month. Project and hydromet data are transmitted over this circuit to everyone interested. Generation schedules are sent to some projects and hourly operational information is sent from projects to others. Regulation instructions are transmitted so that a written copy is available for reference purposes. A merger of the CBTT with the Columbia River Operational Hydromet Management System (CROHMS) is planned to take place in the near future. Eventually we expect Data Speed 40 terminals to used in the CROHMS-CBTT network in place of some of the existing teletypewriters.

At the present time the NPD computer is an IBM Model 360-50 general purpose computer with Model 20 computers located in the District offices. The RCC computer programs are done on this computer and occasionally, frequently even, we have problems which we feel would not have occurred if we had a dedicated computer available for water management purposes. The plans for CROHMS include an IBM 370-155 general purpose computer which is expected to be a big improvement even if it is not a dedicated one.

PLANS FOR IMPROVEMENT

One of the continuing objectives in the RCC is to make improvements. As the system grows this becomes all the more necessary and important. Some of the areas where we need and plan to eventually make improvements are:

a. Systematic organization and training
b. Computer upgrading (CROHMS)
c. CBTT Data Speed Model 40 terminals
d. Reservoir regulation manuals
e. Microfiche data records
f. Documentation of project historical operational data

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TESTING A METEOR BURST DATA ACQUISITION SYSTEM IN ALASKA

by

GARY R. FLIGHTNER 1/

INTRODUCTION

Alaska, the largest State in the United States, contains 571,000 square miles and is more than two times larger than its nearest competitor, Texas. Alaska is the most northerly, easterly, and westerly State in the Union. It contains the highest North American peak, the 20,320-foot Mount McKinley, and one of the longest rivers, the 1,979-mile long Yukon. It sweeps from the steep-cliffed fjords of the southeast to the forests of the interior and the arctic tundra. It has the greatest of the nation's glaciers and a range of weather, which includes the temperate coast (warmed by the Japanese current) and the frigid polar frontier. Alaska's seacoast is longer than the combined seaboards of all other States touching the Atlantic and Pacific Oceans. One-third of the freshwater runoff of the entire Nation is found in Alaska as well as scores of untapped sources of hydroelectric power.

Corps activities in the State of Alaska are as varied and scattered as the State's size and topography. Activities in the State's southeast panhandle region consist of construction of navigation projects to provide safe harbors and moorage basins at small communities, and the construction and study of hydropower developments to furnish the power requirements of the population centers. The first stage of the Snettisham project is now complete and is supplying power to Alaska's capital city, Juneau. The Corps is also busy near the State's largest interior city, Fairbanks, constructing a flood control project to protect the city from floods, of which the most devastating was the August 1967 flood that inundated the entire city and caused damage totaling hundreds of millions of dollars. Federal costs are estimated at $134,000,000, and the project will be operational in about three years. The Corps also aids the State and local communities in the prevention and control of floods. Rivers are "dusted" during the month of April at river reaches where ice jams have historically occurred, resulting in the flooding of riverbank communities. Spring activities also include river flood reconnaissance activities so that flood fights and evacuation can be initiated if required. Military construction and maintenance of military facilities' activities are performed by the Corps over the entire State of Alaska. Planning of future civil works projects includes such potential hydropower projects as Rampart on the Yukon River and the development of two major dams on the Susitna River--the 635-foot-high thin-arch concrete Devil Canyon Dam and an 810-foot-high earthfill Watana Dam.

1/ Chief, Hydrologic Engineering Section, Alaska District, Corps of Engineers

In summary, the Corps' activities in Alaska are numerous, diverse, and scattered over a large remote area. These activities require communication support, which in most areas is not presently available. The meteor burst communication media is one of the alternatives that may provide this communication requirement. To evaluate its potential, several governmental agencies shared in a cooperative testing program in FY 75.

METEOR--SCATTER PHENOMENON

The fact that particles of matter frequently enter the earth's atmosphere from outer space has been known for centuries. In the 1930's, it began to be suspected that these particles, or meteors, created an ionization trail as they passed through the upper atmosphere. It was not until the mid-1950's, however, that attempts were made to use this ionization phenomemon for further meteor research and for long-range VHF communications.

During the late 1950's and 1960's, several governmental and industrial organizations, including the Boeing Company, developed communication systems to operate via reflections from ionized meteor trails. The mode of communication is by reflection off of meteorite trails that are continuously occurring in the region from 50 to 70 miles altitude. Experimental and research data collected by optical and radar techniques have provided considerable insight into the phenomenon of meteors and their effects on the ionospheric region of the atmosphere. It has been established that an average of more than 10^{11} meteors strike the earth's atmosphere each day. The size of these meteors range from less than 40 microns to more than 4 centimeters in diameter.

Approximately five percent of the meteors which strike the earth's atmosphere are members of identifiable groups, which occupy particular orbits within our solar system. These groups form elliptical bands of orbiting particles, which intersect the earth's orbit at various points in its orbital path. The earth's intersection with these bands of particles causes a higher-than-normal rate of influx of meteors, or the creation of "meteor showers," as they are sometimes called. Many of these showers occur at roughly predictable times during the earth's seasons. The remaining 95 percent of the meteors which enter the earth's atmosphere are caused by a more uniform distribution of particulate matter which exists within our solar system. This more general distribution of particles is concentrated in the ecliptic plane of the solar system. Since this is the case, the rate of intersection of the meteors with a particular geographical segment of the earth's atmosphere depends on the time of day, the season of the year, and the particular geographic location. Due to the earth's rotation about its axis, as well as its forward velocity, the greatest number of intercepts with meteors occurs in the early morning hours with a minimum number occurring in evening hours, and this diurnal variation is approximately sinusoidal (Figure 1). The seasonal variation is also approximately sinusoidal with a high in July-August and a low in January-February (Figure 2).

IONIZATION OF THE ATMOSPHERE BY METEORS

As meteors enter the ionosphere, or outer region of the atmosphere, they begin to collide with molecules of the atmosphere. This collision process causes molecules of the meteor to break away from the meteor body, raises the temperature of the meteor, and causes both meteor and atmospheric-molecule atoms to be ionized. The ionized atoms and free electrons left along the path of the meteor as it passes through the ionosphere remain to form an ionized track or trail. This ionized trail is called the meteor trail for the purposes of radio transmission. Meteor trails, consisting of free electrons and ionized atoms, exhibit a conductance, or ability to support electric currents. Optimum size of meteor trails is 6 to 10 meter wave length (30 to 50 MHZ range).

METEOR BURST COMMUNICATIONS SYSTEM

A Meteor Burst Communication System consists of a Master Station and a number of Remote Stations (Figure 3). Operationally, the Master Station transmits a continuous probing signal and when a suitably oriented meteor trail is found between the two stations, the signal is reflected to the Remote Station.

The Master Station can either be at a permanent location, or mounted in a small van for mobility. The Remote Station is small, lightweight, and battery operated.

A typical operational sequence starts when the Master Station continually probes the sky for a suitable meteor trail. The coded digital signal is received at the Remote Station when the meteor trail is properly oriented and is suitably ionized. The Remote Station is in the standby configuration for maximum battery life. When the signal is received, the Remote Station becomes operational, and if the station address is correct, the Remote Station will transmit a block of data to the Base Station over the same meteor trail. When the Master Station receives the remote signal and locks on, the computer will examine the addressing and, if correct, accept the data.

SUMMER METEOR SCATTER TELEMETRY TEST PROGRAM

The National Weather Service (NWS), the Soil Conservation Service (SCS), and the Corps of Engineers (COE) cooperated during July and August 1974 in a summer test of the Boeing Company Meteor Scatter Telemetry System. Approximately 180 hours of testing were performed to identify operation for remote data acquisition applications. The Master Station was located at Elmendorf Air Force Base near Anchorage and one of the Remote Stations was located at the SCS office at Fairbanks, Alaska (Figure 4). A second Remote Station was moved to different locations in Alaska to verify communication operation. The data source for these tests were the devise for Automatic Remote Data Collection (DARDC) multiplexer unit, which is extensively used by the NWS. The stationary Remote Station at Fairbanks transmitted seven DARDC words, while the moving Remote Station transmitted four.

In addition to the DARDC unit, the moving Remote Station had a portable communication unit, permitting limited alphanumeric communications between the Master Station and the Remote Station. Up to sixteen (16) characters can be transmitted in either direction. The moving Remote Station was operated from five different sites in the Fairbanks vicinity, then from selected locations along the Alcan Highway to Delta Junction, Richardson Highway to Thompson Pass, and the Glenn Highway to Anchorage. Also, the Remote Station was flown to Prudhoe Bay with operating stops at Five Mile, Dietrich, and Galbraith Lake Camps.

The daily variation of message reception (seven DARDC words) runs between 80 at the peak hours (6 - 9 a.m.) and 20 at the minimum reception hours (6 - 9 p.m.). These are error-free receptions. Waiting times between messages were measured during the period for minimum activity and a total of 210 messages were received over a five-day period. Of the 210 messages, 97 messages had a waiting period of one minute or less; 27 had a waiting time of two minutes or less; 24 had a waiting time of three minutes or less; and etc., with the longest waiting time being seventeen minutes.

A second Remote Station was moved to a number of typical data sites in Alaska. The significant results were:

(1) Performance between 150 and 275 miles was virtually the same as at 275 miles (Fairbanks Remote Station). Communications were clearly established at each site visited.

(2) Line of sight ground wave operation occurred between 100 and 125 miles.

(3) Both Remote Stations operated on separate meteor trails even with the distance between them as low as 20 miles. Evidence of mutual interference was not indicated at the Master Station.

An additional side trip was made to Prudhoe Bay on the Arctic Ocean. This location is in line with the Remote Station at Fairbanks and the Master Station in Anchorage. Operation communication frequency was approximately twice that of Fairbanks, which is due to a more favorable range for meteor-scatter operation. Peak operation from Prudhoe Bay was 201 error-free messages in the hour between 3:00 a.m. and 4:00 a.m. Enroute to Prudhoe Bay, three operational stops were made at work camps along the pipeline. Approximately 30 error-free receptions were received at each of the pipeline stops during the operational time, which was less than an hour at each site.

The Remote Station was also transported by helicopter about 50 miles from Anchorage to three SCS snow survey courses located in the Chugach Mountains. At each of the sites, ground wave or continuous operation occurred. Thus, at short-range obscured sites, the amount of power used and the shortness of the transmission overcomes multipath conditions and permits operation whenever the Remote Address Station is addressed by the Master Station.

WINTER METEOR BURST TELEMETRY TEST PROGRAM

The February 1975 Winter Test Program of the Boeing Meteor Burst Telemetry System was supported by five Federal agencies which included the Bureau of Land Management (BLM) and the Federal Aviation Administration (FAA), in addition to the NWS, SCS, and COE, who also participated in the summer test. During the testing program, the Base Station was located in Anchorage and remote field units were operated from Fairbanks, the Caribou-Poker Creeks Research Watershed (located approximately 35 miles north of Fairbanks), McGrath, Bethel, Kotzebue, Prudhoe Bay, Dietrich Camp, and Delta Junction, with the first two stations serving as primary source of the data. A total of approximately 40,000 messages were received by the Base Station from the various Remote Stations.

The primary objective of the winter test program was to evaluate the Boeing MBCS under winter conditions in Alaska and to determine if the system was capable of meeting the communication needs of the various participating agencies. Secondary objectives that were established included:

1. To evaluate the seasonal variation rates between the July-August period and the February-March period.

2. To determine the effects of the Alaskan winter climate on the operation of the system.

3. To investigate the communication rate from various locations which were not studied during the summer test period.

4. To determine the adaptability of the system to various kinds of equipment, such as the NWS Automatic Meteorological Observation System (AMOS).

DISCUSSION OF TEST RESULTS

The data rate from the Fairbanks station provided a direct comparison with the summer (July-August) message rate. Figure 5, a copy of one of the graphical presentations prepared by Boeing in its report of the winter test program, shows that the dirunal message rates did vary according to a sine curve with a 4:1 amplitude. However, the relationship for the winter test is not sinusoidal and does not have a 4:1 amplitude as expected. Also, the minimum message rate is much higher than anticipated. The expected 4:1 seasonal variation between summer and winter data is not apparent.

Because the winter characteristics of the Meteor Burst communication media set the limits on the operations of the system, they must be carefully analyzed. Figure 6 is a graphical representation of the diurnal variation in the mean and 90 percentile waiting times for all data obtained during the 30-day winter test period. For most of the day, the mean waiting time is approximately 4 minutes and increases to about 7 minutes during the late afternoon. Also, for most of the day, 90 percent of the messages will be received in 8 minutes or less, and at the worst time of day, 90 percent of the messages will be received in 17 minutes or less.

Because usable meteors occur randomly, the statistical distribution of the waiting times between consecutive messages is an important analysis. Figure 7 shows the relationship of cumulative frequency vs. waiting time for the hourly period with the largest and smallest mean waiting times. The dashed curve, which is for the hourly period with the largest mean waiting time of the worst condition, shows that 18 percent of the messages were received in less than 1 minute, 42 percent in less than 2 minutes, 45 percent in less than 3 minutes, and so on. In one instance, it took 71 minutes to get a good response from the Fairbanks station even though the Base Station in Anchorage continually tried to make contact. All data obtained from the test period were used in defining these relationships including periods of interference from the aurora borealis, improper antenna orientation, etc.

The effects of distance on message rates is shown on Figure 8. The solid curve is the relationship predicted by Boeing Company and the X's are the actual results obtained from the summer test. The expected and observed values are very similar. The circles represent the results from the winter test and no conclusive determination to distance effect is possible. The minimum condition of 40 messages per hour would control the system capabilities.

The data from the Environmental Station indicated that the system operated at temperatures of at least -28^{o}F with no apparent malfunction. The minimum specified operating temperature as supplied by the manufacturer is -22^{o}F.

The only deterrents to Meteor Burst operation during the test period were magnetic disturbances (aurora effects) probably caused by sunspot activity. Based on test data and on tests conducted by others, this condition may exist approximately 10 percent of the time. The effects of the aurora are highly complex and can both enhance and hinder the operation of the system. Since the aurora can result in a serious power drain on remote units powered by a self-contained power source, a protective device should be incorporated into the design of remote units that are a part of an operational system.

CONCLUSIONS

Analysis of the data obtained during the summer and winter tests results in the following conclusions:

1. A seasonal variation in communication rate was observed, yet the 4:1 ratio between summer and winter data was not apparent.

2. A sinusoidal diurnal variation with a 4:1 amplitude was observed in the summer data, but not in the winter data.

3. The anticipated distance effect was observed in the summer data, but no relationship was defined by the winter data.

4. For the worst time period, late afternoon in February, the mean waiting time between consecutive good messages was approximately 7.0 minutes.

5. The effects of the aurora are highly complex and can both enhance and hinder the operation of the system. It is likely that the enhancement effects account for the non-sinusoidal variation in both the seasonal and diurnal relationships as previously mentioned. The adverse effects of the aurora were observed in approximately 10 percent of the hourly operating periods. It appears these can be overcome through slight modification in design and/or operating procedure.

6. The system operated without malfunction at temperatures of at least -28°F.

7. On the average, it took 2.2 attempts by the remote unit to get one good message through to the Base Station.

8. The power consumption measured in terms of watt-hour/received message was extremely small and averaged 66 watt-sec, or 0.02 watt-hour/good message received. (Average value and includes effects of bad messages, incomplete transmissions, etc.)

9. Condition at the remote location had little effect on the operation of the system. Included here were such factors as topographic barriers, antennas, environmental factors, etc., with the exception of metal roofs in the immediate vicinity of remote installations.

10. The system can be readily interfaced to a number of data acquisition devices. The DARDC, AMOS, Boeing's Hydro-Met Unit, and the PCU were successfully interfaced.

11. The data clearly indicated that a meteor burst system, properly designed and operated, can more than adequately meet the data

acquisition needs of the interagency group, including the COE, BLM, USGS, SCS, and NWS.

In reviewing these needs, we would recommend the installation of the following system:

a. A 3000-watt Base Station located in Anchorage--1000 watt transmitter power applied to each of 3 antennas.

b. Sequential polling of the remote units.

c. A multiplexer to route the data to the appropriate office for secondary processing.

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3. Meteor - Scatter Communications, Ball Brothers Research Corporation.

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FIGURES

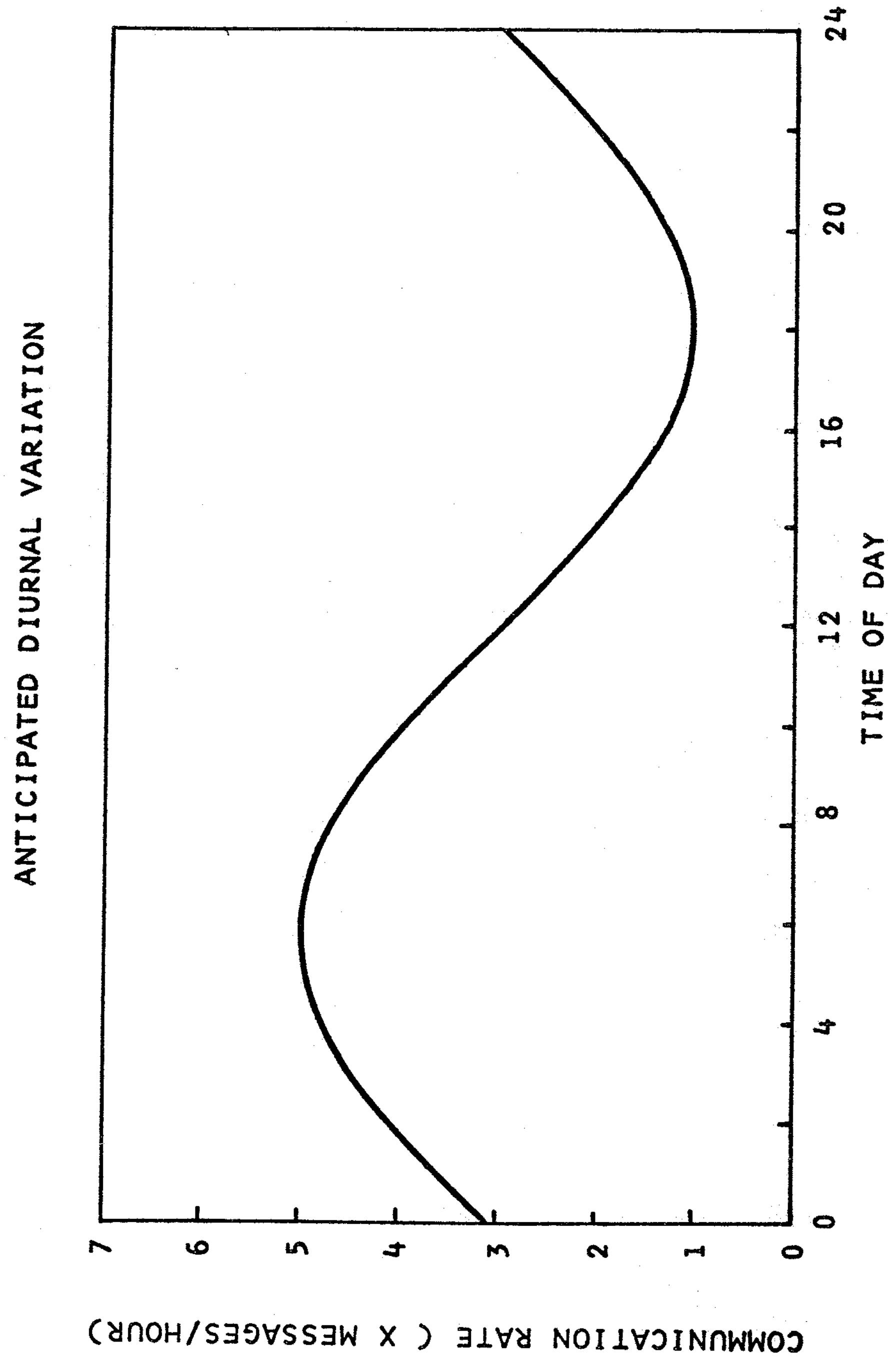


FIGURE 1

Paper 3

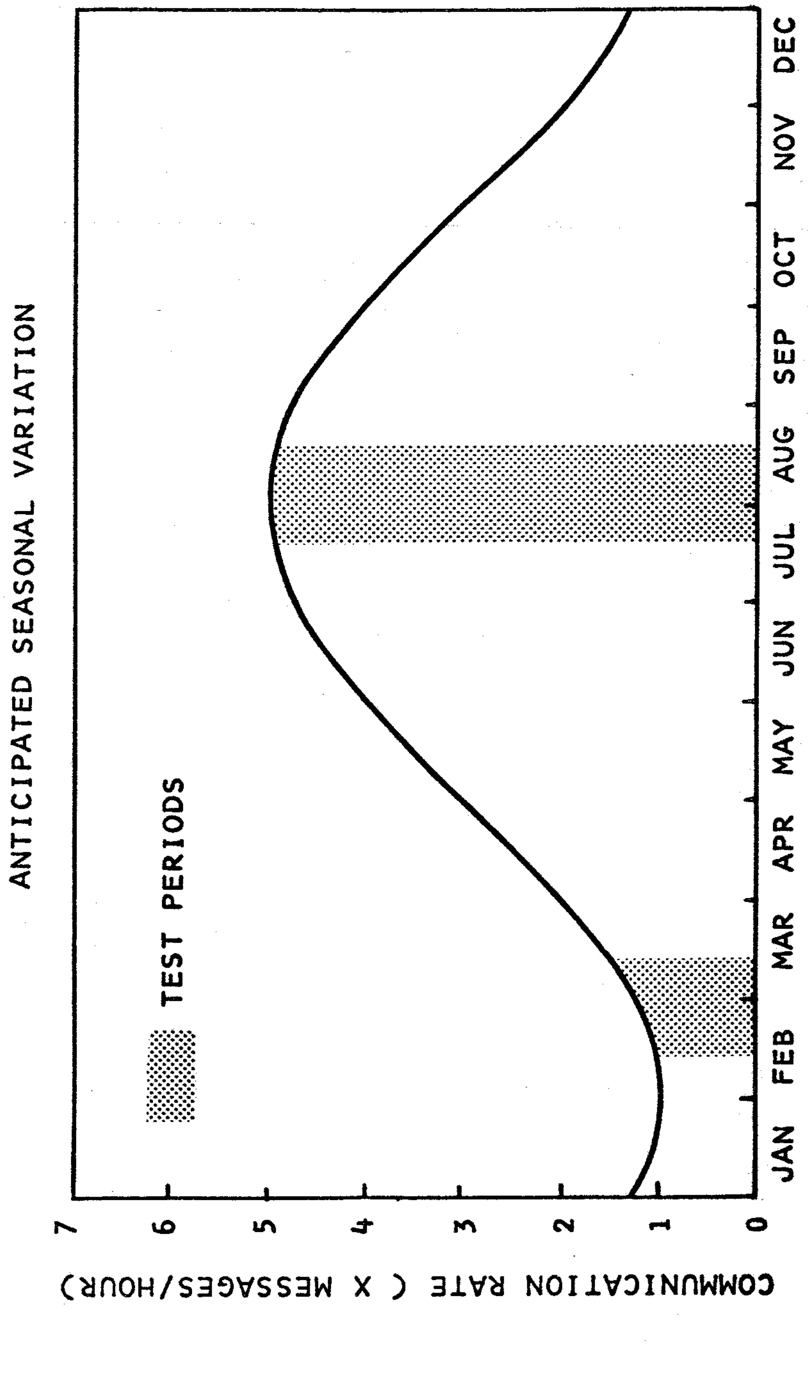
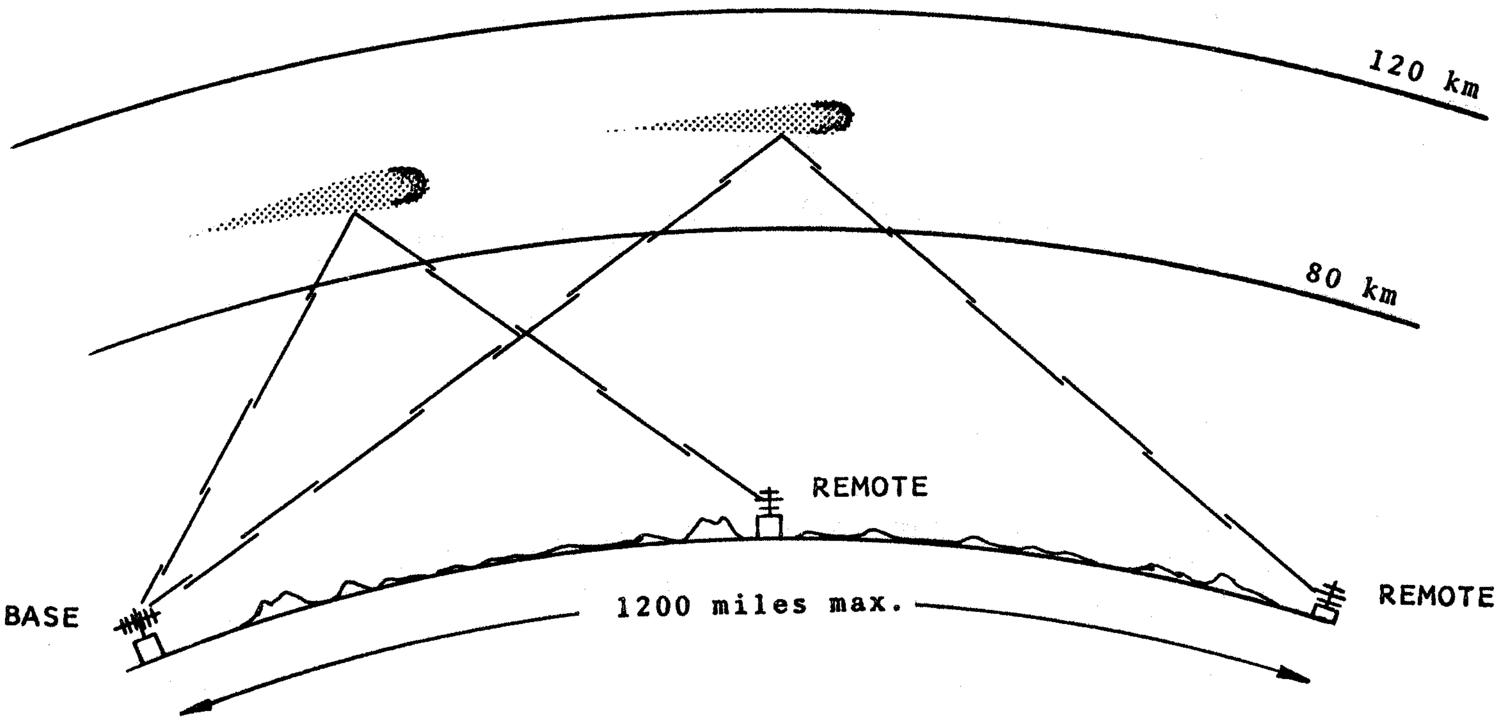


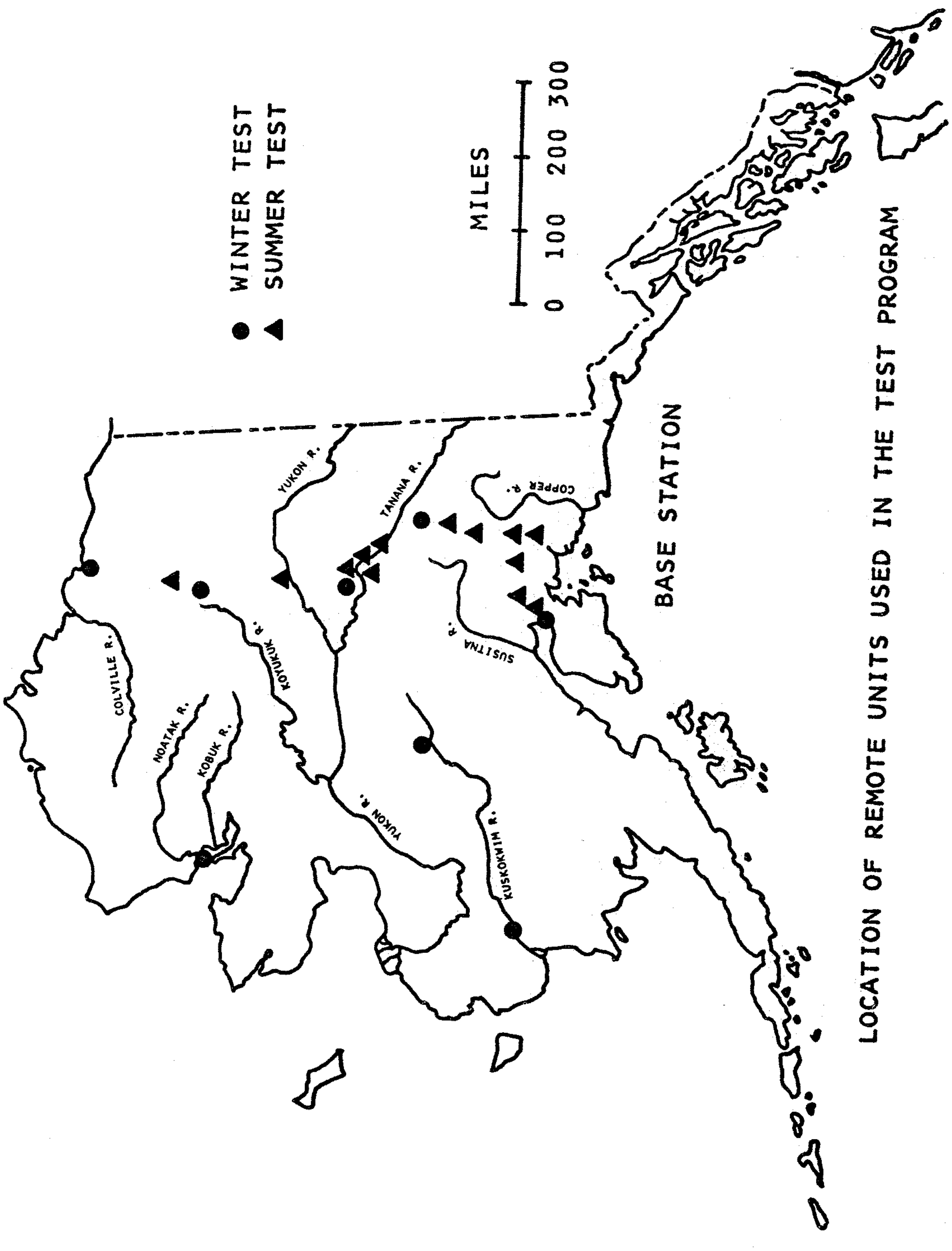
FIGURE 2

Paper 3



METEOR BURST COMMUNICATION SYSTEM

FIGURE 3
Paper 3



LOCATION OF REMOTE UNITS USED IN THE TEST PROGRAM

FIGURE 4

Paper 3

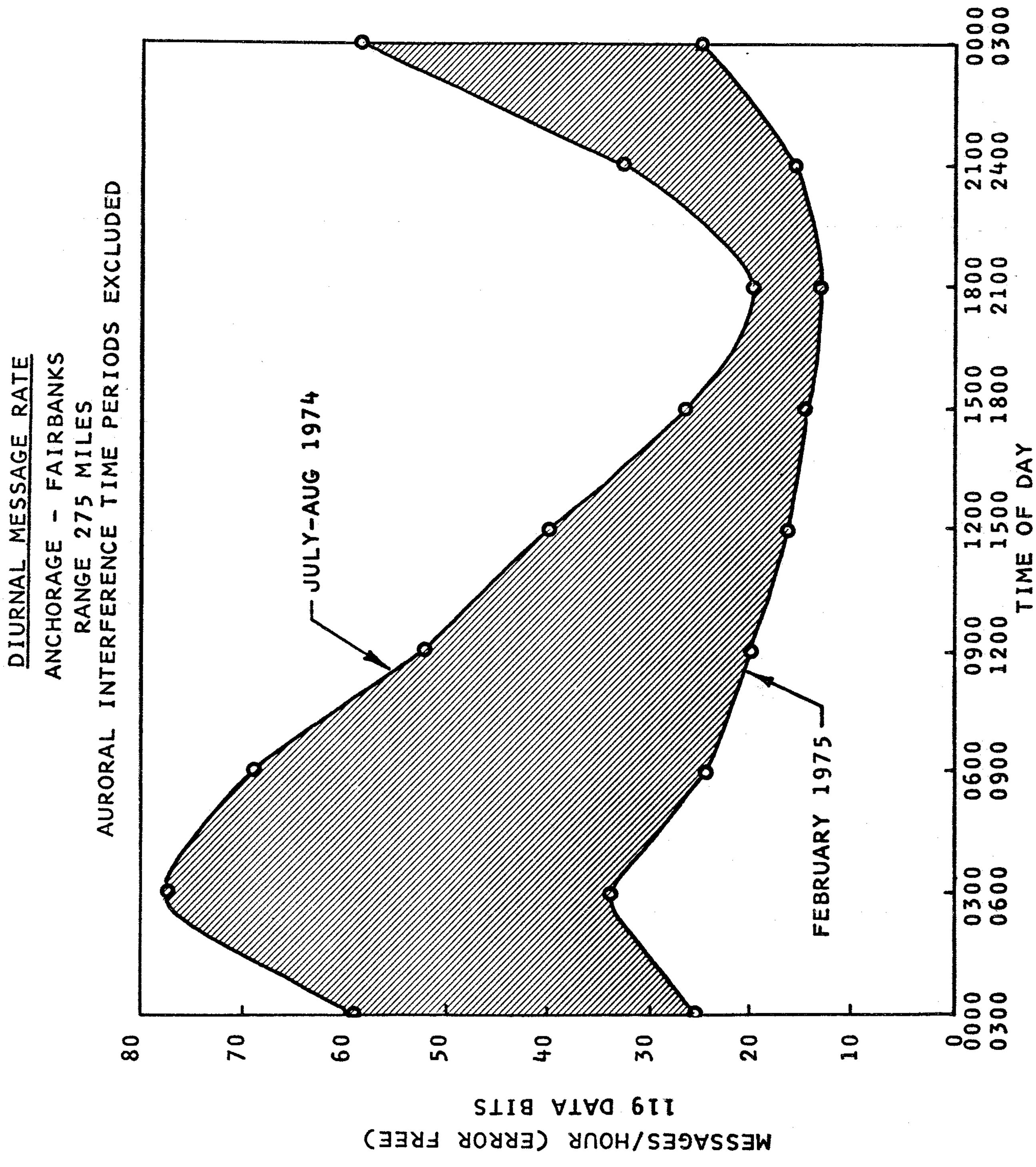
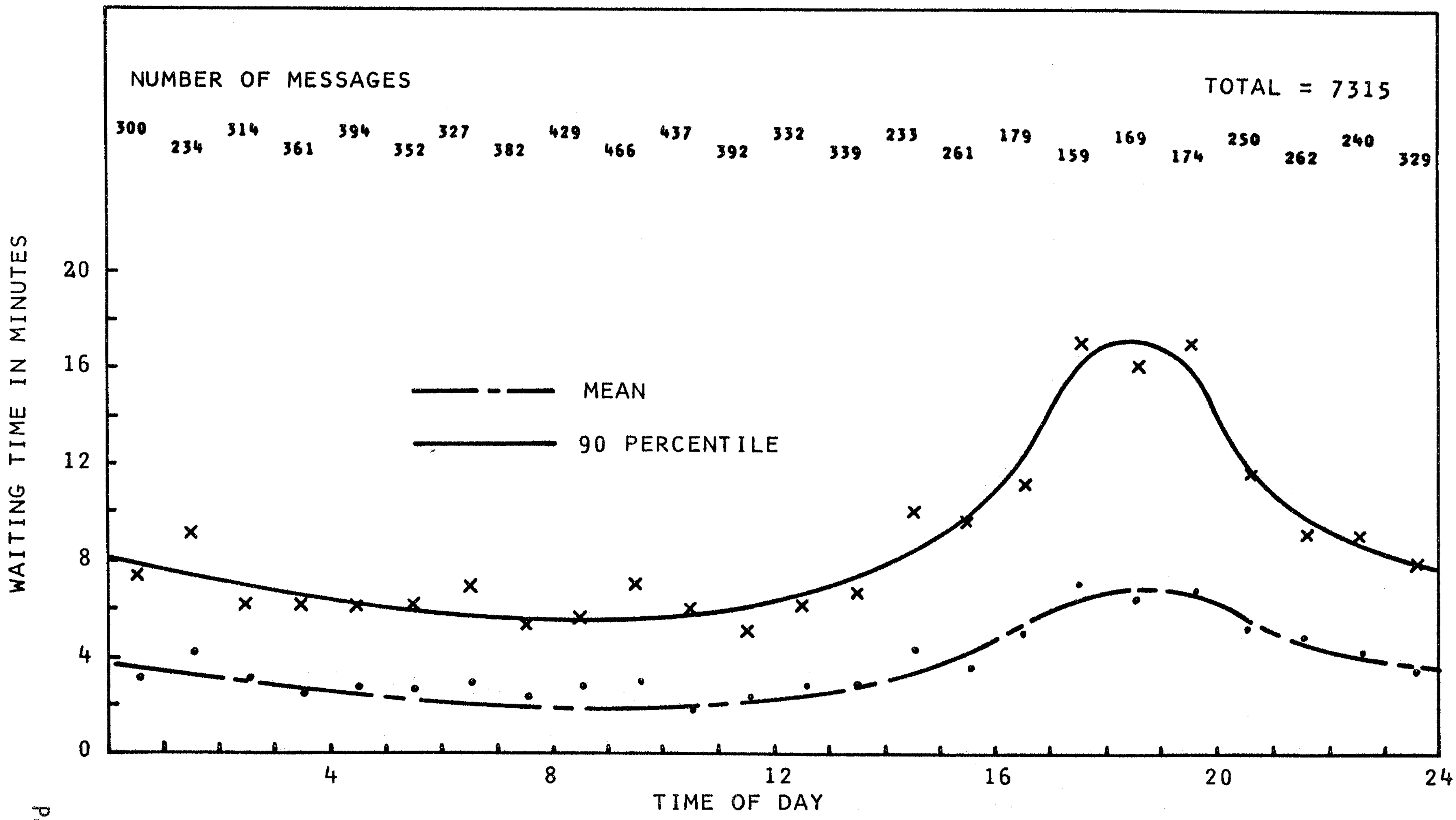


FIGURE 5

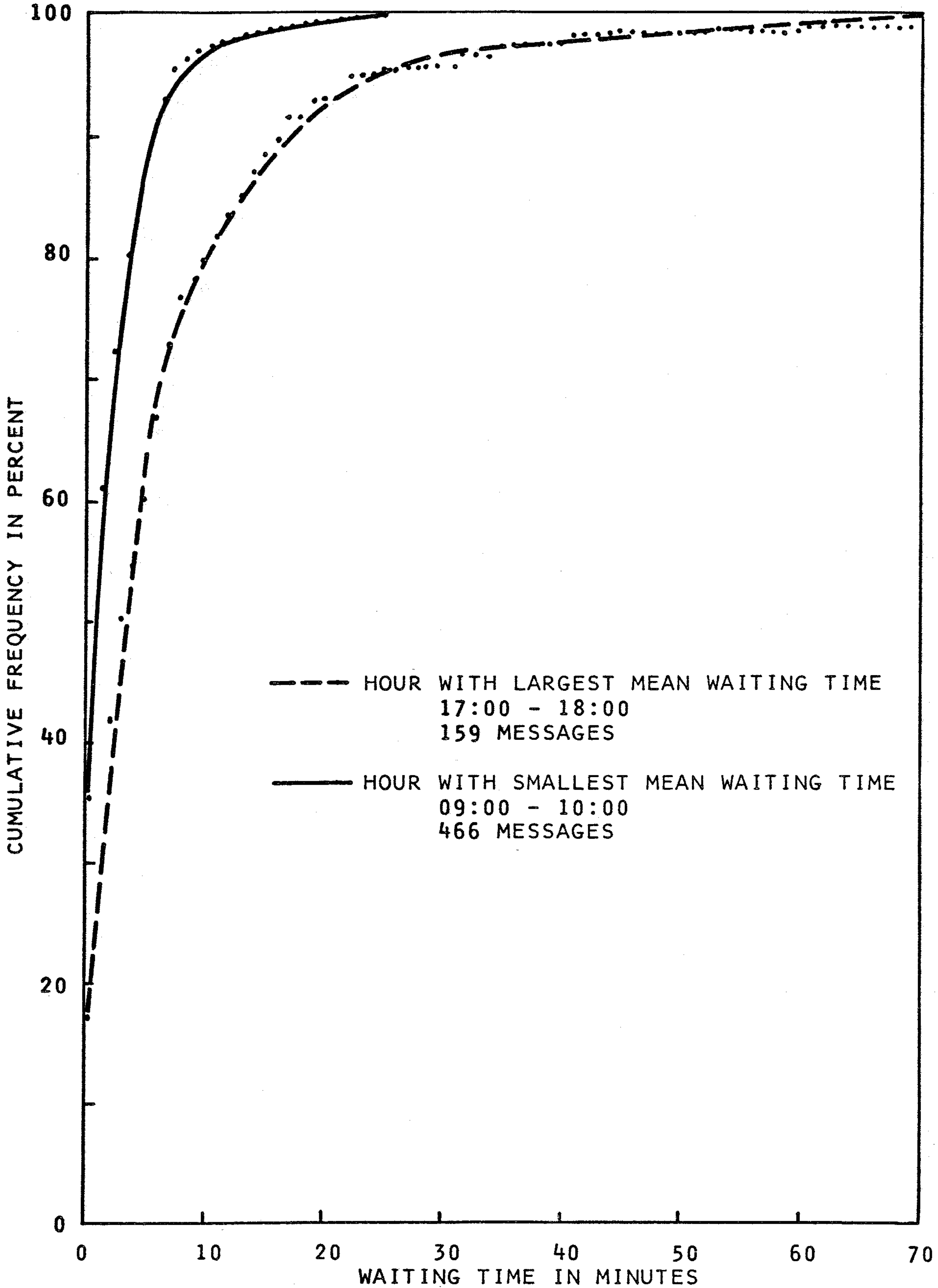
Paper 3



DIURNAL VARIATION IN WAITING TIMES FOR GOOD MESSAGES FROM FAIRBANKS STATION

Paper 3

FIGURE 6



EXAMPLES OF CUMULATIVE FREQUENCY DISTRIBUTION FOR GOOD MESSAGES FROM FAIRBANKS STATION.

FIGURE 7

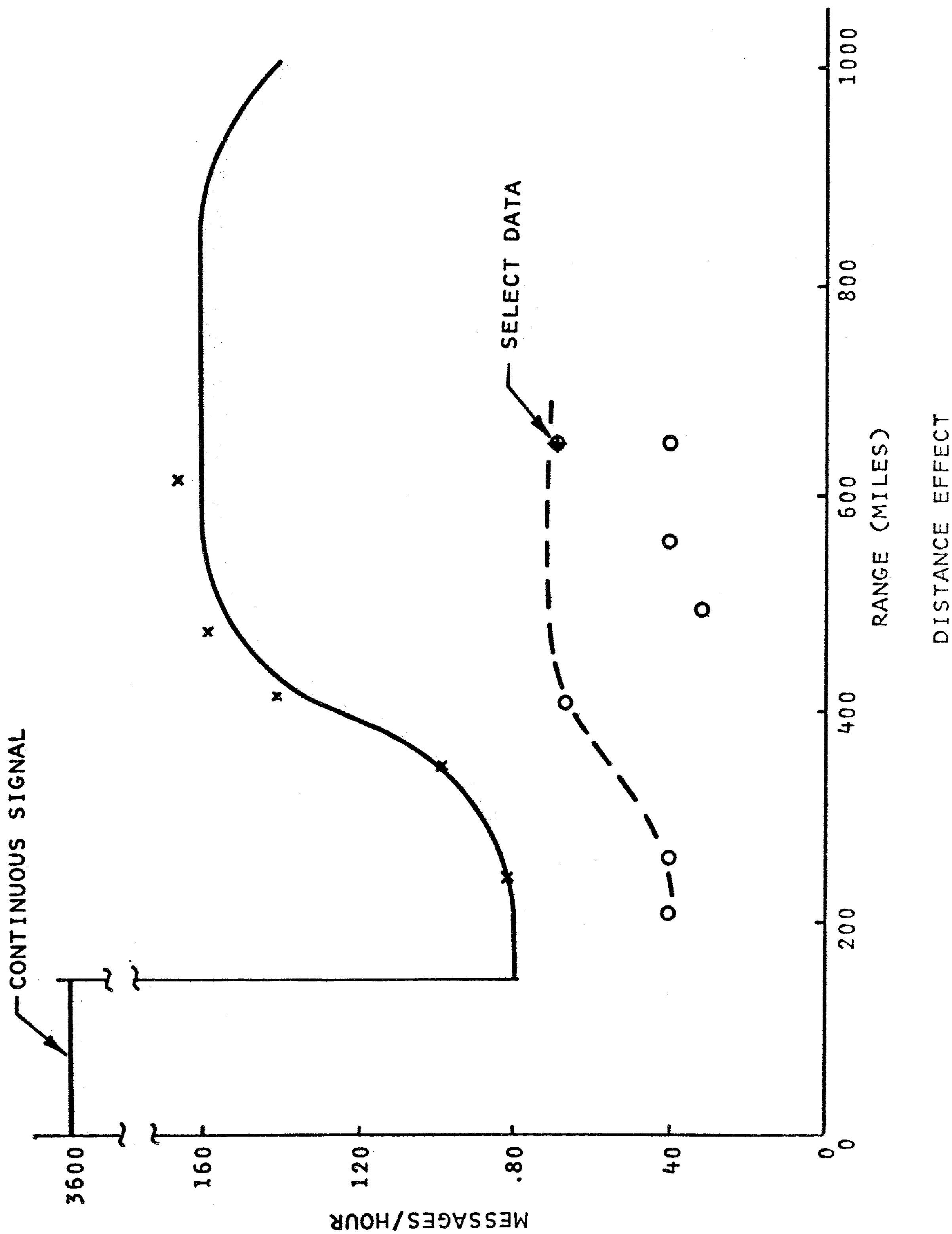


FIGURE 8

Paper 3

PROGRAM FOR LAKE AND RIVER STAGE DATA ANALYSIS AND STORAGE

By

Jerry F. Buehre[1]

This paper will discuss a program developed by the Water Control Section to compute and store the pertinent daily hydrologic information required for project operation and reports. The Kansas City District includes several major drainage basins and, for the purpose of regulation, has been divided along the basin lines as shown on Figure 1. The major basins include the Smoky Hill River basin, containing 20,000 square miles; the Republican River basin, containing 25,000 square miles; the lower Kansas River basin, containing 15,000 square miles; the Marais des Cygnes-Osage River basin, containing 15,300 square miles; and the lower Missouri River basin which contains 35,100 square miles. Within the Kansas City District there are presently eleven operational multipurpose and flood control lakes constructed by the Corps of Engineers and eleven operational lakes including flood control as a project purpose constructed by the Bureau of Reclamation. There are four multipurpose lake projects under construction by the District which are scheduled to be in operation between now and 1980. These lake projects and their status are shown on Figure 2.

For project operation purposes, the District supports 182 stream gaging stations in cooperation with the U.S. Geological Survey and approximately 70 river stage stations under hydro-climatological cooperation with the National Weather Service. In addition, 26 independent stage stations are operated by the District. Within the District there are about 278 river gaging stations for project operation. There are also about 800 rainfall reporting stations within the District boundaries. Data are not received from all of these points during a single event; however, during the recent 1973 flood event, it is estimated that on some days as many as 700 observations were received from the various stations.

Water Control decisions concerning the various lake projects and purposes require timely receipt of the streamflow and rainfall data to provide the best operation for all the various functions, primarily flood control. Therefore, a method for collection, analysis, and storage of the data for further use is necessary for efficient operation. Nearly all of the programs developed by the Water Control Section require the collection and tabulation of daily flow data and data for the preceding time period of as long as 50 days prior.

[1]Chief, Water Control Section, Kansas City District

Therefore, a program has been developed to analyze and print daily hydrologic data for distribution to others as well as store it for use by other programs, such as the forecasting and regulation programs, monthly summaries and RO-1 plotting program, and the project effects program used in computing benefits. Figure 3 shows the relationship of this program to other programs used for water control management and report purposes.

The program has three main subroutines and one small subroutine for timing of saved hydrograph data. The subroutine MAIN sets up input and output files and determines the data base file to use. Two data base files are maintained for computation purposes. Usually the most recent file is used for computation purposes and the oldest file is used as an update file, i.e., the new data base is written over the oldest file each time the program is run. By special control input, the oldest data base file can be used for computation purposes and a new data base file will be created and written over the data base file with the latest data. The data base includes current pertinent data for the lakes; the current pool elevation, precipitation, evaporation, and outflow reported from the project; area-capacity data for each lake; data for RO-1 summary; rating tables for pertinent gaging stations with the latest shift data; and stored hydrographs for the previous 50 days at selected gaging stations. The data base also contains a control which indicates whether to save the hydrograph or not, the date of the last data point entered for the particular station, and the corresponding stage or lake elevation. The current data base contains all of the District lakes in operation, those lakes under construction, and a total of 262 river stations. There are 173 rated river stations where a discharge can be computed for the stage entered, 65 river stations which are not rated but stage data are received, and 24 river stage stations where a lake outflow is read and printed out on the daily information bulletin. This routine sets up six scratch files and calls subroutine RESCMP.

The main purpose of subroutine RESCMP is to print daily lake information bulletins from data collected at the lake projects, compute lake inflow, and update RO-1 data in the data base. The following data are reported for each lake as of 8 a.m. each morning.

a. Pool elevation, feet, m.s.l.

b. Lake outflow, c.f.s.

c. Rainfall, inches

d. Evaporation, inches

e. Lake outflow to canal (as many as two canals), c.f.s.

f. Inflow in addition to river (canal), c.f.s.

The data required for computation of lake data include the above information with the addition of lake identification and the date and time of any outflow changes during the past 24-hour period at each lake. The inflow is calculated based on the following equation.

$I_m = \Delta S + O_m + E$, where

I_m = mean daily inflow, c.f.s.

O_m = mean daily outflow, c.f.s.

ΔS = change in storage during past 24-hour period in d.s.f.

E = lake evaporation in c.f.s.

Mean daily outflow, O_m, is computed by accounting for any outflow changes as entered in the daily lake data. Evaporation, E, is computed using 0.7 of the reported pan evaporation multiplied by the lake surface area corresponding to the reported morning lake elevation. If the computation results in a negative inflow, the inflow is set to zero and continues to the next lake computation. When the computation has been completed for all lake projects, a daily pertinent data sheet is printed as shown on Figure 4. The data required for the monthly lake summary and RO-1 plotting program are stored in the main data base for future use and subroutine STGDIS is called.

Subroutine STGDIS main function is to analyze the stage data entered from the reporting river stations, find a corresponding discharge if required, store the hydrograph data, and set up a daily information bulletin printout. Daily river stage data are entered by river basins as follows:

a. Basin and station identification

b. Morning stage or discharge if a discharge station only

c. Shift control

d. Time and stage of extra readings, if any. As many as three additional stages can be read.

If a shift is required by the control described above due to a change in the stage-discharge relationship, a shift control is read. This control can be applied to the stage-discharge relationship for as many as three different ranges of the rating table. The following information is then read, minimum stage for shift to apply, maximum stage for shift to apply, and shift to apply to the table, in feet, for the range between the minimum and maximum stage. In most cases, we have applied a shift to the entire table; however, the provision to vary the shift for as many as three portions of the table is available, if needed. If it is desired to shift from the base curve as the result of a discharge measurement, it can be done by entering the proper control with the stage data and then the measured discharge and corresponding stage. This causes a new shift to be computed and entered in the data base. Once a shift is used, it is retained in the data base until it is changed or revised by inserting a new shift or discharge measurement. In all cases, the shift is always made from the basic stage-discharge relationship in the data base.

The main data base contains the latest stage-discharge table available; however, if it is necessary to shift the relationship, the shift control is read as follows:

a. Minimum stage for shift to apply in feet

b. Maximum stage for shift to apply in feet

c. Amount of shift in feet

After updating the shift table, the stage-discharge table and shift data are written to the new data base. The shift is applied to the current stage and a discharge determined for the stage and up to three additional stages for the station, if required. The reported stages and corresponding discharges to be printed are then read to a scratch file.

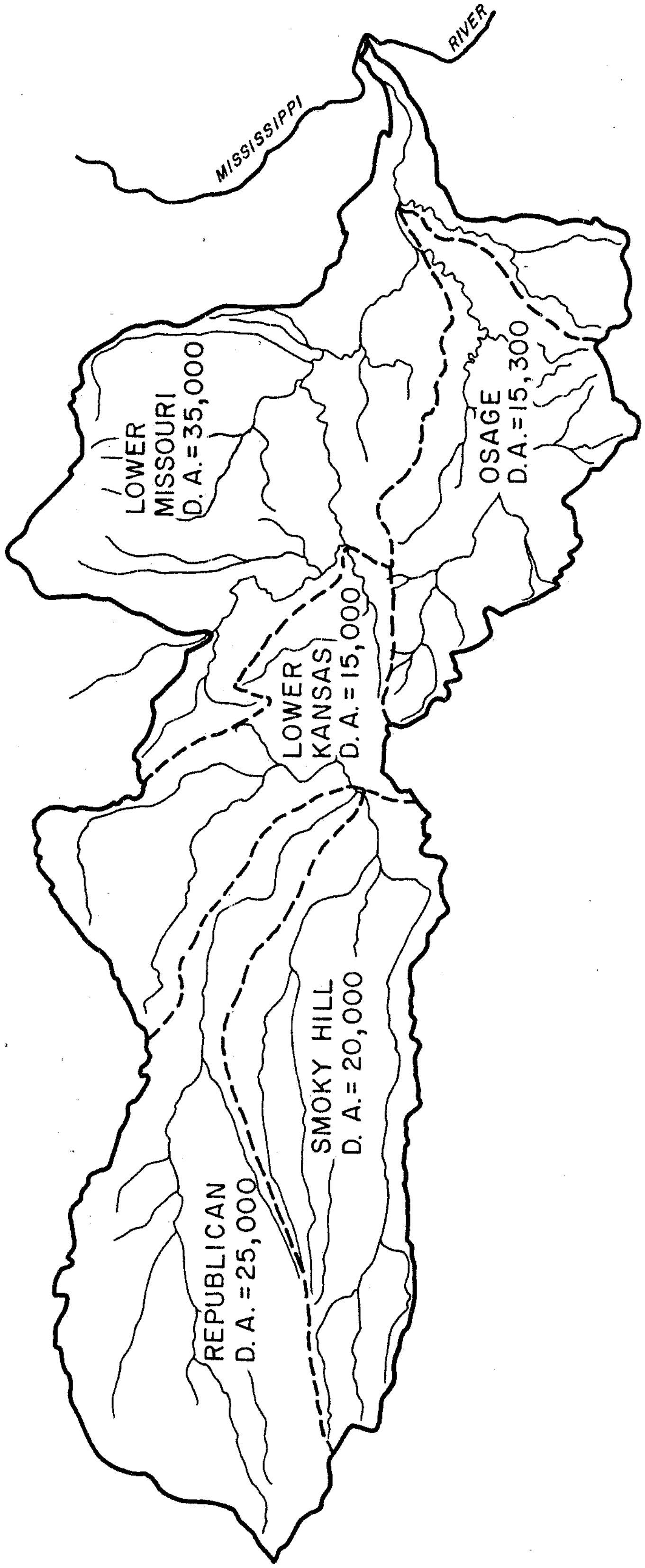
It is optional whether to save a particular station's hydrograph or not. If the station data being processed has a saved hydrograph control, the old hydrograph is read, shifted in the proper time sequence, written to a scratch file, and then written back to the main data base when all data for the run have been processed. In many cases, the data are used for daily information and are of no further use. Where further use of the data is needed for other programs, the discharge data are saved. Hydrograph data are saved for all damage index stations used for determining lake project effects on downstream river stages. When all of the river basin stage data have been read and discharges computed, a printout of the stage and discharge data that were written to the scratch file is made. This printout gives the current stage and corresponding discharge for all key gaging stations where data were available. The file also contains any error messages that may have occurred during the run and the station name, time, stage, and discharge of extra stages which were entered. A typical printout is shown as Figure 5. The STGDIS subroutine calls subroutine DACK.

Subroutine DACK is used to compute a day count for the same hydrograph routine in STGDIS. This routine will compute the number of days that must be inserted in the saved hydrograph when it has been more than one day since the last data run was made or since a stage was received for a particular gaging station. For days of missing data, a (-1) is inserted in the discharge hydrograph. The most recent 50 days of discharge data are saved for use by other programs.

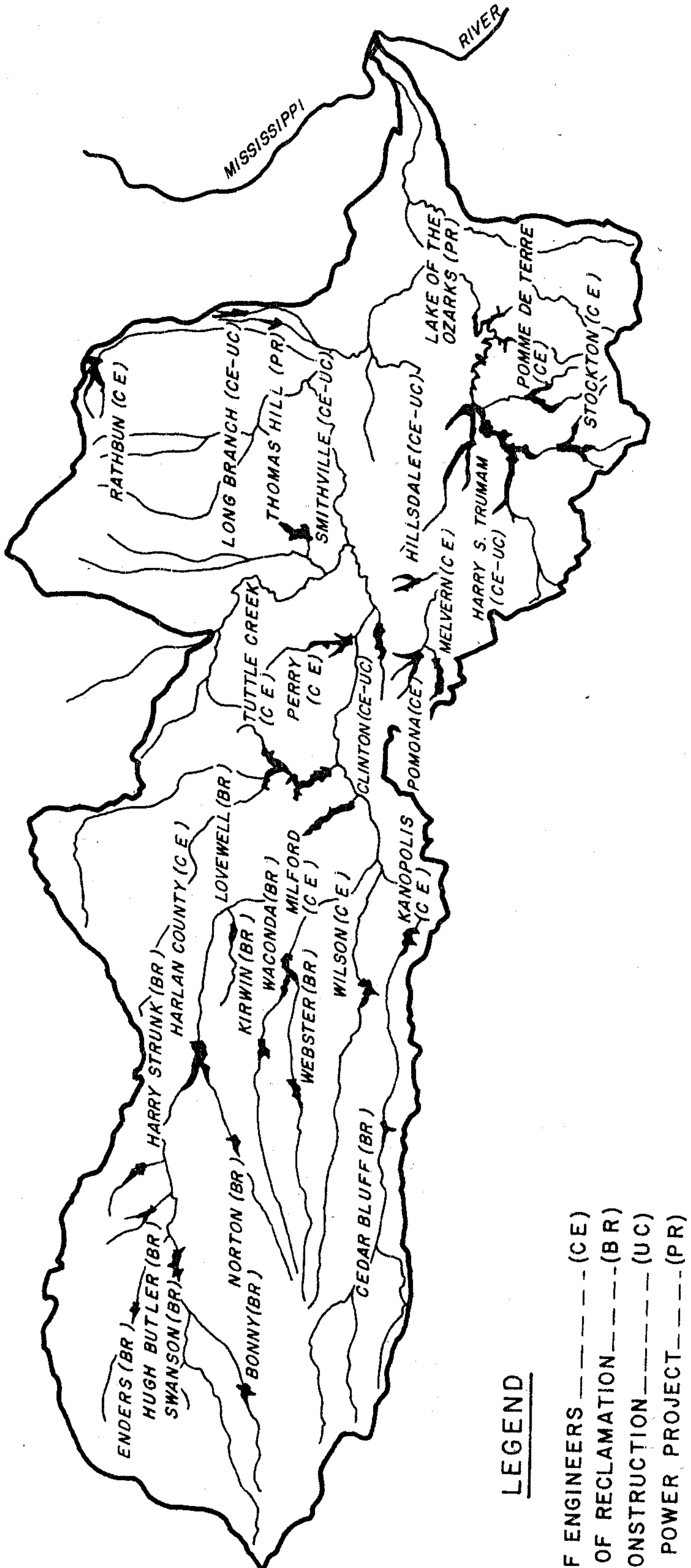
The program is now being run batch; however, we have been experimenting with time-sharing operation of the program. Improvement of the lake inflow computation method needs to be made to enable the use of the stored lake data in the monthly summaries and RO-1 plates because the present method does not result in a balanced accounting of water when negative inflow is computed.

Many of the details of the program have not been discussed, but the program is operational and has been running for a few months. One of the major problems is the data handling required as input to the program. It is hoped that as the data collection process becomes automated, the data can then be analyzed, information bulletin prepared, and data stored for future use in a more efficient and accurate manner. The Bureau of Reclamation is presently in the process of installing a remote control and data collection network in the Kansas River Projects Office at McCook, Nebraska. Data on the Bureau projects are now transmitted to this office by teletype terminals and can be entered into the program with some minor modifications. The National Weather Service is now developing capability to read several river and rainfall reporting stations on a real time basis. Their automated data collection system is planned to rely on the existing telephone system with interrogation of the station by teletype and dialing. The District is presently investigating various alternatives for the collection of hydrologic data on a real time basis.

KANSAS CITY DISTRICT
MAJOR DRAINAGE BASINS
FIGURE I

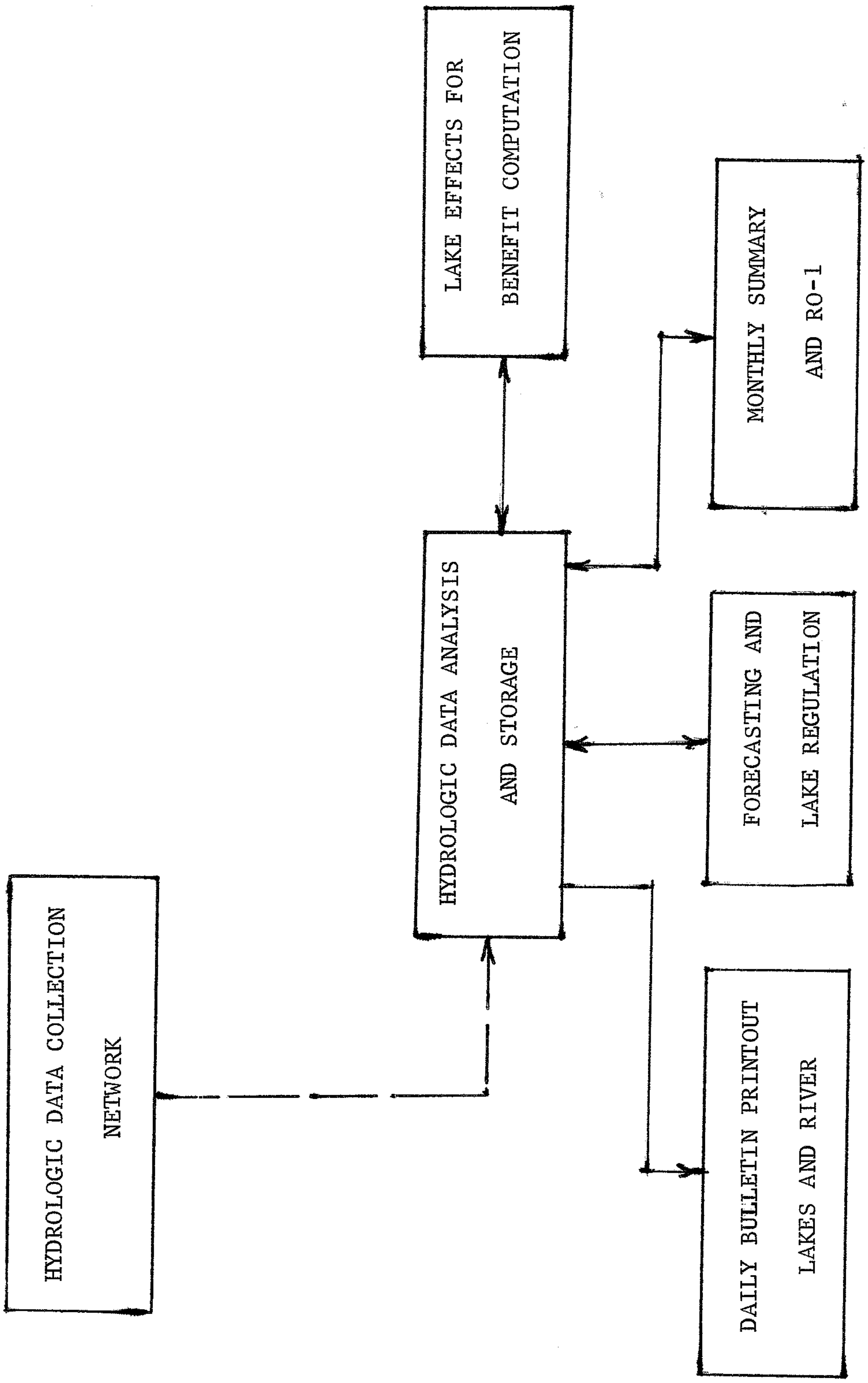


KANSAS CITY DISTRICT
LAKE PROJECTS
FIGURE 2



LEGEND

CORPS OF ENGINEERS ---- (CE)
BUREAU OF RECLAMATION ---- (BR)
UNDER CONSTRUCTION ---- (UC)
PRIVATE POWER PROJECT ---- (PR)



WATER CONTROL PROGRAMS

FIGURE 3

KANSAS CITY DISTRICT, CORPS OF ENGINEERS
HYDROLOGIC ENGINEERING BRANCH-WATER CONTROL SECTION

LOWER MISSOURI RIVER BASIN

LAKE DATA FOR 8AM: 6 MAR. 1975

LAKE NAME	LAKE FLOOD CONTROL INFORMATION ELEVATIONS (MSL) MP	ELEVATIONS (MSL) FC	STORAGE TOTALS (100 AC-FT) MP	STORAGE TOTALS (100 AC-FT) FC	STORAGE TOTALS (AC-FT) (FC-MP)	STORAGE USED FC AC-FT	STORAGE USED FC %	ELEV. (MSL)	STORAGE (AC-FT)	PREC. (IN.)	ELEV. CHNG.	INFLOW (DSF)	DISCH. (DSF)
(REPUBLICAN RIVER BASIN)													
BONNY	3672.0	3710.0	413	1701	128820	0	0	3671.76	40851	-0	.04	46.	5.
SWANSON	2752.0	2773.0	1201	2539	133790	0	0	2743.69	82406	-0	.05	106.	1.
ENDERS	3112.3	3127.0	444	745	30040	0	0	3103.23	30703	-0	0	1.	1.
HUGH BUTLER	2581.8	2604.9	377	866	48851	0	0	2578.38	32458	-0	.01	13.	6.
HARRY STRUNK	2366.1	2386.2	371	893	52172	0	0	2361.67	29606	-0	.07	56.	1.
NORTON	2304.3	2331.4	359	1347	98803	0	0	2278.03	4027	-0	.02	6.	1.
HARLAN COUNTY	1946.0	1973.5	3197	8287	508989	0	0	1941.31	284977	-0	.05	300.	10.
LOVEWELL	1582.6	1595.3	416	921	50460	0	0	1580.68	36232	-0	.03	42.	1.
MILFORD	1144.4	1176.2	4153	11730	757746	11436	1.5	1145.10	426788	-0	.07	891.	300.
(SMOKY HILL RIVER BASIN)													
CEDAR BLUFF	2144.0	2166.0	1850	3769	191860	0	0	2130.80	109043	-0	.01	28.	2.
KANOPOLIS	1463.0	1508.0	552	4256	370434	0	0	1458.78	41743	-0	.26	430.	60.
WILSON	1516.0	1554.0	2478	7785	530710	362	.1	1516.04	248197	-0	-.04	28.	200.
WEBSTER	1892.4	1923.7	773	2613	183959	0	0	1869.10	16630	-0	.03	25.	-0
KIRWIN	1729.3	1757.3	994	3145	215115	0	0	1711.32	32382	-0	.01	13.	-0
WACONDA	1455.6	1488.3	2414	9637	722315	0	0	1455.58	241158	-0	-.01	100.	156.
(LOWER KANSAS RIVER BASIN)													
TUTTLE CREEK	1075.0	1136.0	4253	23670	1941705	30501	1.6	1076.89	455813	-0	.27	2250.	-0
PERRY	891.5	920.6	2432	7651	521880	0	0	889.93	224466	-0	-.01	251.	300.
CLINTON	875.5	903.4	1294	2678	138400	RESERVOIR UNDER CONSTRUCTION							
(PLATTE RIVER BASIN)													
SMITHVILLE	864.2	876.2	1693	2896	120300	RESERVOIR UNDER CONSTRUCTION							
(CHARITON RIVER BASIN)													
RATHBUN	904.0	926.0	2053	5516	346297	0	0	903.33	198064	-0	.02	119.	11.
(LITTLE CHARITON RIVER BASIN)													
LONG BRANCH	791.0	801.0	350	650	30000	RESERVOIR UNDER CONSTRUCTION							
(OSAGE-MARIS DES CYGNES RIVER BASIN)													
MELVERN	1036.0	1057.0	1543	3628	208444	0	0	1034.19	142191	-0	-.03	51.	150.
POMONA	974.0	1003.0	706	2473	176773	0	0	972.21	63656	-0	-.14	134.	400.
HILLSDALE	917.0	931.0	760	1600	84000	RESERVOIR UNDER CONSTRUCTION							
FORT SCOTT	847.2	866.4	823	2355	153200	RESERVOIR UNDER CONSTRUCTION							
STOCKTON	867.0	892.0	8871	16666	779550	150963	19.4	872.79	1038072	-0	-.03	2565.	2980.
POMME DE TERRE	839.0	874.0	2415	6487	407179	33296	8.2	843.04	274859	-0	-.39	805.	2500.
HARRY S TRUMAN	706.0	739.5	12034	52093	4005949	RESERVOIR UNDER CONSTRUCTION							
BAGNELL	630.0	660.0	7087	19268	1218067	1023510	84.0	656.50	1732266	-0	-.37	22662.	32300.

FIGURE 4

KANSAS CITY DISTRICT, CORPS OF _NGINEERS
HYDROLOGIC ENGINEERING BRANCH-WATER CONTROL SECTION

HYDROLOGIC DATA

AM RIVER DATA - 6 MAR. 1975

STATION STREAM	RIVER MILE	FS DATUM	CHG STAGE	DISCH.	STATION STREAM	RIVER MILE	FS DATUM	CHG STAGE	DISCH.
MISSOURI RIVER BASIN-PRIMARY STATIONS									
GAVINS PT. REL. MISSOURI RIVER	811.1			17,100	LEXINGTON MISSOURI RIVER	317.3	22.0 663.5	.1 11.4	
SIOUX CITY MISSOURI RIVER	732.3	36.0 1057.0	.1 16.6	17,600	WAVERLY MISSOURI RIVER	293.4	20.0 645.5	-.1 8.4	37,200
OMAHA MISSOURI RIVER	615.9	19.0 958.2	0 4.7	20,700	GLASGOW MISSOURI RIVER	226.3	25.0 586.1	 10.9	
LOUISVILLE PLATTE RIVER	16.0	9.0 1007.1	.3 5.7	6,870	BOONVILLE MISSOURI RIVER	197.1	21.0 575.0	.2 8.7	48,000
NEBRASKA CITY MISSOURI RIVER	562.6	18.0 905.4	.1 5.8	25,200	JEFFERSON CITY MISSOURI RIVER	143.9	23.0 519.7	-.2 10.1	
RULO MISSOURI RIVER	498.0	17.0 837.2	.2 6.6	26,100	GASCONADE MISSOURI RIVER	104.8	22.0 486.1	-1.1 15.3	
ST. JOSEPH MISSOURI RIVER	448.2	17.0 788.2	0 7.6	29,300	HERMANN MISSOURI RIVER	97.9	21.0 481.4	-1.0 13.1	95,600
LEAVENWORTH MISSOURI RIVER	395.7		.3 6.9		ST. CHARLES MISSOURI RIVER	28.2	25.0 413.6	-1.0 17.8	
KANSAS CITY MISSOURI RIVER	366.1	22.0 715.8	.1 2.5	31,900	ST. LOUIS MISSISSIPPI	180.0	30.0 379.9	-1.0 12.9	194,700
NAPOLEON MISSOURI RIVER	328.7	17.0 680.2	0 5.2						
MISSOURI RIVER BASIN-SECONDARY AND TRIBUTARY STATIONS									
SMITHVILLE PLATTE RIVER		24.0 778.8	.1 11.9	43	BLUE LICK BLACKWATER RIV.	30.8	25.0 593.8	1.6 14.8	1,270
LAKE CITY LITTLE BLUE	10.5	18.0 719.2	0 8.1	140	JEROME GASCONADE	104.7	15.0 657.6	-.4 5.2	4,870
LOWER KANSAS RIVER BASIN-PRIMARY STATIONS									
MILFORD RELEASE REPUBLICAN RV.	7.7			300	FT RILEY KANSAS RIVER	168.9	21.0 1034.7	0 6.2	1,280

NOTE : CHG = STAGE CHANGE IN FEET FROM YESTERDAY.
DISCH. = STREAM DISCHARGE IN CUBIC FEET PER SECOND.
STAGE = FEET ABOVE GAGE ZERO OF WATER SURFACE.
DATUM = FEET ABOVE MEAN SEA LEVEL OF GAGE ZERO.
FS = FLOODSTAGE

FIGURE 5

AUTOMATIC DATA COLLECTION SYSTEM
MISSISSIPPI RIVER LOCKS & DAMS
ST. PAUL DISTRICT
CORPS OF ENGINEERS

November 1975

Paper 5

TABLE OF CONTENTS

FIGURES

I. INTRODUCTION

A. Basin Description - The St. Paul District contains the upper 243 miles of the Mississippi River 9-foot navigation channel. Thirteen locks and dams were constructed to maintain navigation depths throughout this reach, Figure 1. In order to maintain hydraulic operation of the system within the authorized limits, 51 gages are read at least daily, many as often as 6 times a day. Data from 22 additional stations are collected daily to monitor flow conditions on the larger tributaries in the District.

B. Potential for Automation - It was readily apparent that this system would lend itself to automation, both in terms of day-to-day real-time operation and in terms of record keeping and data retrevial. During meetings with the U. S. Geological Survey, to discuss the systematic addition of telephone interrogation equipment to stream gages in the District, it was learned that they had developed an automatic data collection system which we might be able to adopt to our needs without a great expense.

II. PRESENT OPERATION

A. Daily Data Collection - Data is collected each day by radio, beginning at approximately 7:30 a.m. Readings are reported by personnel at each lock, in sequence down the river, and recorded in the office on the "radio log", Figure 2. This routine is generally complete by 8:00 a.m. then relayed, by phone, to the Minneapolis National Weather Service

Office. While this is being done, a second technician is collecting data from stations in the Twin Cities area and from upstream tributaries. The District Office is in St. Paul which is approximately at the head of návigation, however 50 percent of the Mississippi River drainage enters the system at St. Paul.

B. Analysis and Operation - Once the basic river data has been collected, an engineer will compute inflows to each navigation pool. Based upon this and the outlook for the next few days, he will determine the appropriate gate changes and pool operating range for the next 24 hours. These orders are then passed on to each lock and dam by radio. This phase of the operation is usually complete by 9:00 a.m. each day.

C. Other Data Analysis and Compilation - After the daily orders have been given to each lock and dam, other data handling must be done routinely. They are:

1. Daily Bulletin (Figure 3) - a compilation of stage, discharge, and rainfall data sent to the Division Office and 15 others on a local mailing list.

2. Daily Routings - Storage changes are calculated daily for each pool. This is done to compare computed outflow to actual outflow at each dam, which in turn provides us with a spread or discrepancy which must be taken into account by the system operator in his daily inflow computations.

3. Historical file - Either monthly or annually, the data on the "radio logs" are manually typed on to annual summary sheets for permanent record.

II. AUTOMATIC OPERATION

A. Introduction - The automatic data collection network currently being installed by the St. Paul District will enable us to automate all of the preceding functions except the analysis and operation. The flow chart, Figure 4, is a schematic diagram showing the control program with the various input and output features.

B. Automatic Data Collection Network - Figure 5 is a schematic drawing of the automatic data collection network showing the components, the suppliers and unit prices. Bristol Datamaster Division, a subsidiary of American Chain and Cable Co., Inc., supplies the two main components of the telemetry system, the remote unit and the central unit. The remote unit is located in the control house at each lock and the central unit is in the District Office, Reservoir Regulating Section. At the remote station, three parameters will be collected automatically, pool and tailwater elevations along with precipitation. They will be transmitted continuously to the Data Input console via selsyn motors and digital recorders. In addition to the 3 parameters collected automatically, 8 parameters can be input manually to the Data Input Console by means of digital dials. These parameters would include total tainter gate opening, total roller gate opening, and tributary gage readings. The Data Input Console was designed by the U. S. Geological Survey and serves as an interface between the data collection devices and the remote telemetry unit.

The central telemetry unit can be set to automatically interrogate each of the remote units at specific intervals. Ours will be set to 4 hour intervals, similar to present operations. The Central Unit is equipped with a digital clock so that when manual interrogations are made the time will be known and they can be separated from the basic data by the computer program.

Originally, the Central Telemetry Unit was connected to a teletype machine for output. We are linking it to a Texas Instrument Silent 700 ASR. By doing this, the data will be printed both on the keyboard for visual editing and on the cassett tape for computer input.

C. Inflow Computations - Each morning the previous days data would be visually checked for errors and omissions and the necessary changes made on the cassette tape. The data would then be input to the computer via the acoustic coupler for processing. All lock and dam flows and tributary flows would be calculated and output in the Daily River Log, Figure 6, for use by the system operator. Flow routings would also be calculated and the previous four days record output as a running check on the spread or discrepancy between locks and dams, Figure 7.

D. Daily Bulletin - Rainfall and river data now collected in the District Office will continue to be collected in the same manner but input to files through the Texas Instrument keyboard. The Daily Bulletin will be output each day by means of a program developed in the Rock Island District and adopted to our needs and equipment.

E. Present Status of System - The system is only in the preliminary stages of installation and it is at least a year away from being operational. At present, we have the Texas Instrument, the Central Unit and four remote units. The first four units will be installed at Lock and Dams 3, 5, 8, and 10 which are forecast points for the National Weather Service. It is anticipated that the National Weather Service will then install a slave printer to monitor these gages.

F. Problems - The major problem at the present time is communications, both between the remote units and the central unit and between the central unit and the Texas Instrument. The basic system developed by the U. S. Geological Survey uses telephone lines from remote to central. We were hoping to use our existing radio system which is currently being modernized and upgraded; however, the Datamaster equipment appears incompatible with the repeater system. The central unit will transmit a send command to a remote unit and must get a response within 0.1 second or it will go on to the next unit. Our repeater system will not allow a response which is this rapid. We are now in the process of obtaining permission to use a dedicated telephone line.

The primary advantage of the radio over telephone lines is economics. The radio would be virtually free where as the telephone service is estimated to cost approximately $500 per month.

The U. S. Geological Survey is currently building an interface to connect the central telemetry unit to the Texas Instrument terminal.

IV. DATA STORAGE AND RETREVIAL

A. Introduction - The most significant improvement that can be attributed to the automatic data collection network and related automation is not so much in the real-time operation of the lock and dam system as it is in file maintenance and data retrevial. The New Orleans District developed a comprehensive program of historical file maintenance and data handling that has also been modified to meet our requirements and is currently running on the GE 225 system.

B. Data Handling Subroutines - The following two subroutines are part of the New Orleans program and are also operational on our GE 225 system.

1. Annual stage summary (Figure 8) - Daily stages can be recalled for any given year. Data is stored in two parts, the continuous record of daily AM and PM readings and a separate gage description. Both records have the same identification number so that they can be merged for output. We use the U. S. Geological Survey numbering system.

2. Annual Stage Hydrographs (Figure 9) - The same stage data can be printed in hydrograph form. The plotting routine has the capability of plotting both AM and PM readings when they are available.

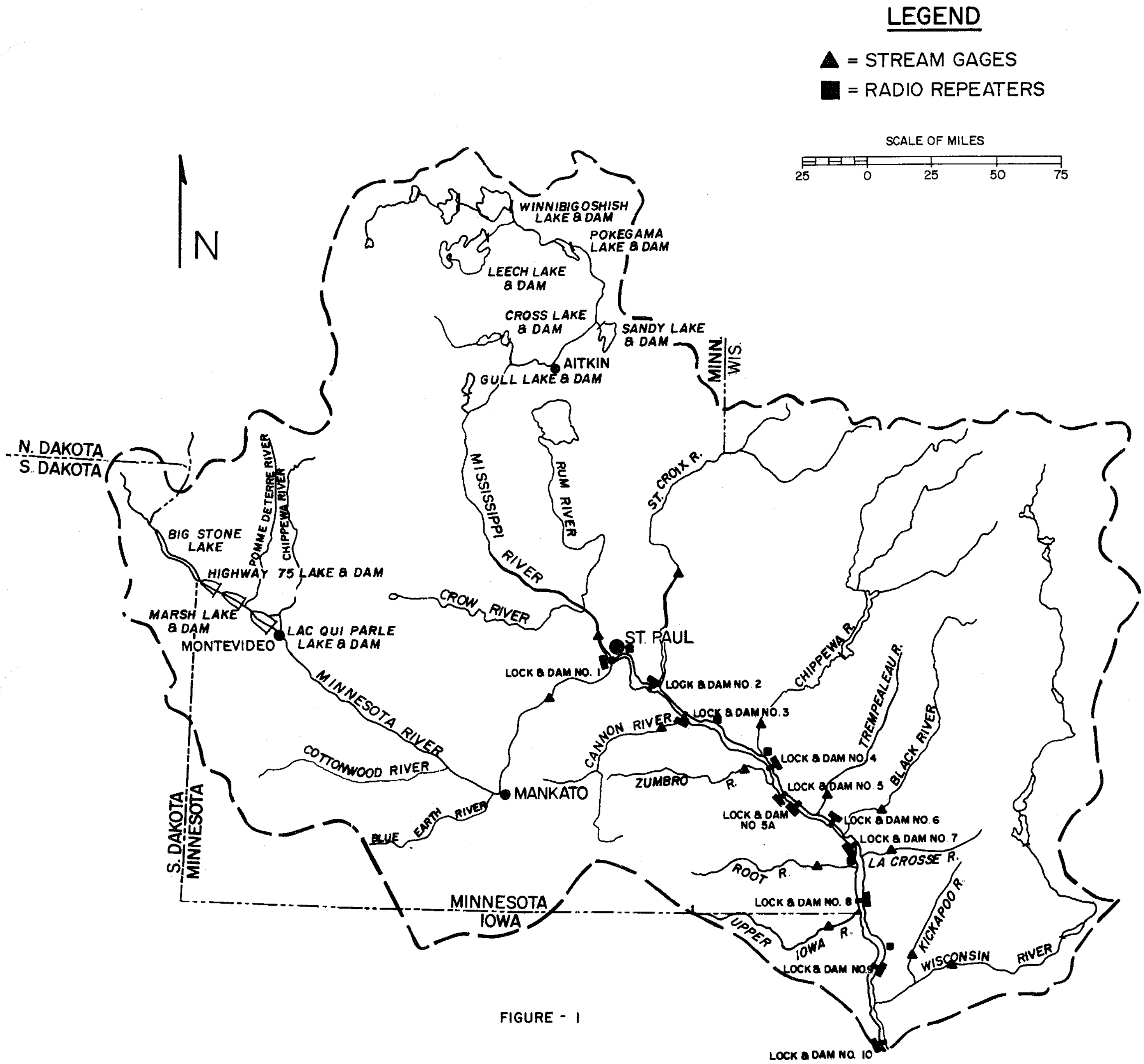


FIGURE - I

UPPER MISSISSIPPI RIVER WATERSHED

ST. PAUL DISTRICT

NORTH CENTRAL DIVISION

CORPS OF ENGINEERS

FROM NOON ____________________ TO 8 AM ____________________

STATION	PREVIOUS DAY 24 HR AVERAGE	12-8 AM AVERAGE DISCHARGE	INST. (8 AM) DISCHARGE
L/D NO. 1 (FORD DAM)			

NOTE:
SHADED NUMERALS ARE REPORTED TO THE WEATHER BUREAU

{1 st - MISS. R. INFO
2 nd - TRIBUTARY STAGES AND DISCHARGES}

STATION		ZERO GAGE		STAGE	ELEVATION	DISCHARGE
MINN.	MANKATO	(1929)	747.92			
	JORDAN		690.00			
MISS.	ANOKA	(1912)	805.02			
	ST. PAUL		684.16			
	C.P. 2 (So. ST. PAUL)				6	

NCS FORM 51
(R OCT 74)

DAM 2		POOL 2	T.W. 2	Q	GATES TAINTER	C.P. 3 PRESC.	
	N					600 +	
	4 PM						
	8 PM						
	MN						
	4 AM					600 +	
	8 AM						
TEMP.		PREC.		WIND	T.V.		

NCAP	CUT	OPEN	FT. T.G.	: SYSTEM OPERATORS ORDERS
FLOW	DEC.	INC.		INFLOW ≈
HOLD POOL		± TO		

DAM 3		POOL 3	T.W. 3	Q	GATES ROLLER	RED WING	
	N						
	4 PM						
	8 PM						
	MN						
	4 AM						
	8 AM					665.13	
TEMP.		PREC.		WIND	T.V.		

NCAP	CUT	OPEN	FT. R.G	: SYSTEM OPERATORS ORDERS
FLOW	DEC.	INC.		INFLOW ≈
HOLD POOL		± TO		

FIGURE 2

Paper 5

U.S. ARMY CORPS OF ENGINEERS — ST. PAUL DISTRICT

DAILY BULLETIN

MISSISSIPPI RIVER — SEE NOTE: * DATA FOR PREVIOUS DAY — DATE ______

LOCATION	MIN W.S. EL.	GAGE-0	F S	NOON	4 P M	8 P M	M N	24 HR	4 A M	8 A M	R F
ST. ANTHONY FALLS									GEN'S.	TEMP.	
UPPER POOL	796.50	700.00	752.00								
LOWER POOL	749.30	700.00	736.00								
LOWER T.W.	723.10	700.00									
DISCHARGE				8-4		4-12			12-8		
L/D-1	LOCKAGES								TEMP.		
POOL	723.10	700.00									
T.W.	687.20	600.00									
DISCHARGE (FORD)				8(4)		4-12			12(8)		

LOCATION	MIN W.S. EL.	GAGE-0	F S	G H	SEC FT	R F
AITKIN *		+ 1182.4	15'			
AITKIN CARD						
FT. RIPLEY		1134.71	10			
ST. REGIS (FORMERLY ST. CLOUD)	24 H R / 8 A M					
ANOKA *		805.02	33'			
COON RAPIDS	830.50					
ST. PAUL *	687.20	684.16	14			
S. ST. PAUL *	687.20	600.00	97			
L/D-2 P.	686.50	600.00				
L/D-2 T.W.	675.00	600.00	685.7 HASTINGS-15			
PRESCOTT	675.00	600.00				
L/D-3 P.	674.00	600.00				
L/D-3 T.W.	667.00	600.00				
RED WING	667.00	665.13	14'			
LAKE CITY	667.00	661.10	677.10			
WABASHA	667.00	660.00	673.50			
L/D.-4 P.	666.50	600.00				
L/D-4 T.W.	660.00	600.00	672.00 (ALMA-TE)			
C.P.-5	660.00	600.00				
L/D-5 P.	659.50	600.00				
L/D.-5 T.W.	651.00	600.00				
L/D.-5A. P.	650.00	600.00				
L/D.-5A T.W.	645.50	600.00				
WINONA	645.50	640.12	13'			
L/D.-6 P.	644.50	600.00				
L/D.-6 T.W.	639.00	600.00				
L/D.-7 P.	638.50	600.00				
L/D. 7 T.W.	631.00	600.00				
LA CROSSE C.P. 8	631.00	600.00				
L/D. 8 P.	630.00	600.00				
L/D-8 T.W.	620.00	600.00				
LANSING	620.00	612.26	18			
L/D-9 P.	619.00	600.00				
L/D.-9 T.W.	611.00	600.00				
Mc GREGOR	611.00	605.30	18'			
CLAYTON	611.00	602.60				
L/D.-10 P.	610.00	600.00	615.00			
L/D.-10 T.W.	603.00	600.00				

5 DAY FORCAST NEXT 5 DAYS AT L/D #10 GUTTEMBURG IOWA

MPLS WEATHER B

LOCATION	STAGE	SEC FT	R F
MPLS -16'			
* MONTEVIDO -14'			
* MANKATO -19'			
ROCH. SEWAGE			
* ZUMBRO FALLS -18			
GAGE 0=626.32 LA CROSSE-12'			
* LA FARGE -12'			
HOKAH -47'			
* LANES BORO			

WISCONSIN VALLEY

LOCATION			
MERRILL -F.S.-11			
WISC. RAPIDS -F.S.-12'			
PETENWELL NOR. FULL P. 932.9			
CASTLE ROCK-NOR. FULL P. 881.4			
NEILSVILLE F.S. 18'			
PORTAGE F.S.-17'			
WISC. DELLS			
PRAIRIE DU SAC			

	STAGE	Q=
MINN. FALLS		
ST. CROIX-24H.		
APPLE R.		
HOLCOMBE (1)		
JIM FALLS		
WISSOTA (2)		
CEDAR FALLS		
MONOMONIE		
STILLWATER P.M.		
STILLWATER A.M.		
ST. CROIX-P		
ST. CROIX-T.W.		

MISC. STATIONS

LOCATION			
LITTLE FALLS			
BLANCHARD			
RAPIDAN (B.E.)			
JORDAN (cords)			
GRAND FORKS, N.D.			
+ COOPERSTOWN			
(GAGE 0=1271.04)			
+ WHAPETON N.D.			
(GAGE 0=942.97)			

RAINFALL STATIONS

LOCATION	R F	LOCATION	R F
MINNESOTA		WISCONSIN	
ALEXANDRIA		ANTIGO	
ARGYLE		BALDWIN	
BEARDSLEY		BIG FALLS	
BEMIDJI		BLAIR	
BENSON		BREED	
BLOOMINGTON		CHIPPEWA FALLS	
BRAINARD		CUMBERLAND	
CALEDONIA		DANBURY	
CANBY		EAU CLAIRE	
CHASKA		EAU PLEINE	
CORRELL		FLAMBEAU RES	
CROOKSTON		GAY MILLS	
DODGE CENTER		GURNEY	
DELANO		HILLSBORO	
DULUTH		LONE ROCK	
FAIRMONT		MADISON	
FAIRBAULT		MARATHON	
GRAND MEADOW		MILWAUKEE	
GRANITE FALLS		NEPCO LAKE	
HARMONY		NEW LONDON	
HIBBING		ONTARIO *	
HINCKLEY		OWEN	
INT'L FALLS		PARK FALLS	
MARSHALL		RAINBOW RES	
MELROSE		REEDSTOWN	
MILACA		REEDSBURG	
MINNESOTA		RHINELANDER	
MOOSE LAKE		RICE LAKE	
MORRIS		SOLDIERS GROVE	
NEW ULM		SOLON SPRINGS	
ONAMIA		SPIRIT DAM	
PARK RAPIDS		SPIRIT FALLS	
REDWOOD FALLS		STEVENS POINT	
REMER		VIROQUA	
RICHFIELD		WAUSAU	
ROCKFORD		WILLOW RES	
ROCHESTER		WINTER-F.K.	
SHOREVIEW		GREEN BAY	
SPRING VALLEY		N. DAKOTA	
ST. FRANCIS		BISMARK	
THIEF R. FALLS		DEVILS LAKE	
TRACY		DICKINSON	
WADENA		FARGO *	
WELLS		GRAND FORKS	
WILLMAR		JAMESTOWN	
WASECA		MINOT	
WORTHINGTON		WHAPETON	
YOUNG AMERICA		WILLISTON	
ZUMBROTA			
		S. DAKOTA	
MICHIGAN		ABERDEEN	
ESCANABA		HURON	
MARQUETTE		LEMMON	
SAULT ST MARIE		MCBRIDGE	
		PIERRE	
CANADA		SOUIX FALLS	
ATIKOKAN		WATERTOWN	
KENORA			
PORT ARTHUR		IOWA	
SOUIX LOOKOUT		CRESCO	
WINNIPEG		DECORAH	
EMERSON		DES MOINES	
		MASON CITY	
MISC.		SOUIX CITY	
		SPENCER	
		WAUKON	
		DORCHESTER	

TRIBUTARY RIVER STATIONS

RIVER	LOCATION	GAGE-0	F S	G H	SEC FT	R F
MINN.	JORDAN *	690.00	20			
MINN.	SAVAGE	600.00	698.00			
MINN.	PEAVEY R.T.	600.00				
CHIPPEWA	DURAND *	+695.27	11			
ZUMBRO	THEILMAN	700.00	39			
WHITEWATER	BEAVER	668.70	7			
TREMP.	DODGE	+663.78	7			
ROOT	HOUSTON	661.00	15			
WISC.	AT MOUTH (E)					
CANNON	WELCH	639.16				
BLACK	GALESVILLE *	+658.43	12			
LA CROSSE	WEST SALEM	668.00	10'			
WISC.	MUSCODA *	+667.05	9'			
CHIPPEWA	* MILAN, MINN.	+959.69				
St CROIX	* GRANTSBURG	848.98				
RED L.K. R.	HIGHLANDING		875-11'			
MISS.	WILLOW BEA.	1200.00				

NOTE: ALL ELEVATIONS, STAGES AND DISCHARGES APPLY TO THE DAY STAMPED ON TOP OF PAGE UNLESS INDICATED BY * WHICH APPLYS TO DAY PRIOR
E-ESTIMATED   M-READING MISSING   + - 1929 ADJUSTMENT   1 - FULL POOL - 45.0
ALL RAINFALL IS FOR 24 HRS. END 7:00 A.M. FOR ABOVE DATE   * - TELEMARK   2 - FULL POOL = 98.0   DATE ______

NCS FORM 130 Revised Feb '74

FIGURE 8

DATA COLLECTION SYSTEM

COMPUTER PROGRAM DIAGRAM

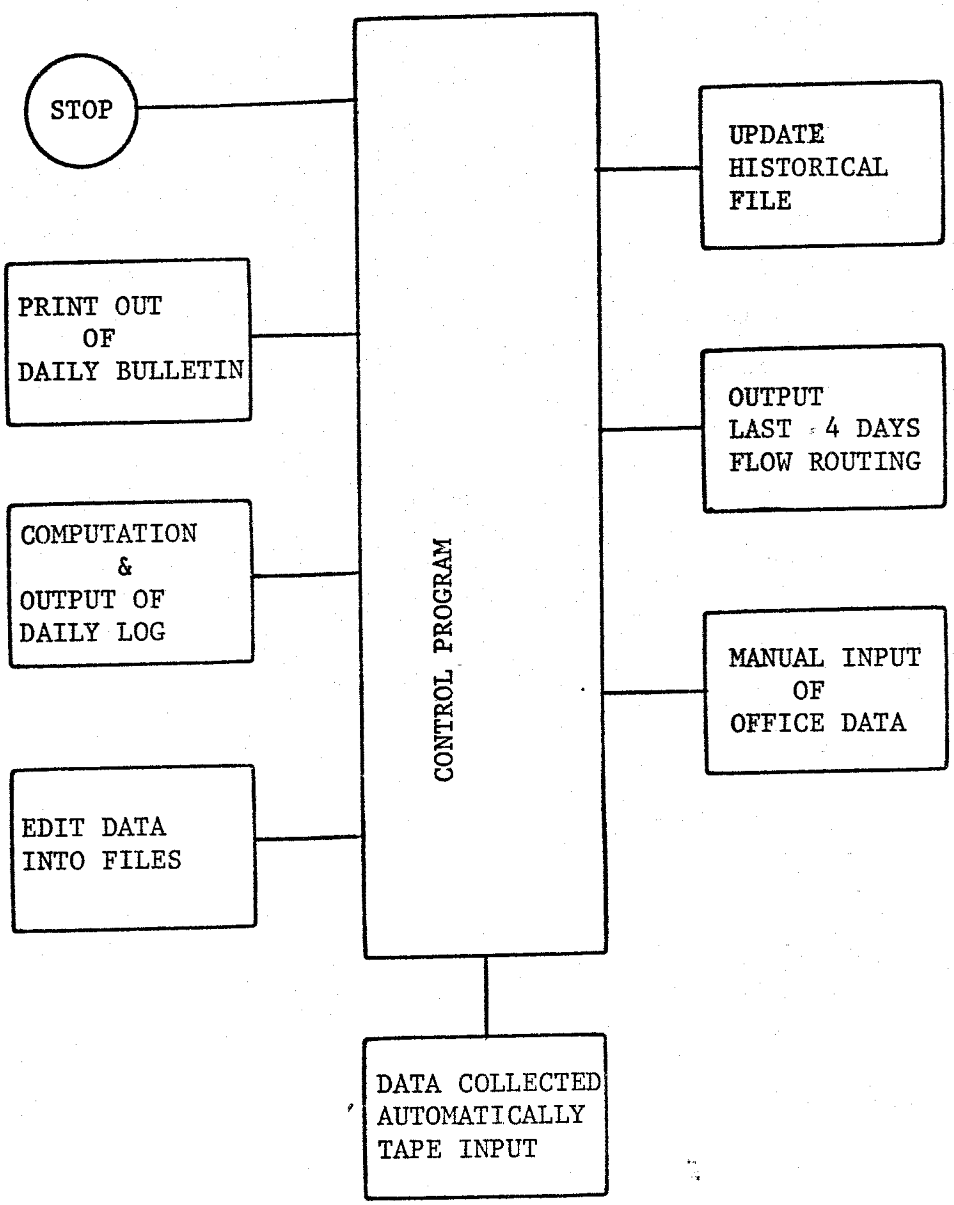
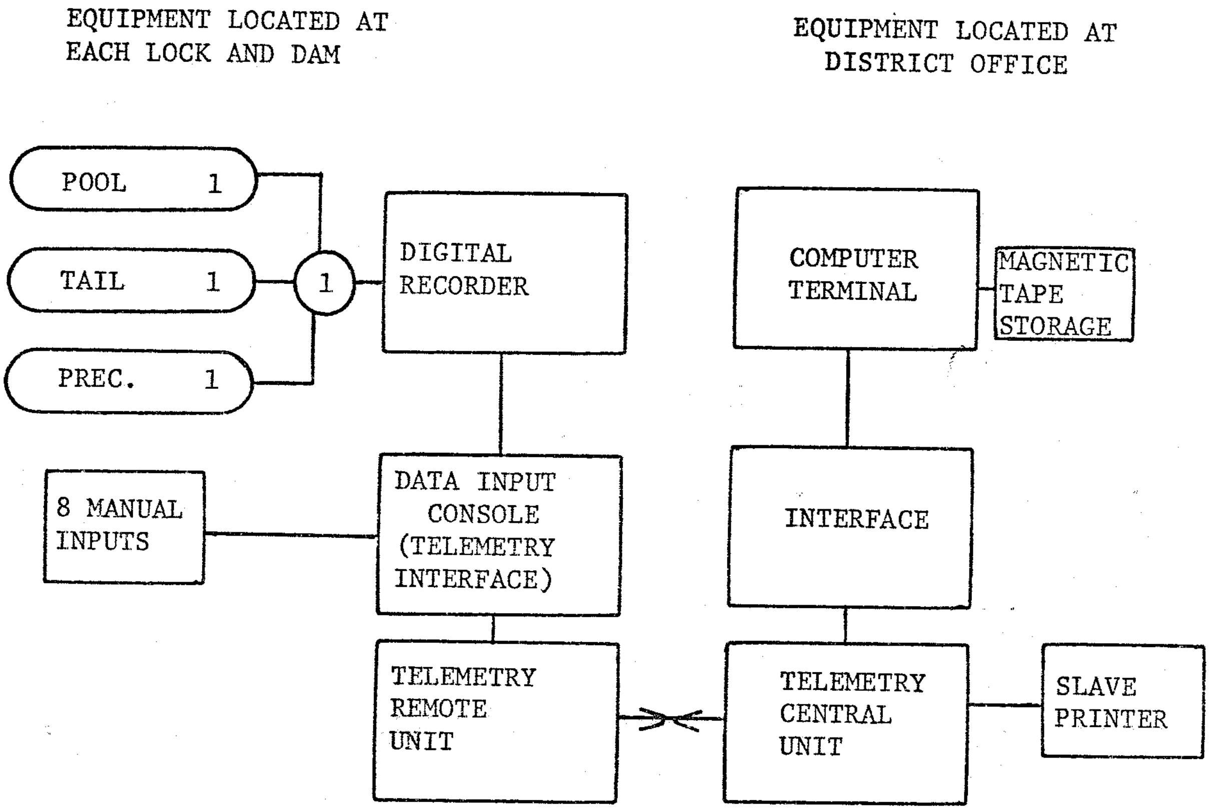


FIGURE 4

AUTOMATIC DATA COLLECTION SYSTEM

BLOCK DIAGRAM OF SYSTEM & COMPONENTS



1 - SELSYN MOTORS

EQUIPMENT COSTS

NAME	COST	MANUFACTURER
REMOTE TELEMETRY	$3500	Datamaster
COMPUTER TERMINAL	$3600	Texas Instruments
SLAVE PRINTER	$6000	Datamaster
CENTRAL TELEMETRY	$6000	Datamaster
DATA INPUT CONSOLE	$1000	U.S.G.S.
DIGITAL RECORDER		
SELSYN MOTORS		

FIGURE 5

DATE 7.: 29.: 75.

STATION	Z-GAGE	STAGE	ELEVATION	DISCHARGE
JORDON	690.00	6.32	696.32	1687.
ANOKA	805.02	4.38	809.40	9830.
ST. PAUL	684.16	3.64	687.80	0.

STATION	PREVIOUS DAY 24 HR. AVERAGE	12-8 AM AVERAGE DISCHARGE	INST. (8 AM) DISCHARGE
L/D NO. 1 (FORD)	10700.	9665.	9670.

****

DAM NO. 2

ELEVATION OF CP-2 (SO.ST.PAUL): 8AM 687.37

L/D 2 DISCHARGE
NO. OF BULKHEADED GATES= 0
ELEVATION OF BULKHEADS= 0.

POOL	TAIL	TAIN	QTAIN	QTAIL	QSP	QBK	QTOT
686.65	675.30	12.0	9084.	10215.	201.	0.	9285.
686.60	675.30	12.0	9061.	10215.	180.	0.	9241.
686.65	675.20	12.0	9130.	9543.	201.	0.	9331.
686.70	675.20	12.0	9153.	9543.	223.	0.	9376.
686.75	675.20	12.0	9175.	9543.	246.	0.	9422.
686.75	675.20	12.0	9175.	9543.	246.	0.	9422.

24 HOUR AVERAGES (8AM-8AM)
QTOT= 9346. QT= 9767.
CLIMATOLOGICAL DATA
TEMPERATURE(F): 72. PRECIPITATION:1.00
WIND SPEED: 15. WIND DIRECTION:SW

[]NCAP []CUT []OPEN FT. TG. :SYSTEM OPERATORS ORDERS
FLOW []DEC.[]INC. INFLOW=

HOLD POOL +OR- TO

DAM NO. 3

	4PM	8AM	CHANGE
ELEVATION OF CP-3 (PRESCOTT):	675.10	675.07	-.03
DISCHARGE:		0.	

L/D 3 DISCHARGE

POOL	TAIL	ROLL	QROLL	QTAIL
673.95	668.67	12.00	15141.	21976.
674.00	668.60	12.00	15338.	21966.
673.97	668.57	12.00	15338.	21964.
673.98	668.56	12.00	15371.	21963.
673.95	668.58	12.00	15289.	21965.
673.95	668.51	12.00	15403.	21961.

24 HOUR AVERAGES (8AM-8AM)
QTOT= 15313. QT= 21966.
CLIMATOLOGICAL DATA
TEMPERATURE(F): 74. PRECIPITATION:0.
WIND SPEED: 3. WIND DIRECTION:E

[]NCAP []CUT []OPEN FT. RG. :SYSTEM OPERATORS ORDERS
FLOW []DEC.[]INC. INFLOW=

HOLD POOL +OR- TO

Figure 6 - Daily River Log

L/D 02 FLOW ROUTING 7/75 4 DAY OUTPUT

DATE	24-HOUR AVERAGE OUTFLOW L/D 01 C.F.S.	JORDON C.F.S.	ACTUAL AVERAGE INFLOW C.F.S.	STORAGE S.F.D.	AVERAGE OUTFLOW C.F.S.	ACTUAL AVERAGE OUTFLOW C.F.S.	DIFF. C.F.S.
26	11700.	2000.	14200.	38800.	14600.	12567.	-2333.
27	10800.	2000.	13300.	38400.	13700.	12100.	-1600.
28	10000.	2000.	12500.	38200.	12700.	11167.	-1533.
29	10700.	1800.	13000.	38800.	12400.	9346.	-3054.

FIGURE- 7

DAILY STAGES FOR 1973

MISSISSIPPI RIVER MAIN STEM.

05378000 MISSISSIPPI R. AT FOUNTAIN CITY, WISC.

LOCATION.--FOUNTAIN CITY, C. OF E. BOATYARD.
GAGE.--TAPE GAGE MOUNTED ON STAND BUILT OVER MANHOLE IN PUMPING STATION AT CENTRAL PART OF BOATYARD, ABOUT 400 FT. BELOW OLD STAFF GAGE.
RECORDS.--1910 TO CURRENT YEAR.
EXTREMES.--MAX. 663.24 APRIL 19, 1969, MIN. 641.35 AUG 31, 1934.
GAGE ZERO.-- 600.00, 1912 ADJUSTMENT.

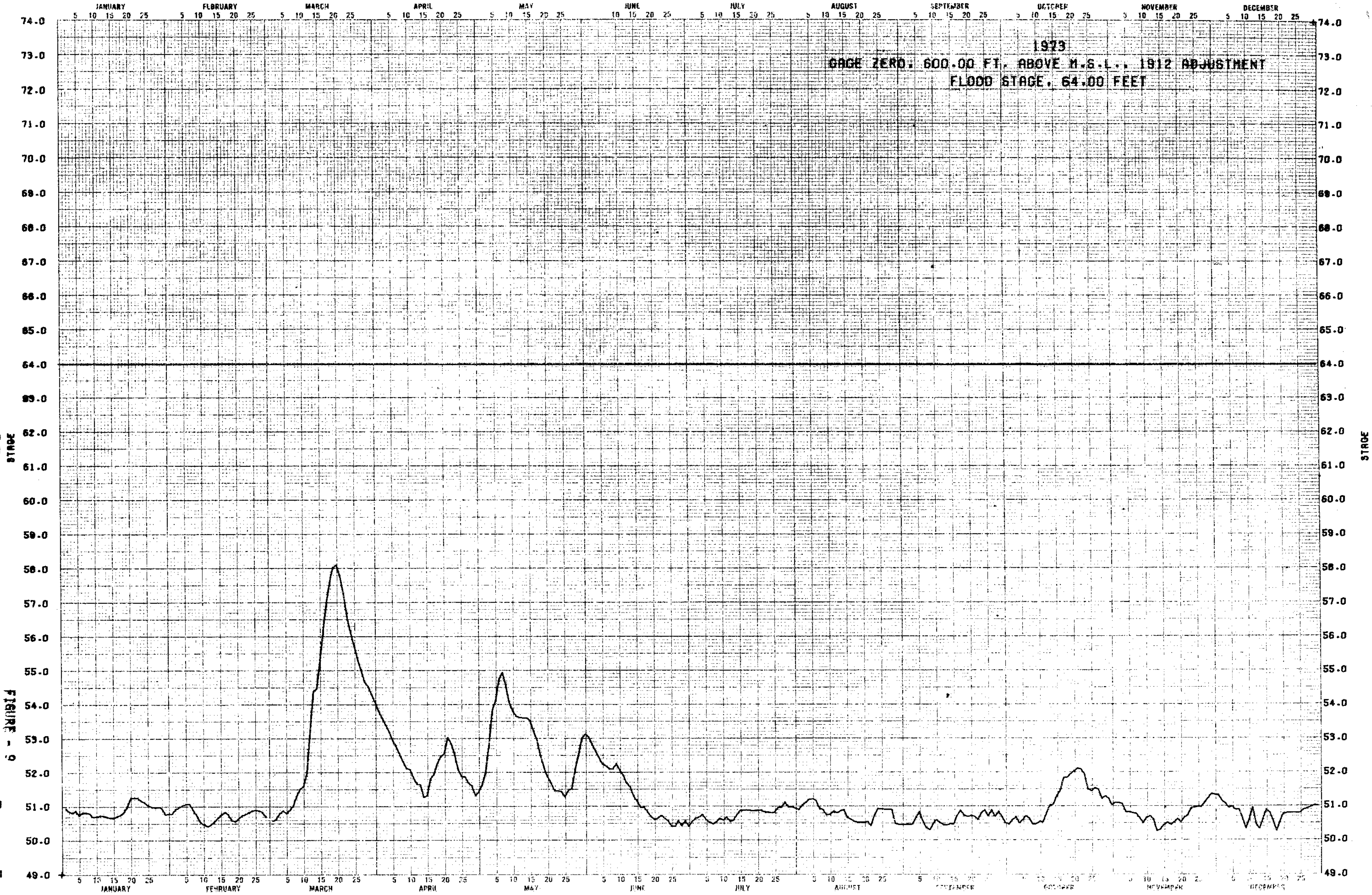
DAY	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1	50.93	50.77	50.56	53.78	51.64	53.03	50.53	50.88	50.44	50.44	51.09	51.32
2	50.83	50.91	50.63	53.58	52.0	52.82	50.64	51.03	50.45	50.58	51.08	51.15
3	50.79	50.98	50.79	53.38	52.84	52.63	50.67	51.11	50.46	50.67	51.04	51.05
4	50.86	51.03	50.86	53.19	53.84	52.41	50.75	51.2	50.65	50.48	50.81	50.93
5	50.73	51.07	50.77	52.92	54.13	52.24	50.58	51.21	50.82	50.57	50.81	50.98
6	50.81	51.06	50.89	52.75	54.76	52.17	50.51	51.15	50.54	50.7	50.78	50.87
	50.8	50.88	51.02	52.52	54.94	52.08	50.46	50.92	50.35	50.62	50.75	50.89
	50.78	50.72	51.31	52.29	54.57	52.09	50.54	50.88	50.28	50.45	50.64	50.63
	50.67	50.52	51.51	52.1	54.06	52.25	50.64	50.73	50.49	50.47	50.49	50.32
10	50.7	50.45	51.59	52.06	53.81	52.04	50.57	50.75	50.59	50.54	50.65	50.58
11	50.72	50.39	51.97	51.81	53.64	51.89	50.68	50.84	50.5	50.51	50.7	50.94
12	50.71	50.46	53.23	51.63	53.61	51.65	50.53	50.79	50.43	50.79	50.6	50.43
13	50.67	50.55	54.39	51.63	53.6	51.55	50.58	50.85	50.43	51.02	50.25	50.3
14	50.65	50.65	54.49	51.25	53.59	51.27	50.75	50.89	50.47	51.06	50.31	50.6
15	50.65	50.75	55.3	51.31	53.52	51.13	50.86	50.65	50.45	51.3	50.43	50.88
16	50.71	50.83	56.25	51.81	53.22	50.95	50.88	50.6	50.71	51.46	50.51	50.78
17	50.75	50.75	57.04	51.95	52.93	50.96	50.88	50.54	50.87	51.83	50.43	50.52
18	50.83	50.57	57.64	52.27	52.46	50.81	50.85	50.51	50.71	51.84	50.51	50.24
19	51.0	50.53	58.04	52.48	52.12	50.67	50.85	50.52	50.67	51.95	50.61	50.51
20	51.24	50.64	58.12	52.55	51.84	50.58	50.86	50.52	50.71	52.03	50.51	50.75
21	51.25	50.73	57.77	53.02	51.67	50.65	50.86	50.53	50.67	52.11	50.66	50.78
22	51.23	50.76	57.27	52.84	51.44	50.73	50.81	50.41	50.59	52.07	50.72	50.79
23	51.15	50.82	56.57	52.52	51.44	50.63	50.81	50.72	50.79	51.92	50.94	50.79
24	51.09	50.86	56.12	52.06	51.41	50.54	50.79	50.91	50.88	51.47	50.95	50.77
25	51.01	50.89	55.73	51.86	51.26	50.39	50.81	50.9	50.69	51.42	50.98	50.79
26	50.97	50.85	55.32	51.86	51.48	50.4	50.95	50.89	50.87	51.52	50.96	50.88
27	50.95	50.81	55.0	51.66	51.52	50.57	50.95	50.89	50.68	51.46	51.09	50.91
28	50.98	50.67	54.65	51.57	52.08	50.41	51.11	50.89	50.83	51.21	51.22	50.95
29	50.93		54.52	51.3	52.52	50.56	50.97	50.47	50.66	51.28	51.35	51.0
30	50.75		54.28	51.43	53.02	50.39	50.97	50.45	50.49	51.21	51.31	50.99
31	50.78		54.05		53.13		50.91	50.44		51.02		50.81

THE FOLLOWING REFER ONLY TO READINGS APPEARING IN THE TABLE ABOVE

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
MEAN	50.87	50.75	54.12	52.25	52.84	51.35	50.76	50.78	50.61	51.16	50.77	50.78
MAX	51.25	51.07	58.12	53.78	54.94	53.03	51.11	51.21	50.88	52.11	51.35	51.32
MI	0.65	50.39	50.56	51.25	51.26	50.39	50.46	50.41	50.28	50.44	50.25	50.24

Figure - 8

MISSISSIPPI RIVER AT FOUNTAIN CITY BOAT YARD



05378000

FILE NO. M - G5A-14/2 - 19

FIGURE - 9

Paper 5

STREAMFLOW FORECASTING BY THE OHIO RIVER DIVISION

By

Dr. Billy H. Johnson[1] and Ronald A. Yates[2]

I. INTRODUCTION AND BACKGROUND

It is suggested that mathematical simulation is an integral part of water control management, a necessary "tool" for the water resources engineer, or water manager, if you will. However, its effectiveness depends upon our ability to utilize that "tool" to perform comprehensive, complex calculations with both speed and precision. This paper will attempt to describe one such model currently utilized by the Ohio River Division (ORD).

At its twenty-seventh meeting on 19 May 1970, the Mississippi Basin Model (MBM) Board authorized a study to develop computer programs along reaches of the Mississippi River and its larger tributaries contained in the MBM. At the thirty-second meeting on 7 January 1971, in a joint effort with the Waterways Experiment Station (WES), the Ohio River Division's area of responsibility was determined to be the lower Ohio River from Louisville, Kentucky to the junction with the Mississippi River at Cairo, Illinois. The result of this effort was to illustrate the applicability of a finite difference representation of the unsteady-flow equations to the flood prediction and river regulation problems in

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ORD. (1) Figures 1 and 2 show some results of the initial model as applied to the 1950 Ohio River flood and the 1973 Ohio River flood. For this application, the Cumberland and Tennessee Rivers were not routed tributaries, and the channel geometric data was not very precise. However, this initial application convinced the Reservoir Control Center that further developmental effort would yield an extremely accurate, diversified, and useful streamflow forecasting model for operational purposes.

It was decided to concentrate the developmental effort at the junction of the Ohio and Mississippi Rivers. Two reasons dictated this decision:

(1) There was a definite need in that area as ORD directs the operation of Barkley and Kentucky Reservoirs on the Cumberland and Tennessee Rivers, respectively, during periods of flooding on the lower Ohio and lower Mississippi Rivers, and

(2) The hydrodynamic complexities in developing an accurate stage and discharge forecast at Cairo, Illinois could only be handled by utilizing the one-dimensional equations of fluid motion and continuity.

II. MODEL DEVELOPMENT

Mathematical Discussion

The phenomenon of unsteady flow in an open channel is described by the St. Venant Equations which represent the conservation of mass and momentum of the fluid.

Continuity: $\partial h/\partial t + 1/B\ \partial(AV)/\partial x - q/B = 0$

Momentum: $\partial V/\partial t + V\partial V/\partial x + g\partial h/\partial x + qV/A + \dfrac{gn^2V|V|}{2.21R^{4/3}} = 0$

where

- h = water-surface elevation
- t = time
- B = surface width
- A = flow area
- V = average velocity
- x = distance
- q = lateral local inflow per unit distance and time
- g = gravitational constant
- n = Manning's resistance coefficient
- R = hydraulic radius

The assumptions that are made in the derivation of the St. Venant Equations are:

a. The flow is assumed to be one-dimensional, i.e., the flow in the channel can be approximated with uniform velocity over each cross section and the free surface is taken to be a horizontal line across the section.

b. The pressure is assumed to be hydrostatic, i.e., the vertical acclceration is neglected and the density of the fluid is assumed to be homogeneous.

c. The effects of boundary friction and turbulence can be accounted for through the introduction of a resistance force which is described by the empirical Manning friction factor equation.

It should be noted that under all conditions of flow the continuity equation must be satisfied. If the storage or pondage effects completely control, as in the case of reservoirs, the momentum equation can be neglected and classical "storage routing" techniques can be used. If friction is also important then the approximation leads to the "kinematic wave" approximation. If along with friction the pressure term is included, the approximation to the St. Venant Equations gives what is known as the "Diffusion model." Incorporating the full dynamic equation gives the complete solution.

The St. Venant equations constitute a hyperbolic system of two nonlinear, first order, first degree partial differential equations to be applied to what is commonly called a mixed initial-boundary value problem. With such a system, assuming the flow regime is subcritical, only one of the two dependent variables, h and V, must be specified at each boundary as a function of time. In practice, since the discharge is given by Q = VA discharge rather than velocity is specified. It is necessary to note that rather than specifying either h or Q, one also has the option of prescribing a rating curve or table of values of water surface elevation versus discharge. This is normally the type of boundary condition utilized at the downstream boundary of a reach of open channel unless the downstream boundary corresponds to a control station. The specification of initial conditions is flexible due to the chracteristic of hyperbolic equations that the solution becomes independent of initial conditions after a sufficient length of time. A steady-flow profile, a flat pool - zero flow profile or a transient profile from previous computations may be utilized in the initial specification of h and Q.

Due to the nonlinearity, the St. Venant equations cannot be solved analytically and thus one must resort to numerical methods such as finite differences. Both implicit and explicit schemes are employed. If an implicit representation is employed, one is confronted with the problem of solving a system of algebraic equations at each time level. Quite often, iterative techniques must be employed. In contrast, in explicit schemes values of the dependent variables at each time level depend only upon values at previous time levels. Thus explicit schemes are very desirable due to the ease with which computations are made. Perhaps the major reason for choosing implicit finite difference schemes over explicit ones is due to stability considerations. In general, implicit schemes are much more stable than explicit ones. An explicit centered difference scheme proposed by Stoker has, however, been found to be sufficiently stable and convergent for the numerical solution of the St. Venant equations if the following relation is satisfied.

$$\left(V + \sqrt{gA/_B}\right) \frac{\Delta t}{\Delta x} \leq 1 - \frac{gn^2 |V| \Delta t}{2.21 R^{4/3}} .$$

Stoker's scheme utilizes a staggered rectangular net as illustrated in Figure 3. With such a staggered net, values of h and V are only computed at every other point on a particular time line, with the points at which computations are made alternating from one time line to the next. A detailed discussion of Stoker's explicit centered difference scheme as applied to the St. Venant equations is presented by Garrison, et al.[2]

Computer Model - SOCHMJ

The digital computer program SOCHMJ (Simulation of Open Channel Hydraulics in Multi-Junction Systems) which has been developed[3] was based upon Garrison's work and thus utilizes Stoker's scheme for the numerical solution of the complete St. Venant equations. Two important features of SOCHMJ are: (1) SOCHMJ can be applied to a system composed of any number of junctions and branches and (2) in SOCHMJ, Manning's n is allowed to vary with elevation as well as with distance along the channel.

Basic input data required by SOCHMJ consist of tables of geometric data, initial values of the water surface elevation and discharge at all grid points on the first two time lines, and the time-dependent boundary conditions which must be prescribed at each open boundary. As previously indicated, at such a boundary either elevations, discharges, or a rating curve may be prescribed. The tables of geometric data consist of top width, flow area, $(\text{hydraulic radius})^{2/3}$ and Manning's n, all as functions of the water surface elevation. Such a table is input at each net point in the system to be modeled. Of course additional data such as the number of junctions, type of boundary conditions prescribed at the boundaries, spatial step employed for each branch, etc. must also be input.

In its present state of development, output can be obtained from SOCHMJ in printed and graphical form. Input data specify those net points at which output is desired as well as the number of time steps between the printouts. Water surface elevations, velocities, and discharges are standard output with the option available of also printing geometric data.

III. SOCHMJ APPLIED TO THE OHIO-CUMBERLAND-TENNESSEE-MISSISSIPPI RIVER SYSTEM

The Physical System

The physical limits of the mathematical model are shown in Figure 4. This system is seen to be composed of three junctions and seven branches. These are:

Junction	Location
1	Ohio-Cumberland Rivers
2	Ohio-Tennessee Rivers
3	Ohio-Mississippi Rivers

Branch	
1	From Golconda to junction 1
2	From Barkley Dam to junction 1
3	The Ohio River between the Cumberland and Tennessee Rivers
4	From Kentucky Dam to junction 2
5	The Ohio River between the Tennessee and Mississippi Rivers
6	From Cape Girardeau to junction 3
7	From Ohio-Mississippi junction to Caruthersville

The discretization of the physical system for application of SOCHMJ is as indicated below.

Branch	Δx, miles	No. of Δx's per Branch
1	4.956	4
2	4.597	6
3	3.319	4
4	3.592	6

Branch	Δx, miles	No. of Δx's per Branch
5	4.560	10
6	5.237	10
7	4.992	24

The geometric tables of top width, flow area and hydraulic depth (all as functions of elevation) required as input data at each net point of the system, were derived from basic storage data obtained from the Mississippi Basin Model in Clinton, MS.

Model Calibration

The calibration of SOCHMJ to the Ohio-Cumberland-Tennessee-Mississippi River system was performed using recorded Mississippi Basin Model values of elevations and discharges for the 1950 flood. The boundary conditions input at the upper model limits shown in Figure 4 were:

a. Elevations at Golconda

b. Discharges at Barkley Dam

c. Discharges at Kentucky Dam

d. Elevations at Cape Girardeau

e. Table of elevations versus discharges at Caruthersville

With the spatial steps indicated above, a common time step of 300 sec was found to be sufficient to yield stable computations.

Model calibration was accomplished by comparing calculated and recorded values of elevation and discharge at several points in the system. Calibration with respect to elevations was performed by varying the values for Manning's n at net points in the neighborhood of the check point. Discharges were brought into agreement at Metropolis on the Ohio River, Thebes on the Upper Mississippi River and at Wickliffe on the Lower Mississippi River by changing the n values at all net points on a particular branch by the same amount. Figure 5 gives recorded and calculated elevations at Cairo. When compared with the results of the initial effort from the MBM Study (Figure 1), definite improvement was realized.

The calibrated model was next applied to the 1973 flood using the recorded field data shown in Figure 6 as input. No additional adjustment of Manning's n was undertaken during this application. Figure 7 illustrates the results at Cairo from such a predictive application of the model. From an inspection of this plot, it is obvious that the calculated results from the model were in very good agreement with the recorded field values. With the model calibrated and verified in a predictive sense, ORD then was ready to begin utilizing the model in the daily operation of Barkley and Kentucky Reservoirs.

IV. APPLICATIONS OF SOCHMJ

The model presently utilized is applicable only to "open" river conditions. (Those conditions in which the locks and dams on the lower Ohio River do not cause a river elevation discontinuity by providing a navigational pool.)

As mentioned previously, input to the model is the stage or discharge time series at Cape Giradeau, Dam 51 (Golconda), and Barkley and Kentucky Dams. Termination of the model is by a rating curve at Caruthersville, Missouri. In the present application discharges are utilized exclusively.

The flow forecast on the Middle Mississippi at Cape Girardeau is obtained from the St. Louis District and verified with that of the National Weather Service River Forecast Center in Kansas City.

The Reservoir Control Center develops the discharge forecast at Dam 51 utilizing a simple deterministic watershed model (4) with inflow into a channel reach routed through channel storage to obtain the routed outflow. The temporal distribution of flow from the local area is then added to the routed outflow to give the total outflow from the reach. Figures 8 and 9 give typical results at Evansville and Dam 51 for one-day ahead forecasts. The basic weaknesses of this model are: (1) Forecasts are only available at Wheeling, St. Mary's, Pomeroy, Huntington, Maysville, Cincinnati, Markland, Louisville, Evansville, Dam 51 (Golconda), and Metropolis (Dam 52). Some of the reaches are 185 miles in length.

Forecasts at intermediate points then are not readily available and, if needed, depend almost entirely on the experience of the forecaster; (2) stage is not output; and, (3) changes in the physical system of the watershed cannot be readily adapted into the model.

During periods of flooding on the lower Ohio and lower Mississippi Rivers, the flow forecasts at Barkley and Kentucky Dam are developed in the Reservoir Control Center, ORD. The reservoirs are operated as a unit during flood periods on the lower Ohio River and lower Mississippi River. The primary objectives of flood control regulation by Barkley and Kentucky Reservoirs are to: (1) safeguard the Mississippi River levee system; (2) reduce the frequency of using the Birds Point-New Madrid floodway; and (3) reduce the frequency and magnitude of flooding of land outside the levees along the lower Ohio and Mississippi Rivers. Figure 10 shows some pertinent data of the projects. Since the reservoirs are connected by an uncontrolled canal, they constitute one of the largest flood control projects in the United States in terms of volume of storage allocated to flood control. However, in terms of the ability to control runoff from the headwater drainage area, these reservoirs are one of the smaller. Therefore, in flood control operation, there is literally no room for mistakes.

Previously, with our manual and simple computerized mathematical models, the RCC could forecast a crest at Cairo within a foot of the observed value. (When I say forecast, I mean knowledge a few days removed from present.) With SOCHMJ, the RCC can forecast within a few tenths of a foot.

In October 1973, the Reservoir Control Center began utilizing SOCHMJ on a daily basis. Its first testing under operational conditions then was the Ohio River and Mississippi River flood of 1974. Results were better than originally contemplated.

During that year, an incident occurred at Kentucky Dam that resulted in a rapid decrease in discharge at the project, and within 36 hours, the discharge was increased just as rapidly. Hourly data was collected through this period at Kentucky tailwater, Barkley tailwater, Dam 51, Smithland, Paducah, Dam 52, Dam 53, and Cairo. Figure 11 shows the input data for this period. The data was run both in hourly and daily format to check the model response to a rapid change in input. Figure 12 shows the results of the comparative runs. At Paducah, the data input as hourly changes gives a closer fit to the observed values, as would be expected. At Cairo, the opposite seems true. However, meaningful conclusions cannot be made because the Cairo gage record was lagging the hydrograph recession and hourly data was not available at Cape Girardeau.

Plots are also available for the 1975 flood on the Ohio and Mississippi Rivers. Figures 13 and 14 show the observed data and one-day ahead forecast for Cairo and Paducah.

The forecast shown is obtained from that simulation of Barkley and Kentucky outflows that best approachs the "optimum" for that particular set of hydrologic conditions. Sometimes as many as five siumlation runs are made before a decision is made on the final outflow schedule for day 1 (present) and the next five days. Day 2, though, might bring an entirely new set of operating conditions that will need to be evaluated.

Although the Reservoir Control Center very seldom will operate based upon anticipated rainfall, we continually evaluate the effectiveness of our present operating plan if additional rainfall should occur. Figure 15 gives the Quantitative Precipitation Forecast for the two succeeding twenty-four hour periods from day 1. These forecasts are obtained daily from the National Meteorological Center. The runoff potential from this anticipated rainfall is evaluated for the local area flows in the junction, with the input flows at Dam 51 and Cape Girardeau increased accordingly. If the operating condition is critical, we then evaluate changes in operating plans that could be implemented immediately upon reports of occurrence of the anticipated rainfall.

A particularly strong advantage of this model is the fact that, if needed, output can be obtained at every net point, for every timestep. A cofferdam was constructed near Smithland, Kentucky, to keep water from the construction area at the lock. During 1975, the Nashville District was daily advised of the five-day forecasts for the Smithland gage so that if flooding of the cofferdam was imminent, it would be controlled by the contractor. Figure 16 shows the accuracy of the model at a station where data is not normally obtained. The same situation will exist this year as the cofferdam for the construction of the dam is not in place. Also, the Corps will begin constructing a 1200 ft. temporary lock at Dam 53 during the next year, 1976, and similar forecast data will be needed at that location.

Another use of SCCHMJ is determining the flood hydrograph if there had been no reservoirs in the Ohio Basin, other factors being "status quo." Figures 17, 18 and 19 are the results for the 1975 flood. It should be pointed out that the estimated damages prevented due to reservoir storage accumulation were $1,786,000 in the lower Ohio and $4,008,000 in the lower Mississippi for the 1975 flood.

V. COSTS OF DEVELOPING & OPERATING SOCHMJ

Developmental costs of SOCHMJ include approximately $36,000 allocated from the MBM study plus an additional $69,500 in RCC funds that modified the initial effort at the junction and extended it to Louisville. It is the RCC's opinion that, in terms of the economic benefit attributable to utilizing SOCHMJ, its developmental costs are minimal. In fact, I submit that the real worth of mathematical models, such as SOCHMJ, is to give the water resources engineer the <u>capability to accomplish the job</u>.

The Reservoir Control Center applies SOCHMJ in both the conversational and batch mode on the INFONET system. Figure 20 gives the relative costs of a run according to the computer system priority. For the conversational mode, the calculated costs for a five-day forecast in the junction area is $50.08. Actual cost is approximately $25.00 because of the 50% discount given government users--a relatively minor amount when compared to the economic benefits attributable to operation of reservoirs for a single flood event.

Each five-day forecast run in the conversational mode can be accomplished in about ten minutes. This includes preparation of the input data form, data input through the terminal, central processor time, and output summarization.

VI. EXTENSION OF SOCHMJ

Figure 21 gives the boundaries of SOCHMJ now being developed by WES under the current contract. To the present 301 miles of river now modeled will be added 454 miles from Dam 51 to Louisville. It is anticipated that this model will be operational by January or February 1976. A subroutine is being built into the model that allows the internal computations to proceed through the water surface discontinuity at a navigation dam. Figures 22 and 23 show an initial application of this routine to a simulated hydrograph at Cannelton Dam. This improvement will allow application of the model to the total flow regime of the Ohio River from Louisville through the Cairo junction. Also, to be included is a subroutine to anticipate gate changes at the navigation dams so that artificial waves are not induced because of over-reaction by a lockmaster.

Presently, WES is working on a proposal to extend SOCHMJ from Louisville to Pittsburgh. The proposal will also evaluate an implicit model as both the RCC and WES feel the current application from Louisville through the junction might be approaching both an economic and a real-time operation limit.

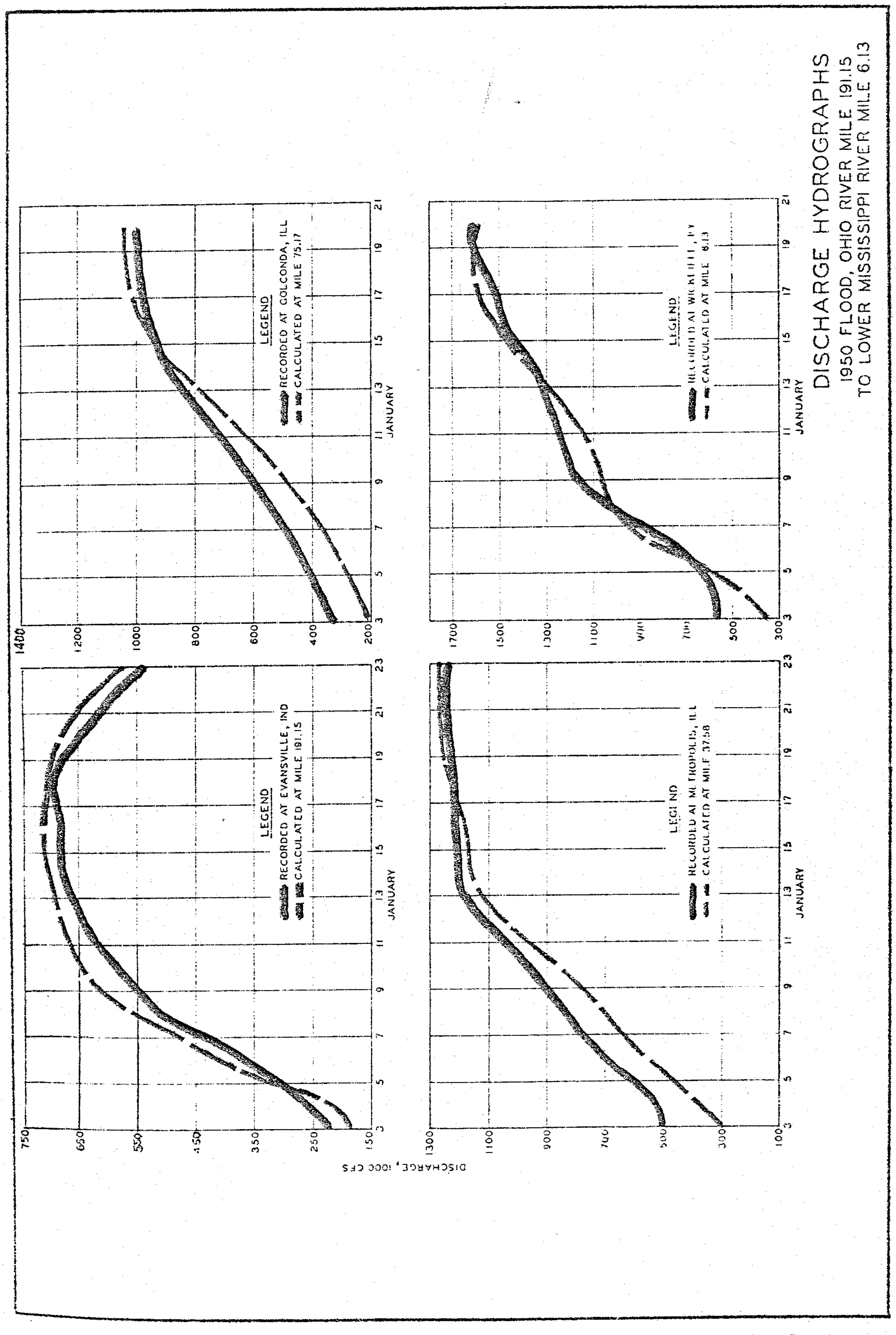
REFERENCES

(1) Johnson, B. H. and Senter, P. K., "Flood Routing Procedure for the Lower Ohio River," Miscellaneous Paper H-73-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MI, June 1973.

(2) Garrison, M. J., Granjo, J.-P, and Price, T.J., "Unsteady Flow Simulation in Rivers and Reservoirs--Applications and Limitations," Journal, Hydraulics Division, American Society of Civil Engineers, VOL 95, No. HY5, Sep 1969, pp 1559-1576, presented at ASCE Hydraulics Division Specialty Conference at Cambridge, MA, 21-23 Aug 68.

(3) Johnson, B. H., "Unsteady Flow Computations on the Ohio - Cumberland - Tennessee - Mississippi River System" Technical Report H-74-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MI, September 1974.

(4) Yates, R. A., "Flood Routing Procedure for the Ohio River," Mississippi Basin Model Board Computer Application Study," June 1973.


DISCHARGE HYDROGRAPHS
1950 FLOOD, OHIO RIVER MILE 191.15
TO LOWER MISSISSIPPI RIVER MILE 6.13
LEGEND
RECORDED AT EVANSVILLE, IND
CALCULATED AT MILE 191.15
LEGEND
RECORDED AT GOLCONDA, ILL
CALCULATED AT MILE 75.17
CALCULATED AT MILE 37.58
CALCULATED AT MILE 6.13
DISCHARGE, 1000 CFS
JANUARY

Paper 6
Fig. 1

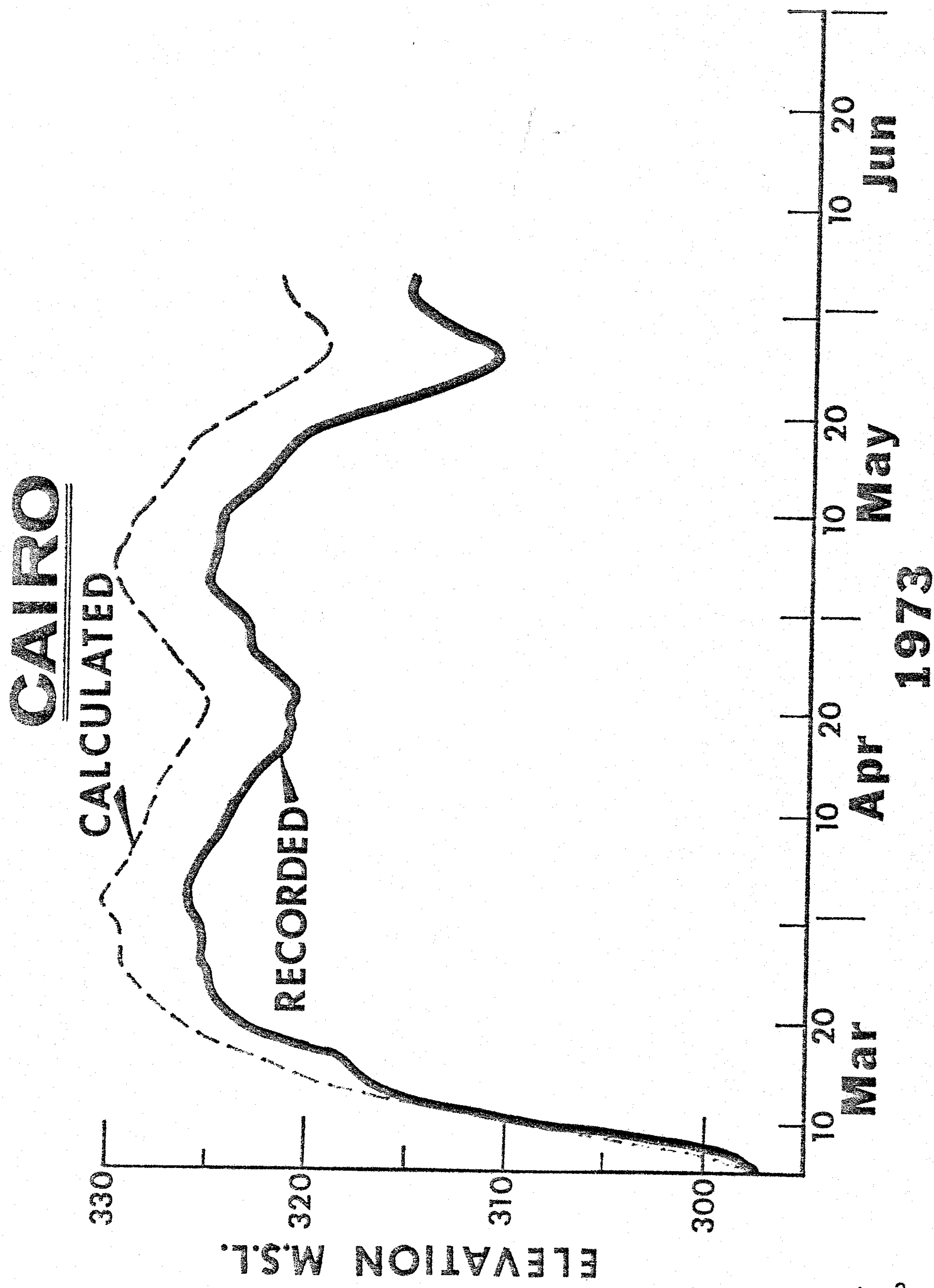
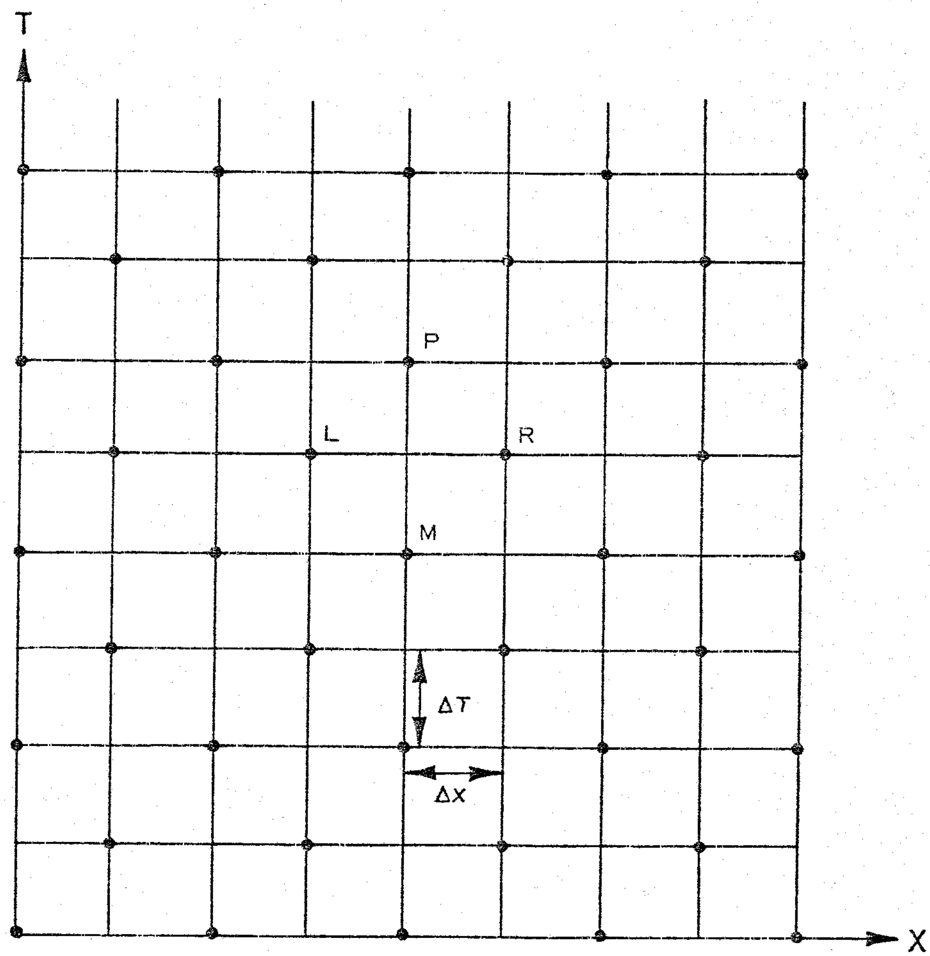

CAIRO
CALCULATED
RECORDED
ELEVATION M.S.L.
330
320
310
300
10
20
Mar
10
20
Apr
10
20
May
10
20
Jun
1973

Fig. 2
Paper 6



$$\partial\phi/\partial x = \frac{\phi_R - \phi_L}{2\Delta x}$$

$$\partial\phi/\partial T = \frac{\phi_P - \phi_M}{2\Delta T}$$

Fig. 3. Staggered centered difference computation net

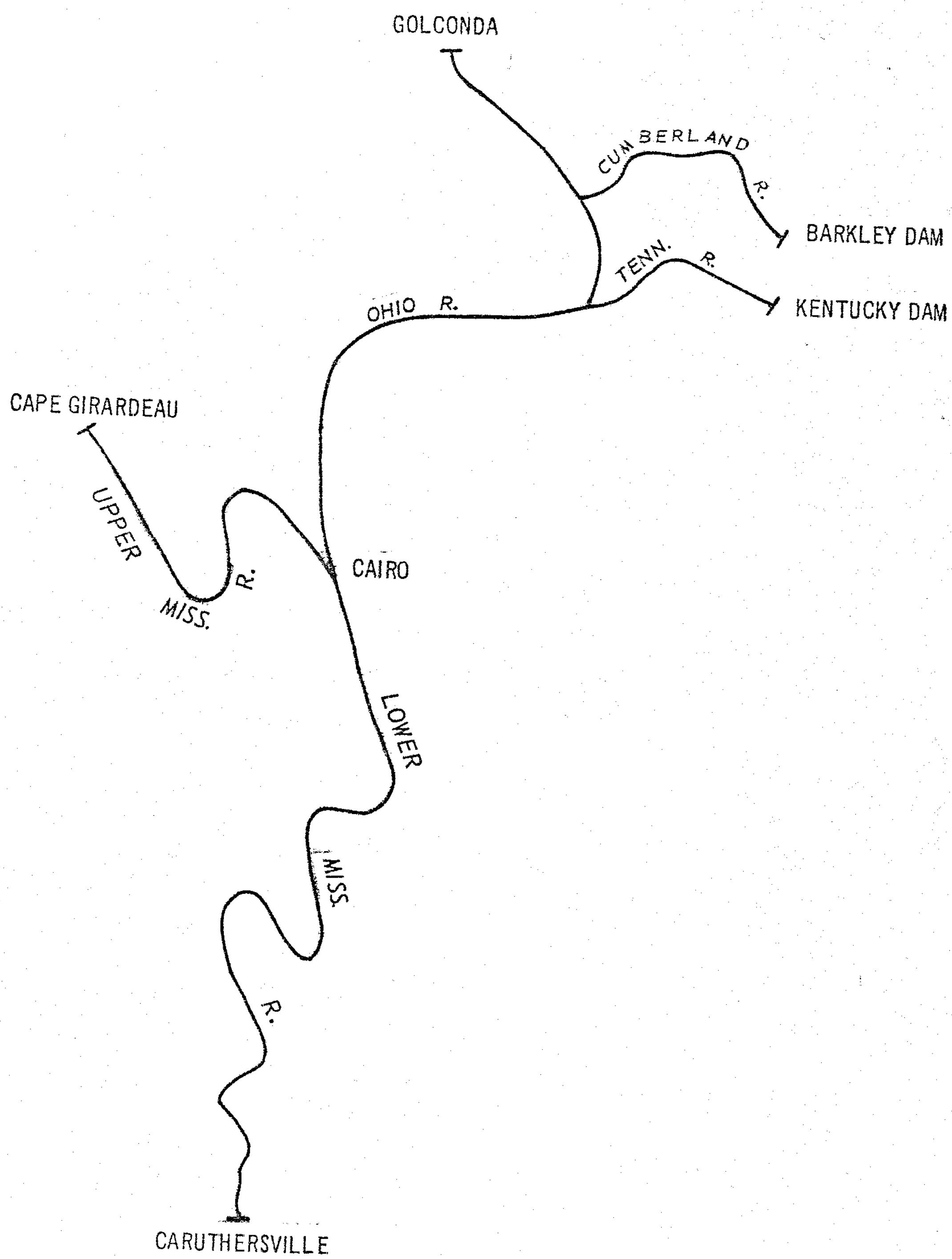


Fig. 4. Physical system modeled by SOCHMJ

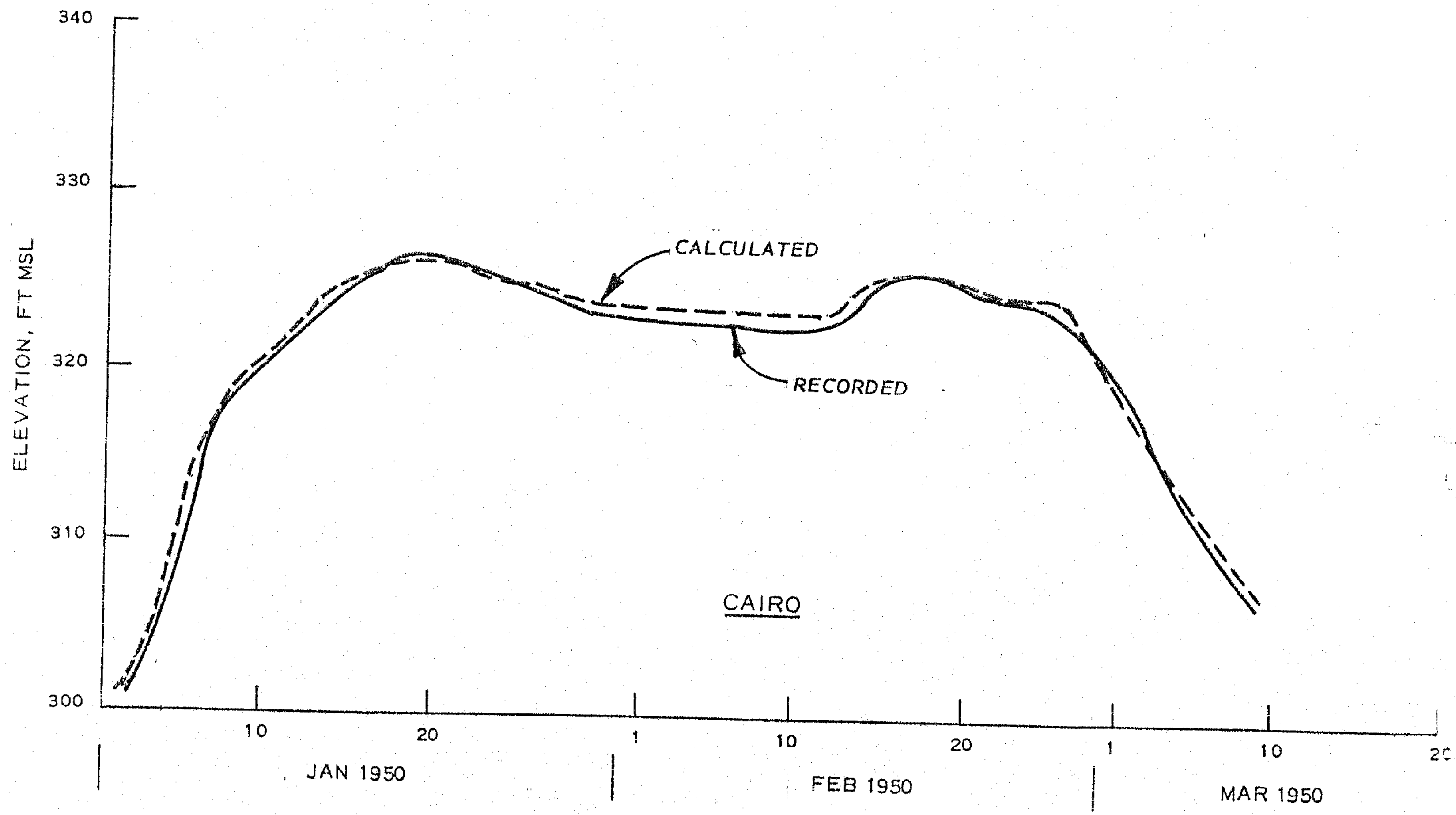


Fig. 5. Elevation hydrographs--1950 flood

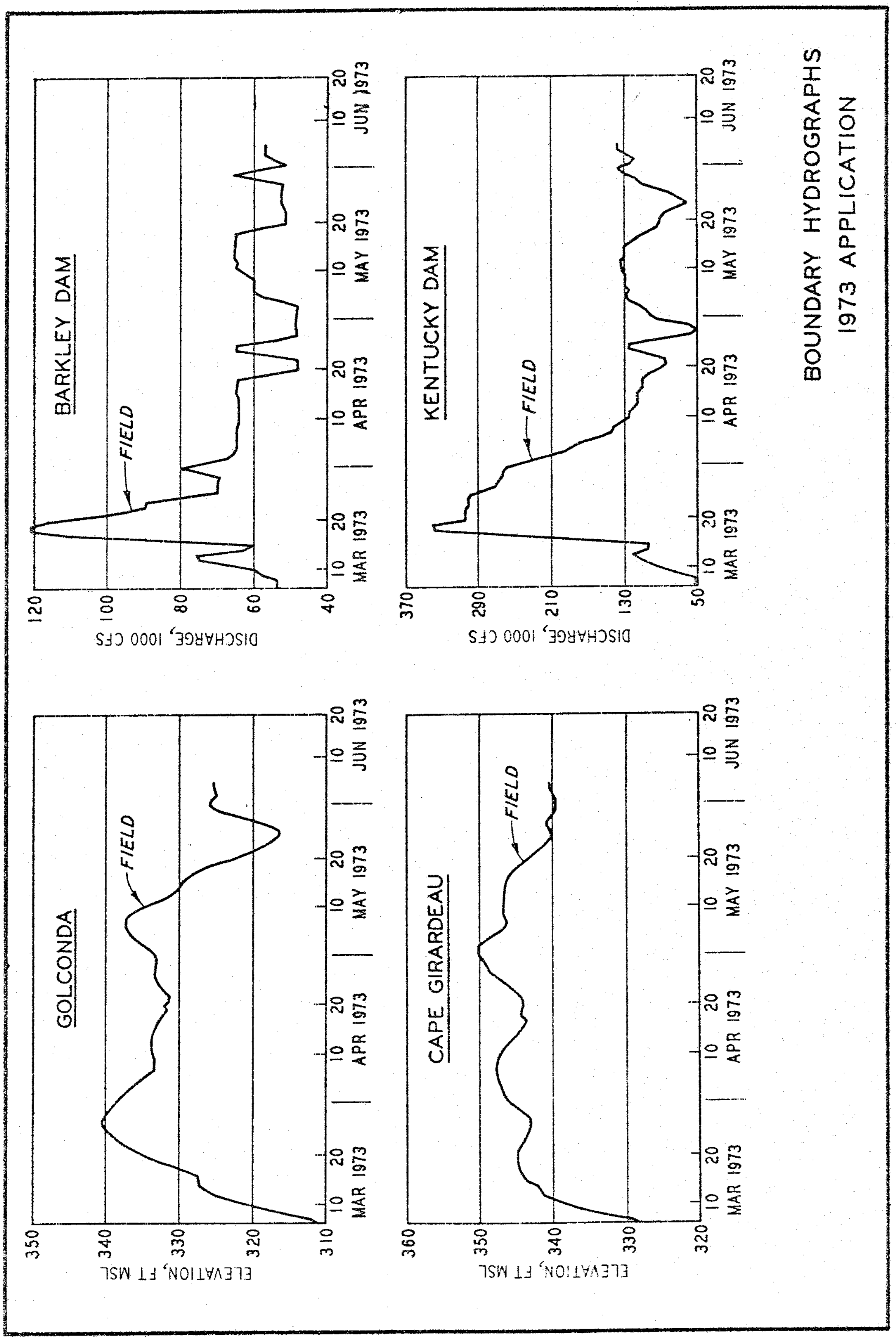

GOLCONDA
FIELD
ELEVATION, FT MSL
350
340
330
320
310
CAPE GIRARDEAU
FIELD
ELEVATION, FT MSL
360
350
340
330
320
BARKLEY DAM
FIELD
DISCHARGE, 1000 CFS
120
100
80
60
40
KENTUCKY DAM
FIELD
DISCHARGE, 1000 CFS
370
290
210
130
50
10 20
MAR 1973
10 20
APR 1973
10 20
MAY 1973
10 20
JUN 1973
BOUNDARY HYDROGRAPHS
1973 APPLICATION

Fig. 6
Paper 6

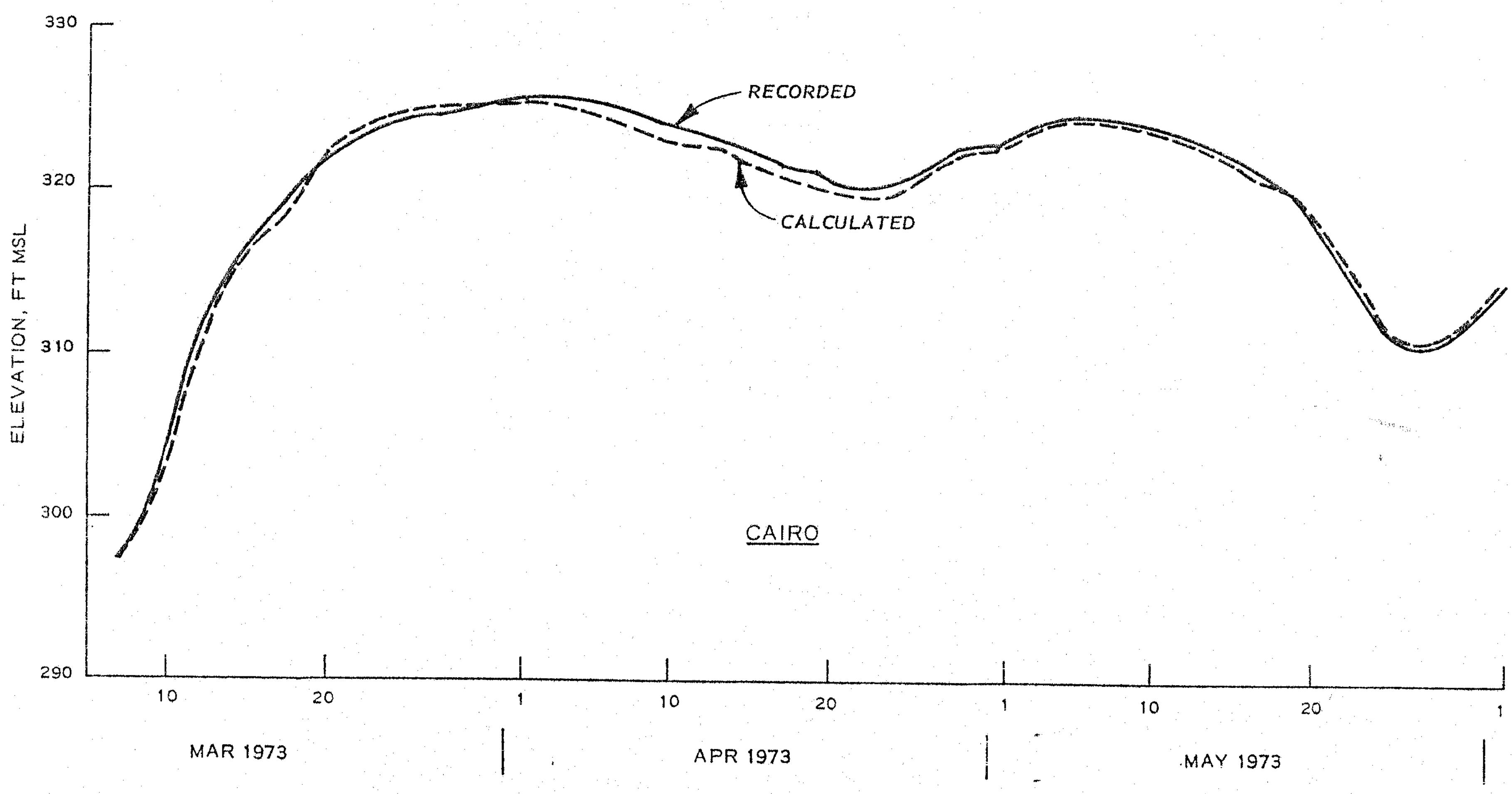


Fig. 7. Elevation hydrographs--1973 flood

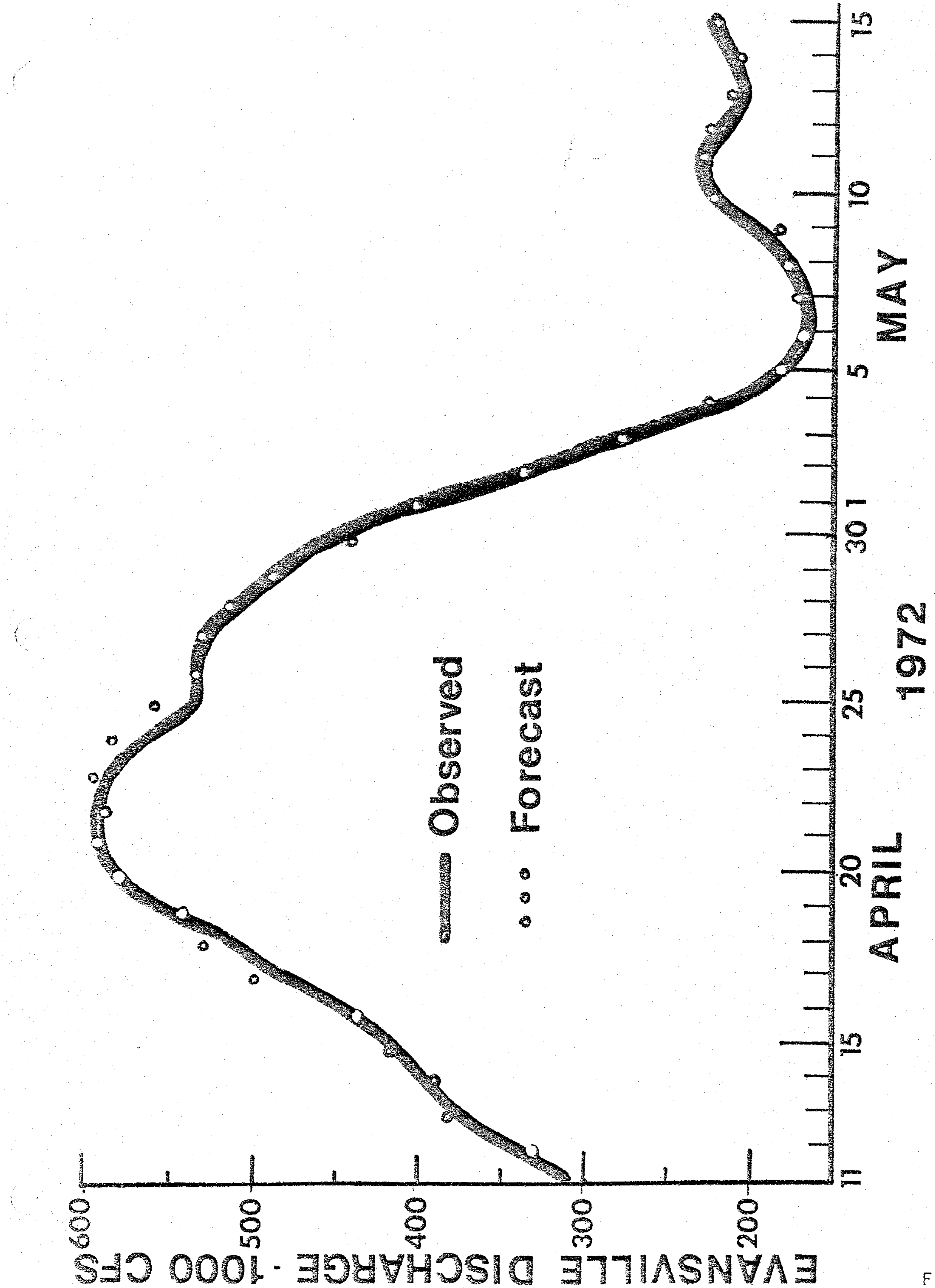
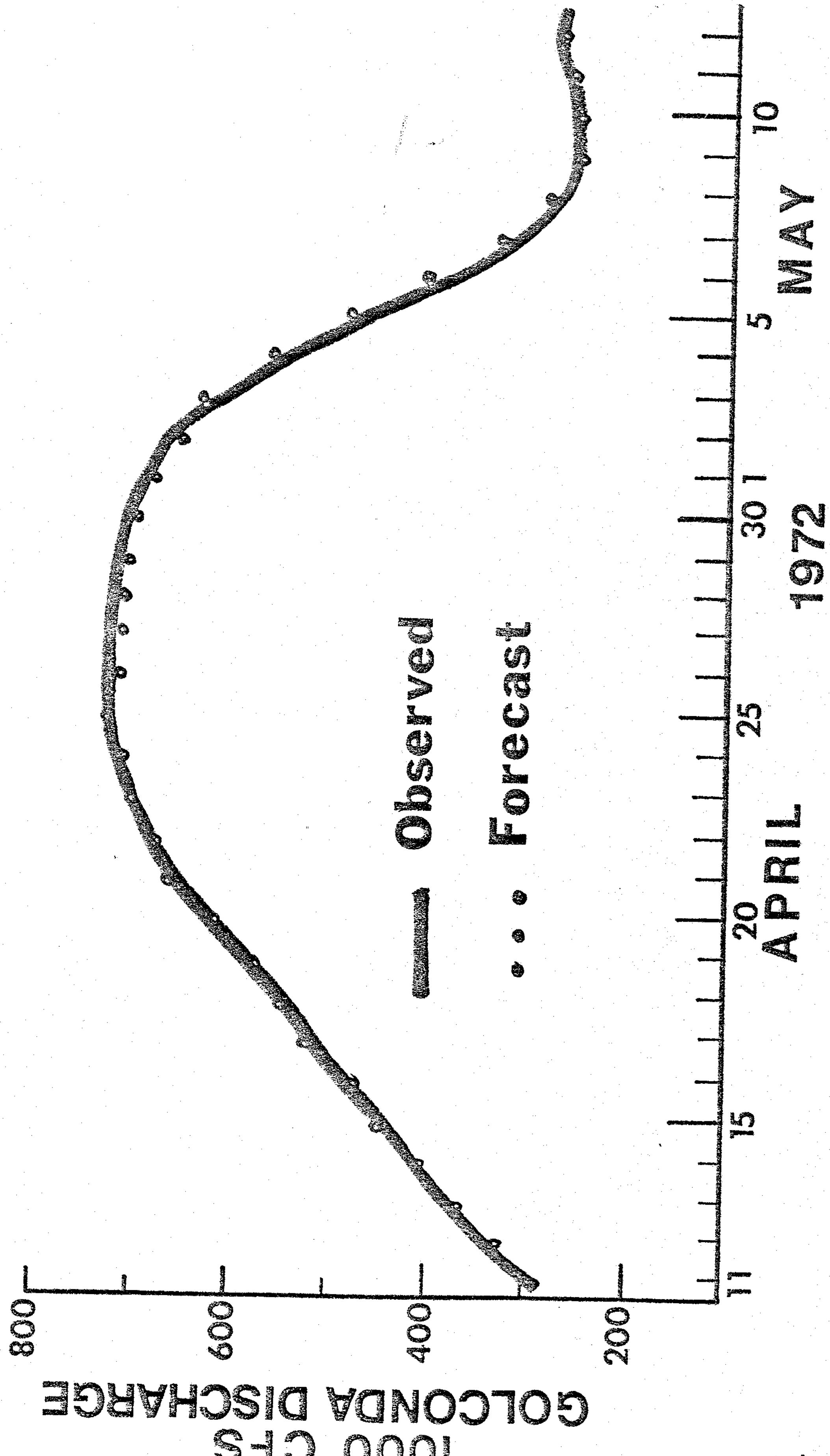

EVANSVILLE DISCHARGE - 1000 CFS
600
500
400
300
200
11
15
20
25
30
1
5
10
15
APRIL
1972
MAY
Observed
Forecast

Fig. 8
Paper 6



Observed
Forecast
1000 CFS
GOLCONDA DISCHARGE
800
600
400
200
11
15
20
25
30
1
5
10
APRIL
MAY
1972

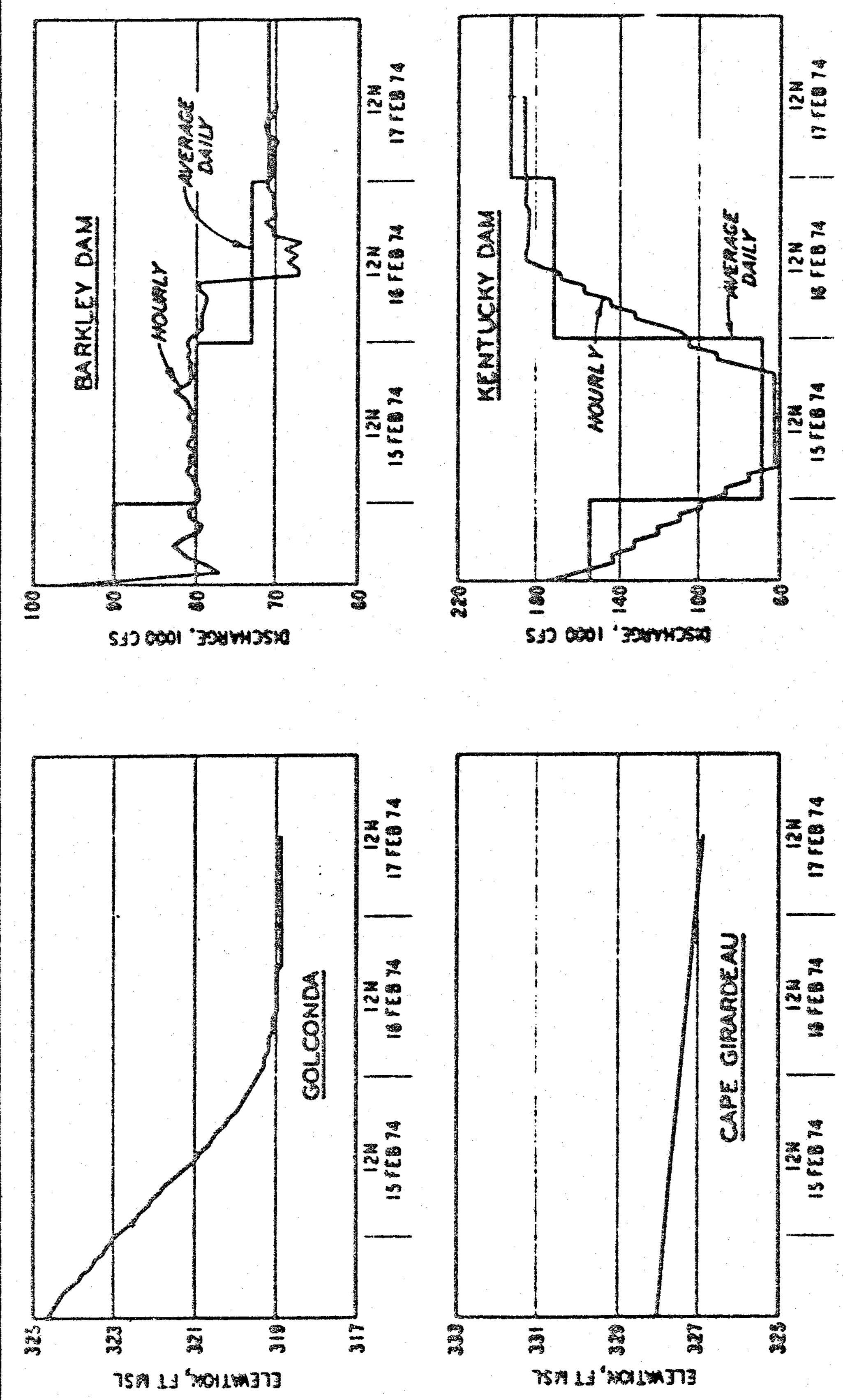
Fig. 9

PERTINENT DATA

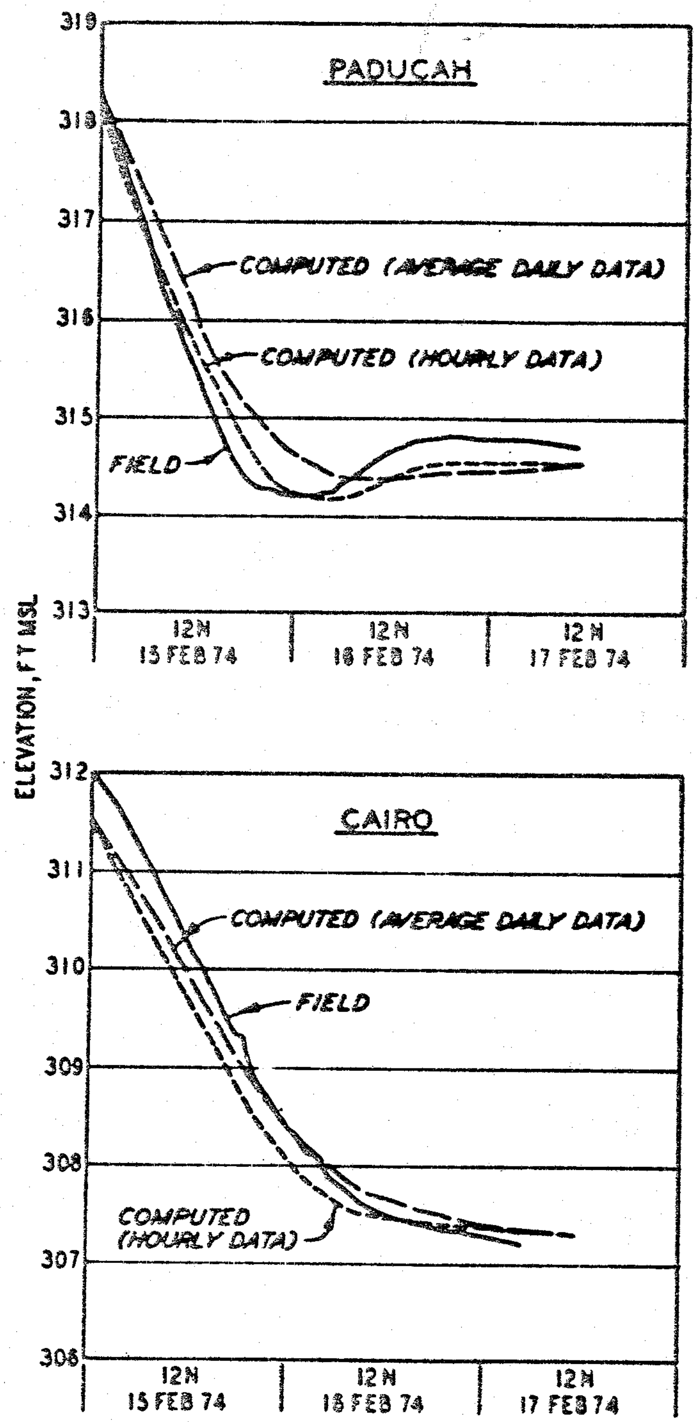
	BARKLEY	KENTUCKY
DRAINAGE AREA, SQUARE MILES	17,600	40,200
UNCONTROLLED AREA ABOVE DAMS	7,800	18,800
RESERVOIR LENGTH, MILES	118	183
STORAGE CAPACITY (Flat pool basis)		
POWER, ACRE-FEET (Inches)	259,000(.62)*	721,000(.72)*
FLOOD CONTROL, A. F. (Inches)	1,472,000(3.5)*	4,010,800(4.00)*
CONSERVATION, A.F. (Inches)	610,000(1.46)*	1,991,800(1.99)*

* Inches of runoff based on uncontrolled area.

Fig. 10
Paper 6



BOUNDARY HYDROGRAPHS
1974 APPLICATION



ELEVATION HYDROGRAPHS
1974 APPLICATION

Fig. 12

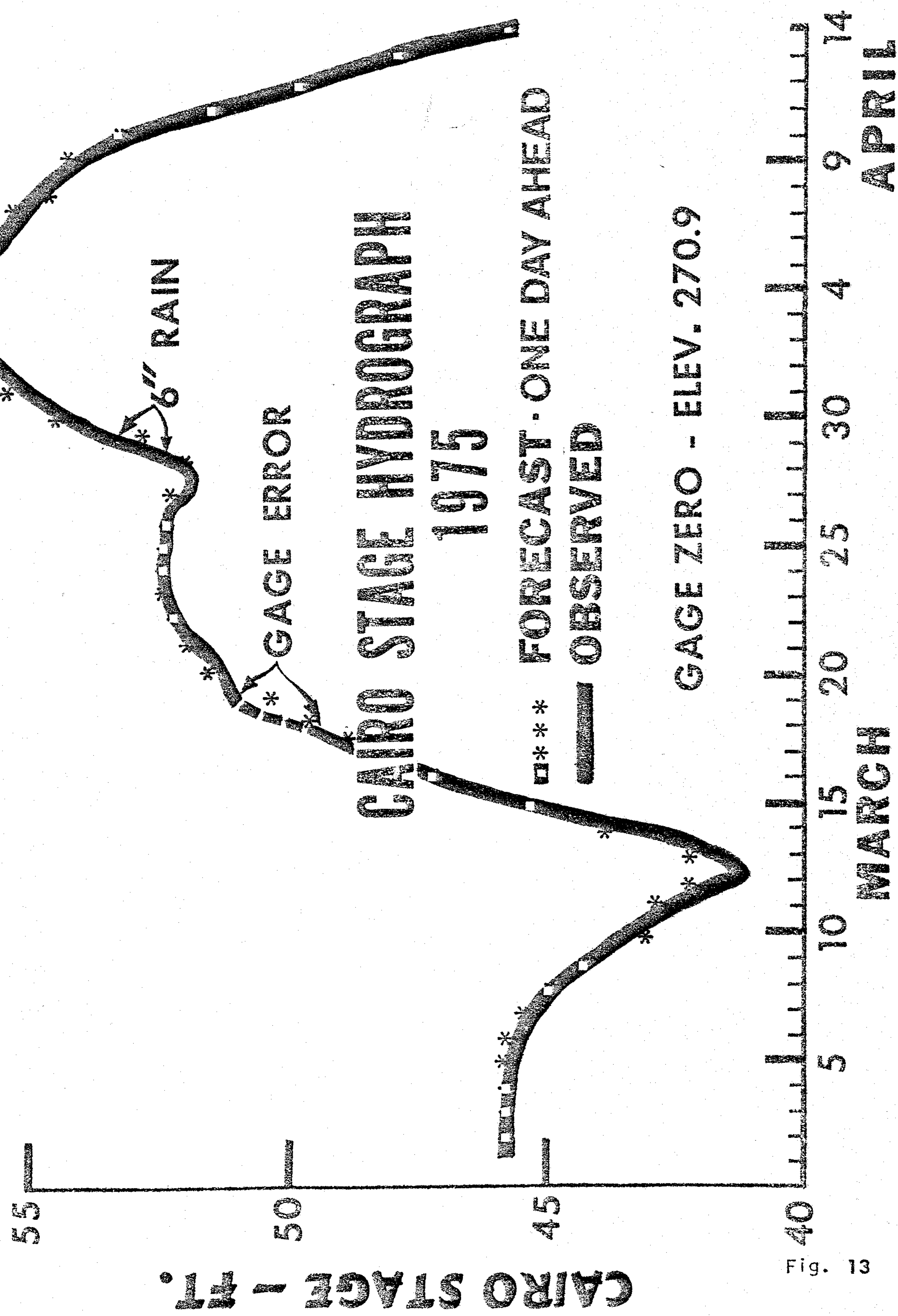

CAIRO STAGE HYDROGRAPH
1975
FORECAST · ONE DAY AHEAD
OBSERVED
GAGE ZERO - ELEV. 270.9
GAGE ERROR
6" RAIN
56.4'
MARCH
APRIL
5
10
15
20
25
30
4
9
14
CAIRO STAGE - FT.
40
45
50
55

Fig. 13

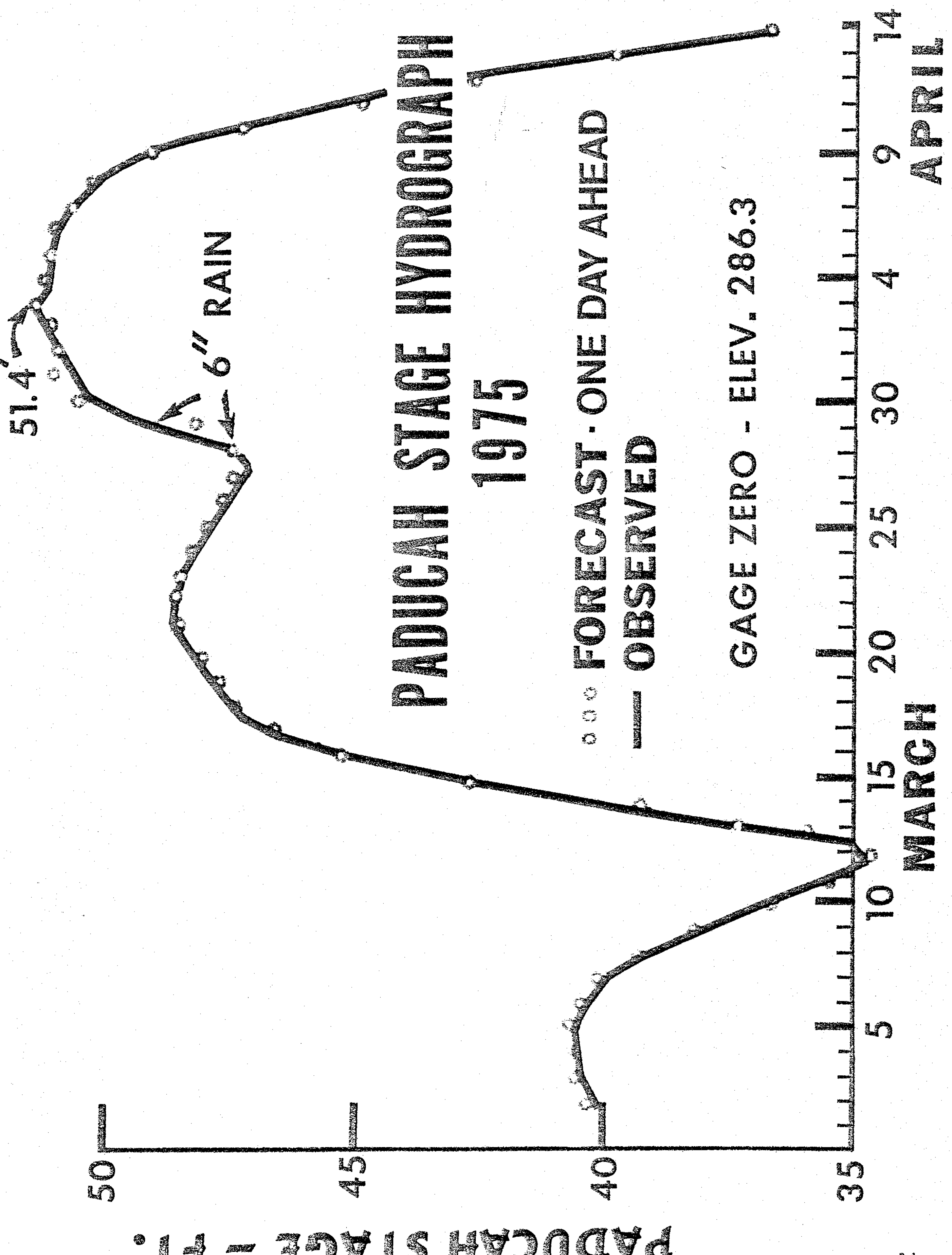
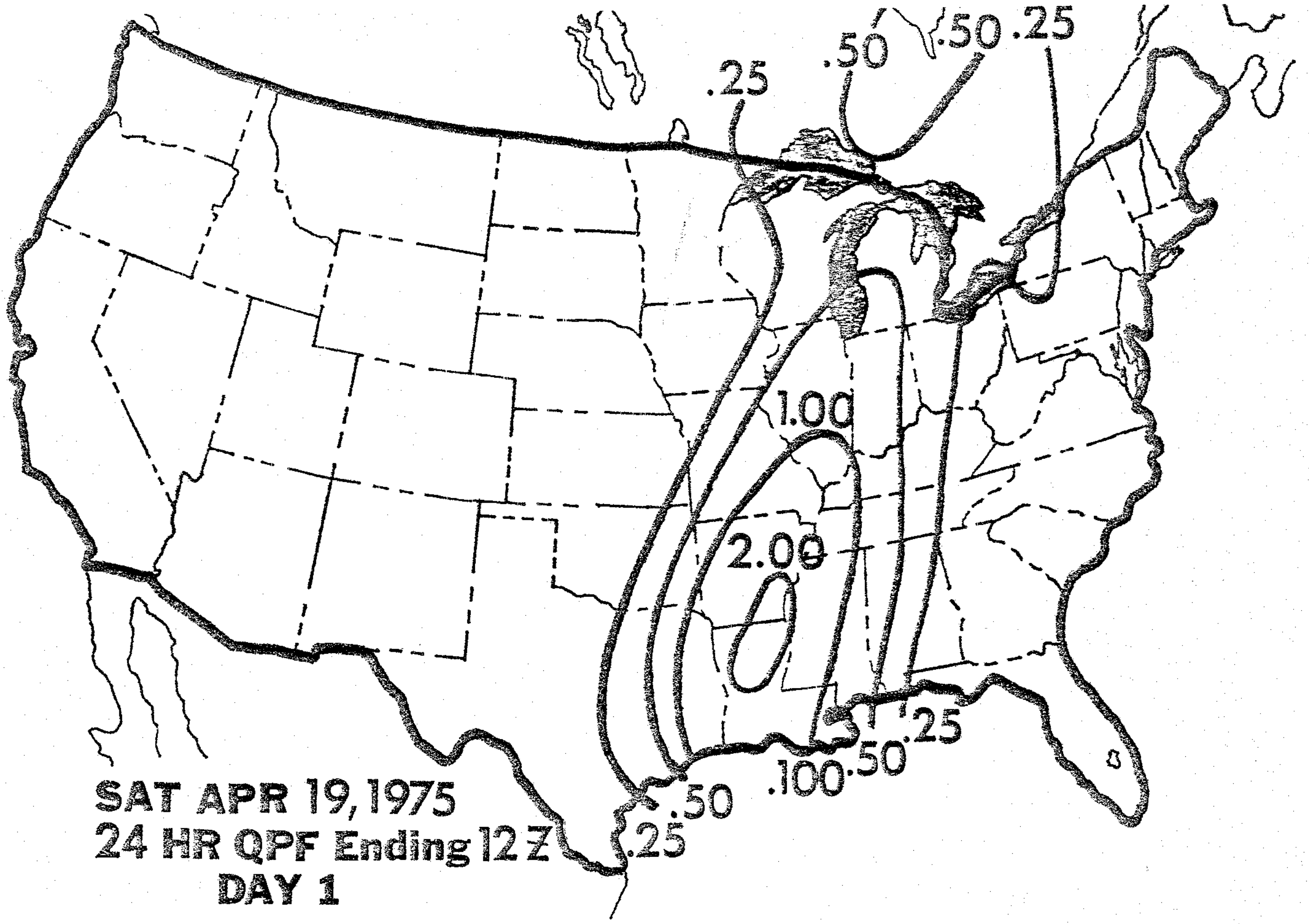

PADUCAH STAGE HYDROGRAPH
1975
FORECAST · ONE DAY AHEAD
— OBSERVED
GAGE ZERO - ELEV. 286.3
51.4'
6" RAIN
PADUCAH STAGE - FT.
50
45
40
35
5
10
15
20
25
30
4
9
14
MARCH
APRIL

Fig. 14



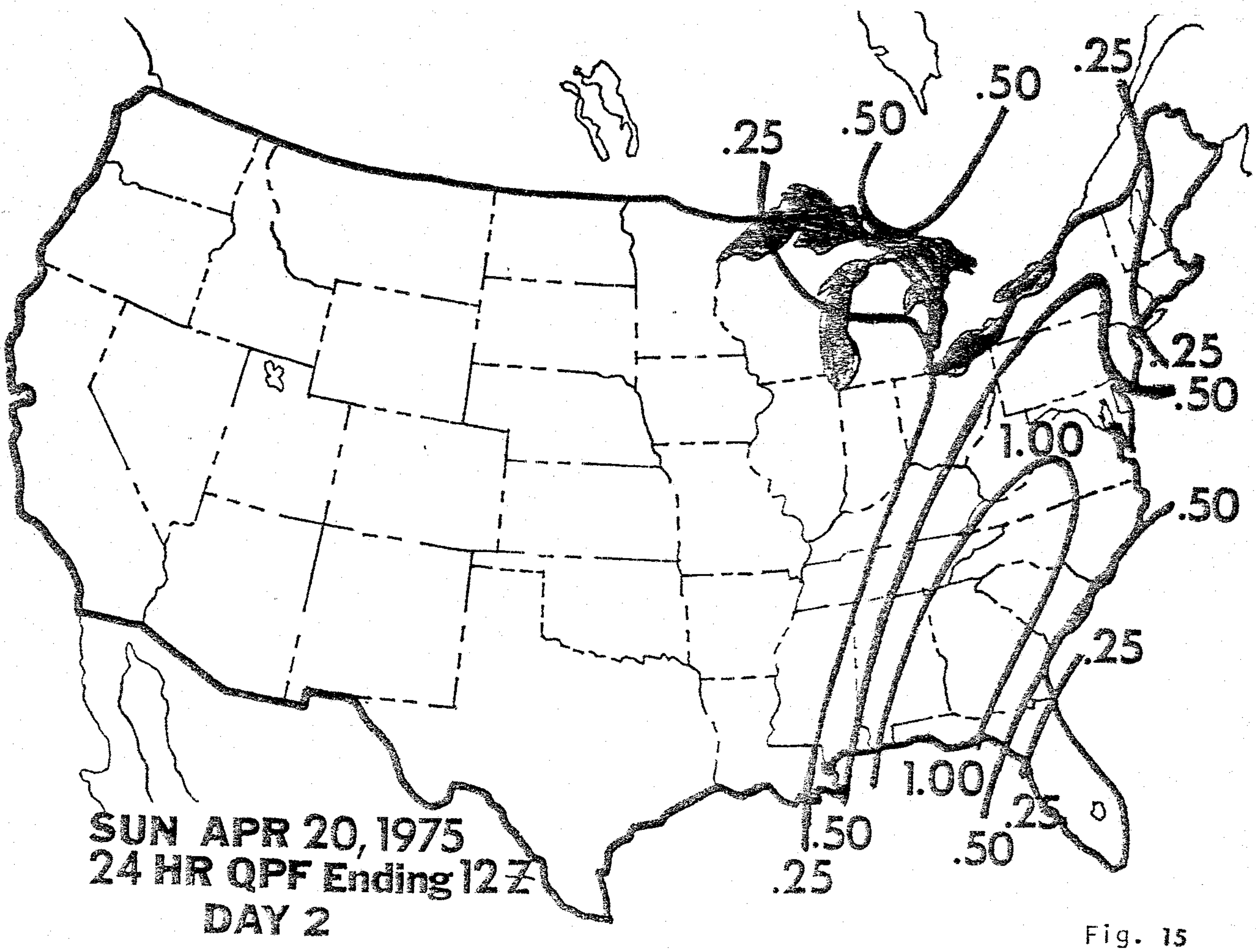


Fig. 15
Paper 6

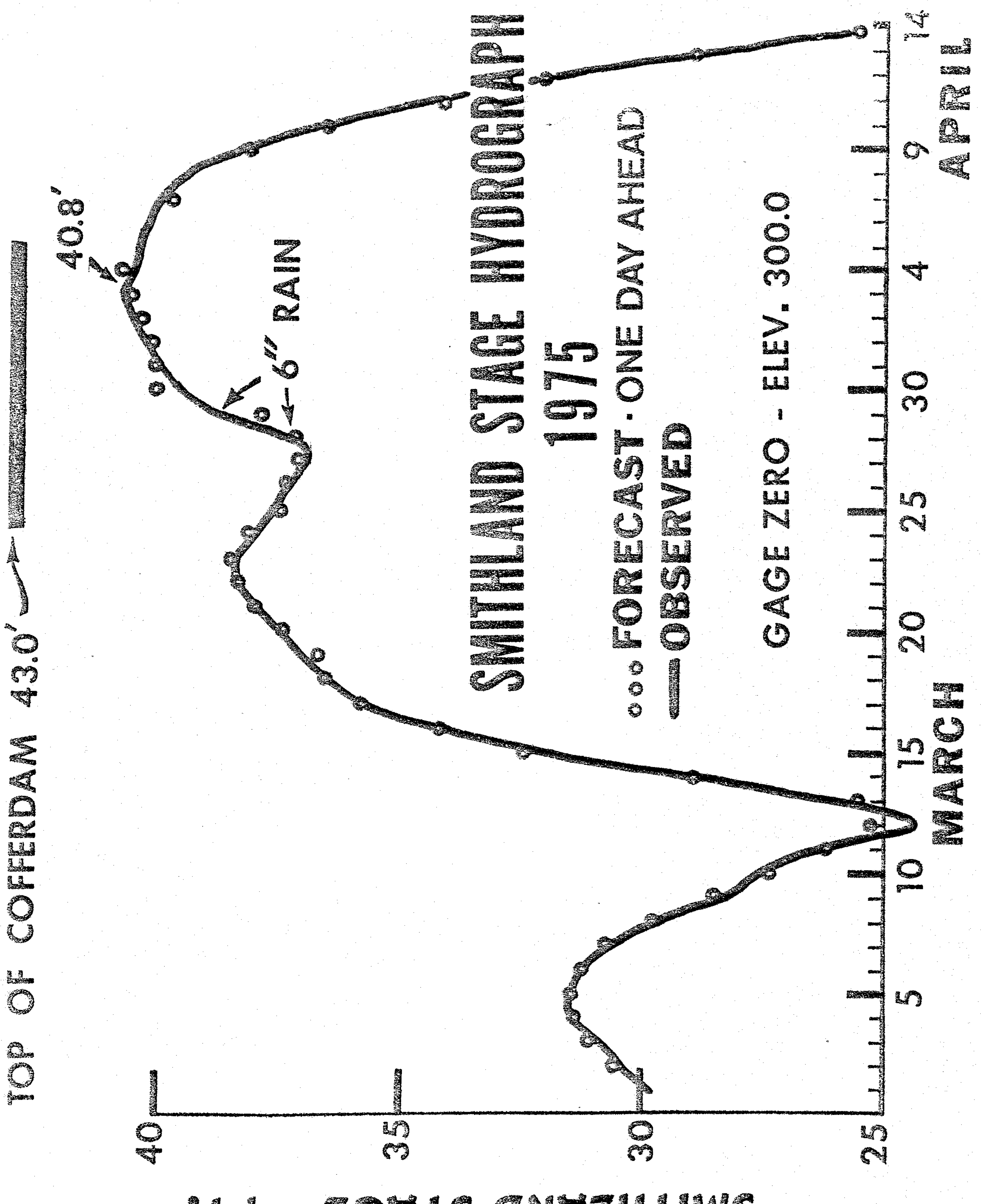

SMITHLAND STAGE HYDROGRAPH
1975
∘∘∘ FORECAST · ONE DAY AHEAD
— OBSERVED
GAGE ZERO - ELEV. 300.0
TOP OF COFFERDAM 43.0'
40.8'
6" RAIN
SMITHLAND STAGE - FT.
40
35
30
25
5
10
15
20
25
30
4
9
14
MARCH
APRIL

Fig. 16

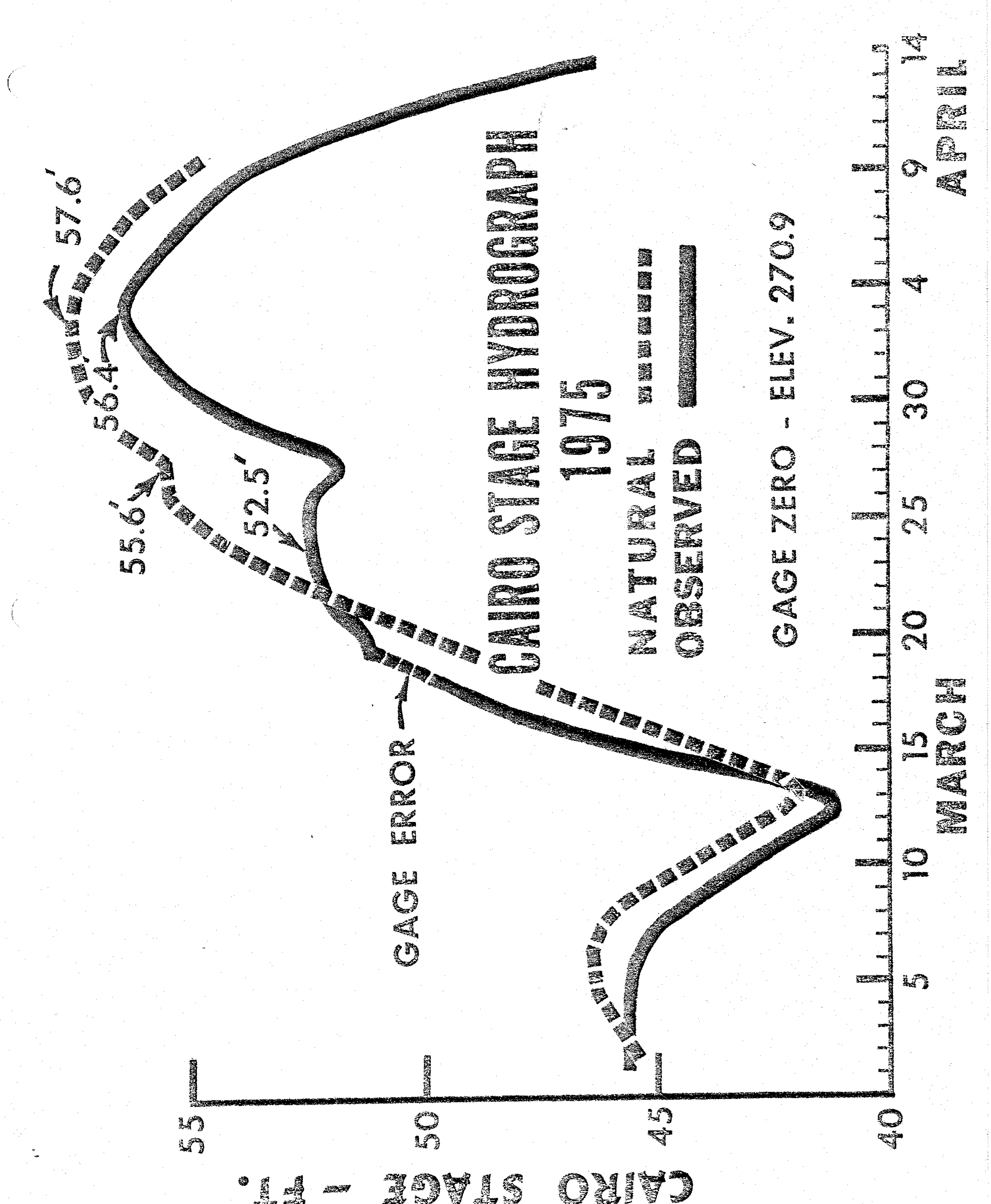


Fig. 17
Paper 6

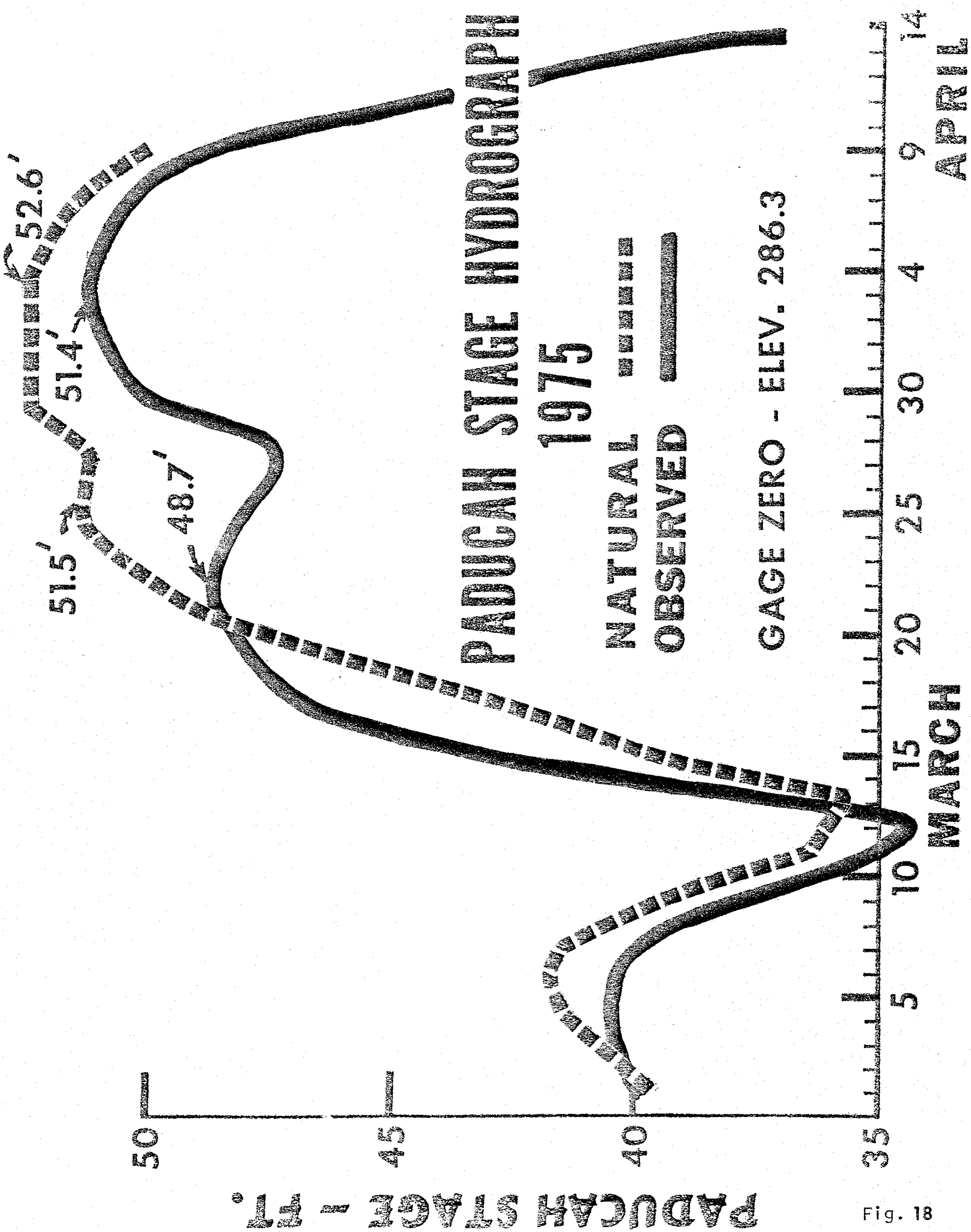

PADUCAH STAGE HYDROGRAPH
1975
NATURAL
OBSERVED
GAGE ZERO - ELEV. 286.3
PADUCAH STAGE - FT.
50
45
40
35
5
10
15
20
25
30
4
9
14
MARCH
APRIL
52.6'
51.4'
51.5'
48.7'

Fig. 18

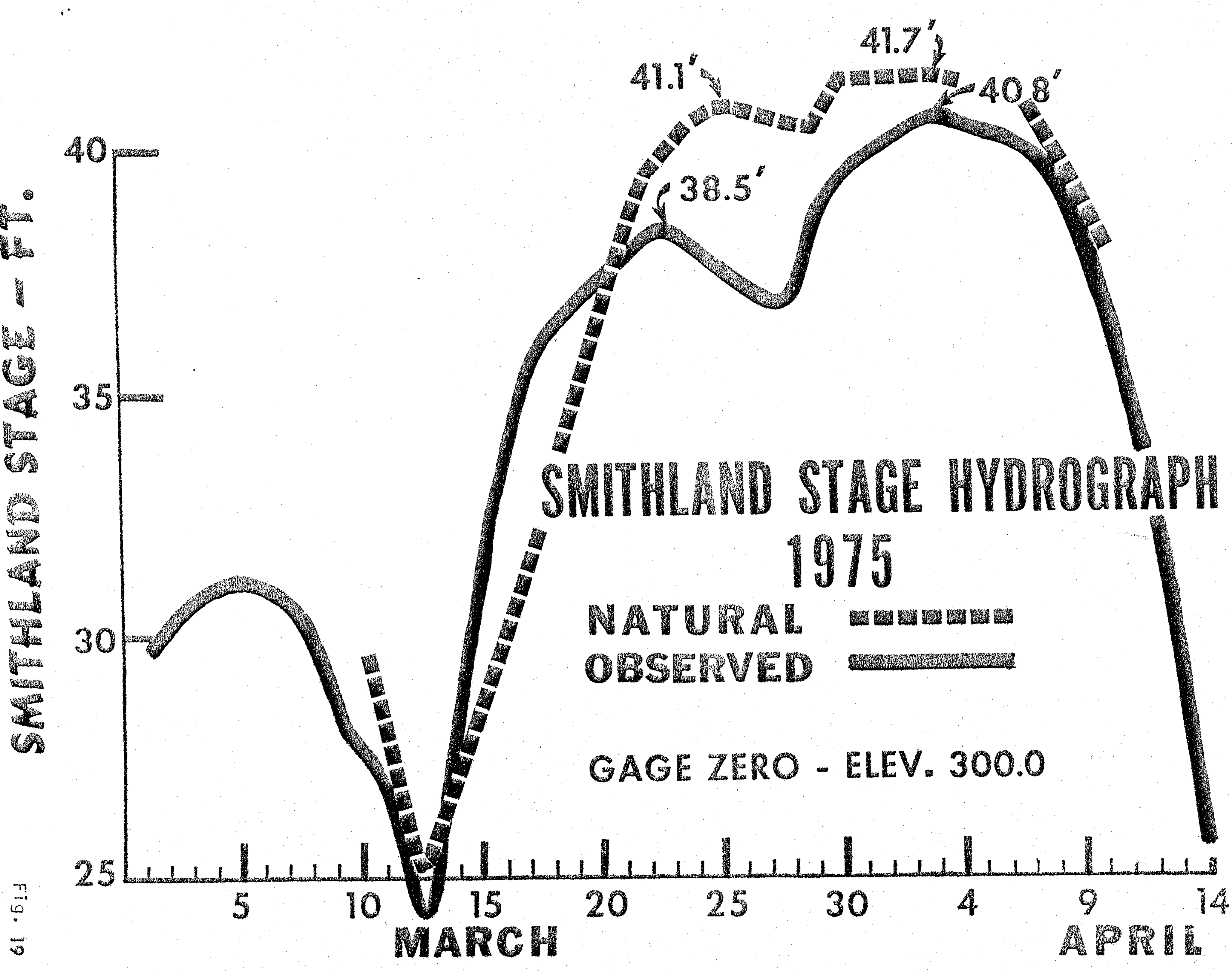

SMITHLAND STAGE HYDROGRAPH
1975
NATURAL
OBSERVED
GAGE ZERO - ELEV. 300.0
SMITHLAND STAGE - FT.
40
35
30
25
5
10
15
20
25
30
4
9
14
MARCH
APRIL
41.1'
41.7'
40.8'
38.5'

Fig. 19

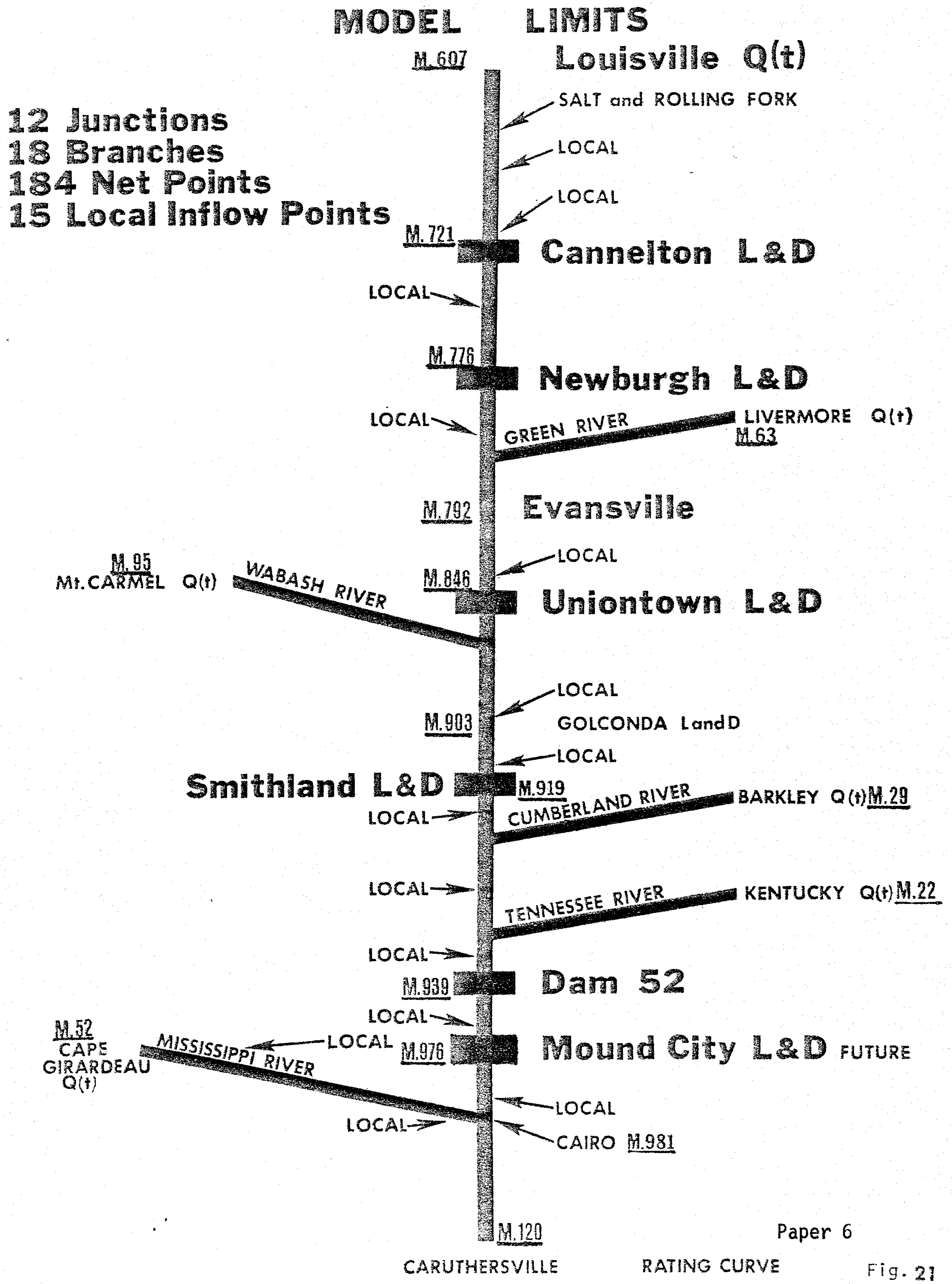
JUNCTION MODEL
COMPUTER COSTS - INFONET SYSTEM

STORAGE	104 PAGES	$.02/PAGE-DAY	$2.08 /DAY
RUN TIME	30 SRU's PER DAY OF RUN		

PRIORITY		**COST**
CONVERSATIONAL	$0.20 / SRU	$6.00 / DAY OF FCST
BATCH 9 (2 HRS)	0.16 / SRU	$4.80 / DAY OF FCST
BATCH 5 (18 HRS)	0.06 / SRU	$1.80 / DAY OF FCST
BATCH 1 (72 HRS)	0.03 / SRU	$0.90 / DAY OF FCST

FOR AN 8 DAY RUN (5 DAY FORECAST)

STORAGE	$2.08
RUN	$48.00
	$50.08 PER RUN



Paper 6

Fig. 21

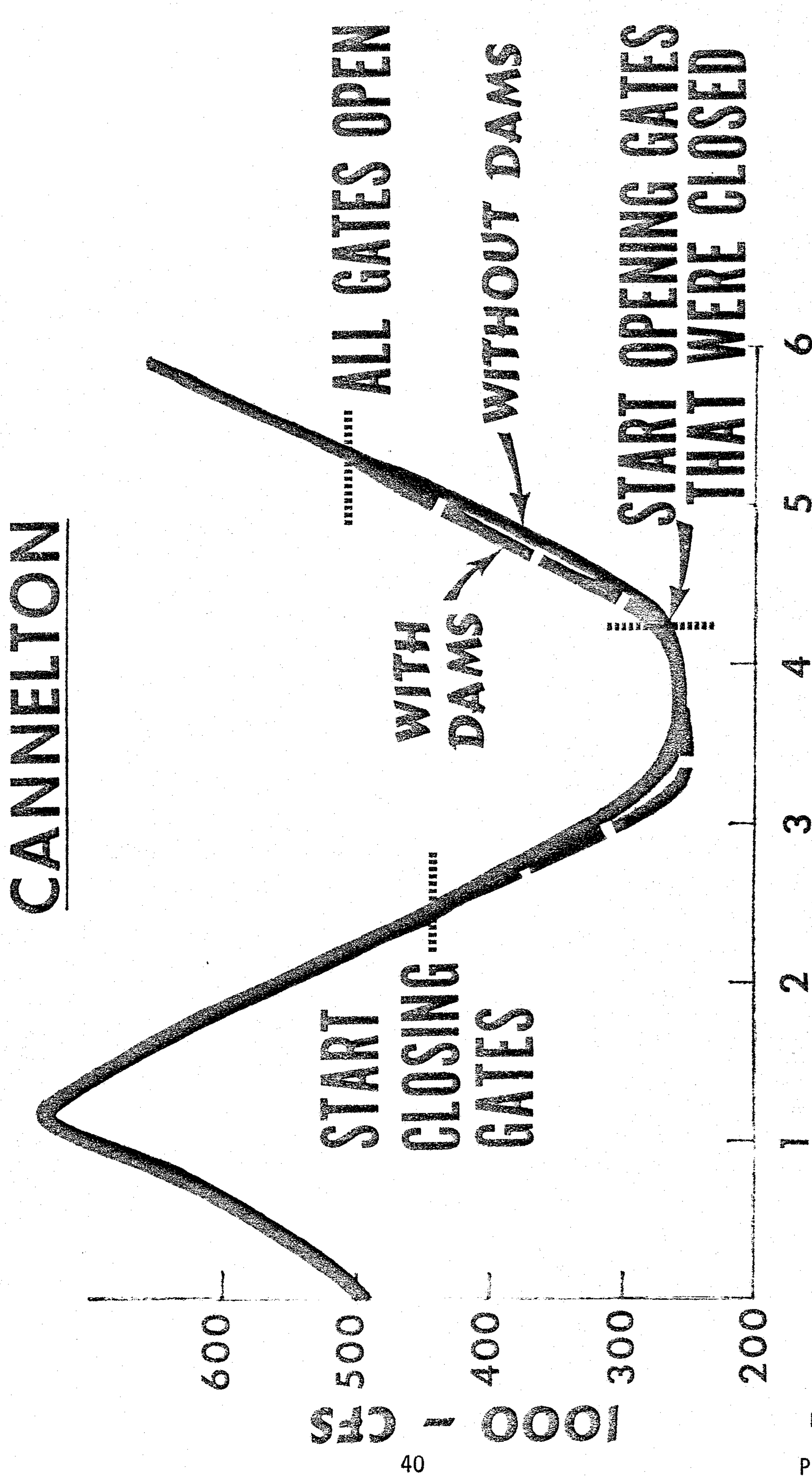
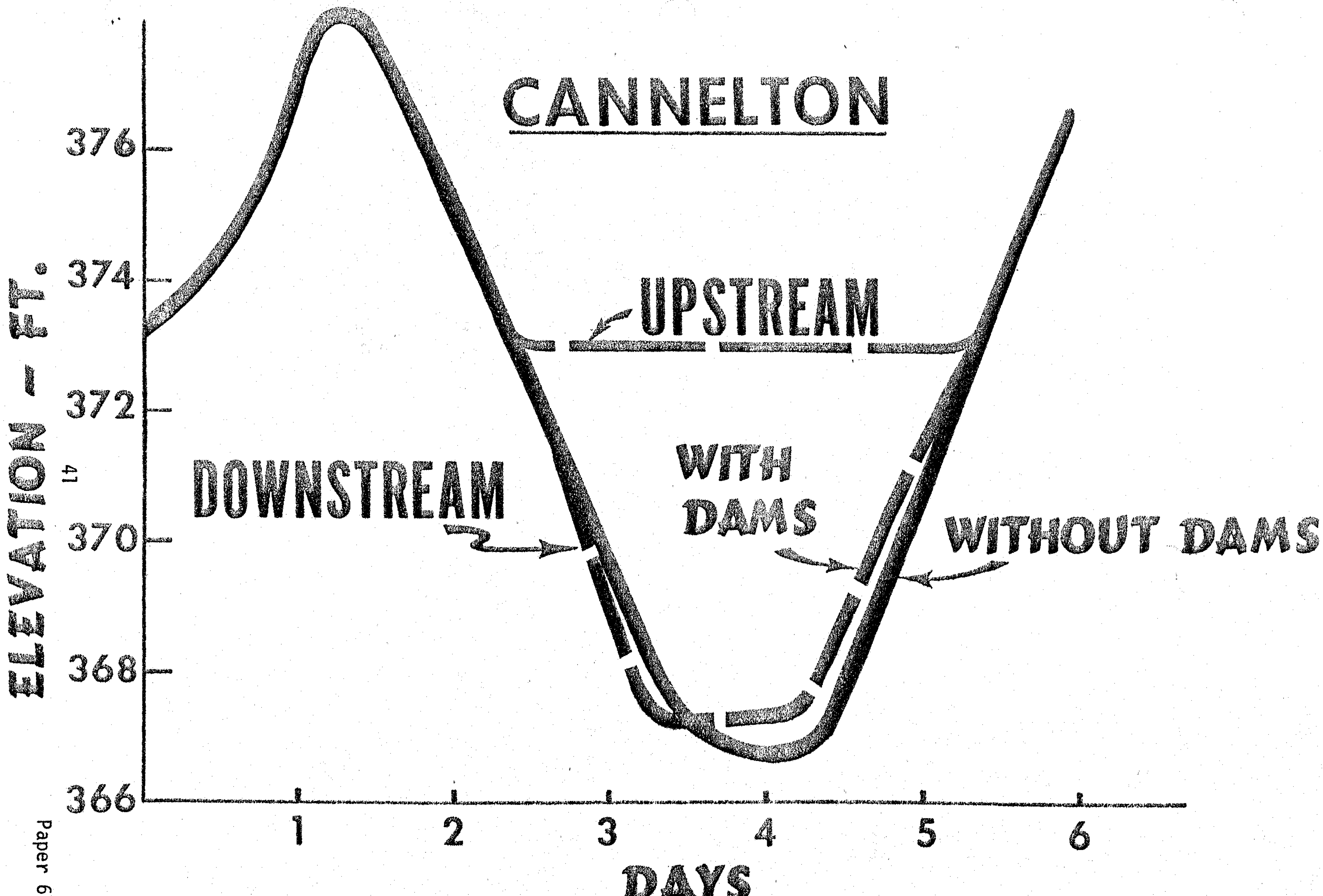

CANNELTON
START CLOSING GATES
WITH DAMS
WITHOUT DAMS
ALL GATES OPEN
START OPENING GATES THAT WERE CLOSED
DAYS
1
2
3
4
5
6
1000 - CFS
200
300
400
500
600

Fig. 22



Paper 6

Fig. 23

RESERVOIR DATA MANAGEMENT - INTERACTIVE GRAPHICAL TERMINAL

By

Daniel J. Barcellos[1]

INTRODUCTION

Utilization of computer technology to assist in the operation of reservoirs requires several phases of data management:

a. Collection of data from remote locations.

b. Storage of data.

c. Expansion of data into engineering units.

d. Presentation of data in useful formats.

This paper offers an approach to the presentation of reservoir data in a practical form by the use of the interactive graphical terminal and a commercial timesharing computer service. Aspects of data storage, data expansion, and methods for the presentation of data to the decision maker are discussed. The discussions are limited to experience gained in the Sacramento District during a one year period of using an interactive graphical terminal and the services of a Computer Sciences Corporation computer.

Data Storage. - Assuming that an automatic reservoir data collection system is in operation, and that the data reaches a central location, the data must be stored. Hardware provisions for such storage are normally a computer disc storage facility as a primary storage device and a magnetic tape facility for backup storage. For this paper the data exists on disc and on magnetic tape at a large commercial computer service.

Storage area on the disc device allocated to reservoir data is herein identified as the data file or data base. The flow of data, into and out of the data file, is controlled with instructions written in the fortran IV computer language.

The size of the data file is kept to a minimum by storing only numeric values. No letters or symbols are stored which require special formatting. The size of the data file is further affected by the type of data stored. This topic is discussed below under the heading "data expansion".

[1]Hydraulic Engineer, Reservoir Regulation Section, SPK.

One line (herein referred to as a record) of data in the data file consists of a row of numbers. The numbers in the record are related to each other by time of collection and by reservoir. The relative position of the numbers within a record indicates what each number represents; whether it be a river stage, pool stage, precipitation reading, etc. Each record is identified by a key. The key is a number computed from the date, reservoir identification number, and the time (if required). The key is used to randomly retrieve a record from the data base for a specific reservoir, date and time.

Data Expansion. - The term "data expansion" is coined for this paper to describe the process of computation needed to convert basic data collected from a reservoir into useful information. An example of the data expansion process is the conversion of reservoir pool stage into the volume and the surface area of water impounded in the reservoir. Further expansion of the data is accomplished by combination with other data to compute regulation parameters; reservoir inflow; etc.

If the reservoir data is being routinely collected and stored on disc and magnetic tape, how is the expansion process to proceed? There are basically three ways to approach this problem. (1) The basic collected data stored on disc can be expanded each time a report is required; (2) the basic data can be expanded at random times with the expanded data stored in additional data files in anticipation of further report requirements; or (3) the basic collected data can be expanded at collection time with both basic and expanded data included in a data file. The three approaches to data expansion are compared on the basis of disc storage requirement, computer computation requirement, and data editing requirement.

The least requirement for disc storage results from storing only the most basic data from which all other information is computed. Going back to the example for the data expansion process, only the pool stage is kept in the data file, with the volume and surface area of water computed from the stage when required for a report. By storing only the basic data the size of the data file is kept to a minimum.

The least requirement for computer computation results from the method that allows expansion of data at random times and the storage of the expanded values in additional data files. The computation requirement is reduced by eliminating repetitive computations for similar reports through retrieval of data from the expanded data files. The requirement is further reduced because report demands may fluctuate, at times reducing the amount of expanded data necessary.

Reduction of computation requirement by establishing additional data files is discounted by the additional editing required. Two of the approaches to data expansion require the expanded data to be stored permanently on disc. An editing change requires not only the change to the basic collected data, but also changes to all the data expanded from it. For example, if the basic datum of reservoir pool stage is found to be in error, then the volume of water, surface area of water, inflow to the reservoir, etc. which are stored in data files, also require corrections. Tracking down all the data requiring corrections can be a massive data handling problem. In the approach where only the basic collected data is stored permanently, the editing change is performed on the value in the data file only; since the expanded data are recomputed from the value in the data file each time they are needed.

The method chosen for data expansion for this paper is the one requiring that only the basic data received from the reservoirs be stored permanently on disc. The minimum requirements for data storage and the ease of data editing influenced the decision.

Another factor to be considered for data expansion is the amount of constant data needed to carry on the expansion process. The constant data is necessary to describe each data conversion relation. In the reservoir pool stage example, the constant data refers to the tables of data used to describe the non-linear relationships of volume versus stage and area versus stage. For computer application it is desirable to reduce the amount of this constant data in storage.

To reduce the amount of constant data required to convert pool stage into areas and volumes a simple mathematical relationship is used. The relationship is the general form polynomial equation adapted for computer implementation. The equation used to compute surface area from stage is of the form -

$$\text{area} = \sum_{i=1}^{\text{degree}+1} (\text{constant})_i \, (\text{Stage above invert})^{i-1}$$

The equation used to compute volume from stage is of the form -

$$\text{volume} = \sum_{i=1}^{\text{degree}+1} \left[(\text{constant})_i \, (\text{Stage above invert})^{i} \right] / i$$

Notice, only one set of constants is needed for both equations. The equation for volume is gotten by integrating the equation of area. The term "degree" refers to the maximum exponent considered for the general form polynomial.

The problem of selecting a degree of the polynomial equation and computing the required constants is solved by using the orthogonal polynomial least-squares curve fitting theory. The theory is available in a computer program form from Computer Sciences Corporation (see reference 1). The program in subroutine form by Computer Sciences Corporation has been included in a special purpose program to compute the constants for a (reservoir pool stage versus surface area) polynomial equation.

Using this program interactively with a graphical computer terminal, degrees of the polynomial are selected for trial computations. Each trial is quickly evaluated by looking at the fitted curve derived from the trial polynomial and the curve of original area versus stage data plotted together. Results for the selected polynomial are normally within one percent difference for the data value and the fitted value over the entire range of data. Exceptions to this accuracy occur when the original curve of area versus stage is not a smooth well defined curve. The least squares computations yield a smooth curve through the data in this case.

In computing the polynomial constants for twelve reservoirs in the Sacramento District, desired curve fitting accuracy was obtained from polynomials with degrees less than ten. This means that ten or less constants are needed for each reservoir to compute surface areas and volumes instead of the tables of data that in a method of conic interpolation included a maximum of 480 values for one reservoir.

The implimentation of the polynomial method of computation results in significant savings in constant data storage. The small deviation of the polynomial values from original data is considered insignificant and well within the accuracy of these original measurements.

Data Presentation. - The discussion of methods for presenting reservoir data assumes that a data collection system; a storage facility for data; and computer solutions for data expansion are implemented. The assumption limits discussion to the hardware and program support necessary to present computed data in various formats. A further constraint placed on this discussion is that the data is presented in "real time". The reason for this constraint is that the ultimate goal of computer application is to provide decision makers with reservoir operating information during flood emergencies.

The hardware utilized for testing various data presentation techniques in the Sacramento District was a Tektronix 4012 graphics computer terminal with a hardcopy device. The terminal was connected via a voice grade telephone link to a Univac 1108 computer operated by Computer Sciences Corporation (CSC) in Los Angeles. This particular computer service was selected for testing because of its sophisticated operating system in support of interactive terminals, and availability considerations.

The programming performed on the interactive computer system was primarily written in the Fortran IV computer language. Support programming written by others is used whenever possible. Some of this support programming consisted of the Tektronix Plot-10 Terminal Control System (see references), the Tektronix Plot-10 Advanced Graphic II package and CSTS Math-Pack Programs supported by Computer Sciences Corporation.

The simplest data presentation format is devised for all the daily average information available for each reservoir. An example of this format is shown in Figure 1. Using the capabilities of the graphics terminal a tabulation of data is displayed on the cathode ray tube of the terminal which offers a more understandable format than is possible on a standard printer or teletype. Upper and lower case characters are written at any location on the terminal display. Lines are drawn around labels and data in such a manner as to clarify their meaning. Using these techniques it is possible to present all the daily average data for each reservoir in a compact form and preserve the readability of the output. This particular method of presentation is currently being used in the Sacramento District on a trial basis in parallel with hand computation methods to verify the data expansion process. It is already proving a valuable tool in verifying hand computations and for forcing a hard look at old computation procedures that "have always been done that way".

During the development of the tabular type output a serious short coming of the 4012 terminal was realized. The terminal allows for printing of only 74 characters on one line. This was deemed insufficient for many projected applications. For example, a short summary of daily reservoir operation data for all the reservoirs was not compact enough to fit on the terminal display because it was necessary to put information for each reservoir on more than one line. It is recommended that in future applications of the graphics terminal that the large screen model graphics terminal be used. The large screen model currently recommended is the Tektronix 4014 graphics terminal It provides capability for 133 characters on one line with 64 lines on the screen.

A graphical plot method of data presentation is applied to a plot of daily average reservoir operation data over the period of a month. An example of the plot is shown in Figure 2. Extensive use of the Tektronix Plot-10 Advanced Graphing II package is made in the application. The package performs most of the numerical scaling and labeling of the plotting axis plus the actual curve plotting. Curves of various types are plotted together on the terminal screen for reservoir inflow, outflow, pool elevation, pool storage, upstream reservoir storage, allowable storage, and basin average precipitation. The terminal proved ideal for this application; however, the detail of the plot was limited somewhat by the size of the 4012 terminal screen. Copies of the plot are made with the hardcopy device on temperature sensitive paper. The quality of the copies is comparable to that of an office photo copy machine.

Conclusions. - The installation of an automatic data collection system is not an end in itself. Consideration must be given to the management of the data once it has been collected. Hand methods of data handling are not always directly transferable to the computer. Therefore, alternate computation procedures and data storage priorities are worth investigating for the sake of efficiency and simplicity.

Experience in the Sacramento District shows that the interactive graphical terminal is currently the best method of presenting numerical data in an understandable form, under the restraint of a "real time" situation. The human being has a difficult time digesting voluminous lists of numbers. The graphical plots presented via the interactive terminal allow him to quickly see trends, data relationships, and obvious data errors. The quick assimilation of data has always been important to those responsible for flood control operation of reservoirs.

References

1. CSTS Math-Pack Programs, Volume 1: Programmers Reference E00164-02, Computer Sciences Corporation, Los Angeles, California, 1971.

2. Plot-10 Terminal Control System, Users Manual (Release #3) Document No. 062-1474-00, Tektronix, Inc., Beaverton, Oregon, 1974.

3. Plot-10 Advanced Graphing II, Users Manual (Release #1) Document No. 062-1530-00, Tektronix, Inc., Beaverton, Oregon, 1973.

Gross Pool
Elev: 473.50 ft
Stor: 160,000 ac-ft

BLACK BUTTE RESERVOIR
DAILY REPORT

9 OCTOBER 1975

POOL EL MDNT (ft)	STORAGE MDNT (ac-ft)	STORAGE CHANGE (ac-ft)	MEAN OUTFLOW (sfd)				MEAN INFLOW (sfd)	STONY CR NR FRUTO (cfs)
			STONY CREEK	SO DIV CANAL	U/S DIV*	TOTAL		
433.24	28247	-165	131	134	0.0	265	185	99

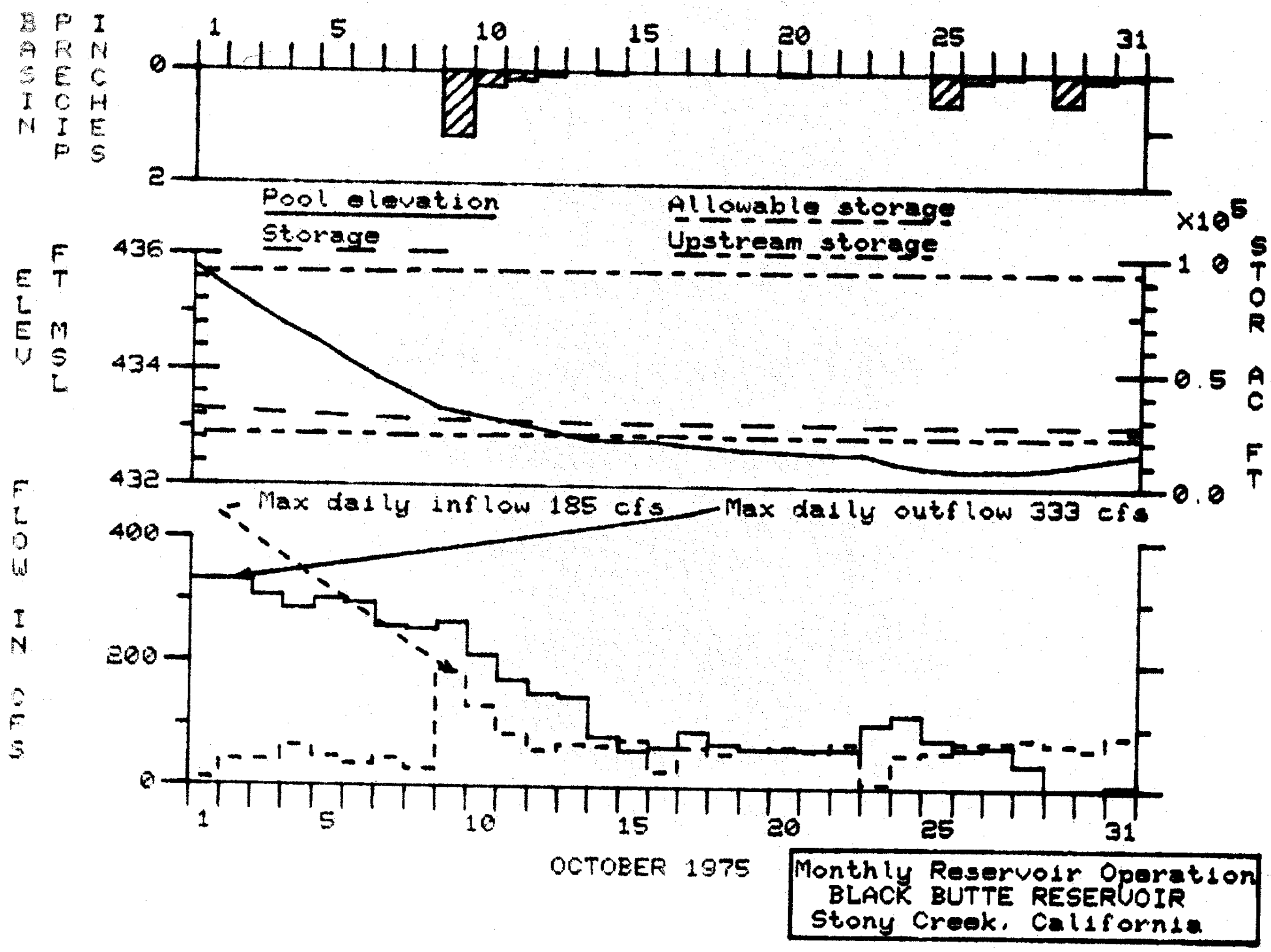
U/S RES (ac-ft)		AIR TEMP @BB DAM (°F)	PRECIPITATION (in)					
			@BB DAM		@STONY GORGE		@EAST PARK	
STONY GORGE	EAST PARK		@OBSN	SEASON	@OBSN	SEASON	@OBSN	SEASON
20870	1181	53	1.53	1.55	1.90	3.07	1.70	2.09

DEMANDS (cfs)			U/S REL (cfs)		RADIO REPORTING PRECIPITATION GAGES (in)							
					LOG SPRG		TROUGH S		NOEL SPR		ALDER SPR	
S/S CANAL	N/S CANAL	D/S GCID	STONY GORGE	EAST PARK	CHG	SUM	CHG	SUM	CHG	SUM	CHG	SUM
120	55	50	55	0	0.4	0.4	2.4	2.4	0.7	0.7	1.7	1.7

ALDER S AIR TEMP (°F)	FLOOD CONTROL COMPUTATIONS							
	BASIN PRECIP		UNADJ REQ'D SPACE	TRANSFER SPACE			ALLOW STOR (ac-ft)	REQUIRED SPACE (ac-ft)
	FOR PERIOD	PRES PARAM		STONY GORGE	EAST PARK	TOTAL		
38.0	1.16	1.16	75000	7500	10000	17500	93400	57500

*Wackerman Ranch diversion above South Diversion Canal measuring weir

Figure 1



BASIN PRECIP INCHES
ELEV FT MSL
FLOW IN CFS
STOR AC FT
Pool elevation
Storage
Allowable storage
Upstream storage
X10^5
Max daily inflow 185 cfs
Max daily outflow 333 cfs
OCTOBER 1975
Monthly Reservoir Operation
BLACK BUTTE RESERVOIR
Stony Creek, California

Figure 2

Use of An Empirical Model in Real-Time Flow Forecasting Saylorville Reservoir 1/

by George E. Johnson 2/
and S. K. Nanda 3/

BACKGROUND

Many computer models have been developed for watershed rainfall-runoff simulation over the recent years. Most of the models use theoretical loss rates to determine the surface runoff and basin response to storms. Many other models are very generalized and attempt to replicate a very large number of basin or reservoir parameters such as water quality, hydropower, temperature regions, etc.

By their nature, some of these models require modification and calibration before application in other regions or even in another watershed. Similarly, some other models are too complex and attempt to perform functions which the user may not desire. The authors of this paper will describe an attempt to develop a model based upon the National Weather Service's rainfall-runoff coaxial relation and basin characteristics of the Des Moines River in Iowa. The model was tested using the record flood of June 1954 as a calibration standard and it was proved successful in actual real-time forecasting situations during the critical period of river closure and final construction activities during the summer and fall of 1975.

1/ Presented at a seminar on Real-Time Operation of Water Resource Projects, The Hydrologic Engineering Center, Davis, California, 17-19 November 1975.

2/ Chief, Water Control Section, US Army Engineer District, Rock Island, Illinois.

3/ Hydraulic Engineer, US Army Engineer District, Rock Island, Illinois.

DESCRIPTION OF THE NEED

Saylorville Reservoir is located on the Des Moines River about 10 miles upstream of the capitol city of Des Moines, Iowa. The upstream drainage basin comprises 5,840 square miles in area. Construction of the reservoir was nearing completion in the summer of 1975. The plan of diversion and closure consisted of diverting the river flow through the single 23-foot diameter circular concrete conduit while low earth cofferdams were to keep flow out of the final-closure construction area located along the left end of the earthfill dam. Consideration of flood potential and the contractor's construction capabilities dictated that the closure not be attempted earlier than about 1 July nor later than 31 July. Until the height and cross section of the last segment of the earthfill could be placed high enough and wide enough to prevent overtopping or failure of the dam cross section, the construction was considered a high-risk venture with possible danger to downstream residents as well as potential property loss hazards.

The Water Control Section of the Rock Island District was charged with the responsibility of providing the Construction Office at the project site with the best possible forecast on streamflow, height of ponding behind the dam and lead time available in which to take positive action to prevent a disaster.

DEVELOPMENT OF THE MODEL

The requirements to be met by the model were:

a. To be able to solve the complex rainfall-runoff relation with a relatively high degree of accuracy for any storm which might occur.

b. To apply the runoff quantities derived in a above to subbasin unit hydrographs in order to develop reasonably accurate flood hydrographs for each subbasin.

c. To rout the various subbasin flood hydrographs downstream through the river channels and combine them at various intermediate points and at the reservoir site to obtain a reservoir inflow hydrograph.

d. To rout the inflow hydrograph through reservoir storage using either uncontrolled or controlled discharges from the outlet in order to determine reservoir elevations.

THE RAINFALL-RUNOFF RELATION

The basin map for the Des Moines River Basin is shown on plate 1. The basin was broken up into 6 subareas, each of which had a USGS stream gaging station located at its downstream limit. Within, or adjacent to, the basin 12 rainfall stations were identified which were considered representative of the basin rainfall. These stations were then weighted using the Thiessen Polygon method to determine weighted rainfall average for each subarea. Plate 2 shows a map which illustrates the division of the total basin into areas of influence of each rainfall gage as defined by the Thiessen Polygons.

The National Weather Service's River Forecast Center in Kansas City furnished the Rock Island District with the NWS Coaxial rainfall-runoff charts which were developed using data from about 1,500 storms. The first portion of the chart (plate 3) utilizes an Antecedent Precipitation Index (API) determined by adding the day's rainfall to 0.9 times the previous day's API. If no rainfall occurred on the current day, the API is simply 0.9 of the previous day's value. The API is entered on the ordinate of the graph and then a point at the intersection of a horizontal line from the API to the appropriate curve for the week of the year is found. A vertical line from that point will intersect the abcissa of the graph at the appropriate Antecedent Index (AI).

The AI value is then utilized on plate 4 to determine the runoff corresponding to the appropriate AI value and the rainfall occurring within the specified period of time (12 hours in this case). The runoff thus derived can be applied directly to the unitgraphs for each subarea, obtaining 12-hour flood hydrographs at each gaging station.

ROUTING AND COMBINING FLOOD HYDROGRAPHS

Once the individual subarea flood hydrographs have been computed they are routed downstream to the reservoir site. The routed hydrographs are combined at several intermediate points along the main stem Des Moines River in order to provide forecasts at various stream gaging stations. The accuracy of the inflow forecast can be checked each day as the flood crest moves toward the reservoir by comparing predicted station discharges with the actual values. A large variance between computed and actual values at upstream stations would provide an indication that the inflow forecast for the reservoir might be in error and adjustments in the forecast are in order. The streamflow channel-routing procedure used in this model is the Tatum routing method. It is a coefficient-type routing procedure which is similar to the Straddle-Stagger routing method.

The Tatum method has consistently yielded good results in the river basins within the Rock Island District and has been used extensively by the District. It was developed by Fred Tatum while he was employed by the Rock Island District.

RESERVOIR ROUTING PROCEDURE

The final point to which the subarea basin flood hydrographs are routed and combined is at the reservoir site. At that location, the routed and combined surface runoff hydrograph is added to a base flow to provide the reservoir inflow hydrograph. The base flow is entered as a single value for the first day of the runoff period and allowed to decay exponentially as a means of estimating the total flood hydrograph of reservoir inflow. A level-pool routing procedure is then employed to determine desired routing outputs. The starting reservoir elevation is entered along with the starting date. The output consists of the date, the period inflow and outflow and the reservoir elevation. Since the model was developed for use when the stream was being diverted through the outlet, the outflows are dependent on reservoir elevation and are assumed to be not regulated by the control gates. The program can be easily modified to use selected outflows rather than uncontrolled outflows for future use under post-construction operation.

MODEL CALIBRATION

The model was calibrated using the record June 1954 flood on the Des Moines River Basin. Lag times were adjusted using the actual observed 1954 discharges at the various gaging stations upstream of the Saylorville project. The hydrograph of daily flows for the June 1954 flood is shown on plate 5. The computed flood hydrograph for this flood is also shown on this plate for comparison. The instantaneous peak discharge actually occurred on 24 June 1954 and reached 60,200 cubic feet per second on the Second Avenue gage at Des Moines. The present gaging station at Saylorville replaces the Second Avenue gage. The computed peak discharge at the Saylorville gage is 59,800 c.f.s. occurring on 24 June.

MODEL USAGE

The model was placed into usage in the spring of 1975 utilizing the Computer Science Corporation's UNIVAC 1108 computer located in Los Angeles. The computer is accessed by remote terminal in the

Water Control Section office in Rock Island. Shortly after the model had been calibrated, a spring flood rise occurred which provided an opportunity for real-time usage testing. This flood was forecasted to crest at the Saylorville gage on 2 May 1975 at a discharge of 19,800 c.f.s. The actual observed flood crest occurred on 2 May and reached a discharge of about 19,400 c.f.s. The flood hydrographs for the predicted and observed values for this flood are shown on plate 6.

Some problems were encountered on the first attempts to use the model. The most frequently occurring and probably the most frustrating problem encountered was that of getting rainfall reports from all 12 of the selected stations. Many times only six or eight of the stations would report the observed rainfall in a timely manner, if they reported it at all. The cooperation of the Des Moines office of the National Weather Service was instrumental in achieving improved reporting.

At the present time a hydrologic telemetering network has been installed and is operating to report river stages by radio to the Saylorville Construction Office. Stations included in the network are:

a. The Des Moines River at Fort Dodge, Iowa.

b. The Boone River at Webster City, Iowa.

c. The Des Moines River at Stratford, Iowa.

d. The Raccoon River at Van Meter, Iowa.

e. Beaver Creek near Grimes, Iowa.

f. The Des Moines River at Saylorville, Iowa, (this gage is also the tailwater gage at Saylorville Dam).

g. The Saylorville Dam pool gage.

h. The Big Creek pump plant ponding stage.

Leupald-Stevens Company in Beaverton, Oregon, is presently modifying four of their 7000 series data recording and transmitting devices to enable rainfall amounts to be monitored by the telemetering network. These devices will be installed at the stream gaging stations located at Fort Dodge, Webster City, Stratford, and Van Meter.

SAMPLE COMPUTER OUTPUT DATA

Plate 7 shows a printout of station rainfall by 12-hour periods at each of the 12 rainfall stations. The bottom portion of the same plate also shows computed average runoff for each of the six subarea basins also by 12-hour periods. The computed flood hydrogrpahs for each of the six subareas tabulated by 12-hour intervals is shown on plate 8. These flood hydrographs are for surface runoff only and do not include any base flow.

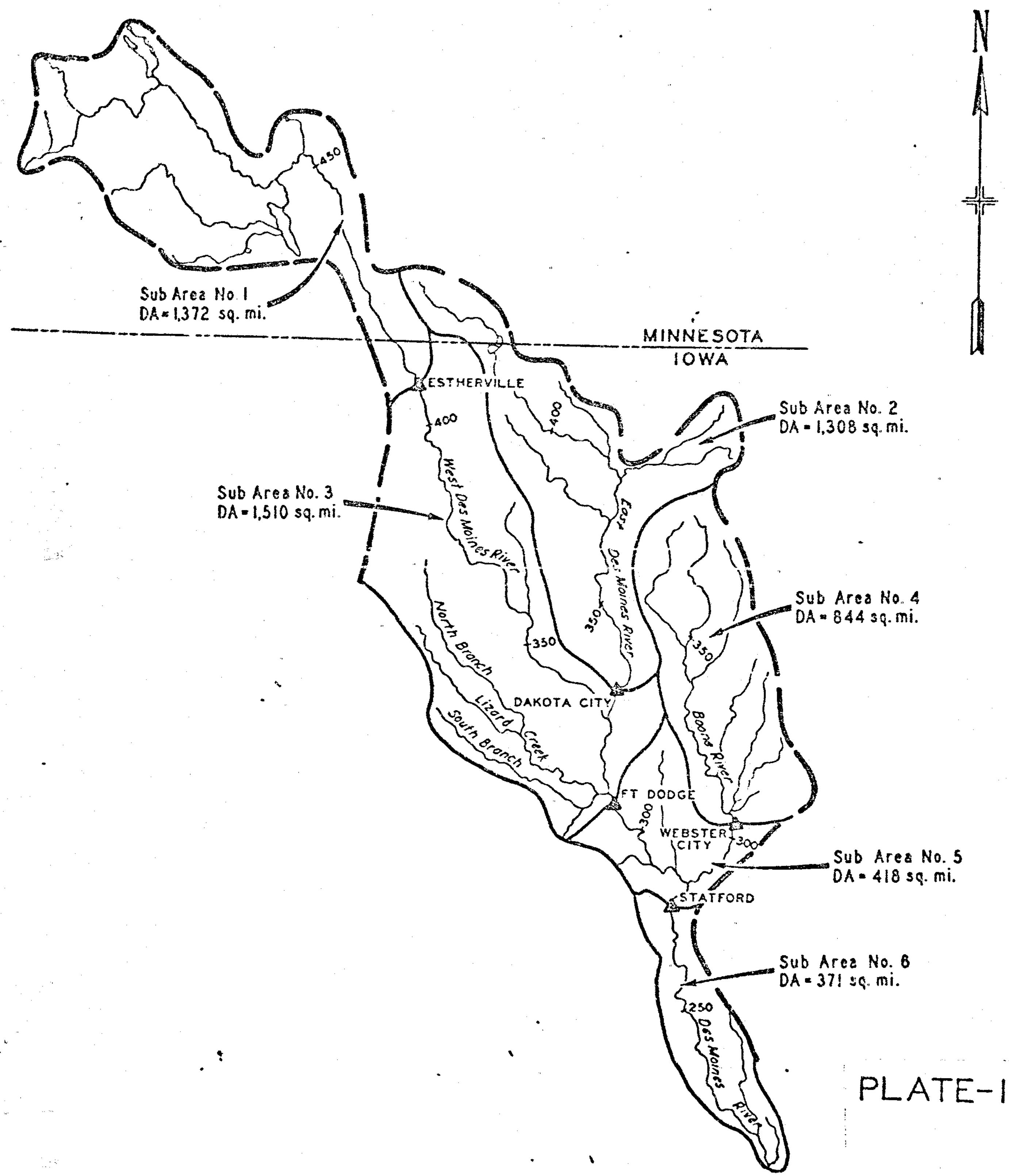
Plates 9, 10, and 11 shows the computed flood hydrographs tabulated by 12-hour interval at the various gaging stations, the same hydrograph as routed to a downstream station and finally a combined hydrograph at each downstream station all the way down to Saylorville. Plate 11 shows the selected first day base flow and the inflow hydrograph to the reservoir. Plate 12 shows the printout from the reservoir routing rout-tine. Shown are the date, inflow, outflow, and elevation of the pool.

The Rock Island District computer terminal is a cathode ray tube (CRT) type terminal. The model uses a conversational mode for inputting data.

CONCLUSION

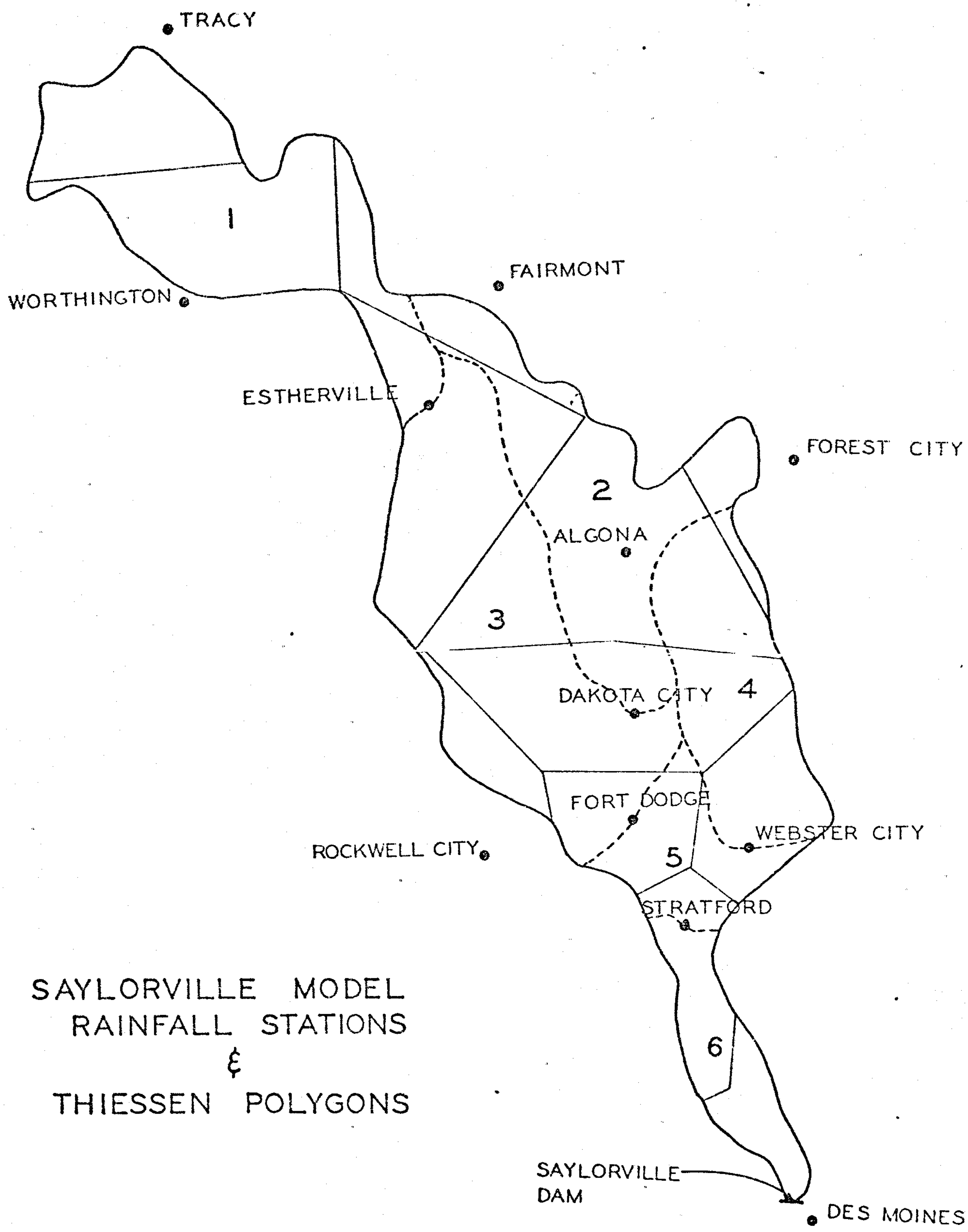
This model has already served the purpose for which it was developed quite well. For single reservoir operation in the future, the model can be modified to accept and use selected outflows as previously mentioned.

The Rock Island District is presently utilizing knowledge and experience gained in working with this model to develop a more sophisticated tandem reservoir model for Saylorville Reservoir and its downstream counterpart, Red Rock Reservoir. This model will feature the ability to use seasonally-varying maximum discharge rates at control points and to utilize seasonally-varying conservation pool levels.



N
Sub Area No. 1
DA = 1,372 sq. mi.
450
MINNESOTA
IOWA
ESTHERVILLE
400
400
Sub Area No. 2
DA = 1,308 sq. mi.
Sub Area No. 3
DA = 1,510 sq. mi.
West Des Moines River
East Des Moines River
Sub Area No. 4
DA = 844 sq. mi.
350
350
350
North Branch
Lizard Creek
South Branch
DAKOTA CITY
Boone River
FT DODGE
300
WEBSTER CITY
300
Sub Area No. 5
DA = 418 sq. mi.
STATFORD
Sub Area No. 6
DA = 371 sq. mi.
250
Des Moines River

PLATE-I



SAYLORVILLE MODEL
RAINFALL STATIONS
&
THIESSEN POLYGONS

PLATE-2

Paper 8

CONVERSION FROM API TO AI

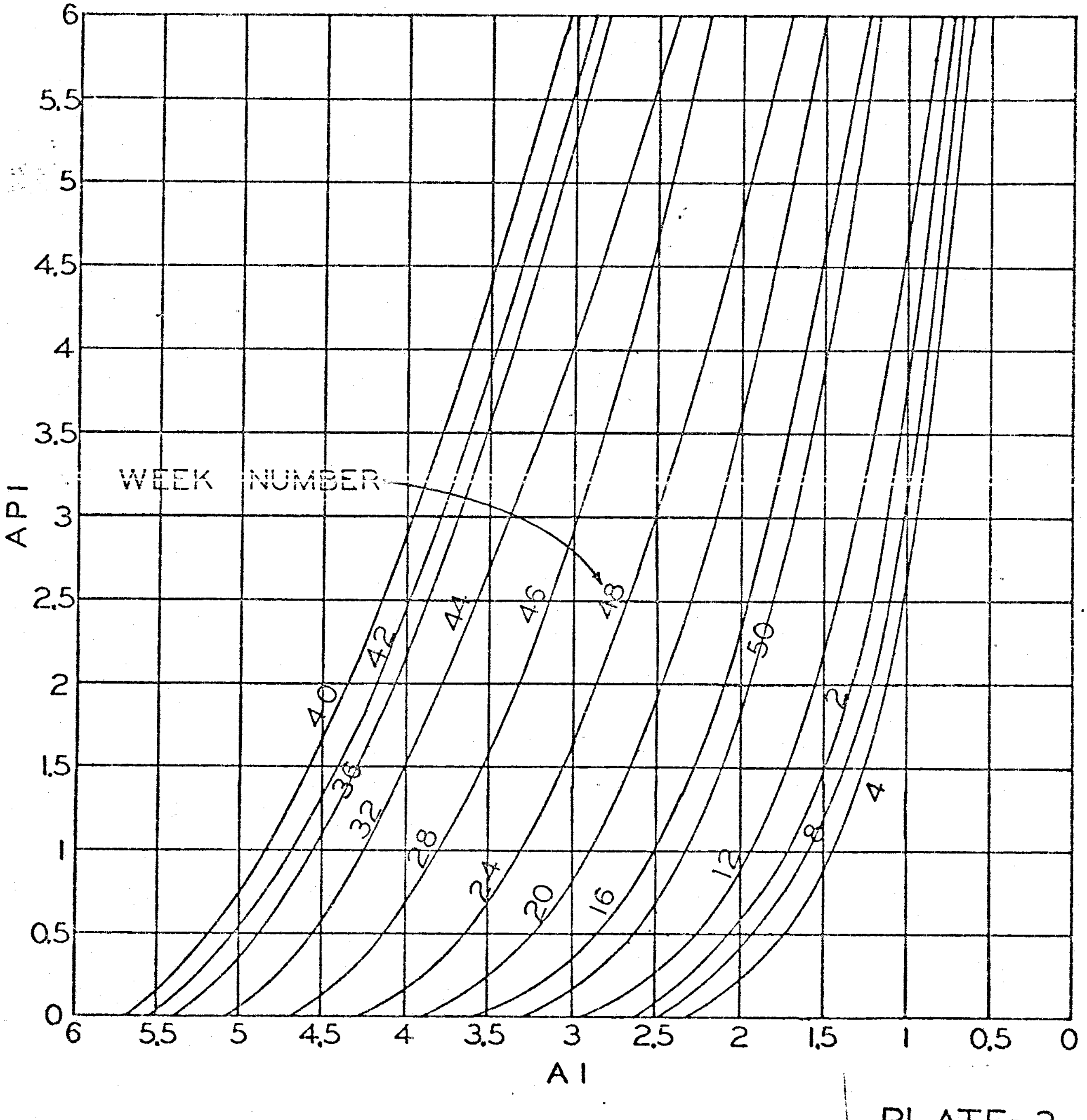


PLATE-3

CONVERSION FROM RAINFALL TO RUNOFF

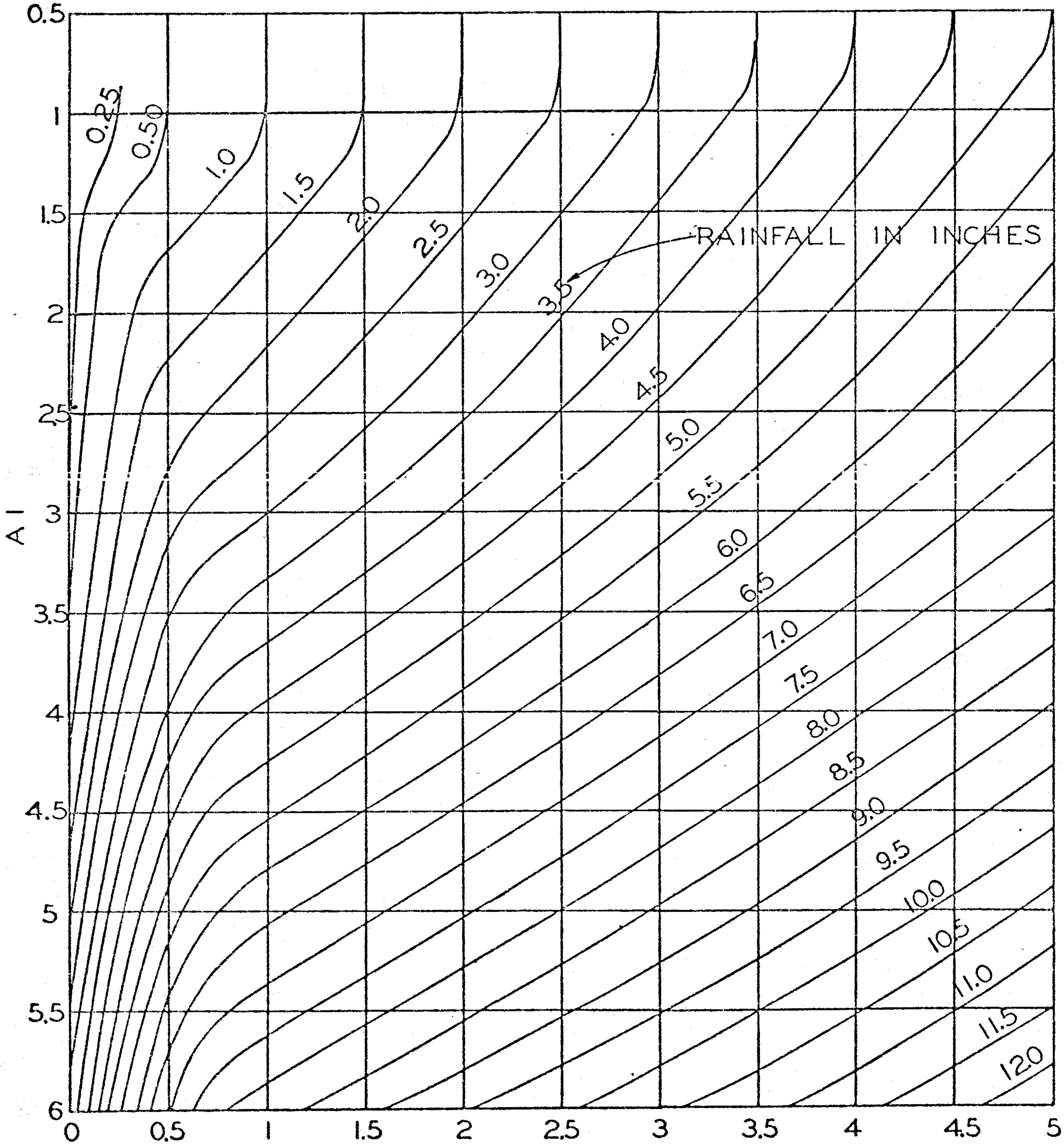


PLATE-4

MODEL CALIBRATION
1954 FLOOD

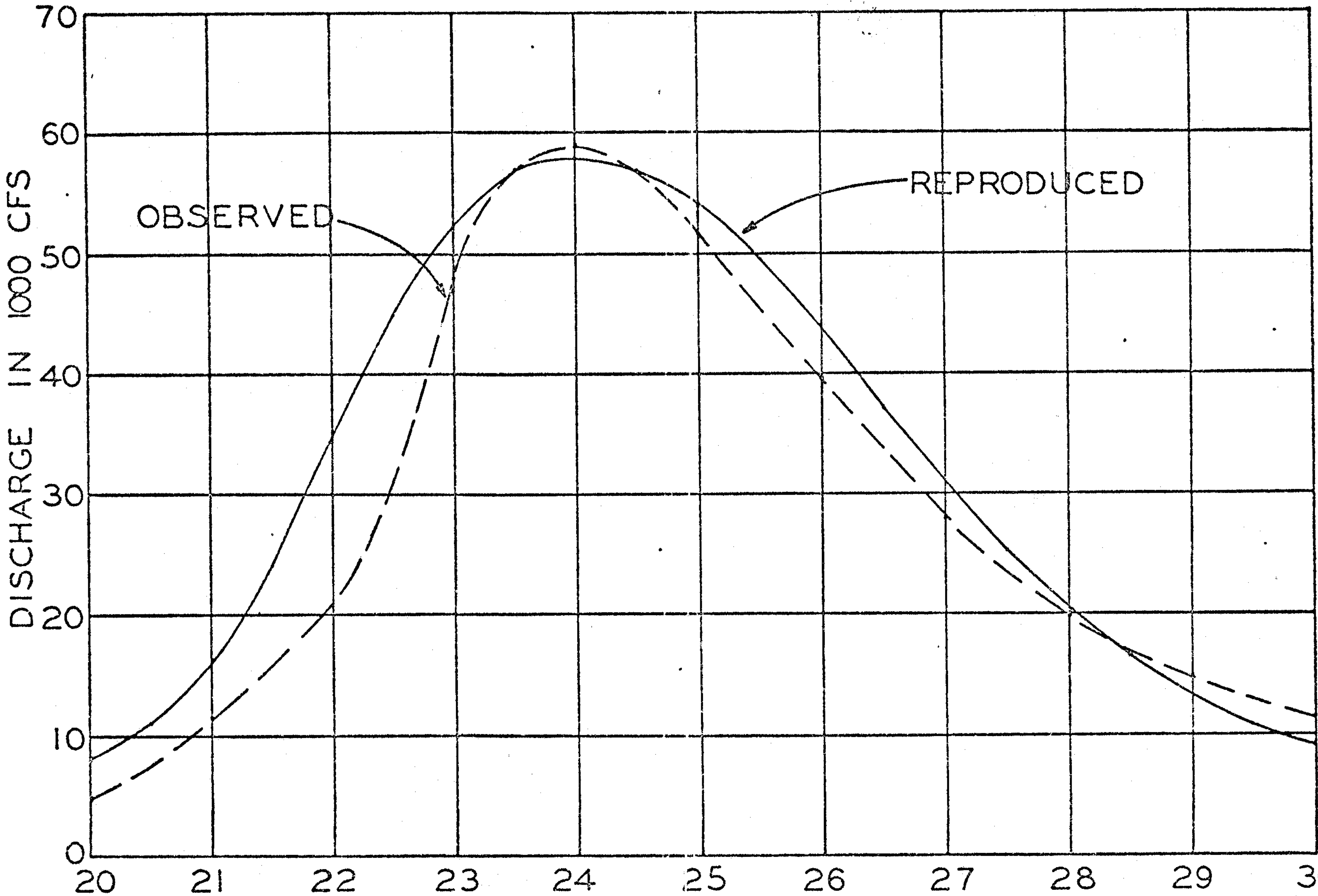


PLATE-5

MODEL FORECASTING

MAY 2, 1975

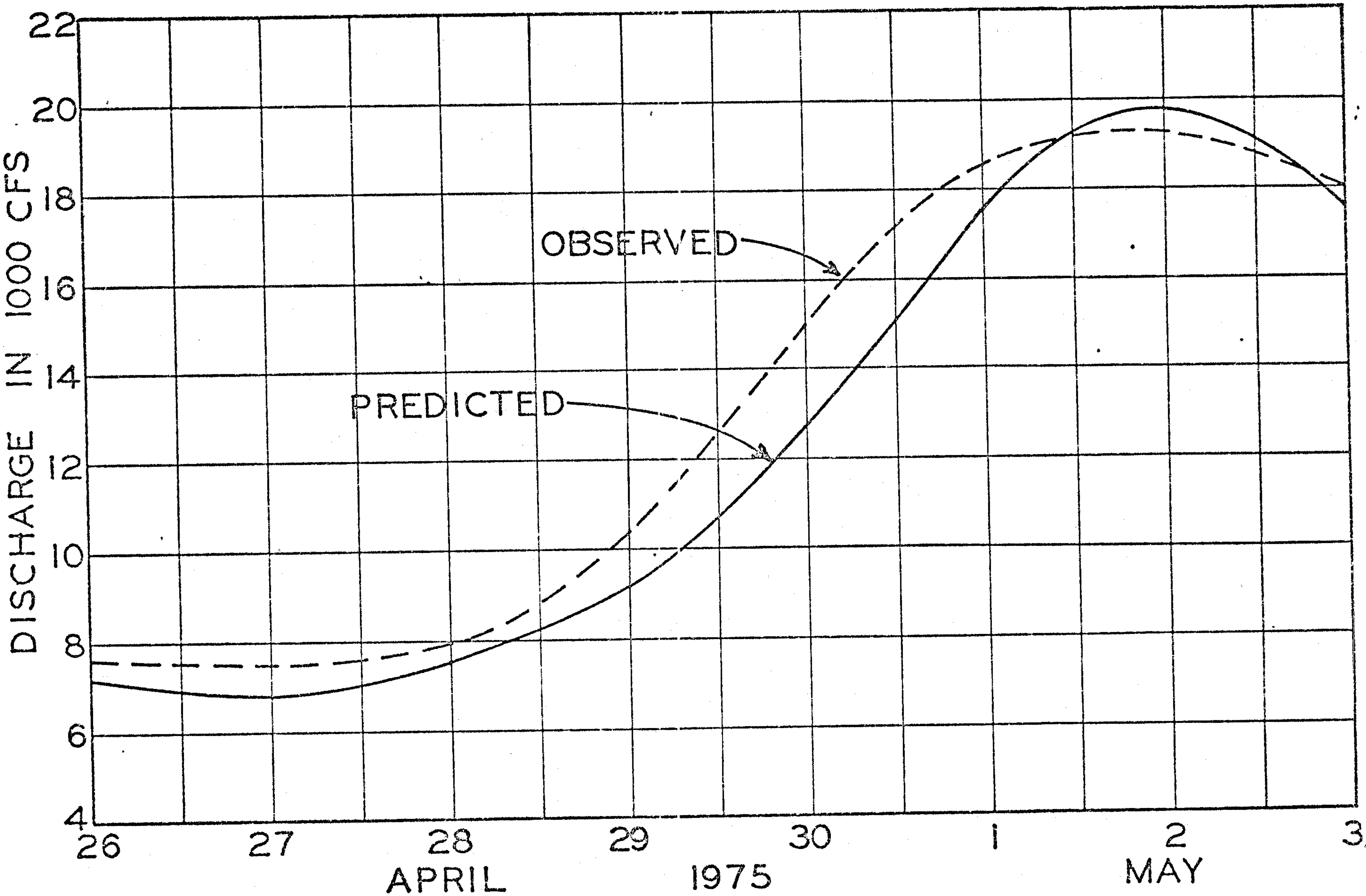


PLATE-6

```
DO MODEL1
INPUT TITLE CARD
TEST RUN
INPUT NUMBER OF RF VALUES & WEEK NUMBER8,24
```

TEST RUN

RAINFALL AS OBSERVED BY 12 STATIONS

STATION 1	.86	0.	.49	.03	.03	.59	.02	.42	0.
STATION 2	.76	0.	1.93	.85	.24	1.11	.01	.58	0.
STATION 3	.76	0.	2.54	1.70	.15	.15	0.	1.50	0.
STATION 4	.55	0.	3.90	0.	1.30	0.	0.	1.23	0.
STATION 5	1.77	2.51	2.12	.18	.72	.08	0.	.83	.10
STATION 6	.93	6.16	.72	0.	.88	0.	0.	1.29	0.
STATION 7	1.56	.72	.09	.91	1.39	0.	0.	1.03	.06
STATION 8	2.15	0.	.64	0.	.07	0.	0.	3.18	.41
STATION 9	1.61	0.	0.	.90	.04	0.	0.	5.02	0.
STATION 10	2.70	0.	0.	0.	0.	0.	0.	2.60	.08
STATION 11	2.15	0.	0.	0.	0.	0.	.29	2.09	.11
STATION 12	2.71	0.	0.	0.	0.	0.	.06	.16	.20

COMPUTED AVERAGE RUNOFF BY BASINS AND PERIODS

TIME	BASIN1	BASIN2	BASIN3	BASIN4	BASIN5	BASIN6
1	0.	2.52	.19	.99	0.	0.
2	.32	.69	.47	.05	.05	0.
3	.09	.04	.04	.03	0.	0.
4	.03	.27	.35	.17	.01	0.
5	.12	0.	0.	0.	0.	0.
6	0.	0.	0.	0.	0.	.03
7	.11	.51	1.12	.85	2.60	.26
8	0.	0.	0.	0.	.02	.03

STOP

Paper 8

PLATE-7

FLOOD HYDROGRAPHS FOR SIX BASINS AT 12 HOUR INTERVAL

TIME	BASIN1	BASIN2	BASIN3	BASIN4	BASIN5	BASIN6
1	0.	2520.	247.	396.	0.	0.
2	157.	9006.	1390.	1604.	40.	0.
3	521.	18949.	3271.	4052.	186.	0.
4	978.	27888.	5525.	7543.	234.	0.
5	1575.	31424.	7932.	10459.	211.	0.
6	2202.	30437.	9632.	8814.	174.	54.
7	2655.	26482.	11429.	7299.	2211.	598.
8	2991.	22710.	13779.	6575.	9784.	1290.
9	3160.	20271.	15578.	7141.	11897.	1138.
10	3177.	17948.	16374.	8739.	9165.	860.
11	3140.	15296.	16721.	9967.	6816.	633.
12	2974.	12502.	15436.	7502.	5072.	468.
13	2746.	9888.	12278.	5095.	3726.	343.
14	2510.	7623.	9705.	3121.	2722.	255.
15	2257.	6104.	7621.	2192.	2035.	187.
16	2033.	4824.	5969.	1491.	1506.	139.
17	1845.	3791.	4707.	983.	1110.	103.
18	1668.	2959.	3723.	672.	819.	74.
19	1490.	2340.	2911.	452.	608.	55.
20	1335.	1061.	2238.	257.	475.	42.
21	727.	655.	1569.	181.	316.	32.
22	522.	500.	1225.	126.	263.	23.
23	427.	316.	840.	76.	184.	16.
24	202.	245.	661.	51.	131.	10.
25	180.	194.	515.	34.	79.	3.
26	0.	0.	0.	0.	1.	0.

STOP

PLATE – 8

ESTHERVILLE FLOWS

0.	157.	521.	978.	1575.	2202.	2655.	2991.	3160.
3177.	3140.	2974.	2746.	2510.	2257.	2033.	1845.	1668.
1490.	1335.	727.	522.	427.	202.	180.	0.	

ESTHERVILLE FLOWS ROUTED TO FTDODGE 8STEPS

0.	1.	7.	37.	128.	322.	642.	1079.	1584.
2090.	2524.	2841.	3026.	3086.	3041.	2912.	2723.	2503.
2276.	2059.	1858.	1659.	1436.	1169.	881.	622.	422.
273.	159.	77.	28.	6.	1.	0.		

DAKOTACITY FLOWS

2520.	9006.	18949.	27888.	31424.	30437.	26482.	22710.	20271.
17948.	15296.	12502.	9888.	7623.	6104.	4824.	3791.	2959.
2340.	1061.	655.	500.	316.	245.	194.	0.	

DAKOTACITY FLOWS ROUTED TO FTDODGE 4STEPS

2520.	2925.	5168.	10645.	18451.	25516.	29205.	29053.	26448.
23229.	20377.	17823.	15233.	12582.	10077.	7940.	6256.	4944.
3895.	3010.	2160.	1363.	802.	512.	358.	251.	154.
64.	12.	0.						

FTDODGE FLOWS

247.	1390.	3271.	5525.	7932.	9632.	11429.	13779.	15578.
16374.	16721.	15436.	12278.	9705.	7621.	5969.	4707.	3723.
2911.	2238.	1569.	1225.	840.	661.	515.	0.	

COMBINED FLOW AT FTDODGE

2767.	4316.	8446.	16207.	26511.	35470.	41276.	43910.	43611.
41692.	39622.	36100.	30537.	25374.	20739.	16820.	13686.	11170.
9082.	7308.	5587.	4247.	3078.	2342.	1754.	873.	576.
337.	171.	77.						

PLATE-9

FTDODGE FLOWS ROUTED TO STRATFORD 6STEPS

2767.	2791.	3001.	3872.	6213.	10792.	17676.	25812.	33421.
38982.	41885.	42394.	41160.	38643.	35018.	30524.	25741.	21255.
17370.	14143.	11500.	9305.	7429.	5805.	4432.	3322.	2432.
1656.	951.	406.	109.	14.				

WEBSTERCITY FLOWS

396.	1604.	4052.	7543.	10459.	8814.	7299.	6575.	7141.
8739.	9967.	7502.	5095.	3121.	2192.	1491.	983.	672.
452.	257.	181.	126.	76.	51.	34.	0.	

WEBSTERCITY FLOWS ROUTED TO STRATFORD 4STEPS

396.	472.	927.	2210.	4485.	7108.	8721.	8627.	7684.
7173.	7586.	8434.	8563.	7320.	5326.	3554.	2355.	1590.
1075.	720.	475.	305.	197.	131.	86.	55.	30.
12.	2.	0.						

STRATFORD FLOWS

0.	40.	186.	234.	211.	174.	2211.	9784.	11897.
9165.	6816.	5072.	3726.	2722.	2035.	1506.	1110.	819.
609.	475.	316.	263.	184.	131.	79.	1.	

COMBINED FLOW AT STRATFORD

3163.	3303.	4113.	6316.	10909.	18073.	28608.	44223.	53003.
55320.	56296.	55900.	53448.	48690.	42379.	35584.	29206.	23664.
19052.	15338.	12291.	9872.	7810.	6067.	4597.	3378.	2462.
1668.	954.	406.						

PLATE-10

STRATFORD FLOWS ROUTED TO SAYLORVILLE 4STEPS

3163.	3172.	3257.	3650.	4827.	7460.	12233.	19815.	30296.
41573.	50068.	54325.	55565.	54897.	52357.	47933.	42127.	35762.
29604.	24121.	19489.	15684.	12591.	10065.	7969.	6205.	4573.
2795.	1132.	211.						

SAYLORVILLE FLOWS

0.	0.	0.	0.	0.	54.	598.	1290.	1138.
860.	633.	468.	343.	255.	137.	139.	103.	74.
55.	42.	32.	23.	16.	10.	3.	0.	

COMBINED FLOW AT SAYLORVILLE

3163.	3172.	3257.	3650.	4827.	7514.	12831.	21105.	31434.
42433.	50701.	54793.	55908.	55152.	52544.	48072.	42230.	35936.
29659.	24163.	19521.	15707.	12607.	10075.	7972.	6205.	4573.
2795.	1132.	211.						

INPUT BASE FLOW AT SAYLORVILLE ON 1ST DAY

6000.

COMBINED FLOW AT SAYLORVILLE WITH BASE FLOW

8671.	8228.	7899.	7911.	8738.	11105.	16128.	24132.	34212.
44983.	53042.	56942.	57881.	56963.	54207.	49599.	43631.	37123.
30839.	25247.	20516.	16620.	13445.	10845.	8679.	6854.	5169.
3342.	1634.	672.						

STOP

PLATE - II

INPUT BEGINNING POOL LEVEL 800.

TIME(DAYS)	INFLOW(CFS)	OUTFLOW(CFS)	ELEVATION
0.	0	0	800.00
.5	8671	3938	810.70
1.0	8228	4988	813.79
1.5	7899	5566	815.49
2.0	7911	5864	816.37
2.5	8738	6177	817.29
3.0	11105	6654	818.69
3.5	16127	7509	821.08
4.0	24131	9000	825.00
4.5	34212	10531	829.38
5.0	44983	12149	834.51
5.5	53041	13761	840.29
6.0	56941	14956	845.94
6.5	57881	16068	851.15
7.0	56962	16949	855.83
7.5	54206	17705	860.04
8.0	49598	18177	863.41
8.5	43630	18572	866.03
9.0	37122	18879	868.00
9.5	30839	19091	869.32
10.0	25247	19214	870.10
10.5	20515	19256	870.41
11.0	16620	19248	870.35
11.5	13445	19200	870.00
12.0	10844	19101	869.38
12.5	8678	18969	868.56
13.0	6853	18812	867.58
13.5	5168	18632	866.45
14.0	3341	18429	865.19
14.5	1633	18209	863.64
15.0	671	17974	861.96

PLATE – 12

MISSOURI RIVER MAIN STEM RESERVOIR SYSTEM EVAPORATION ESTIMATES

by

Maurice A. Clare*

Evaporation from reservoirs regulated by the Corps of Engineers can be a significant factor in the overall accounting of the water supply available for regulation including processes such as the development of inflows and estimating regulation effects of particular projects. Additionally, as consumptive uses of the available water supply increase, valid estimates of evaporation effects upon this supply increase in importance in order that an equitable distribution and accounting of the water supply can be made. Average annual lake evaporation depths vary from less than two feet in cool and humid portions of the United States to over seven feet in hot arid regions. The Missouri River main stem reservoirs, located in the temperate northern states of Montana, North and South Dakota, have an annual evaporation depth averaging about three feet. Location of the projects is indicated on Figure 1. The surface area of the total reservoir system approximates one million acres, resulting in an average evaporation rate approaching 5,000 cfs. During some months the evaporation rate will exceed 10,000 cfs, and for a short period of time, will occasionally be at twice this rate. It is not unusual for evaporation from the main stem reservoirs to exceed inflows into the reservoirs for an extended period of time. Therefore, the development of reasonable evaporation estimates becomes a significant factor in the regulation of the reservoirs.

This presentation is an attempt to describe the procedures selected for use in developing evaporation estimates for the main stem reservoirs and the rationale for the selection.

Evaporation from a body of water cannot be measured directly; therefore, all estimates must be based on indirect measurements or by empirical processes. These include the following methods:

Water Budget. This method requires the measurement of all inflow to the body of water as well as outflow. The difference between inflow and outflow is considered to be the evaporation. However, extremely accurate measurements of all inflow and outflow components are required for valid results. While satisfactory for the development of pan evaporation depths, the method becomes entirely impracticable for use in reservoir evaporation estimates except under controlled research situations. Perhaps the most notable use of the water budget method was during the studies at Lake Hefner, Oklahoma, performed cooperatively by the USGS and the Weather Bureau in the 1950's.[1]

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Energy Budget. This method measures the change in energy stored in the lake by means of thermal surveys, measures the advection of energy by processes other than evaporation, and assumes the residual energy advection is due to the evaporation process. Measurements of all advected energy is required; therefore, there is considerable expensive instrumentation required and the necessary calculations are voluminous. Although the method is more practicable than the water budget method, it is largely limited to research situations. During the 1960's, in cooperation with the USGS, energy budget studies were made at Garrison Reservoir; however, the results were not satisfactory. With such a large reservoir, both in area and volume, resulting in marked variations of pertinent elements of any particular time, it appeared that the definition of appropriate energy budget parameters was not successful.

Mass Transfer. This is a quasi-empirical method of computing evaporation making use of an equation in the form:

$$E = Ku\,(e_o - e_a)$$

where
- E is evaporation
- K is mass-transfer coefficient
- u is wind velocity
- e_o is saturation vapor pressure as derived from water surface temperature
- e_a is air vapor pressure

The mass-transfer coefficient should be obtained by calibration of the particular lake at which evaporation estimates are derived and one means of calibration is by utilization of energy budget procedures as a calibration source. This was one of the purposes of the U.S.G.S. investigations at Garrison Reservoir. Instrumentation is necessary to obtain humidity, air temperature, water surface temperature and wind velocity. For small lakes the mass-transfer approach appears to be most promising in obtaining reliable evaporation estimates, and those most familiar with evaporation research have recommended its use in the development of continuing evaporation estimates.[(2)] However, in large reservoir projects, such as exist along the main stem of the Missouri River, there is so much variation in the pertinent parameters through the reservoir area that the necessary calibration for use of this method is impracticable. Long term averages were however investigated and it appeared that a value of 0.734 for a daily mass-transfer -- wind velocity coefficient was reasonable for these reservoirs with evaporation and vapor pressure given in inches.

Pan to Lake Evaporation Relationship. One of the findings of evaporation research during the 1950's and 1960's was that the average annual Class A pan to lake evaporation coefficients varied from between 0.60 to above 0.80 over the United States. The annual coefficient applicable to the region of the Missouri River main stem reservoirs is about 0.70, provided that the pan installation is representative of the area.[(3)] However, on a seasonal basis the pan to lake coefficient may vary markedly with this variation apparently resulting from seasonal variations of energy advection to the pan installation and the fact that seasonal variations of lake surface temperatures differ markedly from seasonal variations in the pan water temperature. If the seasonal variations in the pan to lake coefficient were known, a pan installation would provide a most convenient basis for estimating lake evaporation.

Hydro-meteorologic data collected adjacent to the reservoir sites that could be utilized in the development of evaporation estimates included the following:

Class A pan evaporation - generally extending during the months of April through October.
Air temperature.
Relative humidity.
Release temperature.

Effort was therefore directed toward development of procedure that would utilize these data for evaporation estimates thereby precluding additional instrumentation. The development of seasonal pan to lake coefficients appeared to be the most promising method of computation during periods pan data were available. Solution of an empirical mass-transfer equation also appeared to be most practical during the periods pan observations were not available.

The first steps in the investigation were to examine available evaporation pan data and determine an appropriate mass-transfer coefficient that combined normal wind velocity with the coefficient. The process consisted of developing a complete ten-year record of monthly pan evaporation depths for each of the six main stem reservoir projects. Missing pan data were computed by a mass-transfer equation, assuming that pan water temperature would be equal to the air temperature utilizing trial coefficients. Average annual pan evaporation developed for each location was then compared to the Class A pan averages developed for the region as developed by the National Weather Service and shown on Figure 1.[3] After examination of several trial coefficients, developed averages consistent with published averages at four of the six projects were obtained. Through further analysis it was concluded that the differences noted at two projects were due to unrepresentative pan data and that the mass-transfer coefficient that appeared applicable to four of the six reservoirs was valid. The factor for adjusting the unrepresentative pan data was then defined.

It was reasoned that the major factor influencing a seasonal distribution of pan to lake evaporation coefficients was the difference experienced between lake surface temperatures and pan temperatures. Unfortunately, lake surface temperature records for the main stem projects are very meager and a program for collection of such data would constitute a significant effort. However, an estimate of normal seasonal lake surface temperatures, consistent with estimated average annual evaporation, was developed in the following manner:

a. Records of release temperature are available. Utilizing a six-year period (1967-1972) the normal release temperature throughout the year was determined. Experienced release temperatures were usually very near the developed normals with departures seldom exceeding 3 degrees F.

b. Utilizing Weather Service records, normal air temperatures for the reservoir area were developed.

c. It was assumed that there would be very little temperature gradient in the reservoir during the period release temperatures were experiencing a falling trend, and the mean surface temperature would approximate the release temperature. These conditions occurred during the late summer through early winter period.

d. All of the main stem reservoirs freeze over each winter. The normal dates of freeze-up and ice break-up were determined. When the reservoir is frozen and normal air temperatures are below 32 degrees F., a normal surface temperature equal to the air temperature was assumed. With air temperatures above 32 degrees F. with an ice cover, the surface temperature was assumed to be at 32 degrees.

e. Using developed normal monthly pan evaporation, the evaporation pan mass-transfer equation was solved (assuming pan temperature equal to normal air temperature) to develop the normal vapor pressure, e_a, for each month.

f. An assumption was made as to possible lake surface temperatures during the period extending from the time the ice went out in the spring until the time that release temperatures began to fall. Utilizing the resulting annual surface temperature sequence, the pan mass-transfer equation was again solved (assuming pan temperature equal to the selected normal lake surface temperature and developed normal vapor pressure) for evaporation. Resulting annual pan evaporation was compared to previously developed normals. The water surface temperature profile was modified in a reasonable manner as necessary and evaporation recomputed until computed annual pan evaporation equalled normal annual pan evaporation. The resulting normal water surface temperature profile was very consistent with actual surface water temperature observations that are available.

An example of the observed normal air temperature and release temperature profiles, together with the developed normal lake surface temperature profile is given on Figure 2.

Seasonal pan to lake evaporation coefficients were then assumed to be based on the relationship developed between normal monthly pan evaporation and the monthly pan evaporation estimated to occur (based on mass-transfer analysis) with pan temperatures equivalent to lake surface temperature. This required a pan evaporation mass-transfer coefficient reflecting the average 0.7 pan to lake relationship. Figure 3 presents the monthly pan coefficients developed for each of the six main stem reservoirs.

Based on the discussion given above, estimates of evaporation from each of the six Missouri River main stem reservoirs are made as follows:

a. When evaporation pan data are available, the seasonal pan to lake coefficients are utilized.

b. The mass-transfer equation with a daily coefficient of .734 is used when pan evaporation data are not available. The saturation vapor pressure, e_o, is based on the developed normal lake surface temperature while the air vapor pressure, e_a, is based on observed temperature and relative humidity data obtained from around the reservoir area.

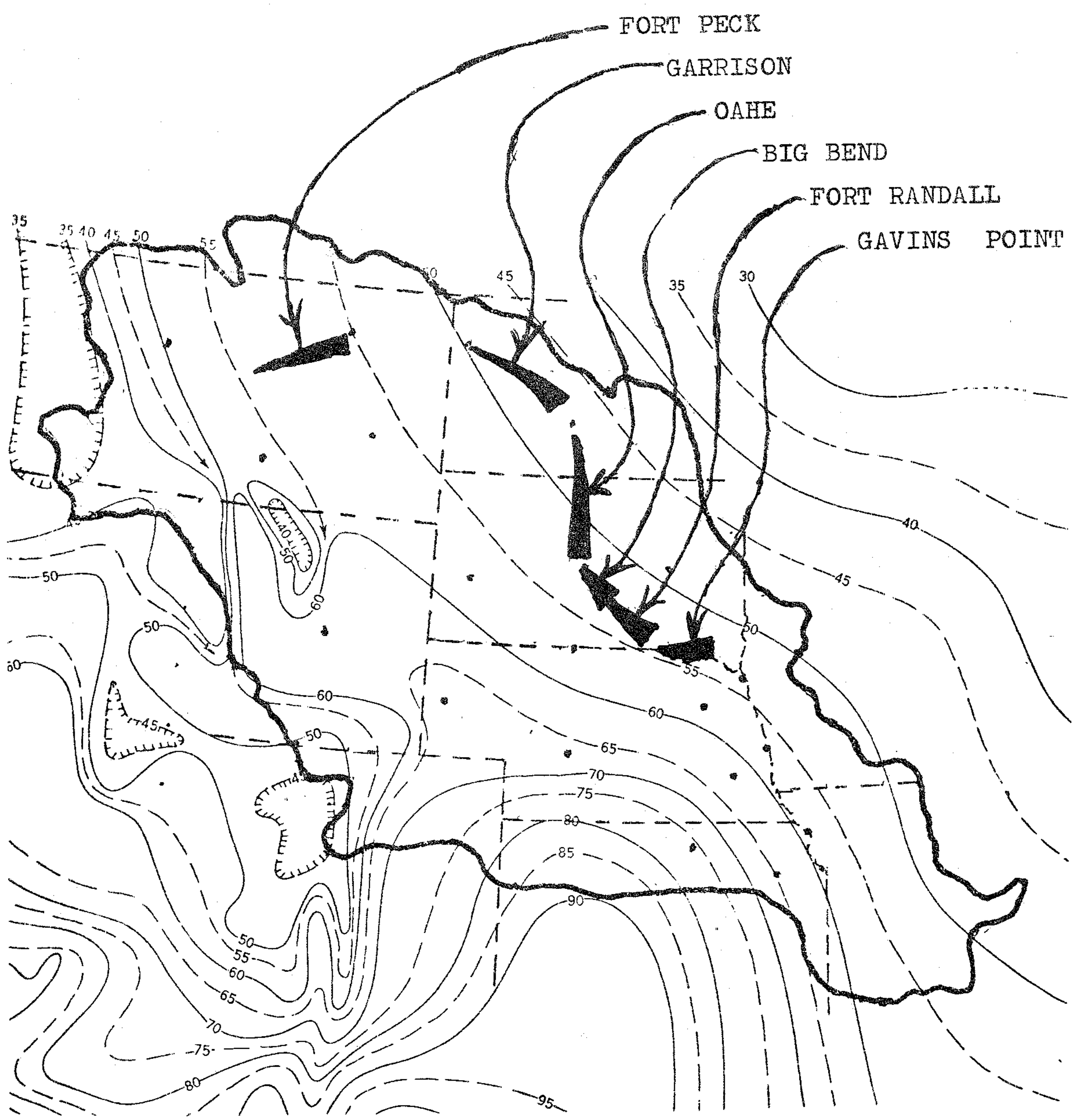
In summary, the method adopted for use in developing the estimates of evaporation from the main stem reservoir is based on the use of variable evaporation pan-to-lake coefficients and a mass-transfer equation. The method appears to be consistent with evaporation research available at this time and requires a minimum of instrumentation. Extremely rough water budget estimates indicate that the evaporation estimates derived by the procedures are quite reasonable -- much more so than by use of a constant pan coefficient through the year. Annual estimates derived by the process are also consistent with the averages published in current literature.

REFERENCES:

1. U. S. Geological Survey, "Water-Loss Investigations -- Lake Hefner Studies", U. S. Geological Survey Prof. Paper 269, 1954.

2. U. S. Geological Survey, "A Practical Field Technique for Measuring Reservoir Evaporation Utilizing Mass-Transfer Theory", U. S. Geological Survey Prof. Paper 272-E, 1962.

3. U. S. Department of Commerce, Weather Bureau, "Evaporation Maps for the United States", Technical Paper No. 37, 1959.

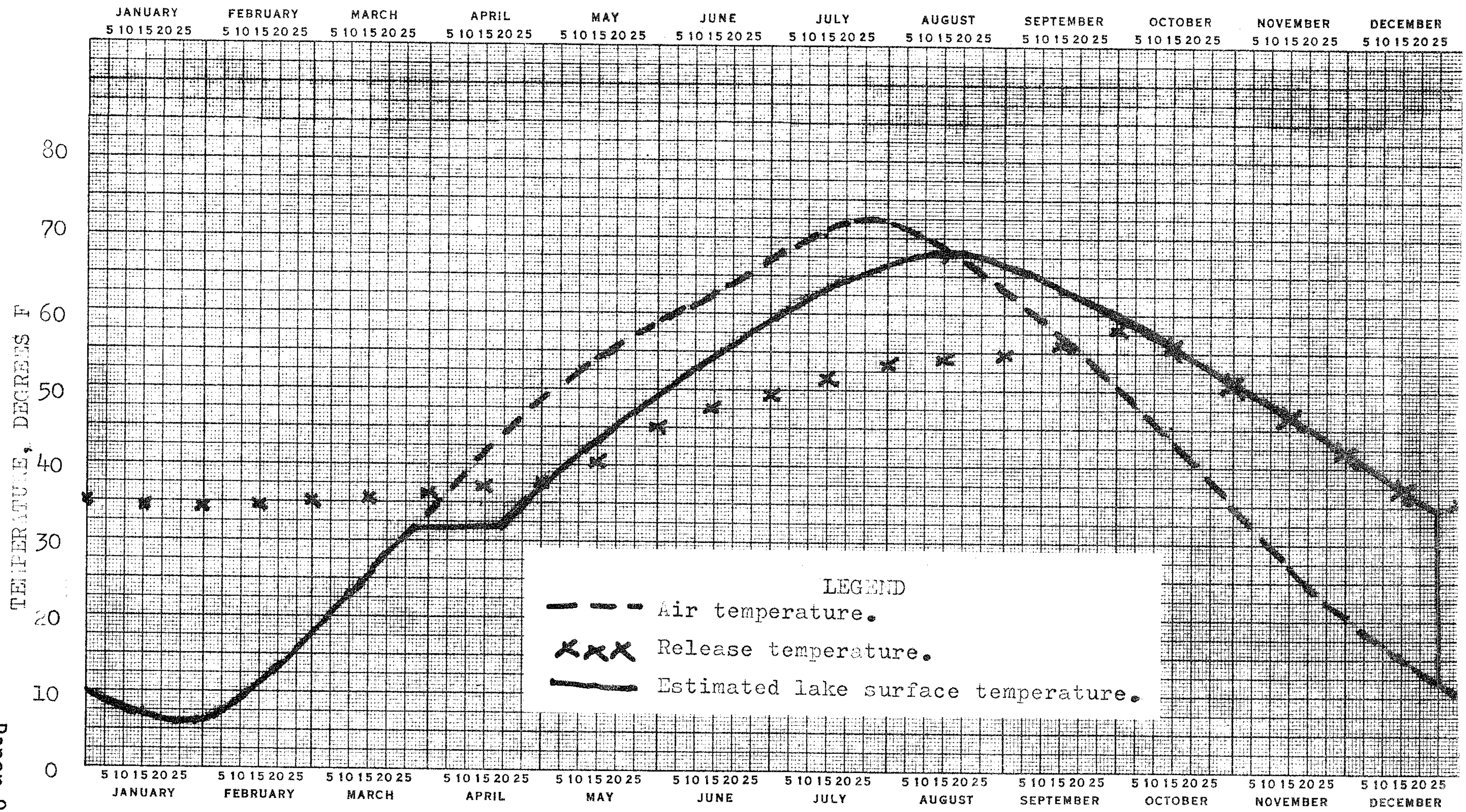


AVERAGE ANNUAL CLASS A PAN EVAPORATION IN INCHES
(Period 1946-1955)

From U.S.W.B. Technical Paper No. 37
Missouri River Basin

Paper 9

Figure 1



GARRISON RESERVOIR, SEASONAL TEMPERATURES

Paper 9
Figure 2

MISSOURI RIVER MAIN STEM RESERVOIR SYSTEM

Normal
Pan to Lake
Evaporation Coefficients

Month	Fort Peck	Garrison	Oahe	Big Bend	Fort Randall	Gavins Point
January	1.28	0.70	0.73	0.63	0.70	0.70
February	0.70	0.70	0.56	0.63	0.70	0.70
March	0.60	0.70	0.49	0.54	0.63	0.62
April	0.11	0.14	0.13	0.47	0.19	0.53
May	0.22	0.20	0.16	0.35	0.32	0.53
June	0.32	0.21	0.18	0.39	0.37	0.53
July	0.39	0.26	0.22	0.53	0.42	0.56
August	0.64	0.64	0.50	0.70	0.78	0.70
September	1.21	1.13	0.89	0.82	1.31	0.93
October	1.32	1.44	1.19	1.05	1.42	0.97
November	2.57	3.74	2.22	1.52	1.62	1.59
December	4.22	5.04	3.42	1.36	1.39	1.57

These coefficients are applicable to the pan installations currently in operation in conjunction with the projects. They make allowances for the fact that the Oahe and Big Bend installations are not considered to be representative installations. If pan evaporation is available, lake evaporation depths are estimated by application of the above coefficients.

Figure 3

MANAGEMENT OF THE SAD HYDROPOWER SYSTEM

By

Albert G. Holler, Jr.[1]

INTRODUCTION

The South Atlantic Division hydropower system involves ten Corps' multiple purpose reservoirs and 24 non-Federal projects in three river basins. Two of the basins are completely within the Mobile District boundaries and the third basin is entirely within the Savannah District boundaries. The Corps' hydropower dams include Carters, Allatoona, Jones Bluff, and Millers Ferry in the Alabama-Coosa River Basin, Buford, West Point, Walter F. George, and Jim Woodruff in the Apalachicola River Basin, and Hartwell and Clark Hill in the Savannah River Basin. The location of these projects is shown on Figure 1. The non-Federal projects are run-of-the-river which have little effect on the overall conservation operations of the system. The sale of power from the Mobile and Savannah Districts hydropower projects is handled under one contract negotiated and administered by the Southeastern Power Administration (SEPA). Weekly power declarations from these projects are coordinated with SEPA by the Division Hydropower Branch and the Mobile District and the Savannah District reservoir regulation units. There are two SAD hydropower dams within the Wilmington District boundaries that are not a part of the system.

In managing the SAD hydropower system, consideration must be given to all project purposes while meeting hydropower contract commitments. The South Atlantic Division is in the process of developing a hydropower systems analysis computer model which will include all project purposes. The program will be used mainly for hydropower studies needed during negotiation periods. However, real time operation during floods and long term reformulation studies will also be included in the program capability.

SYSTEM COMPONENTS

Alabama-Coosa River Basin

1. Carters Dam

Carters Dam is 27 miles above the mouth of the Coosawattee River in northwest Georgia. It is approximately 60 miles north of

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Atlanta and about 50 miles southeast of Chattanooga, Tennessee. A reregulation dam is located about 2 miles downstream from the main dam. Carters is a multiple purpose project with principal purposes of flood control, power, and recreation. The main dam at Carters contains four hydrogenerating units with a total nameplate rating of 500,000 KW and is operated as a peaking power plant. Two of the generators are reversible and are used as motors to pump water from the reregulation pool to the main pool. The reregulation dam is used to reduce fluctuations downstream due to the peaking power operation, to store water for the pumping operation, and to maintain a minimum dependable outflow from the project.

The main dam is rock fill with a maximum height of 445 feet. Other features of the main structure are the powerhouse, a low level sluice for reservoir evacuation, and a gated spillway. The reregulation dam is a 47 foot high rock fill structure with a gated spillway for reregulation of powerwaves.

2. Allatoona Dam

Allatoona Dam is situated on the Etowah River in northwest Georgia, close to Atlanta. It is a multiple purpose project with principal purposes of flood control and power. Other project benefits include regulation of stream flow, public recreation, and fish and wildlife conservation. The increased flow during low flow periods increases power production at the Alabama Power Company plants on the Coosa River and aids navigation on the Alabama River. The project consists of a concrete gravity dam with gated spillway, earth dikes, a 74,000 KW power plant and appurtenances which are operated from the Carters Powerhouse, and a reservoir extending about 28 miles up the Etowah River at full summer level power pool.

3. Jones Bluff Lock and Dam

The Jones Bluff Project is located in the south central part of the State of Alabama on the Alabama River. It is approximately 15 miles east-southeast of Selma and 35 miles west of Montgomery. The project consists of a concrete gravity type dam with gated spillway supplemented by earth dikes, a navigation lock, and a 68,000 KW power plant. The dam forms a reservoir which covers an area of 12,510 acres at normal pool elevation.

4. Millers Ferry

The Millers Ferry Project is located 142 miles above the mouth of the Alabama River in the southwestern part of the State of Alabama.

The project consists of a concrete gravity dam with gated spillway, earth dikes, a navigation lock and control station, and a 75,000 KW power plant. The reservoir extends 105 miles upstream with a normal pool at elevation 80 feet msl.

Apalachicola River Basin

1. Buford Dam (Lake Lanier)

Buford Dam is located on the Chattahoochee River about 35 miles northeast of Atlanta. The project consists of an earth dam supplemented by earth dikes, an unpaved chute spillway, an 86,000 KW power plant and appurtenances, and a reservoir extending about 44 miles up the Chattahoochee. Normally, the project is operated as a peaking plant for the production of hydroelectric power. The primary purpose of the project is flood control. The power plant is also remotely operated from the Carters powerhouse.

2. West Point Dam

West Point Dam is on the Chattahoochee River upstream from Walter F. George Lock and Dam and downstream from Buford Dam. It is a multiple purpose project with major purposes of flood control, hydroelectric power production, recreation, fish and wildlife development and stream flow regulation for downstream navigation. The dam is a concrete gravity type structure with rolled earthfill embankments. In addition, the project includes a powerhouse, a non-overflow section, a gated spillway located in the main river channel and a left embankment retaining wall which supports the earth embankment on the east abutment. The power plant and spillway gates are operated remotely from the control room of the George Powerhouse via a microwave link between the two projects.

3. Walter F. George Lock and Dam

The Walter F. George Lock and Dam is located on the Chattahoochee River bordering the States of Alabama and Georgia. The project consists of a powerhouse (130,000 KW), a gated spillway, a lock (88 ft. lift), and earth dikes extending to high ground on both banks. The reservoir extends 85 miles and provides a 9 foot minimum depth for navigation.

4. Jim Woodruff Lock and Dam (Lake Seminole)

Jim Woodruff Lock and Dam is located about 1,000 feet downstream from the area where the Chattahoochee and Flint Rivers unite to form the Apalachicola River. The project consists of a dam, an 82 x 450 foot lock, a 30,000 KW power plant and appurtenances, and a conventional concrete gravity-type fixed-crest spillway. The reservoir extends up the Chattahoochee River to George W. Andrews Lock and Dam and 47 miles up the Flint River. The power plant at Jim Woodruff supplies energy and capacity to the Florida Power Corporation under terms of a contract negotiated and administered by SEPA. The output of the plant, which operates continuously, varies with changes in the inflow.

Savannah River Basin

1. Hartwell Dam

The Hartwell Dam is located on the Savannah River along the Georgia-South Carolina boundary line. It consists of a concrete dam flanked by earth embankments, a powerhouse (264,000 KW) in the west flood plain immediately below the dam and a switchyard adjacent to the powerhouse. A 66,000 KW unit will be added to the powerhouse to bring the ultimate plant capacity to 330,000 KW. The concrete dam rises about 204 feet above the streambed. The gated spillway is a concrete gravity ogee type located on the concrete portion of the dam. Approximately 21 percent of the drainage area above the Hartwell Dam is controlled by the Duke Power Company's Keowee-Toxaway Project.

2. Clark Hill Dam

This project is also on the Savannah River immediately below Hartwell Dam and 22 miles upstream from Augusta, Georgia. The project consists of a combination concrete and rolled earth dam, a powerhouse (280,000 KW) and a concrete gated spillway. The reservoir area is 71,000 acres at full pool which is 15,150 acres larger than the Hartwell Reservoir area.

SYSTEM COMPONENT OBJECTIVES

Carters Dam. The main dam is operated to provide flood control, power generation and to maintain a dependable downstream flow. Other objectives are fish and wildlife conservation and public recreation.

Allatoona Dam. The primary purpose for the project is flood control. However, the project is normally operated as a peaking plant for the production of hydroelectric power. The project also benefits navigation, water supply, and pollution abatement.

Jones Bluff Lock and Dam. The primary function of the Jones Bluff Project is to provide a navigable channel. The other major function of the project is the generation of hydroelectric power. Design criteria for stability against overturning and sliding make it imperative that the difference between headwater and tailwater elevations not exceed 47 feet at any time.

Millers Ferry Lock and Dam. This multiple purpose project serves as a major unit of the Alabama River navigation system and provides a hydroelectric plant for the generation of power. Other purposes for the project are public recreation and fish and wildlife conservation.

The reservoir is maintained at or near the approved normal pool elevation (80 fmsl) to provide the navigation channel. The pool may be drawn down to elevation 79 fmsl for hydropower production, if necessary. Fluctuations up to elevation 81 fmsl are permitted at times for mosquito control, fish propagation and other worthy purposes. Design criteria for stability against overturning and sliding of the project make it imperative that the difference between headwater and tailwater elevations never exceed 48 feet.

Buford Dam (Lake Lanier). Another multiple purpose project, Buford Dam provides flood control, navigation flows, and hydropower. Because of its proximity to Atlanta and clean water, Lake Lanier has boomed as a recreational attraction. Its annual visitation rate exceeds any other Corps reservoir.

West Point. Normally the West Point Project will be operated as a peaking plant for the production of hydroelectric power. Other project purposes include navigation, water supply, and water quality. However, another major purpose of the project is flood control.

Walter F. George. Navigation and the generation of hydroelectric power are the principal objectives in the operation of the project. Secondary regulatory objectives include recreation, fish propagation, mosquito control, aquatic plant control and drift removal. Flood control is not a project feature. Reservoir elevations are normally maintained at or below the top of the power pool. However, to provide the needed flexibility for peaking power operation, the pool level is fluctuated between elevation 184.5 and 186.5 during the principal flood season, December through April. Because of navigation requirements, the pool is not drawn down below elevation 184. From May through November, the pool is maintained between elevations 184 and 190 fmsl.

Jim Woodruff Lock and Dam (Lake Seminole). This is a multiple purpose project created primarily to aid navigation in the Apalachicola River below the dam and in the Chattahoochee and Flint Rivers above the dam, and to generate electric power. Secondary benefits include public recreation, regulation of stream flow, and fish and wildlife conservation.

Hartwell and Clark Hill Dams. Storage in the reservoirs is allocated to flood control, navigation, and hydropower. Reservoir operation is dependent upon the various combinations of reservoir stages and runoff and requires a flexible method of operations. Coordination with the Keowee-Toxaway Project of the Duke Power Company is required.

SYSTEM CONSTRAINTS

Hydropower Production Requirements

1. Georgia Power Company and Alabama Power Company

These two power companies' generating and transmission systems are interconnected to form an integrated system permitting the flow of power between systems. Georgia Power is connected to the Allatoona, Buford, George, Clark Hill and Hartwell Projects. Alabama Power is connected to the Millers Ferry and George Projects. The existing contract which was negotiated by and is administered by SEPA concerns the sale of energy from these projects. Negotiations are underway to include Carters, Jones Bluff and West Point in the contract. At present, they are being operated under short term amendments to the existing contract. The following capacity and energy amounts from the contract are made available to Alabama Power and Georgia Power:

- Up to 750,000 kilowatts of monthly dependable capacity
- The total energy at Allatoona and Buford
- The total energy at Millers Ferry
- Approximately 85% of the total energy at George
- Approximately 1/2 of the total energy at the Clark Hill Project
- Approximately 1/2 of the total energy at Hartwell
- Up to 395,000 kilowatts of monthly dependable capacity in excess of the amount shown above.

Monthly capacities vary with the season and are in accordance with the following schedule:

Monthly Capacity Availability (Kilowatts)

Month	Georgia Power	Alabama Power	Total
January	561,000	119,000	680,000
February	561,000	119,000	680,000
March	561,000	119,000	680,000
April	561,000	119,000	680,000
May	596,000	124,000	720,000
June	622,000	128,000	750,000
July	622,000	128,000	750,000
August	622,000	128,000	750,000
September	596,000	124,000	720,000
October	561,000	119,000	680,000
November	561,000	119,000	680,000
December	561,000	119,000	680,000

The figures shown above represent an overall commitment to be met from all projects, provided, however that such capacity and energy are divided among the respective projects in amounts acceptable to the two power companies.

Each week, minimum declarations of energy are made available to Georgia Power and Alabama Power from the projects. Monthly energy amounts are shown below:

TOTAL (MWH)

Month	Georgia Power	Alabama Power	Total
January	14,040	3,960	18,000
February	11,778	3,322	15,100
March	11,778	3,322	15,100
April	11,778	3,322	15,100
May	13,572	3,828	17,400
June	15,990	4,510	20,500
July	17,316	4,884	22,200
August	20,670	5,830	26,500
September	16,458	4,642	21,100
October	14,430	4,070	18,500
November	14,040	3,960	18,000
December	14,040	3,960	18,000

2. South Carolina Public Service Corporation

This corporation supplies electric power to Central Electric Power, Incorporated, a generation and transmission cooperative composed of 15 rural electric cooperatives. One hundred and five thousand KW of dependable capacity are made available to the corporation each month from Clark Hill Reservoir. Weekly minimum amounts of energy made available to the corporation are as follows:

Month	MWH	Month	MWH
January	4,600	July	5,600
February	4,400	August	5,600
March	4,100	September	5,000
April	4,400	October	4,600
May	4,700	November	4,500
June	5,100	December	4,700

3. Duke Power Company

This company is served from the Hartwell and Clark Hill power plant. Service is made available at Hartwell in the amount of 124,000 KW of dependable capacity and 1/2 of the total energy. Clark Hill provides the power company with 21,000 KW of dependable capacity and approximately 7½ percent of the total energy. Minimum energy to be made available by months at the two projects is as follows:

Month	Hartwell MWH	Clark Hill MWH	Total MWH
January	3,300	560	3,860
February	3,000	510	3,510
March	3,000	510	3,510
April	3,000	510	3,510
May	3,000	510	3,510
June	3,000	510	3,510
July	2,500	420	2,920
August	3,900	660	4,560
September	3,900	660	4,560
October	3,900	660	4,560
November	3,600	610	4,210
December	3,900	660	4,560

4. Alabama Electric Cooperative, Inc.

This cooperative has a contract to obtain capacity and energy from the Walter F. George Project. Under the terms of the contract, 22,000 kilowatts of dependable capacity are made available to the cooperative except under extremely high water conditions. In addition, 50,000 KW are available to meet necessary scheduled and emergency outages. The minimum weekly declaration of energy available to the cooperative from George is in accordance with the following table.

Month	KWH/KW	Month	KWH/KW
January	34	July	34
February	34	August	34
March	17	September	34
April	17	October	17
May	17	November	17
June	34	December	34

Flood Control

Operations at the Carters Project reduce peak flood heights on the Coosawattee River and the lower Oostanaula River and also reduce flood inflows to the reservoir formed by the Alabama Power Company's Weiss Dam on the Coosa River. Bankfull flow downstream is 6,600 cfs. The reservoir is used to store the inflow during major flood with no outflow in excess of that which can be retained in the reregulation dam. When the Carters pool exceeds elevation 1099.0 fmsl, an induced surcharge operation schedule is followed.

The flood control operation of Allatoona Dam is to alleviate floods along the lower Etowah River and to reduce flood stages at Rome, Georgia. When reservoir inflows cause the pool to rise above the top of the power pool, normal power operations may be continued as scheduled unless modified by the Corps for flood control. The channel capacity below the dam is 9,500 csf. Should predictions indicate that anticipated runoff from a storm will appreciably exceed the flood control storage, an induced surcharge operation is used. The operation of Allatoona and Carters for flood control involves an integrated operation in order to provide maximum flood protection for Rome, Georgia.

There is no flood control storage in the Jones Bluff or the Millers Ferry Projects. During periods of high flow, the Jones Bluff Reservoir is maintained at elevation 125 fmsl by passing the inflow through the spillway gates and/or the power plant. Once spillway and power plant capacity is reached, a free overflow condition prevails.

During floods, Millers Ferry Dam is operated so that the pool will not appreciably exceed elevation 80 fmsl at the dam by passing the inflow through the spillway gates and/or the power plant until discharge capacity is exceeded. The power plant is inoperative when the tailwater exceeds 70.5 fmsl.

Operation of Buford Dam for flood control is in accordance with instructions issued by the reservoir control section in the Mobile

District office and releases depend on stages forecast for the Chattahoochee River below the dam. Channel capacity below Buford Dam is 10,000 csf. The reservoir provides for full flood control storage up to 15 feet above normal pool. This storage will completely control the standard project flood occurring on a full power pool.

Flood control releases from West Point Dam depend on the pool level and on stages forecast for key downstream stations. The flood control plan provides for the reservoir to be drawn down ten feet in the fall season. A floodway below the dam makes it possible to have flow downstream from the dam of up to 25,000 csf. An induced surcharge operation is used for pool elevations above top of power pool.

Generally, an induced surcharge operation is used during flood conditions at Walter F. George. The induced surcharge schedules are designed to eliminate the necessity for sudden large increases in outflow peaks to values equal to or less than those which would occur under natural river conditions. Downstream channel capacity is 60,000 csf.

There is no flood control storage available in the Jim Woodruff project and the pool level at the dam is normally maintained between elevations 76.5 and 77.5 fmsl. When spillway and power plant capacity is exceeded during a flood a free discharge condition prevails and natural flood elevations are approximated. The power plant becomes inoperative when the tailwater reaches elevation 70 fmsl.

The amount of 293,000 acre-feet of flood control storage is available in Hartwell Lake. When the lake is above elevation 665 fmsl, an induced surcharge operation is used.

There are 390,000 acre-feet of flood control storage in Clark Hill Reservoir. The purpose for the flood control operations is to obtain the maximum reduction in damages at and below Augusta.

Low Flow Requirements

The Carters Reregulation Dam maintains an outflow of not less than 240 csf. Allatoona Dam provides a minimum discharge of 200 csf. As long as water is available, a minimum mean daily outflow of 7,000 csf from Millers Ferry and 6,000 csf from Jones Bluff will be maintained. The Alabama Power Company is committed to release a minimum seven-day total of 32,840 csf from their plants on the Coosa and Tallapoosa Rivers upstream from Jones Bluff and Millers Ferry.

During off-peak periods, enough flow must be released from Buford Dam to provide a flow of 650 cfs at Atlanta. Consideration must also be given during low flow periods to the release of water from Buford (up to an average weekly release of 1600 csf - prime flow) to help maintain navigable depths in the Apalachicola River below Jim Woodruff Dam.

During off-peak periods, the West Point Dam maintains a continuous flow of 675 csf.

Continuous releases are not required from the Walter F. George Project. However, downstream water requirements for navigation are carefully considered in establishing release schedules.

REAL TIME ASPECTS OF SYSTEM OPERATION

Manual radio reporting networks have been installed in the Oostanaula and Etowah River basins. These networks facilitate power and flood control operations of Carters, Buford, and Allatoona dams. The data from these stations is transmitted directly to operators at Carters. The stations report on call from the manual system and are interrogated at least 6 times each day. The manual interrogation system for Allatoona and Buford is located at Carters power plant control room to provide interim data until an automatic three-plant interrogation and recording system is installed. The automatic system will provide hourly data for Carters, Allatoona, and Buford, all of which are controlled from Carters. The system will be integrated into the supervisory control and data acquisition (SCADA) system which will provide daily printout of hourly readings. It will also provide updates of hourly readings upon request from the District and/or Division.

An automatic radio reporting hydrologic network is located in the Alabama River Basin above Millers Ferry Dam to facilitate the operation of Millers Ferry and Jones Bluff Projects. Rainfall, river stage, pool, and tailwater data are telemetered to operators at the Millers Ferry powerhouse who control operations at both projects. This system is being integrated into the power plant SCADA system which will provide daily printout of hourly readings to the District and Division.

An automatic radio reporting hydrologic network is located above West Point Dam for river and rainfall data. Data is transmitted through a relay station at West Point Dam to the operators at the Walter F. George powerhouse who control operations at both projects. The National Weather Service operates a DARDC (Device for Automatic Remote Data Collection) system in the river basin above the projects. These data are also used in the project management.

Two river gage stations have been equipped with telemarks and unattended station relays to permit interrogation by telephone for the operation of Jim Woodruff Dam. These data are supplemented by reporting rainfall and river stage stations at the upstream projects. These stations will be integrated into an automatic interrogation and recording system which will also report hourly readings for the previous day to the District and Division and an hourly update upon manual request from District and/or Division.

Radio reporting stations are not located above Hartwell Dam at the present time. Reservoir and tailwater elevations are relayed to the Clark Hill Project by radio and to the Savannah District office by telephone.

An automatic river stage radio reporting system is in operation above Clark Hill Dam. Gage heights are transmitted to the dam at regular intervals. The dam tender reports the gage reading to the District office by radio.

OPTIMIZATION OF SYSTEM OPERATION ON LONG TERM AND REAL TIME BASES

Subsystem Operations

1. System operation of the Apalachicola River Basin Projects

Existing authorizations call for a nine-foot navigation channel extending up the Apalachicola and Chattahoochee Rivers to Columbus, Georgia. Above Jim Woodruff Lock and Dam, this channel is provided by the Woodruff, George W. Andrews, and Walter F. George pools. However, below Jim Woodruff there are 108 miles of open river channel between the dam the the Gulf of Mexico. Flow requirements to maintain a 9-foot channel vary from about 10,500 to 15,000 csf depending on the status of maintenance dredging. Releases are needed from upstream reservoirs considerably in excess of the flow requirements to meet power contract commitments. In order to assure maximum release from Buford, West Point, and Walter F. George storage for navigation without jeopardizing the projects' capability to meet power contract requirements, a system operation plan has been developed for the Apalachicola River Basin projects. The plan establishes zones within the storage prisms at Buford, West Point, and Walter F. George which indicate the maximum flow that can be maintained in the river by storage withdrawals for any given level in the storage reservoirs. All releases are made through the power plants and all power scheduling is on a weekly basis. Utilization of pondage starts with Woodruff and proceeds upstream depending on what the subsystem study shows to be the proper balance among the reservoirs at a particular time.

2. System Operation of the Alabama-Coosa River Basin Projects

Power discharges from Millers Ferry Lock and Dam are reregulated by the downstream Claiborne Lock and Dam project. Flows from Claiborne provide navigable depths in the downstream channel. The navigable channel below Claiborne has been designed to provide a 9-foot depth with a flow of 8,500 csf. Low flows below Claiborne are lower and more frequent than anticipated. Presently there are about 45 days each year when the flow is less than 8,500 csf. More flow is needed to maintain project depth and prevent excessive dredging costs. It could be provided from increased minimum flows from the Alabama Power Company projects on the Coosa and Tallapoosa Rivers and the provision of enough storage in Millers Ferry and Jones Bluff Reservoirs to regulate the increased flows. One proposed operating plan is to operate Millers Ferry and Jones Bluff projects as run-of-the-river plants during a low flow period. Three feet of stored water would be used from Claiborne to further increase flows. This type of operation would result in some loss in power revenue. A determination of this loss will be made and compared with operation benefits.

3. System Operation of the Savannah River Basin Projects

Allocations of flood control storage, drawdown schedules for seasonal flood control, and discharge schedules are made to provide the greatest practicable degree of flood protection in the basin. Close coordination of reservoir operations at the Hartwell and Clark Hill Projects is required for flood control as the two projects are operated as one system with coordinated timing of releases.

System Operation

Operation and Management of the SAD hydropower system considers all project purposes. The overall hydropower commitments must be met. Establishment of hydropower commitments must maximize the sale of hydropower with the constraint of assuring that all other project purposes can be met. Flows should be sufficient to provide authorized navigation depths at all times. Stored flood waters must be emptied as quickly as conditions allow. Reservoir pool elevations should be held as steady and as close to schedule as possible for recreation interests, which are considerable, and to inhibit shoreline erosion. Changes in system operation and in reservoir elevations are often considered for additional water supply, hydropower, and navigation.

System Analysis Methodology

While operating decisions have been effectively made by District and Division personnel, a combined reservoir systems analysis study is underway. A system analysis study which considers all project purposes is being designed to maximize overall system benefits and insure that individual project benefits are met to the fullest possible extent. The system analysis study will be used in the reformulation and modification studies of existing projects resulting from changes in water use requirements. The capability to analyze the effects of proposed changes in project purposes on the total system is needed.

Several existing Corps systems analysis models that produce real time and long term data are being considered for this study. These models are:

1. North Pacific Division's Hydropower System Regulation Analysis Program (HYSYS) which performs a sequential river and reservoir routing and regulates a set of hydropower projects to meet a system power load according to a specified schedule or to system power peaking requirements.

2. HEC-5C, Simulation of Flood Control and Conservation Systems, which simulates the sequential operation of a system of reservoirs of any configuration for short interval historical or synthetic floods or for long duration non-flood periods or for combinations of the two.

3. NPD's Streamflow Synthesis and Reservoir Regulation Model (SSARR) which consists of a generalized watershed model, a river system model and a reservoir regulation model.

Because the SAD hydropower system involves three separate river basins, some modification to existing programs will be needed. Consideration is also being given to starting a new model specifically for this system.

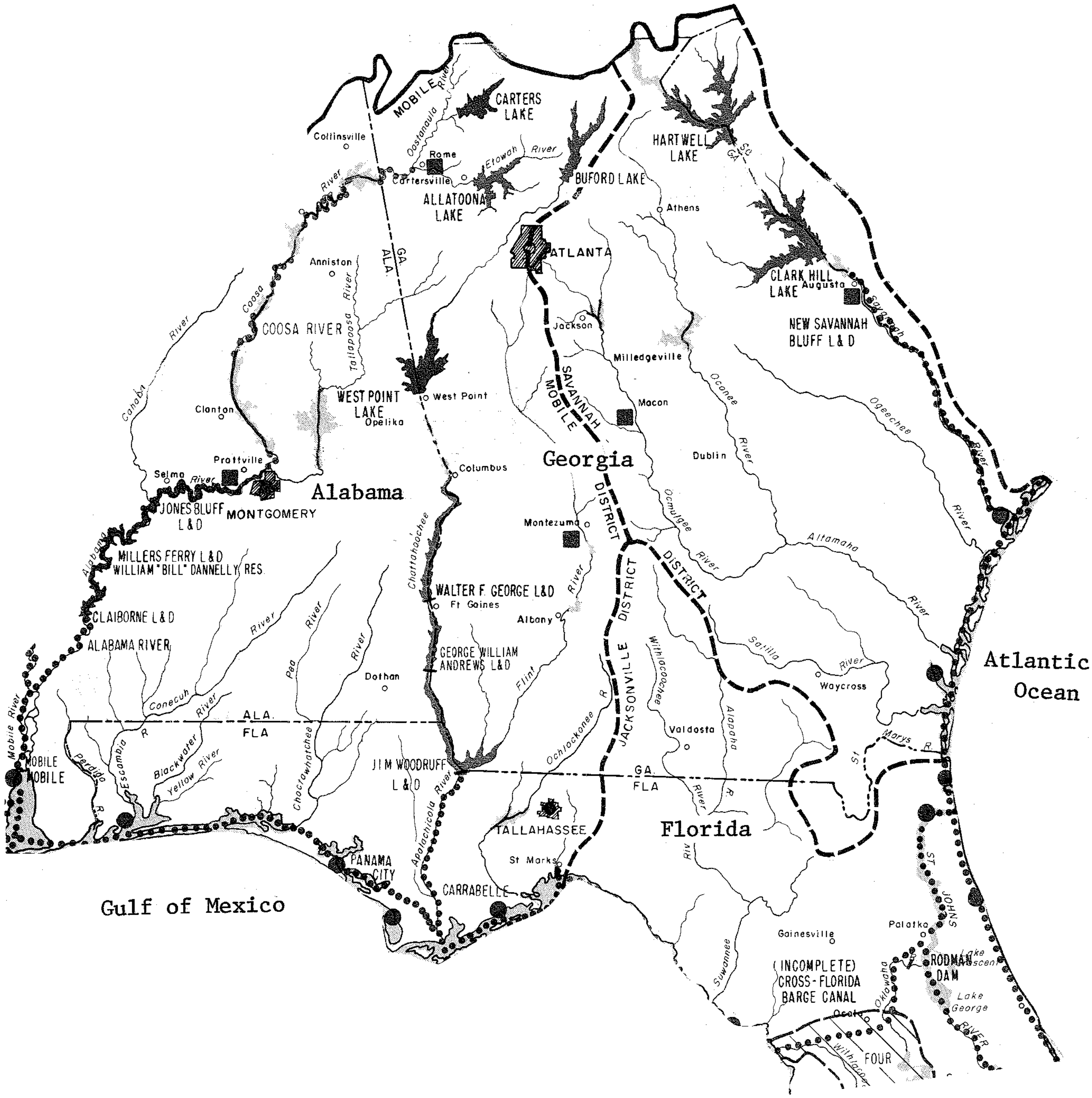


Figure 1. Location of the SAD hydropower system projects.

THE USE OF LANDSAT DCS IN RESERVOIR MANAGEMENT AND OPERATION

S. Cooper and J. L. Horowitz

New England Division, Corps of Engineers
Waltham, Massachusetts

1. INTRODUCTION

The New England Division, Corps of Engineers (NED) has been participating in the NASA LANDSAT experiments since the summer of 1972, when LANDSAT-1 was launched. NED is assessing the possible future usefulness of orbiting satellites such as LANDSAT in the operation of its water resources systems used to control floods. Both the data collection and imaging systems have been studied. This paper reports on the data collection aspects of these studies.

2. BACKGROUND

Since the Industrial Revolution in the 1800's, the rivers of New England have been developed to supply water for power and transportation. As new means of transportation became more economical, both railroad and highway systems were built along the banks of the rivers to service the expanding needs of the industrial, commercial and urban centers. Structures, such as buildings, roads, bridges and dams have restricted floodways to such an extent that considerable property and environmental damages have occurred during moderate and major floods. Notable floods of November 1927, March 1936, September 1938 and August 1955 demonstrated the need for flood control to prevent these natural catastrophes.

At the direction of Congress, the U.S. Army Corps of Engineers developed a comprehensive plan of protection for each river basin after careful analysis of all water resources. Protective works generally consist of a combination of channel improvements, dikes and/or floodwalls at major damage centers augmented by upstream flood control reservoirs. Many of these reservoirs contain additional storage reserved for other uses such as water supply, conservation and recreation. The Corps has built 35 flood control reservoirs, 37 local protection projects and 4 hurricane barriers in New England at a total investment of some $300 million.

To achieve optimum operating benefits from this comprehensive protection system, the New England Division requires hydrologic data such as river, reservoir and tidal levels, wind velocity and direction, barometric pressure and precipitation. In the past this data was collected from field observation and relayed via telephone or voice radio. It took several hours to compile and assess the data in this manner. With the need for timely and reliable information increasing, the Corps began development of new methods of data collection. In 1970, the Automatic Hydrologic Radio Reporting Network was placed in operation. This ground-based radio relay system consists of 41 remote reporting stations, and a central control at Division Headquarters in Waltham, Massachusetts. The network, under computer programmed control collects and analyzes, in real time mode, information from in situ sensors which is essential for flood regulation. The remote reporting stations are strategically located in five major river basins and at key coastal points, with each contributing to a detailed, comprehensive hydrologic picture. In June 1972, NASA contracted with NED to evaluate the possibility of using satellites such as LANDSAT for the purpose of collecting environmental data from Data Collection Platforms (DCP's) versus conventional methods of data relay. DCP's have now been installed at 26 locations throughout New England, many at existing U.S. Geological Survey gaging stations.

3. THE LANDSAT DATA COLLECTION SYSTEM

The present configuration of the LANDSAT Data Collection System (DCS) in New England is shown in Figure 1. The system relays hydrometeorological information such as river stage, rainfall and water quality parameters to NED within 45 minutes of acquisition by NASA's Goddard Space Flight Center, Greenbelt, Maryland, via a teletype link supplied by NASA for this purpose.

Based on three years' experience with this 26 station network in New England, NED has found real time data collection by orbiting satellite to be both reliable and feasible. Orbiting satellite systems can be designed that are more flexible, easily maintained and less expensive than conventional ground-based means. Satellite systems retain most of the advantages of ground-based radio relay, while obviating the need for the latter's costly relays and repeaters. Also, because the satellite platforms are small in size, they may be relocated at low cost offering a new capability to support special or changing requirements. The only drawbacks with the LANDSAT Data Collection System for NED operational purposes are the frequency of data reports (four to six times daily) and the 45 minute time lag for data receipt by NED. However, it should be understood that the present LANDSAT system has been only an experiment, to test the feasibility of data collection by orbiting satellite. An operational system could be designed involving more than one satellite, to increase the frequency of data reporting; also, satellite ground receiving stations could be constructed at all major user locales, such as NED to permit direct and immediate receipt of the information, rather than the relay of data from NASA or some other agency.

Based on its LANDSAT experience, NED has endorsed the institution of a satellite data collection system on a Corps-wide basis or a nationwide system with other Federal and State agencies, whether it be of the orbiting type with which we have experimented, or the geostationary kind, for which evaluation is not yet available. A questionnaire sent to all Corps of Engineers offices in July of 1973 indicated that nearly 4,000 fully automated data collection stations would be required Corps-wide over the next five years for the relay of hydrologic information for watershed management activities. This alone would be far more than required for an economical Corps-wide operational orbiting satellite system as compared with ground-based methods.

4. THE NED GROUND RECEIVING STATION

Since any operational satellite configuration should include ground receiving stations at all major user locales, NED, with NASA support has constructed and is now testing an inexpensive semiautomatic and easily maintained ground receive station or Local User Terminal (LUT) as a follow-up to its original study. This will enable the Division to receive hydrometeorological data from data collection platforms in the field directly at its headquarters in Waltham, Massachusetts with no time delays. The software to drive the antenna system is being developed with the intention that the antenna operate in an unattended mode automatically over nights and during weekends and holidays, with a Data General mini computer controlling all processes. A diagram of the overall facility is shown in Figure 2.

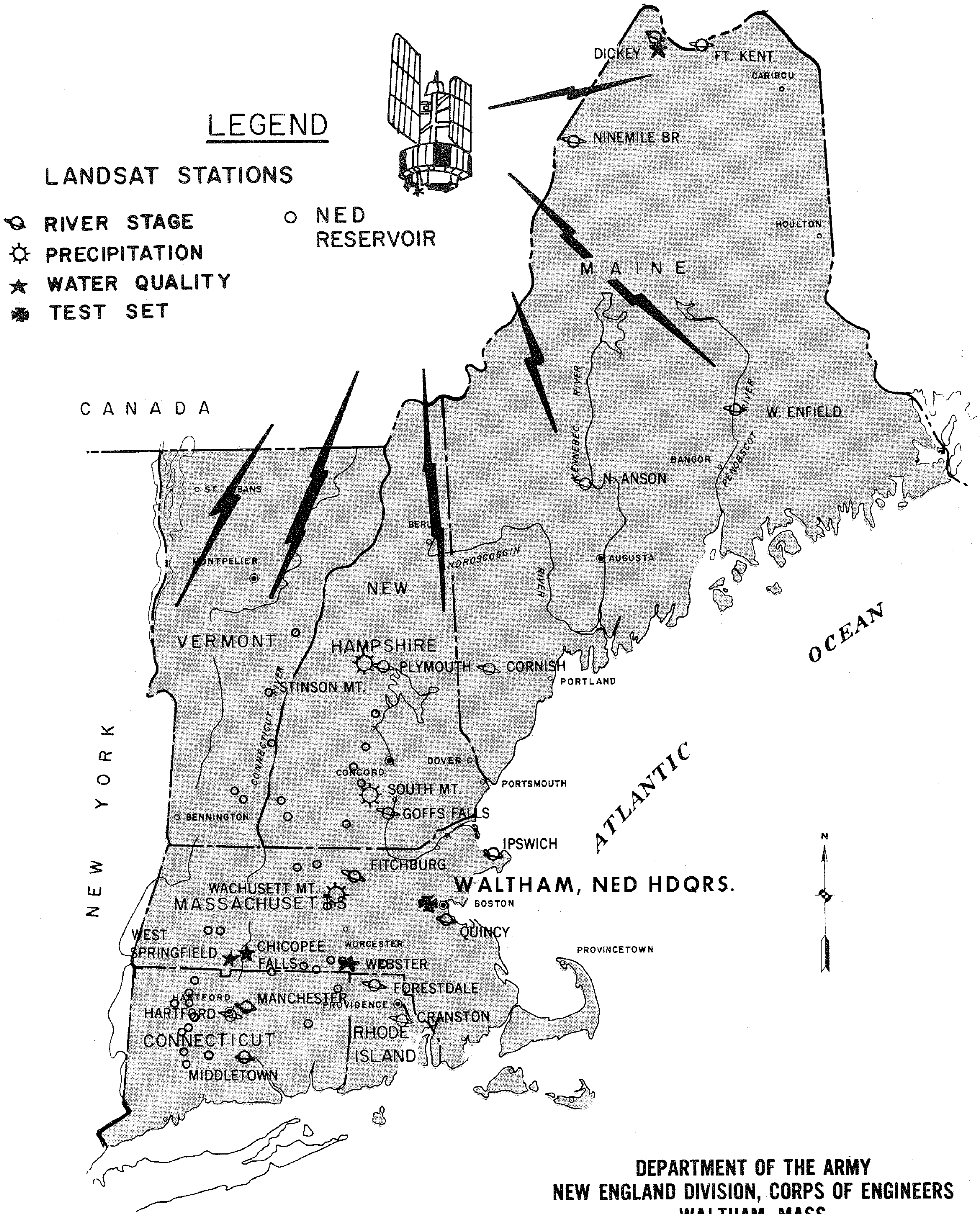
The mini computer is a very active component of the LUT. It periodically interrogates radio station WWVB for the correct Universal Time, controls the 15-foot diameter antenna and acquires data virtually simultaneously by multitasking programs. By accurately knowing the time of day and the satellite's precise predicted position, the computer easily keeps the satellite within the antenna's three degree receiving beam width. Current plans call for the total slave mode of operation, i.e., tracking depends on the computer being informed correctly. However, we are also developing software autotracking packages which will be more versatile. With these, if for some reason the satellite were outside the antenna's receiving beam, the computer would execute a search for it and order changes in antenna direction and movement to bring it back into view.

As of 1 October 1975 the LUT had already tracked LANDSAT numerous times, but not on a regularly scheduled basis. Valid data is being acquired, however, and we expect soon to be in a quasi-operational mode.

5. CONCLUDING REMARKS

Investigation of the LANDSAT Data Collection System as well as the associated study of LANDSAT imagery at the New England Division is part of an overall and expanding Corps of Engineers R & D program to assess potential remote sensing capabilities for operational watershed management purposes. It is the feeling at this office that the LANDSAT Data Collection System has already made a significant contribution toward the goals embodied in this program.

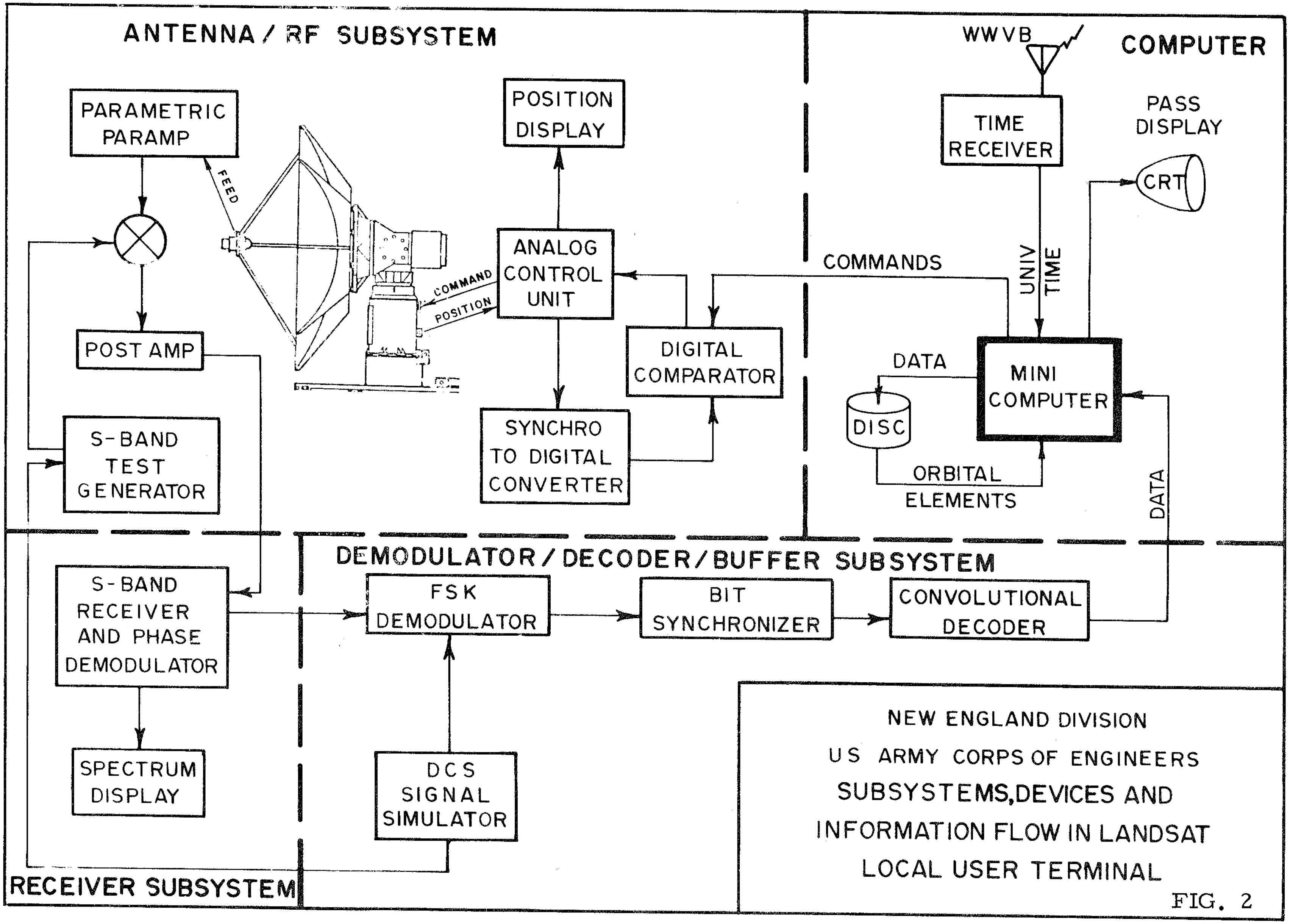
LANDSAT–2 DATA REPORTING STATIONS



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.
AUGUST 1975

Paper 11

FIG. 1



NEW ENGLAND DIVISION
US ARMY CORPS OF ENGINEERS
SUBSYSTEMS, DEVICES AND INFORMATION FLOW IN LANDSAT LOCAL USER TERMINAL

FIG. 2

SUMMARY

THE USE OF LANDSAT DCS IN RESERVOIR MANAGEMENT AND OPERATION

S. Cooper and J. L. Horowitz

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Waltham, Massachusetts

The New England Division, Corps of Engineers (NED) has been participating in the NASA LANDSAT experiments to assess the possible future usefulness of orbiting satellites such as LANDSAT in the operation of its water resources systems used to control floods.

Based on three years' experience with a 26 station network in New England, NED has found real time data collection by orbiting satellite relay to be both reliable and feasible. Orbiting satellite systems can be designed that are more flexible, easily maintained and less expensive than conventional ground-based means.

NED endorses the institution of a satellite data collection system on a Corps-wide basis or a nationwide system with other Federal and State agencies, whether it be of the orbiting type with which we have experimented, or the geostationary kind for which evaluation is not yet available. Any operational satellite configuration should include ground receiving stations at all major user locales for direct receipt of satellite information, rather than the relay of data from NASA or some other agency. Therefore NED, with NASA support, has constructed and is now testing an inexpensive, semiautomatic and easily maintained ground receive station as a follow-up to its original study.

TECHNIQUES FOR REAL-TIME OPERATION OF FLOOD CONTROL RESERVOIRS IN THE MERRIMACK RIVER BASIN

by

Bill S. Eichert,[1] John C. Peters,[2] and Arthur F. Pabst[3]

INTRODUCTION

This paper contains a description of the techniques that are under development at The Hydrologic Engineering Center for providing decision criteria on a real-time basis for operating the five flood control reservoirs in the Merrimack River basin. Techniques under development include:

a. testing of alternative streamflow forecasting models

b. application of computer program HEC-5C, Simulation of Flood Control and Conservation Systems, to develop decision-criteria for system operation on a real-time basis

c. use of computer terminals to enable analysis of alternative forecast and/or decision criteria in both batch mode and interactive applications.

Each of these techniques will be discussed following a brief description of characteristics of the basin, reservoir system and automatic data collection network.

CHARACTERISTICS OF BASIN AND RESERVOIR SYSTEM

The Merrimack River basin is located in east central New England and extends from the White Mountain area of New Hampshire southward into the northeast portion of Massachusetts. The basin is 134 miles long north to south and up to 68 miles wide east to west, the drainage area is about 5,000 square miles, 3,800 of which are in New Hampshire.

The average annual precipitation for the Merrimack River basin varies from about 60 inches in the headwaters to about 40 inches in the southern section, with a basin average of about 45 inches. Precipitation is fairly evenly distributed throughout the year. During winter months the precipitation is mostly in the form of snow, with amounts averaging 70 to 100 inches or more in the north to 45 to 60 inches in the southern areas.

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Floods can occur during any season. The two greatest basin-wide floods occurred in March 1936 and September 1938. The 1936 event resulted from two periods of heavy rainfall about a week apart associated with significant snowmelt. The 1938 flood resulted from intense hurricane rainfall which occurred after a week of almost continual rain.

The reservoir system in the Merrimack basin consists of five reservoirs, all of which are operated almost exclusively for flood control. Drainage areas and flood control storage capacities for the reservoirs are as follows:

Reservoir	Drainage Area sq. mi.	Flood Control Storage ac. ft.	Flood Control Storage in.
Franklin Falls	1,000	150,600	2.8
Blackwater	128	46,000	6.7
Hopkinton	382	70,100	6.5
Everett	64	85,500	
MacDowell	44	12,800	5.4

A schematic diagram of the reservoir system is shown in figure 1. The Hopkinton and Everett reservoirs are joined by a canal to enable diversion from Hopkinton to Everett during flood events. Franklin Falls reservoir has a relatively limited flood control capacity because it was originally anticipated that another reservoir would be developed upstream.

Population centers in the basin are located, for the most part, along the main stem of the Merrimack River. For computer simulation runs described in this paper, the locations of Franklin Junction, Concord, Manchester and Lowell were treated as damage centers (see Figure 1). The first three centers are in New Hampshire; Lowell is in northern Massachusetts. A major proportion of the total damages occurs at Lowell. The travel time from Franklin Falls to Lowell is about 30 hours.

PRESENT REGULATION PROCEDURES

Because all reservoirs except Franklin Falls contain about 6 inches of flood control storage, a large flood or a series of smaller ones can be stored in four of the reservoirs without spilling. Consequently, current operation procedures require that the outlet works of MacDowell, Blackwater, Hopkinton, and Everett reservoirs be shut off early in a flood event. Because of the limited storage capacity at Franklin Falls, flows are passed through this reservoir up to the channel capacity of 18,000 cfs. Inflows to Franklin Falls are estimated from reservoir rate-of-rise curves; future inflows are based on flows at an upstream location called Plymouth. Once flows at Plymouth have peaked, a release rate for Franklin Falls is established based on the estimated flood volume.

DATA COLLECTION SYSTEM

The New England Division has established a comprehensive data collection network in the Merrimack River basin as well as in several other New England basins. An automatic radio reporting network supplies information on rainfall and river stage directly to a computer in the Control Center in the Division office in Waltham, Massachusetts. Under computer-programmed control, reporting stations can be interrogated singly or as a group at automatically selected time intervals ranging from six hour to one hour periods based on the amount of river flow or rainfall. At present, river stage is reported from ten locations and precipitation from three locations. Also the New England Division is assessing the use of orbiting satellites for relaying information from data collection platforms. Four of these platforms are currently in use in the Merrimack basin.

FORECASTING TECHNIQUES

Streamflow forecasting may be accomplished using a variety of techniques. A relatively simple technique would involve relating the stage at a downstream location to the stage at some earlier time at an upstream station. A relatively sophisticated technique would involve modeling the precipitation-runoff process continuously involving all aspects of the hydrologic cycle deemed significant. Accompanying this range of forecasting sophistication is a range of required data. A simple gage relationship requires only stage or discharge as a function of time. A precipitation-runoff model may require precipitation, water equivalent of a snow pack, air temperature, dewpoint, wind velocity, insolation, albedo, soil moisture, frost depth as well as streamflow discharge. Forecasting by a simple model is severely limited in its capability to provide information very far into the future. The sophisticated precipitation-runoff model may utilize forecasted meteorological conditions and provide forecasts of runoff as far as is reasonably possible into the future.

The selection of a particular technique will depend on a realistic assessment of the actual data available, the accuracies of the forecast method, and the use that will be made of the forecasted streamflows.

Streamflow Extrapolation Forecasting

A method for making relatively short-term forecasts that utilizes a minimum amount of data would be a useful tool to complement a more complete precipitation runoff model. Initial efforts were directed to development of a simple forecast tool that would require only observed streamflow at various locations in the basin. The technique developed is termed Streamflow Extrapolation Forecasting. In essesnce it is no more than taking observed flows at stream gaging locations, extending them into the future by gage relationships, recessing the flows from that point on, and routing them down the basin.

This technique will be described by use of a simple illustration. Given only the observed flows at stream gage locations A, B, C and D, (Figure 2a) up until the current time, the problem is to estimate the future flows at sites A and B.

The following series of steps would be taken:

(1) Extend the current flows at A, 3-hours into the future based on the immediately preceding flows at A and nearby station C. (Figure 2b)

(2) Recess this hydrograph at A for flows beyond 3 hours into the future. This will provide the forecasted flows at A. (Figure 2b)

(3) Route the forecasted flows at A down to B. (Figure 2c)

(4) Extend the current flows at B, 6-hours into the future based on the immediately preceding flows at B and nearby station D. (Figure 2c)

(5) Subtract the routed flows from A from the extended flows at B yielding incremental local flows between A and B. (Figure 2d)

(6) Recess the incremental local flow beyond 6-hours into the future. This will be the forecasted incremental flows between A and B. (Figure 2d)

(7) Add these incremental flows to the flows routed down from above yielding the forecasted flows at B. (Figure 2e)

(8) Continue downstream as required using the preceding steps.

The gage relationships used in steps 1 and 4, to extend flows at a given station, should be developed from historical data. Multiple regression analysis may readily be used to establish these relationships. A future flow at station A may be correlated to current and past flows at stations A and C. The time span for extending flows (i.e., 3 hours, 6 hours) will depend on the size, shape and other characteristics of the basins.

Such a procedure which assumes little or no future precipitation or snowmelt input, yields the minimum future flows in the river system. If future additional precipitation or snowmelt occur the actual flows will exceed the forecasted flows.

Such a forecast can provide a firm basis for establishing that a reservoir will surely fill. It may not give a long lead time as to when a reservoir will fill; but it is free from the uncertainities of having to assess average basin precipitation with sparse data, calculations of snowmelt, or of establishing losses for frozen or partly frozen ground.

The forecasted discharges are interfaced to the reservoir operation model through a data file. This allows the forecast model to run independently of the operation model. As a separate program the forecast can be made once and then several operation policies may be evaluated using the discharges. It is not necessary to fit both the forecast and operation models in computer core at the same time, or to resort to overlays. Any other forecast technique may be used by simply having the alternative model write the appropriately formatted discharge file.

Evaluation of Forecasts

In order to choose between alternative forecasting techniques it is necessary to establish a measure of forecast accuracy. The measure should reflect the error over a certain finite period into the future, recognizing that the error over such a span will change as one proceeds through the event. In addition it would be desirable to aggregate the error over a series of several floods rather than to accept a given technique only on its reproduction of one event.

When historical data are available the Streamflow Extrapolation Forecasting program will provide information on the relative error (eq. 1) and the standard error (eq. 2) over a future time span selected by the user

$$RELERR_i = \frac{\sum_{j=i+1}^{i+\ell} \frac{|Q_j - QOBS_j|}{QOBS_j}}{\ell} \qquad \text{eq. 1}$$

$$STOERR_i = \sqrt{\frac{\sum_{j=i+1}^{i+\ell} (Q_j - QOBS_j)^2}{\ell}} \qquad \text{eq. 2}$$

where Q is forecast discharge
QOBS is observed discharge
i is current time index
ℓ is future span length of forecast

Such errors can be evaluated at each time period as one steps through the event. The relative errors for the 1936 flood for the Merrimack River at Lowell, MA for two forecast conditions are shown in figure 3. Beneath the actual cumulative local flow hydrograph are shown the errors in a forecast based only on recessed flows at upstream locations, and the errors when station flows are first extended into the future by gage relationships and then recessed. Relatively good agreement was obtained for short term forecasts using this technique.

Another indication of the adequacy of the forecast can be measured in terms of the efficiency of system operation by comparing the results of operation of historical floods with and without forecasts. One measure of this efficiency that may be useful for flood control operation is a comparison of expected (or average) annual flood damages (AAD) using historical flows and forecasted flows. Such a comparison is made in Table 2 where the average annual damage (AAD) for run F-1 is about the same for locations 8, 9, and 10 as the runs using measured streamflow (runs J and 17). The AAD for location 11 is considerably higher for run F-1 than for either run J or 17 indicating additional improvement in forecasting procedures is needed for the location that is substantially further downstream.

RESERVOIR OPERATIONAL MODEL

Basic Objectives of Model

The HEC-5C program was initially developed to assist in planning studies required for the evaluation of proposed changes to a system and to assist in sizing the system components for flood control and conservation requirements. However, the program can also be used in studies made immediately after a flood to calculate the preproject conditions and to show the effects of existing and/or proposed reservoirs on flows and damages in the system. Special features have been added to the program to make it useful for real-time applications. The program logic is designed to minimize flooding as much as possible and yet empty the system as quickly as possible while maintaining the proper balance of flood control storage among the reservoirs.

The above objectives are accomplished by simulating the sequential operation of various system components of any configuration for short interval historical or synthetic floods or for long duration nonflood periods, or for combinations of the two. Specifically the program may be used to determine:

a. Releases from reservoirs during flood emergencies based on local flow forecasts furnished to the program.

b. The evaluation of operational criteria for both flood control and conservation for a system of reservoirs.

c. The influence of a system of reservoirs, or other structures on the spatial and temporal distribution of runoff in a basin.

d. The expected (or average) annual flood damages (AAD), system costs, and excess flood benefits over costs.

e. Flood control and conservation (including hydropower) storage requirements of each reservoir in the system.

f. The determination of the system of existing and proposed reservoirs or other structural or nonstructural alternatives that results in the maximum net benefit for flood control for the system by making simulation runs for selected alternative systems.

Basic Data Requirements

The input data requirements for any basin for HEC-5C can be minimal for preliminary planning studies or detailed for modeling existing systems. The minimum data requirements are as follows:

a. General Information (4 cards)

(1) Title cards for job (3 cards)

(2) Six miscellaneous items including the number of periods of flow data, time interval of flows, etc.

b. Reservoir Data (4 cards per reservoir)

(1) Reservoir capacities for top of conservation and top of flood control elevations.

(2) Downstream control points for which reservoir is to be operated.

(3) Reservoir storage/outflow tables.

c. Control Point (including reservoirs) Data (3 cards per control point)

(1) Identification number and title

(2) Operating channel capacity

(3) Channel routing criteria (Muskingum, modified Puls, Working R/D, Tatum, or Straddle-Stagger)

d. Flow Data

Inflow or local flow data (or observed flows and reservoir releases) for each control point for one or more historical (including forecasted flows) or synthetic floods.

Optional data on flood damages may also be used by inputting peak discharge-damage data where flood damages are directly related to maximum stage (or discharge) obtained during a flood event.

General Operational Criteria for Model

a. Reservoirs are operated to satisfy constraints at individual reservoirs, to maintain specified flows at downstream control points, and to keep the system in balance. Constraints at individual reservoirs are as follows:

(1) When the level of a reservoir is between the top of conservation pool and the top of flood pool, releases are made to attempt to draw the reservoir to the top of conservation pool without exceeding the designated channel capacity at the reservoir or at downstream control points for which the reservoir is being operated.

(2) Releases are made equal to or greater than the minimum desired flows when the reservoir storage is greater than the top of buffer storage, and or equal to the required flow if between level one (top of inactive pool) and the top of buffer pool. No releases are made when the reservoir is below level one. Releases calculated for hydropower requirements* will override minimum flows if they are greater than the controlling desired or required flows.

(3) Releases are made equal to or less than the designated channel capacity at the reservoir until the top of flood pool is exceeded, then all excess flood water is dumped if sufficient outlet capacity is available. If insufficient capacity exists, a surcharge routing is made. Input options permit channel capacity releases (or greater) to be made prior to the time that the reservoir level reaches the top of the flood pool if forecasted inflows are excessive.

(4) The reservoir release is never greater (or less) than the previous period release plus (or minus) a percentage of the channel capacity at the dam site unless the reservoir is in surcharge operation.

b. Operational criteria for specified downstream control points are as follows:

(1) Releases are not generally made (as long as flood storage remains) which would contribute to flooding at one or more specified downstream locations during a predetermined number of future periods except to satisfy minimum flow and rate-of-change of release criteria. The number of future periods considered is the lesser of the number of reservoir release routing coefficients or the number of local flow forecast periods specified on input data.

(2) Releases are made, where possible, to exactly maintain downstream flows at channel capacity (for flood operation) or for minimum desired or required flows (for conservation operation). In making a release determination, local (intervening area) flows can be multiplied by a contingency allowance (greater than 1 for flood control and less than 1 for conservation) to account for uncertainty in forecasting these flows.

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*No Corps hydropower projects are in the Merrimack River Basin.

c. Operational criteria for keeping a reservoir system in balance are as follows:

(1) Where two or more reservoirs are in parallel operation above a common control point, the reservoir that is at the highest index level, assuming no releases for the current time period, will be operated first to try to increase the flows in the downstream channel to the target flow. Then the remaining reservoirs will be operated in a priority established by index levels to attempt to fill any remaining space in the downstream channel without causing flooding during any of a specified number of future periods.

(2) If one of two parallel reservoirs has one or more reservoirs upstream whose storage should be considered in determining the priority of releases from the two parallel reservoirs, then an equivalent index level is determined for the tandem reservoirs based on the combined storage in the tandem reservoirs.

(3) If two reservoirs are in tandem (one above the other), the upstream reservoir can be operated for control points between the two reservoirs. In addition, when the downstream reservoir is being operated for control points, an attempt is made to bring the upper reservoir to the same index level as the lower reservoir based on index levels at the end of the previous time period.

Use of Contingency Allowance and Foresight

Two key input items are used in determining reservoir releases based on downstream flooding as discussed under "operational criteria for downstream control points." These factors are the number of future time periods (IFCAST) that should be checked for possible future flooding (called forecast periods) and the contingency allowance (CFLOD) which is multiplied times the cumulative uncontrolled downstream flow to account for uncertainty in forecasts. For simulation of historical floods (where flows are known for duration of flood) a contingency factor of 1 and an infinite forecast period could be used in order to operate with maximum foresight. However, these assumptions would not simulate "real world" conditions where large errors in forecasting future streamflows are possible. These two key factors, for the simulation of historical floods, should be selected so that the operational efficiencies in the planning mode will approach the expected efficiencies under flood emergency conditions. During flood emergencies these factors should be used to insure that the forecasting errors do not cause reservoir releases to be made which will cause major unnecessary flood damages. The sensitivity of the system to different values of these two factors can be determined by simulating the operation and resulting flood damages for a series of different sized flood events for the system. The difference between the average annual damages (AAD) for various combinations of these factors will help to evaluate the sensitivity of these factors. Table 1 illustrates how the reservoir system responds to these factors. The adopted value for the number of forecast periods (IFCAST) was four and the adopted contingency factor (CFLOD) was 1.2. Because a time interval of 3 hours was used, the duration of the adopted forecast period was 12 hours.

TABLE 1

AVERAGE ANNUAL DAMAGE VS FORECAST PERIOD
AND CONTINGENCY FACTOR USING HISTORICAL FLOWS

RUN	CFLOD	IFCAST	AAD (in $1000)
1	1.0	2	1191
2	1.0	4	1083
3	1.0	6	1048
4	1.0	10	1070*
5	1.2	2	1116
6	1.2	4	1046
7	1.2	6	1032
8	1.2	10	1052*
9	1.2	20	1099*
10	1.4	2	1082
11	1.4	4	1047
12	1.4	6	1056*
13	1.4	10	1090*
14	1.6	4	1075

*One would expect these values to be less than the previous values. They are not because for the larger events a long forecast period causes reservoir releases to be diminished relatively early in the event. When the reservoirs eventually go uncontrolled, the resulting flooding is greater than would have occurred if the releases had not been diminished early in the event. The increase in damages in the larger events exceeds the decrease in damages for the smaller events.

TABLE 2 - AVERAGE ANNUAL DAMAGES SUMMARY - FORECASTED FLOW

Run	Flood	Total AAD $1000	AAD Location				AAD LOC 11 Flood Ratios					Description
			11	10	9	8	1.4	1.0	.8	.7	.6	
J*	1936	1046	817	146	61	22	373	199	114	122	9	Base Run "J" - Actual Flows CFLOD = 1.2
F1	1936	1345	1120	146	58	21	491	184	118	221	105	Recession Forecast Flows Base Run, CFLOD = 1.2
F2	1936	1286	1052	151	61	23	500	198	113	188	52	Recession Forecast Flows Base Run, CFLOD = 1.4
F3	1936	1307	1072	152	61	22	500	191	117	196	68	Extension Forecast Q Base Run, CFLOD = 1.2
17*	1936	1304	1088	140	60	16	349	199	140	271	128	Base Run, 18,000 rel-res 1, w/o op 11, pre-rel

*From Table 4 using measured streamflows instead of forecasted streamflows

Use of Forecasted Flows

If flow forecast models are available, the same type of simulation runs can be made to determine the proper forecast period and contingency factor by using forecasted streamflow for one or more historical floods and one or more ratios of those floods to calculate the average annual damages. In most cases, the best operation should occur where the average annual damages are a minimum. Table 2 illustrates results using forecasted flows. The adopted values for the forecast period (IFCAST) were four 3-hour periods and contingency factors (CFLOD) were assumed as 1.2 and 1.4 respectively. Results indicate forecast flows are generally adequate for locations 8, 9, and 10, but not at location 11.

The adopted values for historical floods (where future flows are known) should not necessarily be the same as the adopted values during flood emergencies since the forecasted flows will not be the same as the observed historical flows. In general, the contingency factor for historical floods should be selected to produce the same AAD as the run using forecasted flows. The number of periods of foresight should be the same regardless of the source of flows.

OPERATIONAL CRITERIA FOR MERRIMACK BASIN

An essential task associated with computer simulation of the Merrimack reservoir system is evaluation of input parameters for computer program HEC-5C to obtain the most desirable operation of the system. Some of the key input parameters are listed in Table 3. Alternative operation criteria were evaluated by determining average annual damages based on spatial and temporal runoff variations associated with the March 1936 and September 1938 flood events. While average annual damage is a useful criteria for selecting operating policies, other factors such as legal and political considerations must also be used in the evaluation. The procedure used in HEC-5C for estimation of average annual damages is as follows:

a. Ratios are determined for application to selected historical flood events that are representative of the full range of frequency of flood occurrence. For example, ratios of 1.4, 1.0, 0.8, 0.7 and 0.5 were applied to inflow and local flow hydrographs for the March 1936 flood to obtain five floods for which system operation was to be simulated. Frequencies associated with peak discharges for the five floods were determined from frequency curves for unregulated flows for locations where damages were to be computed.

b. Reservoir system operation is simulated for each flood, that is, for each set of inflow and local flow hydrographs obtained by applying ratios to hydrographs for a historical event. Frequencies associated with peak "regulated" discharges are assumed to be the same as the "unregulated" frequencies. Figure 4 illustrates natural and regulated frequency curves for Lowell. Points on the regulated frequency curve in figure 4 represent peak discharges resulting from system simulation for a specific set of operation criteria.

c. Damage-discharge relations input to the computer program enable determination of dollar damages corresponding to the peak discharges at each damage center for each flood. Figure 5 illustrates the damage-discharge relation that was used for Lowell. This is an approximate relation that will be updated in the future.

d. Damage-frequency relations are established for each damage center for both natural and regulated conditions. Figure 6 illustrates these relations for Lowell. The computer program integrates the area below the damage-frequency curves to obtain average annual damages.

The average annual damage calculation is influenced by the distribution of runoff for the historical "pattern" storms. Some characteristics of runoff production for the 1936 and 1938 floods can be ascertained from the hydrographs in Figures 7 and 8. These plots show discharge per square mile for inflow to Franklin Falls reservoir, unregulated flow on the Contoocook river at Penacook and uncontrolled local flow (runoff from all areas downstream from nearest upstream reservoirs) at Lowell.

The 1936 flood reflects high runoff production over the entire Merrimack basin. The flatness of the peak for local flow at Lowell reflects the relatively slow responsiveness of this portion of the basin. Lowell is a key location because a large proportion of total damages occurs there. Consequently, an objective in operating the reservoir system is to try to avoid "building on" the local peak at Lowell.

Figure 8 indicates that runoff production from the Contoocook was relatively high for the 1938 event. However, this portion of the Merrimack basin is 'controlled' with four of the five reservoirs.

TABLE 3

INPUT PARAMETERS FOR HEC-5C

1. Number of periods of future (forecasted) flows that will be used to determine reservoir releases.

2. Contingency factors

   These are ratios to be applied to flows in determining reservoir releases; factors are used to account for limited knowledge of future flows beyond the forecast period.

3. Control points for which reservoirs are to be operated.

4. Rate-of-change-of-release criteria for reservoirs.

5. Channel capacity criteria for control points.

6. Minimum release vs reservoir elevation criteria.

7. Pre-release criteria

   a. Whether or not pre-releases will be permitted. A pre-release is a flood-producing reservoir release that is made when the reservoir level is below the top of flood control pool. The release is based on the anticipated flood volume exceeding available capacity.

   b. Reservoir elevation that pre-releases will be geared to.

   c. Magnitude of pre-release permitted (can be specified as a function of reservoir elevation).

RESULTS OF AVERAGE ANNUAL DAMAGE RUNS

In setting up average annual damage runs for HEC-5C, a variety of approaches could be used in selecting floods and flood ratios. For example, one or more ratios of a number of different historical events could be incorporated in a single average annual damage computer run. Another approach is to determine average annual damages for separate sets of ratios applied to individual historical events. Results of these individual average annual damage runs could be weighted depending on how representative individual storms are of the overall flood-producing characteristics of the basin. Two separate sets of simulation runs were made to compute average annual damages for the Merrimack basin. As indicated previously, one set is based on using five ratios of the 1936 event. A second set uses five ratios of the 1938 event.

TABLE 4 AVERAGE ANNUAL DAMAGE SUMMARY

	Run	Flood	Total AAD $1000	Change from Base $1000	Change from Base Percent	AAD Location 11	AAD Location 10	AAD Location 9	AAD Location 8	AAD LOC 11 (Lowell) Flood Ratios 1.4	1.0	.8	.7	.5	Description
A	0	1936	1046	--	--	817	146	61	22	373	199	114	122	9	Base Run[1]
B	1	1936	1074	+ 28	+ 2.7	837	151	63	23	388	211	109*	120*	9	Base Run, w/o RD cards - Res 1 - See Note 2
C	4	1936	1059	+ 13	+ 1.2	841	139*	59*	20*	355*	201	127	148	10	Base Run, RD cards - Res 1 - High Q
D	6	1936	1068	+ 22	+ 2.1	833	149	63	23	384	210	110*	120*	9	Base Run, RD cards - Res 1 - Low Q
E	7	1936	1048	+ 2	+ .2	812*	150	63	23	376	198*	108*	121*	9	Base Run, No Pre-release
F	10	1936	1210	+164	+15.7	994	137*	58*	21*	358	190*	122	214	110	Base Run, w/o operating for loc 11
G	13	1936	1089	+ 43	+ 4.1	839	156	67	27	397	220	95*	118*	9	Base Run, No Pre-release, w/o RD cards
H	14	1936	1207	+161	+15.4	984	141*	60*	22	355*	184*	121	214	110	Base Run, No Pre-release, w/o operating for 11
I	16	1936	1048	+ 2	+ .2	820	145*	61	22	373	199	117	122	9	Base Run, Res 1, level 2 = El 389
Z	17	1936	1304	+258	+24.7	1088	140*	60*	16*	350*	199	140	271	128	Base Run, 18,000 rel-res 1, w/o op Lowell, pre-rel
T	19	1936	1081	+ 35	+ 3.3	858	143*	59*	21*	373	212	128	135	10	Base Run, Pre-Release with Recession
										Flood Ratios 2.0	1.43	1.15	1.00	.70	
A	2	1938	1161	--	--	853	214	67	27	595	186	52	20	0	Base Run
B	3	1938	1164	+ 3	+ .3	853	216	68	27	600	189	48*	16*	0	Base Run, w/o RD cards - Res 1
C															Base Run, RD cards - Res 1 - High Q
D															Base Run, RD cards - Res 1 - Low Q
E	8	1938	1142	- 19	- 1.7	837*	212*	65*	28	592*	174*	51*	20	0	Base Run, No Pre-release
F	9	1938	1193	+ 32	+ 2.8	915	195*	60*	23*	566*	160*	94	93	2	Base Run, w/o operating for loc 11
G	11	1938	1168	+ 7	+ .6	853	217	67	31	604	186	47*	16*	0	Base Run, No Pre-release, w/o RD cards
H	12	1938	1192	+ 31	+ 2.7	915	194*	59*	24*	565*	160*	94	94	2	Base Run, No Pre-release, w/o operating for 11
I	15	1938	1187	+ 26	+ 2.2	891	201*	68	27	483*	193	86	111	18	Base Run, Res 1, level 2 = El 389
Z	18	1938	1200	+ 39	+ 3.3	922	195*	60*	23*	566*	161*	95	97	3	Base Run, 18,000 rel-res 1, w/o op Lowell, pre-rel

NOTES: 1 Base Run - Operating for loc 11, RD cards as shown note 2, Pre-releases, Contingency factor = 1.2, Forecast Period = 12 hours.

2 RD Cards for Res 1 specify min emergency releases as function of reservoir storages.

% F.C. Stg	32	49	75	100 (El 389)	109 (El 394)	112 (El 396)
Base Run Q	0	6,000	12,000	18,000	18,000	30,500
High Q	0	18,000	18,000	18,000	18,000	30,500
Low Q	0	0	6,000	18,000	18,000	30,500

* Improvement over base conditions.

Results of the average annual damage runs are summarized in Table 4. The "base" runs, labeled A in Table 4, were made with HEC-5C input parameters specified as follows.

a. A forecast period of 12 hours; that is, discharges up to 12 hours in the future were considered in determining reservoir releases.

b. A contingency factor of 1.2 was applied to local flows for purposes of reservoir release determination.

c. The reservoir system was operated for all control points shown in figure 1.

d. Rate-of-change of release criteria were specified so as not to be a constraint on releases from Franklin Falls reservoir.

e. Fixed channel capacities were specified for all control points based on information supplied by the New England Division.

f. A table of values for minimum permissible release as a function of reservoir elevation was specified for Franklin Falls reservoir as shown on Table 4 (note 2).

g. Pre-releases were permitted; at Franklin Falls reservoir, pre-releases were made if inflows to the reservoir during the 12-hour forecast period would cause the reservoir level to rise above elevation 394 (5 feet above the spillway crest).

The discharge-damage relation for Lowell (Figure 5) that was input to HEC-5C had a maximum discharge ordinate of 180,000 cfs. Because discharges larger than 180,000 cfs were used in the damage analysis, the discharge-damage relation was extrapolated by the computer program as shown in Figure 5. The effect of using the alternative extrapolation, also shown in Figure 5, on average annual damages was less than 2% for both natural and regulated flows.

Operation criteria for average annual damage runs other than the base runs are summarized in **Table 5**. Some observations pertaining to results of the average annual damage simulation runs are as follows:

a. Average annual damages based on floods patterned after the March 1936 flood are of approximately the same magnitude as average annual damages based on floods patterned after the September 1938 flood.

b. Operational criteria used for the base run (run A) produced the lowest average annual damages for floods patterned after the March 1936 flood; operational criteria that does not utilize the pre-release option (run E) produced the lowest average annual damages for floods patterned after the September 1938 flood and only slightly more damage for the 1936 flood.

c. Of the order of 75% of the total average annual damages occurs at Lowell on the basis of the approximate stage-damage relationship for that location.

d. Damages associated with very large floods account for a major proportion of average annual damages; this is illustrated in figure **9** which shows the relation between percent of average annual damage and recurrence interval at Lowell for the base run for floods patterned after the March 1936 flood, (e.g., 45% of average annual damages occur under regulated conditions from floods having a recurrence interval of 300 years or greater).

e. Significant discharge and damage reduction at the 1000 year flood level is due to the surcharge storage available in the reservoirs due to the limited discharge capacity of the uncontrolled spillways since the flood control storages were exceeded very early in the largest floods.

TABLE 5

OPERATION CRITERIA FOR SIMULATION RUNS

Run	Criteria
B	Same as for base run, except minimum release not specified for Franklin Falls reservoir.
C	Same as for base run, except relatively large minimum releases were specified (as a function of reservoir elevation) for Franklin Falls reservoir (see note 2 of Table 4 for values).
D	Same as for base run, except relatively small minimum releases were specified (as a function of reservoir elevation) for Franklin Falls reservoir.
E	Same as base run, except pre-releases were not made.
F	Same as base run, except reservoir system was not operated for Lowell.
G	Same as base run, except pre-releases were not made and minimum releases for Franklin Falls reservoir were not specified.
H	Same as base run, except pre-releases were not made and reservoir system was not operated for Lowell.
I	Same as base run, except reservoir elevation of 389 was used at Franklin Falls reservoir for pre-release determination.
J	Same as base run, except minimum releases of 18,000 cfs from Franklin Falls reservoir were specified, system was not operated for Lowell.
K	Same as base run, except the pre-release option was modified to include volume of recession of hydrograph past the period of forecast.

OUTPUT DISPLAYS

Batch Mode vs Interactive Mode

Execution of a computer program in batch mode requires that all input for a computer run be supplied to the computer prior to program execution. An interactive-mode execution is where the user can interact with the computer during the execution of a job. As used in the application described herein, the interactive mode is used to selectively print out data from an output file of the system operation that has been generated in batch mode. This enables the user to review any portion of the output that he desires. The output file can be permanently saved and interrogated at future times from one or more computer terminal sites.

While output displays from high-speed line printers used in the batch mode can provide any amount of output desired, the level of output must be specified prior to making the computer run. Presently, after a run has been made, output not previously requested can only be obtained by making another complete simulation run. An alternative method would be to save the output file and print in batch mode by writing a special program. If the turn-around time is adequate (say less than 30 minutes) and the program execution cost is small, then the batch mode is the best way to get the necessary output assuming that a high-speed printer is available. Where batch mode turn-around times are long, or high-speed printers are not readily available, the slow-speed terminals can be an effective way of obtaining a limited amount of information rapidly. After looking at selected data through the slow-speed terminal the output can then be directed to a high-speed printer if desired.

High-Speed Printer Output

The subroutine PROUT in HEC-5C is used to print output for the high-speed printer (batch mode), slow-speed teletype terminal (interactive mode) and cathode ray tube terminals (interactive mode). All output devices can be used to print any combination of types of output (see samples on Figures 10-11) except for the graphical plots (see Figure 17) available with cathode ray tube terminals. Printer plots (Figure 12) can be requested by the batch mode printers. The types of output that can be requested are as follows:

OUTPUT DESCRIPTION

* Input Card Listing
* Input Flows
* Input Data for System Specification
* Output - Normal Sequential (by control point)
* Output - Reservoirs - by Period
* Output - Reservoir Releases - by Period
* Output - Reservoir Regulation Summary - Single Flood
* Output - Reservoir Regulation Summary - All Floods
* Output - Hydrologic Efficiencies
* Output - Computer Check for Possible Errors
* Batch Economic Summary
* User Designed Output - Results by Period
* User Designed Output - Summary

Interactive Terminal

While the batch mode selects desired output by input cards, the slow-speed terminal asks the user questions, as illustrated in figure 13, concerning what type of function (see Figure 14) should be performed next (output from operation, modify input deck HEC-5C, etc.). If operational output is desired, the type of data (option number of Figure 14) and possibly variable codes and locations, on Figure 15, as well as the output mode (plot, tabulate or save on tape) are required to be specified to the computer. Data can be input to the computer through the terminal by depressing the appropriate terminal keys or (for certain CRT's) by touching an electronic pen to the appropriate instruction (see Figure 15) on a selection list (menu) which lies on a graphic tablet. After the tabulated output (see Figure 16) or plotted results (Figure 17) are obtained, additional data can also be selected and printed as desired. Any combination of data types (reservoir outflow, storage, elevation, downstream flow, etc.) for any location can be tabulated or plotted. Depending on the size of the paper (or screen) up to 10 different items can be tabulated side by side and up to 5 different time dependent variables can be plotted on a single graph. Two different scales can be used for the plots as shown on Figure 17 where the inflow and outflow are plotted on the left scale (discharge) and the reservoir level on the right scale. When all of the desired output has been displayed, the data can be transferred to the line printer at a nearby batch location.

Updating and Reoperation of System

In the batch mode, data cards for the forecast program are read by the card reader and an output file of the forecasted basin-wide flows is obtained. The HEC-5C data cards can be loaded at the same time or at a later time, and the system simulation is performed for the duration of the forecasts and the output is directed to the line printer. Any desired change in the forecast or operation requires a few new data cards and rerunning either the operation model or both the forecast and operation models. Where adequate computer turn-around is available this process can be accomplished in 30 minutes or less.

With a slow-speed terminal the same cycle can be accomplished by using the keyboard or the electronic pen. In addition, high quality graphical displays of the status of the system can be obtained. After the operational data has been displayed, function 5 of figure 14 can be selected and an interactive program called REVISE will allow the revision and/or execution of the data files of either the forecast or operation models. It is then possible to once again display selected output. A diagram of this process is shown as figure 18.

COMPUTER SYSTEMS REQUIRED FOR REAL-TIME OPERATION

In order to reap the benefits of the real-time operation tools described previously, access to digital computer equipment is a necessity. This access may take several different forms depending on characteristics of the data acquisition system, the forecasting and operation models used, and available communications equipment. Real-time water resource operations may make use of only in-house equipment, only remote site equipment, or some combination of each.

The first basic function to be accomplished is that of acquisition of available data. Information required may include observer collected data reported by voice communications and analog or digital signals received by appropriate equipment. Such information may be received over dial-up phone lines, dedicated phone lines, radio repeating links, or satellite repeating links.

When the volume of data is great it is obviously desireable to have the data recorded directly on a medium that may be read by machines. Even more desireable is to have the whole data acquisition function occur under control of a mini-computer. If this is done the mini-computer may also perform several functions such as, error checking, data reduction, permanent logging, report generation, etc. In most cases a mini-computer dedicated to such functions would be required.

Once the data in reduced form (i.e., discharge, precipitation, etc.) is available, the forecast may be performed. Computer equipment for this function will vary depending on the size of the forecast program. Some forecasting models may execute well on the same mini-computer used for data acquisition. Others would require such extensive reprogramming in order to operate on a mini-computer that other alternatives would be desirable. In some cases additional large scale in-house computers may be available. In most cases such facilities will not be directly available to district offices. The use of larger capacity remote site computing becomes very attractive in such a situation. A portion of the reduced data may be sent to the remote site directly by the mini-computer, or read in from paper tape, or magnetic cassette. The forecast may then be performed at the remote site with results returned to the local site and/or saved for future reference at the remote site.

A similar situation exists for executing large system operation programs. Again remote site computing offers a cost effective solution. The forecasted flows may be passed on to the operation model through common access to a data file. Results from the operation routine may then be returned to the local site, or be held for future reference. Display of the results may be performed by the in-house mini-computer, or be handled directly from the remote site.

Problems of using a remote site computing center around two factors; (1) guaranteed access to facilities twenty-four hours per day, three hundred and sixty-five days per year, and (2) reliable communications and power supply even under extremely adverse conditions. It is impossible to guarantee access to any one facility at all times. It is possible however, to have access to several remote facilities and thus provide as high a level of "guaranteed" access as deemed necessary. Communications and power supply which involve ground lines are quite vulnerable to interruption during major storm activity. Backup power supply may be easily supplied by emergency generators.

Backup communications based on a ground network may require an individual to physically carry necessary data to an alternate input site outside the area affected by the communications interruption. An attractive backup for communications which is rapidly being developed is to communicate to the remote site by way of a satellite link.

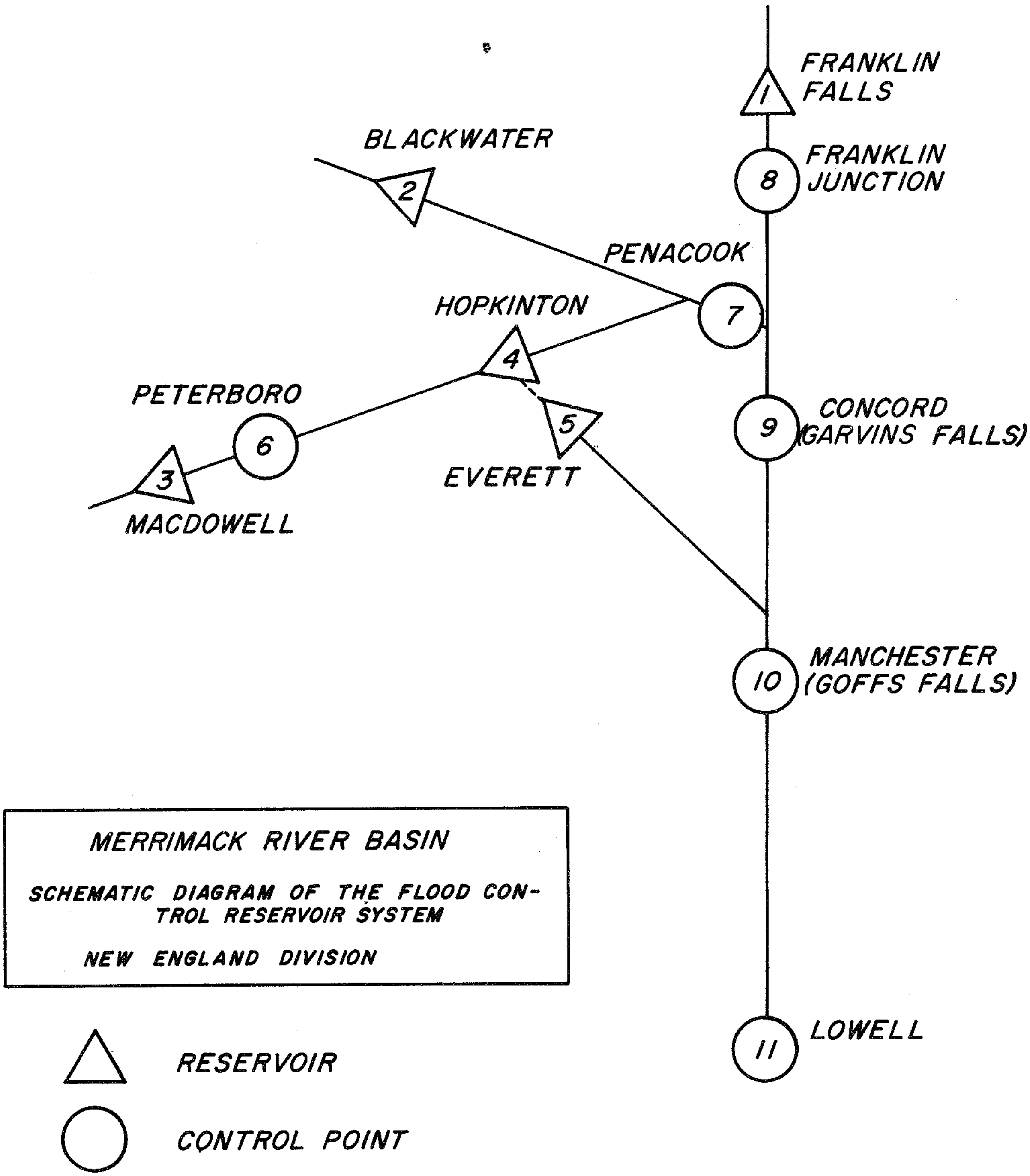
The use of large scale machines for the forecasting and operations aspects of real-time operation are attractive because of the ease of updating or improving the models used. Other modeling techniques may be quickly compared and substituted for those currently in use. When mini-computers are used for executing large programs, changes to the program often entail major restructuring of overlays.

FUTURE WORK

It is planned to implement the forecast-operation-display capabilities described in this paper at the Control Center of the New England Division in the near future. The next step will be to interface these capabilities with the existing automated data collection system.

Alternative forecasting techniques other than the streamflow extrapolation procedure described herein will be tested. Application of the computer program Streamflow Synthesis and Reservoir Regulation (SSARR) developed by the North Pacific Division is anticipated.

The entire procedure for real-time simulation will be thoroughly tested and "fine-tuned" once it is operational at the Control Center.



Paper 12

FIGURE 1

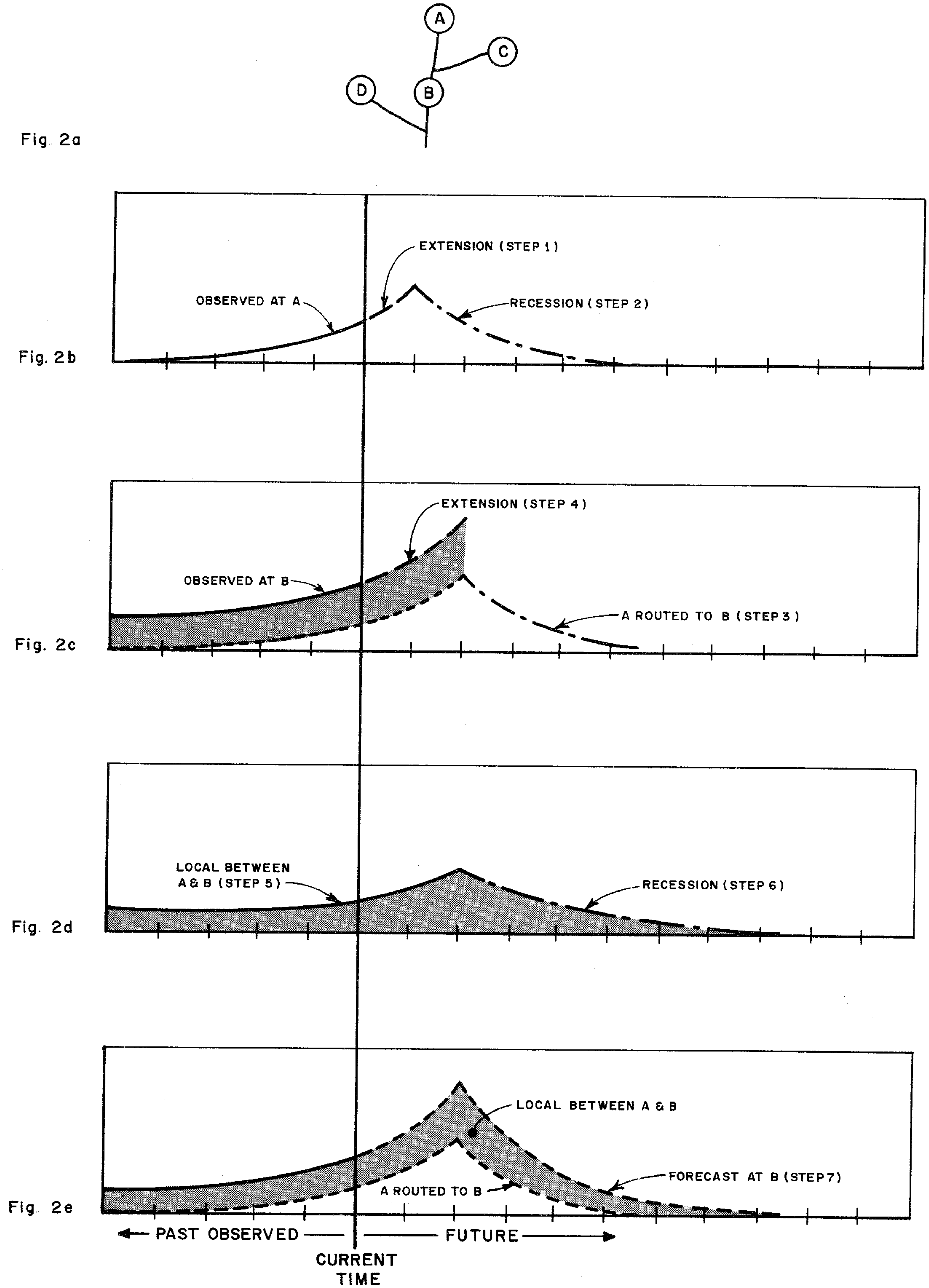
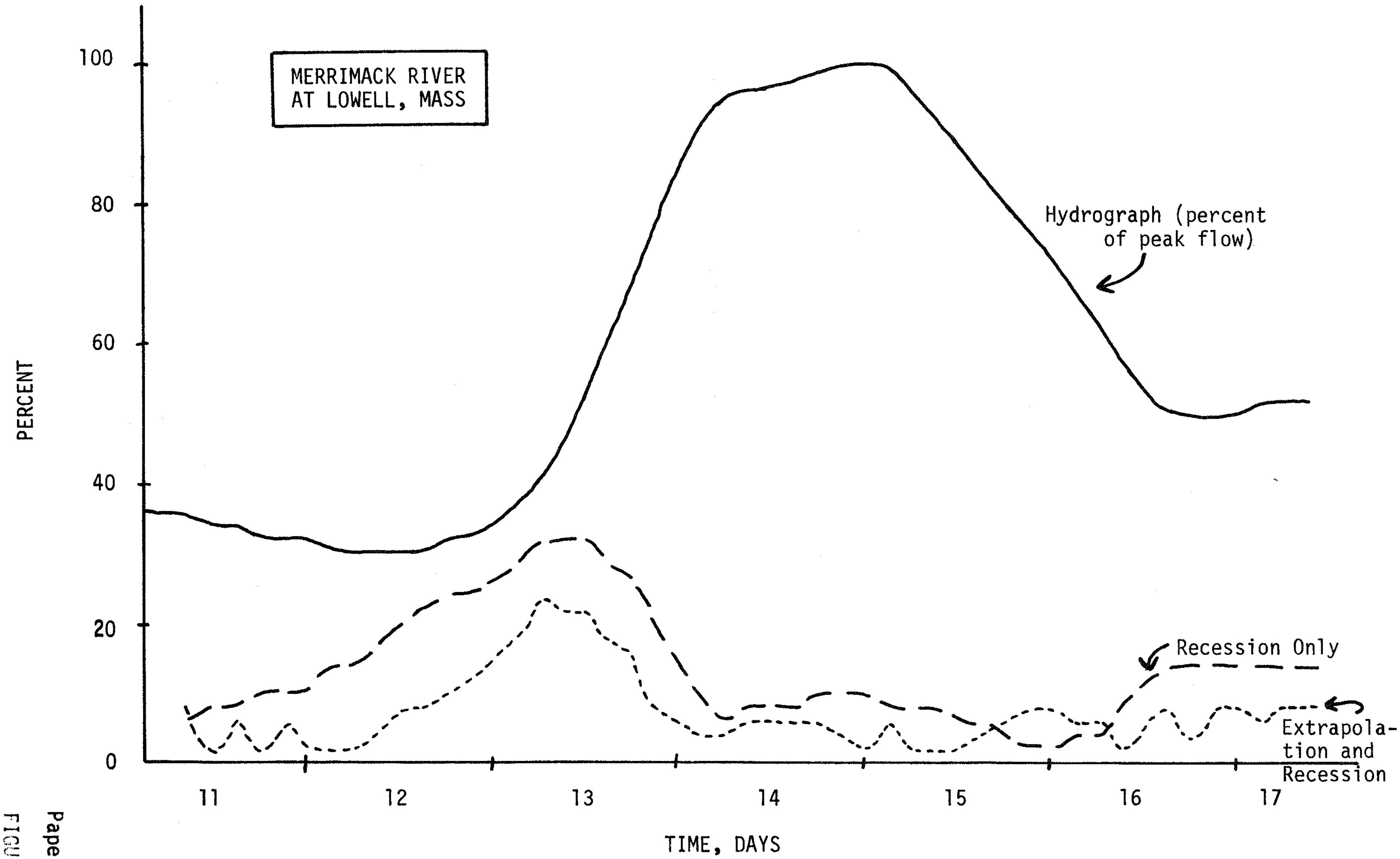


FIGURE 2

Paper 12

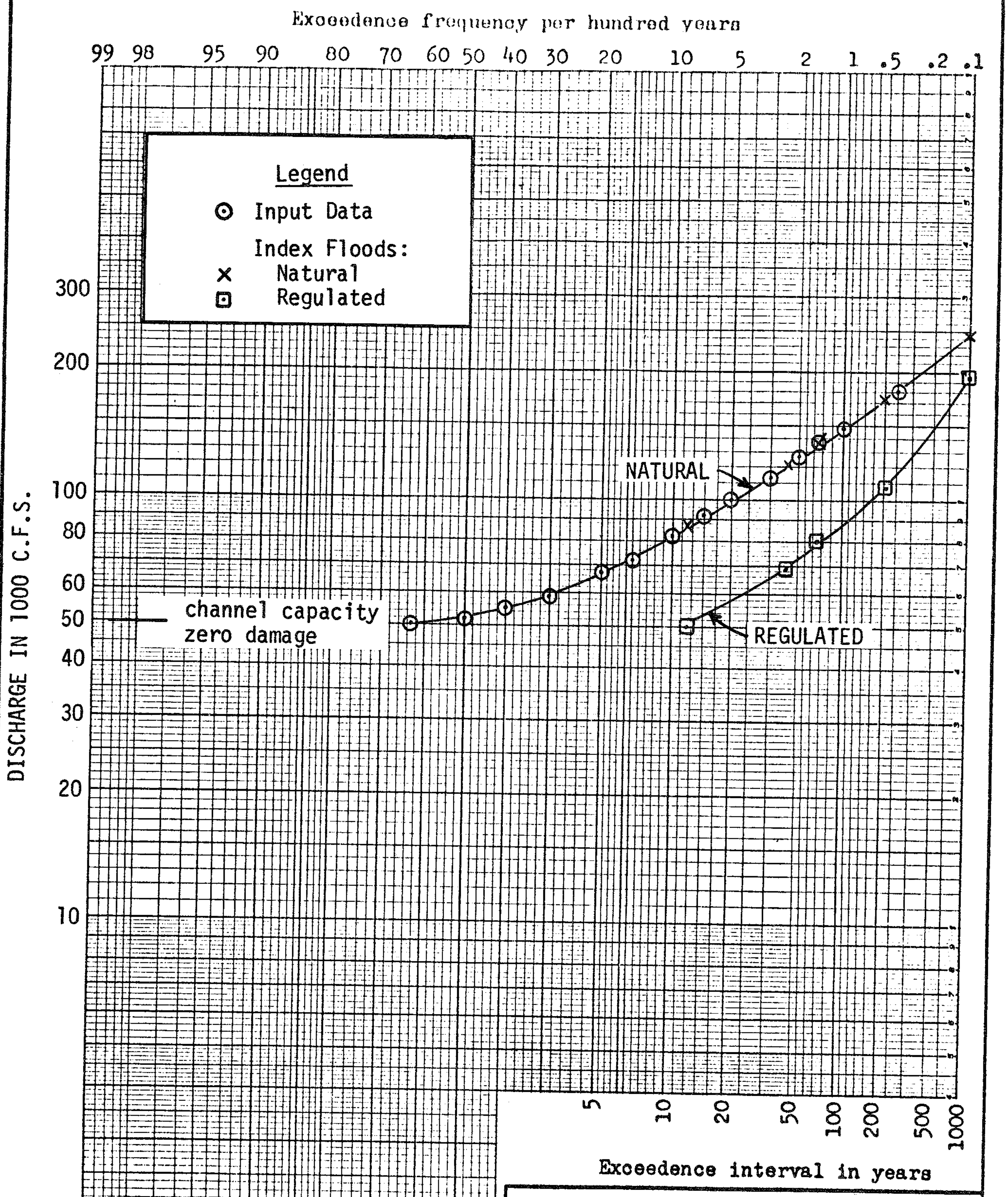


<u>Average Relative Forecast Error Over Four Future Periods</u>

Example: At midnight on the 13th, while on the rising limb, the discharge over a span of four future periods (until noon on the 14th) was predicted within 18% by recession only or within 8% by extrapolation and recession.

Paper 12

FIGURE 3

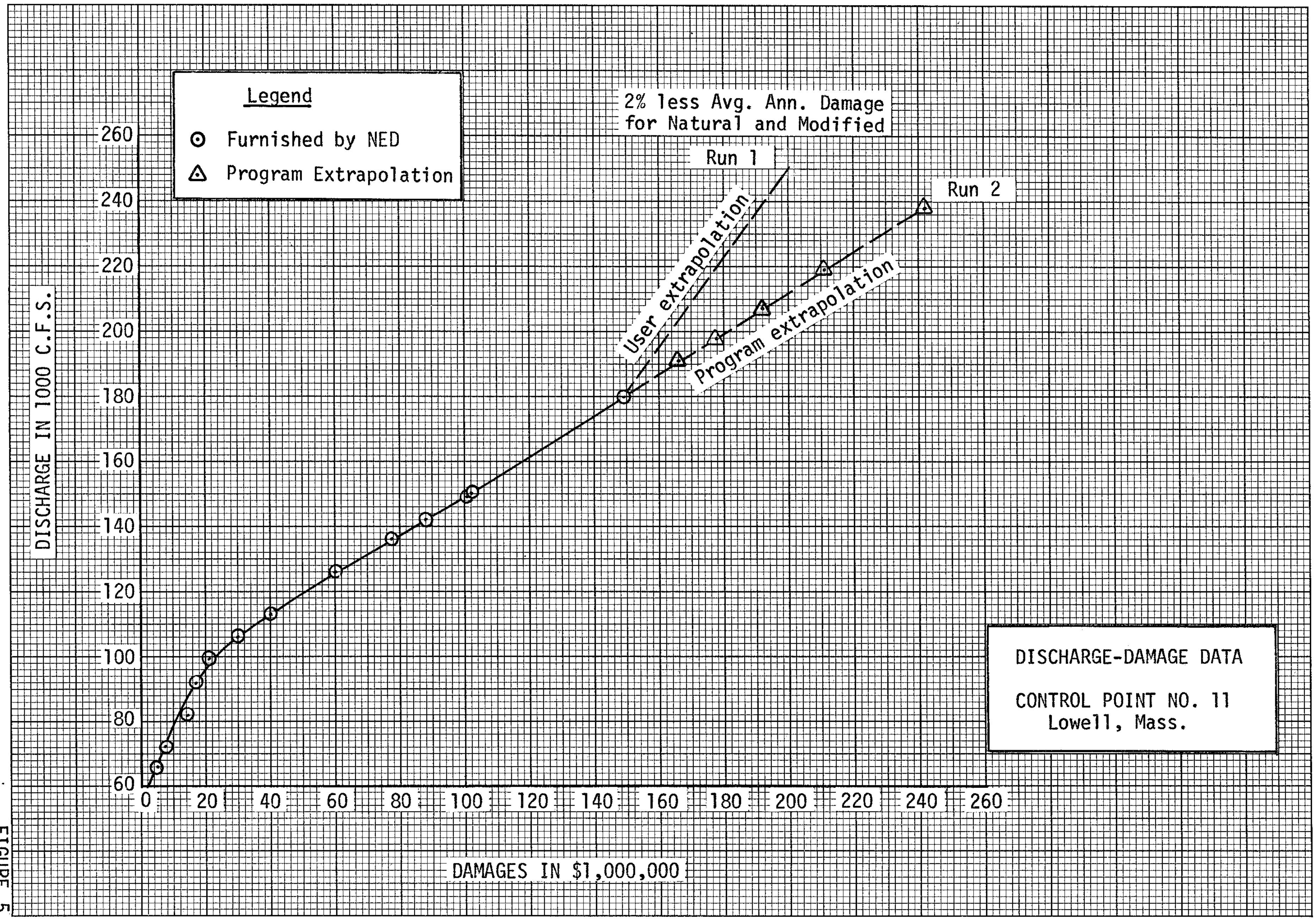


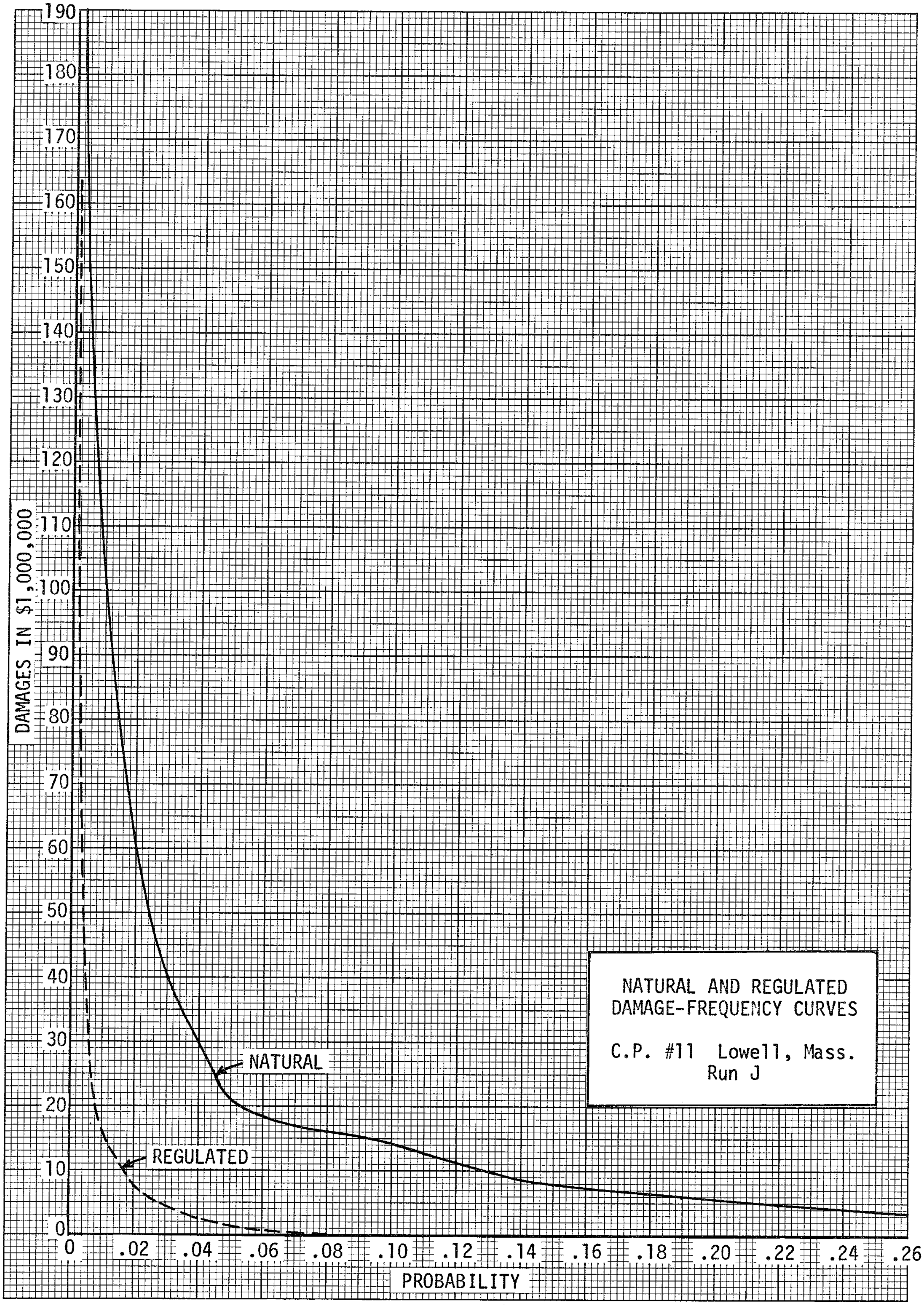
NATURAL AND REGULATED
DISCHARGE-FREQUENCY CURVES

C.P. #11 Lowell, Mass.
Run J

Paper 12

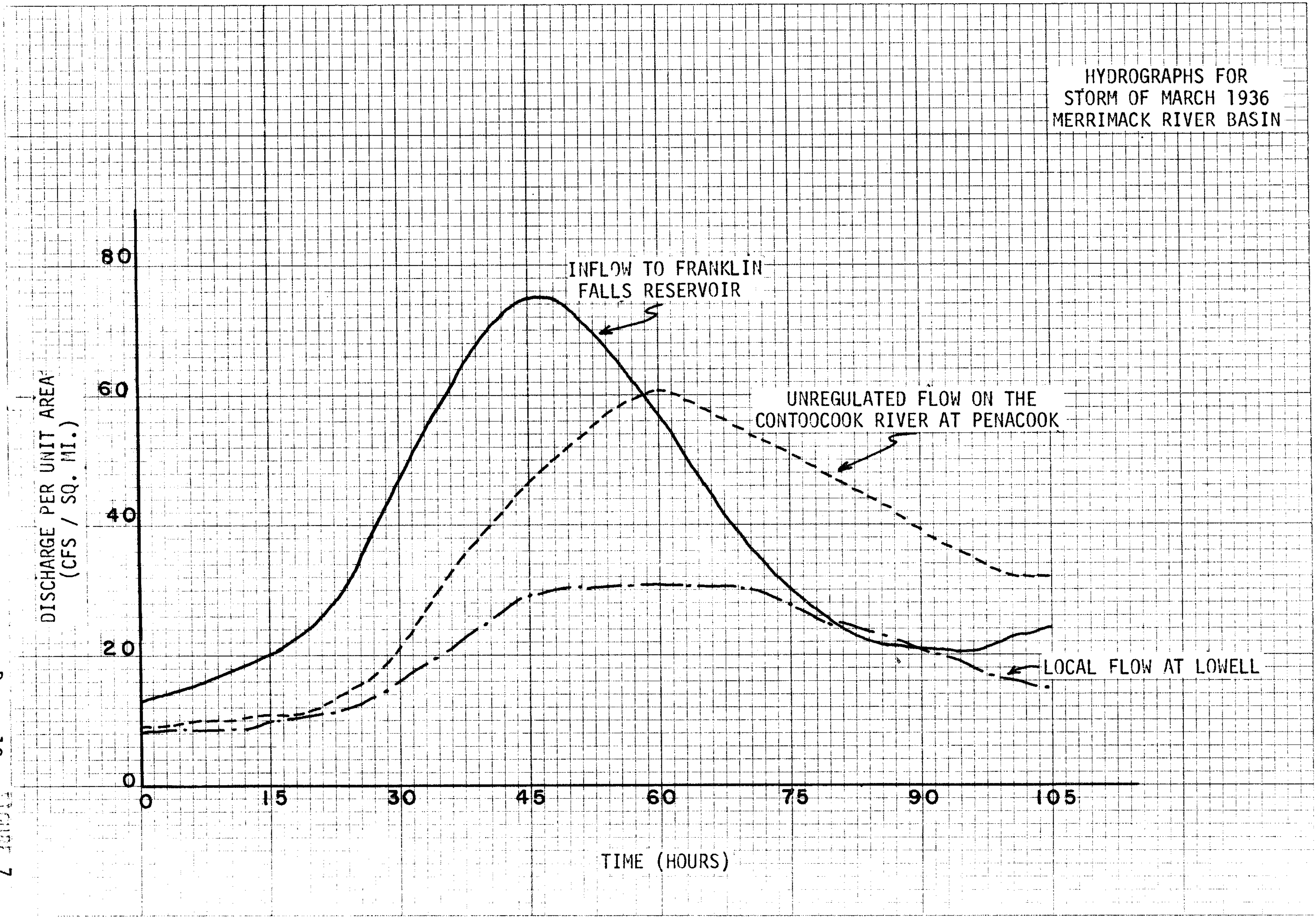
FIGURE 4


Legend
Furnished by NED
Program Extrapolation
2% less Avg. Ann. Damage
for Natural and Modified
Run 1
Run 2
User extrapolation
Program extrapolation
DISCHARGE IN 1000 C.F.S.
DAMAGES IN $1,000,000
60
80
100
120
140
160
180
200
220
240
260
0
20
40
60
80
100
120
140
160
180
200
220
240
260
DISCHARGE-DAMAGE DATA
CONTROL POINT NO. 11
Lowell, Mass.



DAMAGES IN $1,000,000
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
.02
.04
.06
.08
.10
.12
.14
.16
.18
.20
.22
.24
.26
PROBABILITY
NATURAL
REGULATED
NATURAL AND REGULATED
DAMAGE-FREQUENCY CURVES
C.P. #11 Lowell, Mass.
Run J

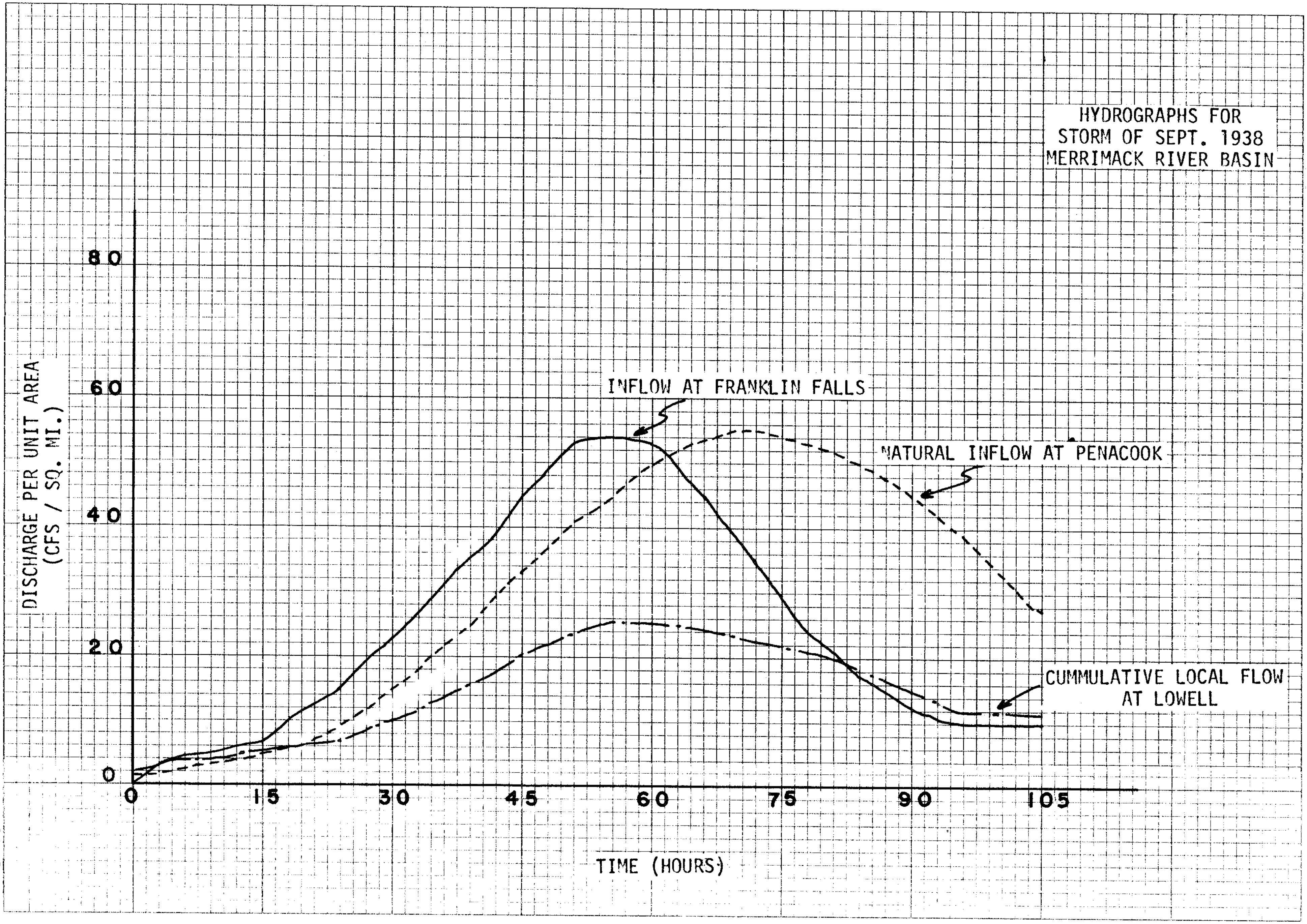
FIGURE 6



Paper 12

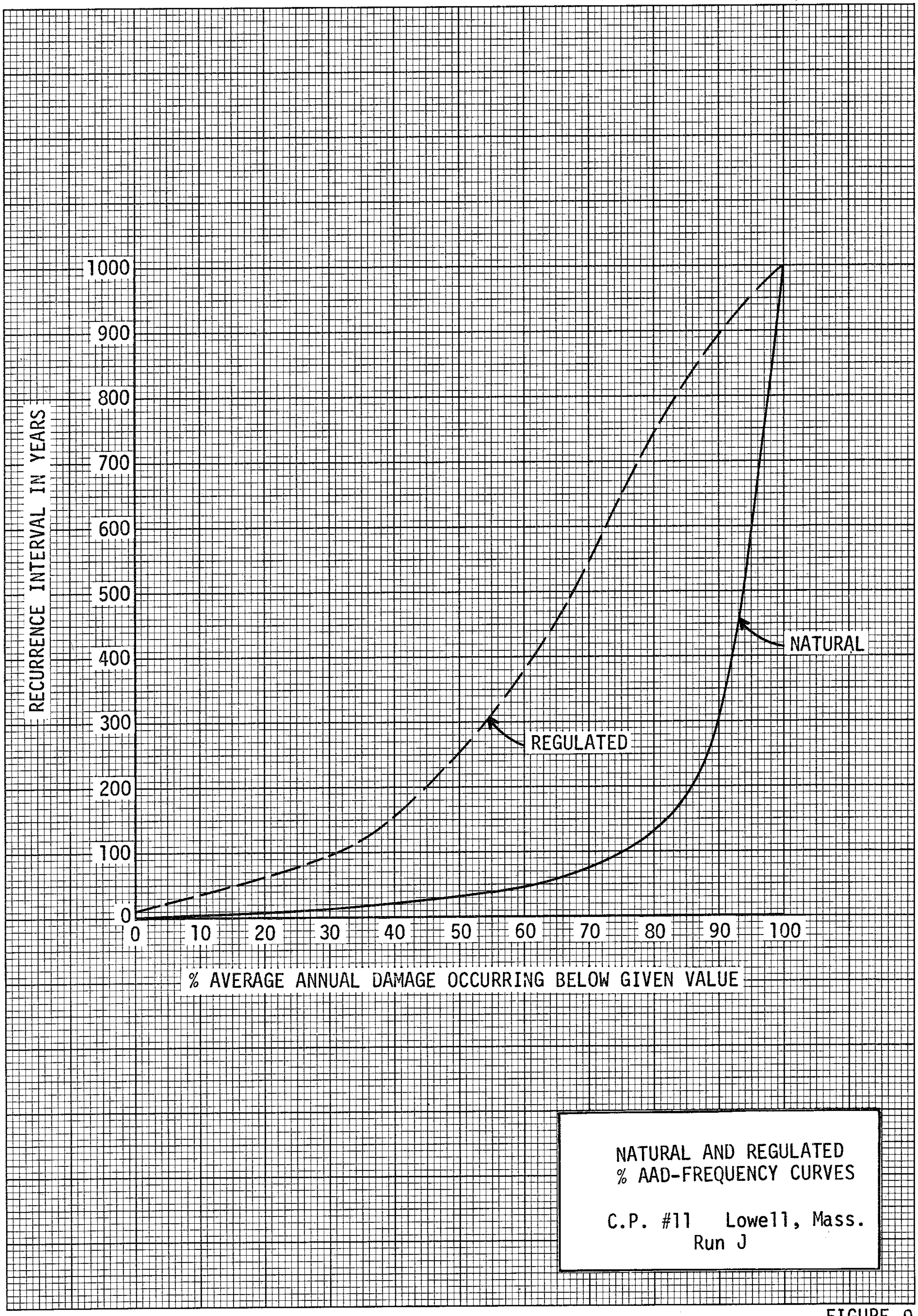
FIGURE 7

46 0702



Paper 12

FIGURE 8



RECURRENCE INTERVAL IN YEARS
0
100
200
300
400
500
600
700
800
900
1000
0
10
20
30
40
50
60
70
80
90
100
% AVERAGE ANNUAL DAMAGE OCCURRING BELOW GIVEN VALUE
REGULATED
NATURAL
NATURAL AND REGULATED
% AAD-FREQUENCY CURVES
C.P. #11 Lowell, Mass.
Run J

FIGURE 9

SAMPLE HEC-5C OUTPUT
SINGLE FLOOD SUMMARY

SINGLE FLOOD SUMMARY COPY= 1
COMPUTATION INTERVAL IN HOURS= 3

MERRIMACK RIVER BASIN - FLOOD CONTROL OPERATION
1936 STORM DAMAGE IN THOUSANDS OF DOLLARS
5 RESERVOIRS

***** FLOOD NUMBER 2 *****

LOC=		NONRESERVOIRS	MAX REG Q	MAX NAT	MAX UNC	Q BY RES	NO FLOODS TOTAL	VOL FLOODS TOTAL	NO F… FR…
6	6	PETERBOROUGH	5963.	8881.	5312.	651.	34.	71513.	
7	7	PENACOOK	25212.	46800.	9425.	15787.	15.	99079.	
8	8	FRANKLIN JUNC	63748.	83000.	7200.	56548.	30.	460102.	
9	9	GARVINS FALLS	60440.	121068.	18990.	41450.	28.	571844.	
10	10	GOFFS FALLS	73486.	143606.	45971.	27515.	29.	578207.	
11	11	LOWELL	109208.	172939.	88519.	20688.	33.	1414560.	

LOC=		RESERVOIRS	STOR1	MAX STG	MAX LEVEL	EXCEEDED TOP F.C. 1ST PER.	LAST PER.	MAX INFLOW
3	3	MACDOWELL	1453	16074	2.102	24	49	3950
4	4	HOPKINTON	24000	78987	2.022	38	49	17827
2	2	BLACKWATER	9220	45999	2.000	49	49	10850
1	1	FRANKLIN FALL	50000	184953	2.262	24	49	75800
5	5	EVERETT	10000	94028	2.035	37	49	23963

MAX SYSTEM STORAGE= 420041

FIGURE 10 Paper 12

SAMPLE HEC-5C OUTPUT USER DESIGNED

USER DESIGNED OUTPUT

LOC NO=						3.	4.	2.	1.	5.
PER	HR	DY	MO	YR	DW	OUTFLOW	OUTFLOW	OUTFLOW	OUTFLOW	OUTFLOW
1	3	1	0	0	1	650.00	7000.00	2253.20	0.00	0.00
2	6	1	0	0	1	0.00	7000.00	2237.53	1200.81	0.00
3	9	1	0	0	1	0.00	7000.00	2222.10	1640.56	750.00
4	12	1	0	0	1	0.00	7000.00	2206.89	6140.56	903.79
5	15	1	0	0	1	0.00	7000.00	2191.91	10640.56	902.28
6	18	1	0	0	1	0.00	7000.00	2177.16	15140.56	901.79
7	21	1	0	0	1	0.00	0.00	0.00	17059.57	901.84
8	24	1	0	0	1	0.00	0.00	0.00	12559.57	0.00
9	3	2	0	0	2	0.00	0.00	0.00	8059.57	0.00
10	6	2	0	0	2	0.00	0.00	0.00	3559.57	0.00
11	9	2	0	0	2	0.00	0.00	0.00	3345.24	0.00
12	12	2	0	0	2	0.00	0.00	0.00	4181.03	0.00
13	15	2	0	0	2	0.00	0.00	0.00	5080.57	0.00
14	18	2	0	0	2	0.00	0.00	0.00	6037.64	0.00
15	21	2	0	0	2	0.00	0.00	0.00	6753.43	0.00
16	24	2	0	0	2	0.00	0.00	0.00	7608.50	0.00
17	3	3	0	0	3	0.00	0.00	0.00	8737.69	0.00
18	6	3	0	0	3	0.00	0.00	0.00	4237.69	0.00
19	9	3	0	0	3	173.56	0.00	0.00	15338.09	0.00
20	12	3	0	0	3	652.44	0.00	0.00	18433.30	0.00
21	15	3	0	0	3	650.00	0.00	0.00	19598.83	0.00
22	18	3	0	0	3	650.00	0.00	0.00	20878.23	0.00
23	21	3	0	0	3	650.00	0.00	0.00	22960.18	0.00
24	24	3	0	0	3	650.00	0.00	0.00	30545.48	0.00
25	3	4	0	0	4	650.00	0.00	0.00	40829.16	0.00
26	6	4	0	0	4	650.00	0.00	0.00	55388.84	0.00
27	9	4	0	0	4	650.00	0.00	0.00	60728.52	0.00
28	12	4	0	0	4	650.00	0.00	0.00	60548.33	0.00
29	15	4	0	0	4	650.00	0.00	0.00	57320.74	0.00
30	18	4	0	0	4	650.00	1720.98	0.00	52793.29	0.00
31	21	4	0	0	4	650.00	4888.30	0.00	47590.02	582.51
32	24	4	0	0	4	650.00	7784.88	0.00	42397.26	1338.48
33	3	5	0	0	5	650.00	9422.69	0.00	38125.96	1500.00
34	6	5	0	0	5	650.00	10732.69	0.00	33811.98	1500.00
35	9	5	0	0	5	650.00	9184.52	0.00	30500.00	1500.00
36	12	5	0	0	5	650.00	11031.11	0.00	30500.00	1500.00
37	15	5	0	0	5	650.00	11960.31	0.00	30650.23	1500.00
38	18	5	0	0	5	650.00	14171.50	0.00	22852.04	1516.55
39	21	5	0	0	5	650.00	14745.91	0.00	22520.00	1536.38
40	24	5	0	0	5	650.00	15243.95	0.00	22976.00	1547.42
41	3	6	0	0	6	650.00	15523.89	0.00	23400.80	1556.16
42	6	6	0	0	6	650.00	15692.83	0.00	19503.20	1560.89
43	9	6	0	0	6	650.00	15746.59	57.47	24860.00	1561.97
44	12	6	0	0	6	650.00	15688.07	781.98	24860.00	1559.13
45	15	6	0	0	6	650.00	15528.73	1307.58	24860.00	1552.79
46	18	6	0	0	6	650.00	15291.80	1307.58	24860.00	1543.86
47	21	6	0	0	6	650.00	14996.47	1307.58	24860.00	1533.04
48	24	6	0	0	6	650.00	14655.65	1307.58	24500.00	1520.74
49	3	7	0	0	7	650.00	14275.82	1307.58	23700.00	1513.40

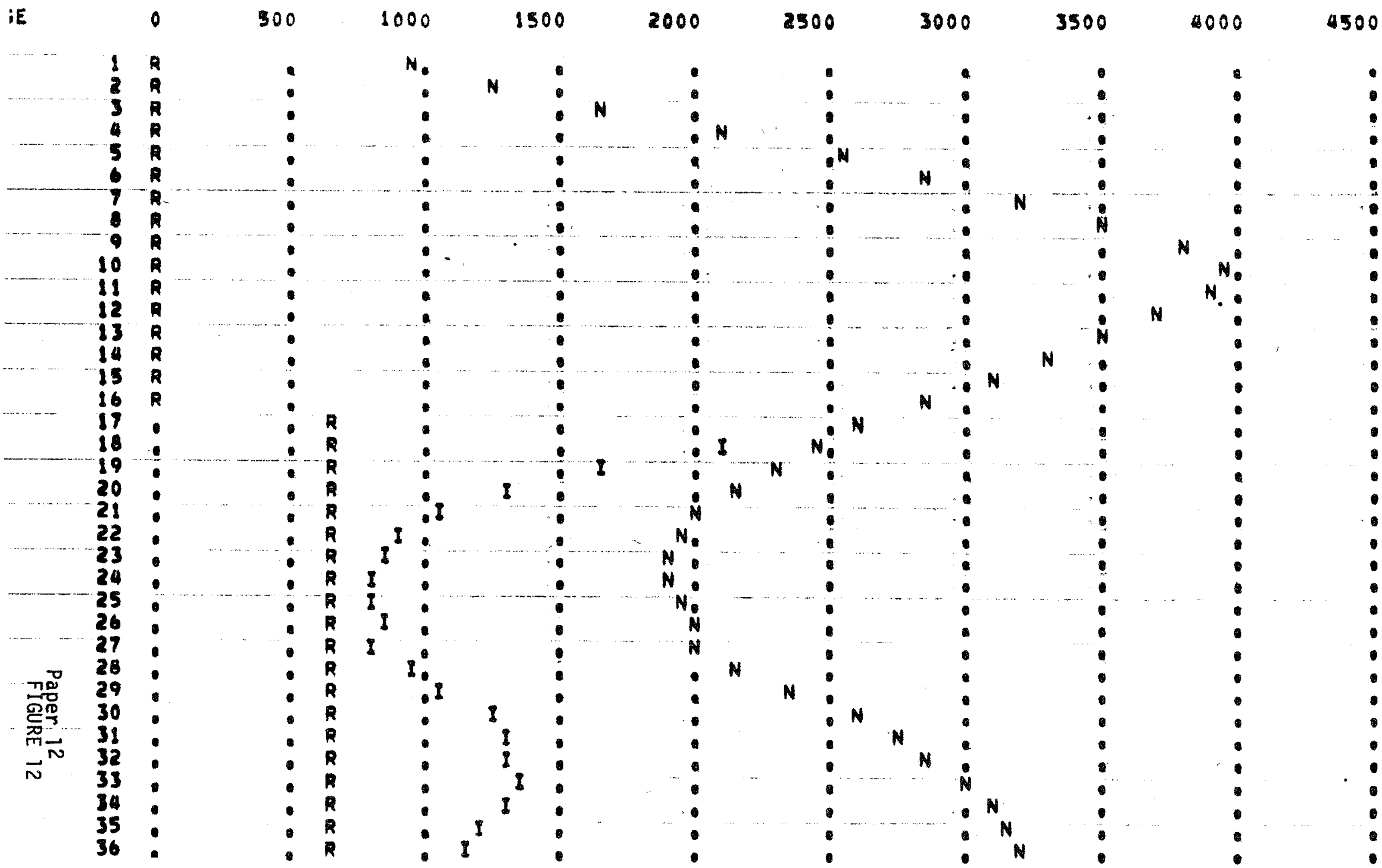
FIGURE 11
Paper 12

POINTS (BY PRIORITY)=R=REGULATED,N=NATURAL,L=LOCAL(CUM),I=INFLOW

MAX= 650 3950 3950 =1000

SAMPLE HEC-5C OUTPUT
PRINTER PLOTS

3 MACDOWELL MX= 3 CH CAP= 650 HOUR=18,DAY=15,MON= 0,YEAR=1



Paper 12
FIGURE 12

INTERACTIVE HEC-5C OUTPUT
"QUESTIONS AND REPLIES"

```
'prout
specify interactive mode and file name   1 ned
********

enter a  ?  when you are uncertain a required response
key the terminal attention key (ESC) once to return to a major branch point or
key the terminal attention key twice in succession to return to the system

********

data file available
 key in  1  to enter data via tablet 1

          * * * BRANCH POINT * * *

key in desired function

2

function no. 2    user determined output format. . .

select period or summary type results  1.  -1.  -11  or ?

1

enter combination(s) or ?. then return carriage

   9.02     9.04     9.01

select output mode p. t. s. or ?

t

copy page and return(
```


FIGURE 13

INTERACTIVE HEC-5C MENU
FUNCTION SELECTION AND FUNCTION 1 OPTIONS
(PRE-FORMATTED OUTPUT TYPES)

YES	NO	?	DELETE ENTRY	DELETE LINE	KEYBOARD	ESCAPE ESC	CARRIAGE RETURN

* * * B R A N C H P O I N T * * *

FUNCTION	DESCRIPTION
1	DISPLAY DATA AND/OR RESULTS USING STANDARD OPTIONS
2	DISPLAY RESULTS BY USER SPECIFIED CONTROL POINTS AND VARIABLE CODES
3	TRANSFER THE MOST RECENT TIME-SHARE REQUESTED OUTPUT TO THE LINE PRINTER
4	TERMINATE PROCESSING
5	MODIFY HEC-5 INPUT DATA DECK USING THE REVISE PROGRAM
6	DISPLAY OUTPUT WITH BATCH OPTIONS (INVOKE THIS FUNCTION ONLY ONCE)

FUNCTION NO. 1

OPTION	DESCRIPTION
1	display input 1=basic input data
2	display input 2=flow data
4	display input 4=summary of input
5	display normal sequential output by control point

FIGURE 14

INTERACTIVE HEC-5C MENU
FUNCTION 2 (USER DESIGNED OUTPUT)

FUNCTION NO. 2

1	if you want data by period
-1	if you want summary data for reservoirs
-11	if you want summary data for nonreservoirs

VARIABLE CODES

.01	cum local q	.07	min required q	.13	level	.19	q by us res divs
.02	natural flow	.08	shortage	.14	equivalent level	.20	flood by res
.03	diversion q	.09	inflow	.15	power required	.21	evaporation
.04	regulated flow	.10	outflow	.16	power generated	.22	elevation avg
.05	min desired flow	.11	eop storage	.17	channel capacity	.23	pow. shortage
.06	shortage	.12	case=loc typ	.18	q space avail.		

SUMMARY CODES

SUM	MAX	MIN	PD. OF MAX	AVG

SELECT OUTPUT MODE

P	T	S

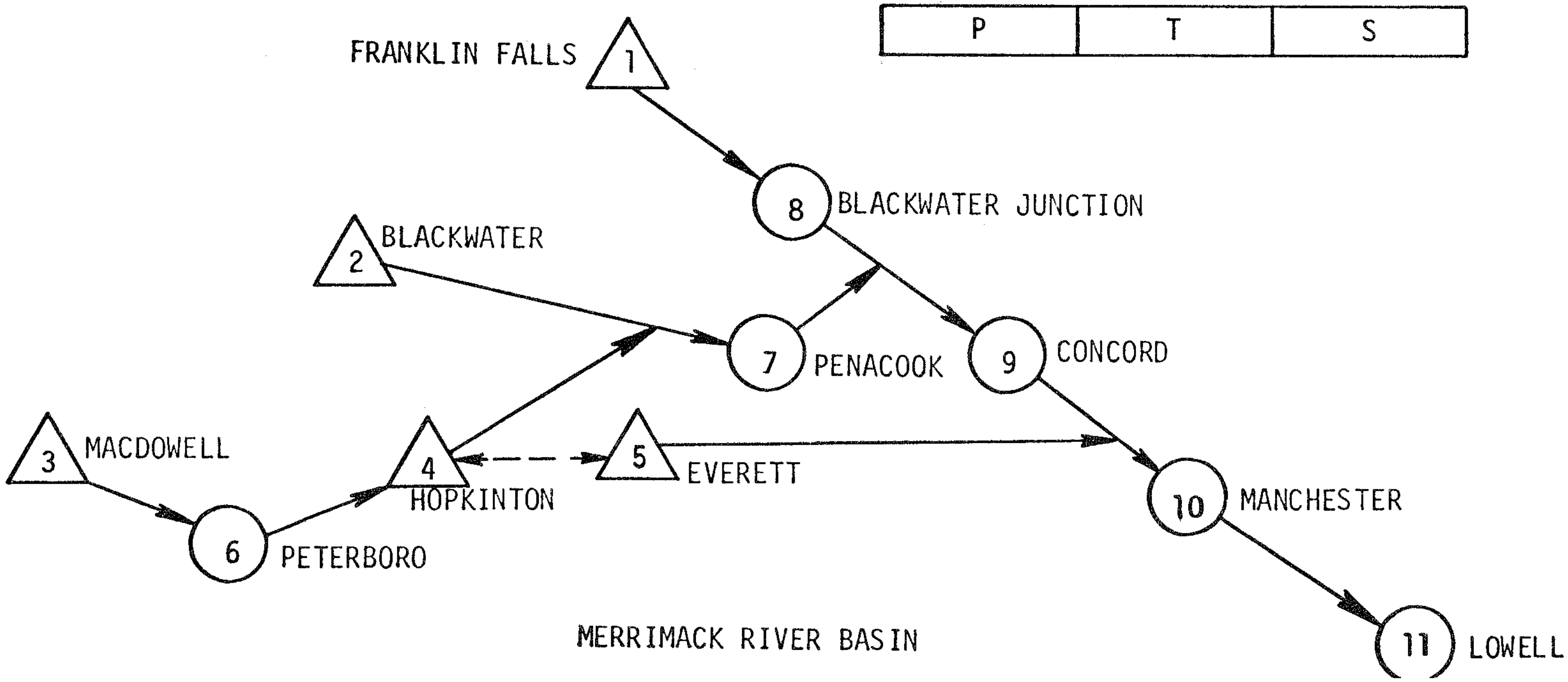


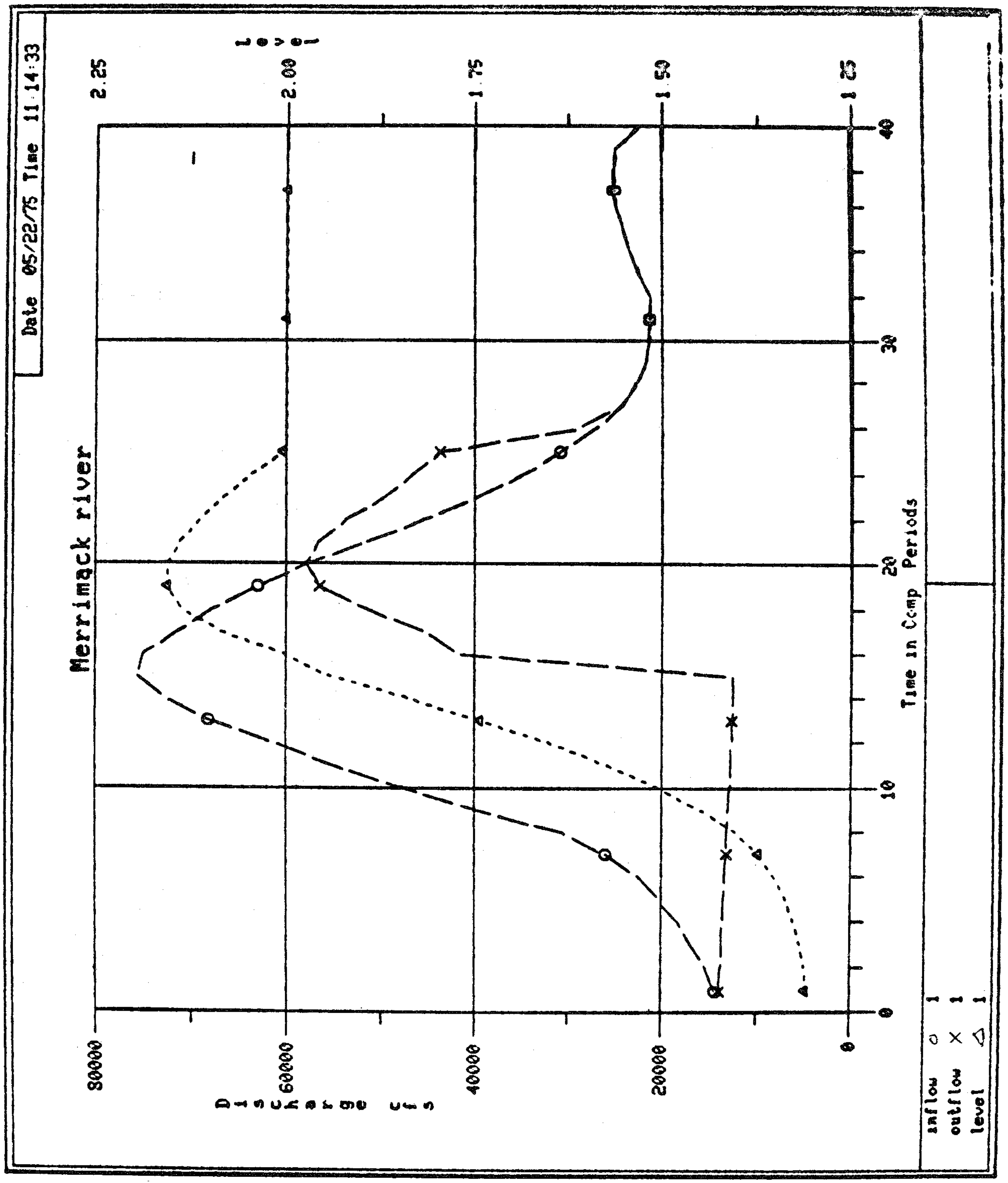
FIGURE 15
Paper 12

INTERACTIVE HEC-5C OUTPUT
SAMPLE - USER DESIGNED TABULATION

LOC NO=					9.	9.	9.
PERIOD	MO	DY	YR	HR	NATURAL	REGULATE	CUM LOCA
1	0	5	0	6.	25000.00	27611.16	7044.40
2	0	5	0	9.	26200.00	28692.85	8129.07
3	0	5	0	12.	26350.00	28672.54	8122.66
4	0	5	0	15.	27200.00	29074.60	8555.46
5	0	5	0	18.	28550.00	29142.04	9289.33
6	0	5	0	21.	29900.00	28339.55	9942.61
7	0	6	0	0.	32300.00	28568.95	11330.91
8	0	6	0	3.	34800.00	28543.40	12282.41
9	0	6	0	6.	38200.00	28226.55	13160.84
10	0	6	0	9.	43000.00	28006.42	13850.17
11	0	6	0	12.	48650.00	26980.24	13369.93
12	0	6	0	15.	55700.00	25647.24	12354.63
13	0	6	0	18.	66600.00	26626.40	13533.07
14	0	6	0	21.	82000.00	31347.73	18397.67
15	0	7	0	0.	91000.00	30074.13	17242.24
16	0	7	0	3.	98000.00	28296.41	14630.53
17	0	7	0	6.	110500.00	35262.40	18211.06
18	0	7	0	9.	118000.00	41495.53	18854.33
19	0	7	0	12.	121500.00	46804.25	17815.91
20	0	7	0	15.	121000.00	51689.89	15273.35
21	0	7	0	18.	116500.00	56630.69	11256.75
22	0	7	0	21.	113000.00	64164.27	10226.90
23	0	8	0	0.	106000.00	70323.76	9983.74
24	0	8	0	3.	99200.00	74775.14	10118.43
25	0	8	0	6.	95950.00	77410.36	10181.78
26	0	8	0	9.	91600.00	78038.71	10012.88
27	0	8	0	12.	85800.00	76627.35	9926.28
28	0	8	0	15.	80100.00	73444.46	9748.16
29	0	8	0	18.	71600.00	68048.84	7966.18
30	0	8	0	21.	68500.00	64268.92	7705.23
31	0	9	0	0.	65850.00	62292.93	8839.77
32	0	9	0	3.	62200.00	59402.61	8569.46
33	0	9	0	6.	59500.00	57408.08	8695.51
34	0	9	0	9.	58300.00	56840.88	9739.17
35	0	9	0	12.	57050.00	56277.35	10320.90
36	0	9	0	15.	57800.00	57579.72	12394.33
37	0	9	0	18.	55300.00	55795.46	11141.33
38	0	9	0	21.	54800.00	55749.99	11532.48
39	0	10	0	0.	56800.00	57949.69	14119.64
40	0	10	0	3.	57800.00	59204.33	15692.39
41	0	10	0	6.	57800.00	59542.84	16275.32
42	0	10	0	9.	58700.00	60706.01	17679.72
				SUM =	2854600.00	2031584.50	503516.84
				MAX =	121500.00	78038.71	18854.33
				MIN =	25000.00	25647.24	7044.40
				MPER=	19.00	26.00	18.00

FIGURE 16

INTERACTIVE HEC-5C PLOTS
SAMPLE - RESERVOIR DATA



Paper 12

FIGURE 17

GENERAL SCHEMATIC
REAL TIME-DATA PROCESSING
MERRIMCAK BASIN

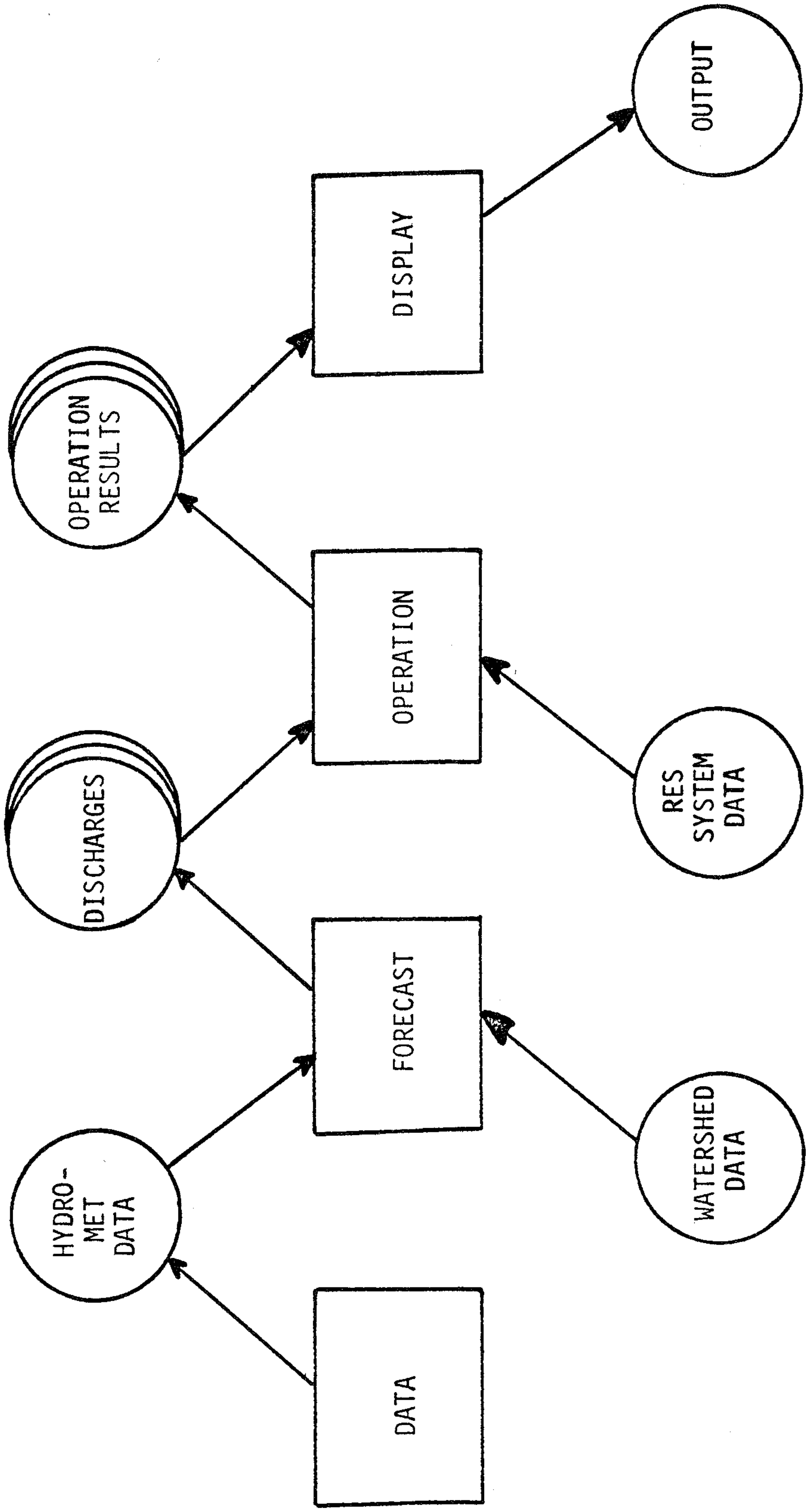


FIGURE 18
Paper 12

REAL-TIME OPERATION OF THE ILLINOIS RIVER SYSTEM 1/

By
Joseph Raoul, Jr. 2/

SYNOPSIS

An assessment is made of the present operation of the Illinois River System following a description of the system. Emphasis is put on the need to develop a hydrologic model of the system to improve its operation and satisfy the two constraints imposed on the system, i.e., Lake Michigan upstream and the Mississippi River downstream. The need for real-time inputs for more efficient outputs is stressed.

General: I have not had any real practical exposure to real-time operation of reservoir systems nor have I dealt with the problem in any extensive theoretical manner. As I understand it, in today's jargon, "real-time" denotes the processing of information or data in a sufficiently rapid manner as to influence a physical response in the system being monitored or controlled. In a slightly different context, it can mean the analysis for and execution of water control decisions for both normal and emergency conditions, based on prevailing hydrometeorological conditions and constraints, to achieve efficient management of water resources systems.

For quite a while, I have pondered over the concept. For presentation at this Seminar I had envisioned, in the light of today's tools for optimization, linear and dynamic programming techniques and the ever-increasing technological

1/ Presented at "Seminar on Real-Time Operations of Water Resources Projects" on 17 November 1975, at HEC, Davis, California.

2/ Chief, Hydrologic Engineering Section, Water Control Center, North Central Division, U.S. Army Corps of Engineers.

advances to devise some kind of optimum release policy for a reservoir system. This would be done over some specified release periods, to determine a set of ending storages known to be adequate for future system operation. In other words, given a system of reservoirs, based on previous observations and other considerations, an acceptable storage vector space would be determined; then given some potential state of the system and some typical behavior of the system, a set of policies would be determined on which to base the operation of the system automatically.

It appeared that the idea was too theoretical and would not lend itself to any immediate application to any Corps water control system. In searching through the literature, I have found that the concept had already been developed by Becker and Yeh. They have used a dynamic programming routine for the selection of an optimal reservoir storage policy path through a specified number of policy periods. A linear programming routine for period by period optimization of the system was used. Their procedure was applied satisfactorily to a real system of reservoirs and hydroelectric facilities associated with the Shasta and Trinity subsystems of the California Central Valley Project.

Interestingly enough, in the final report submitted to the Office of Water Resources Research, Department of the Interior, by the Center for Research in Water Resources, University of Texas, the same concept has been considered for the real-time management of the Trinity River Basin System.

Be that as it may, I had, for the purpose of this Seminar and because of the time-frame available to me, to resort to a project deserving immediate considerations: The real-time operation of the Illinois Waterway System.

DESCRIPTION OF THE SYSTEM

The Illinois Waterway extends from Lake Michigan at Chicago to the Mississippi River at Grafton, Illinois, over a distance of 330 miles. The natural drainage area of the Waterway is about 29,000 square miles, of which 25,000 square miles are within NCD's Chicago District boundary. This includes about an 800 square miles of former Lake Michigan basin drainage which was added to the Illinois River when the flow of the Chicago and Calumet Rivers was reversed. It also includes certain drainage areas intercepted along the Lake Michigan shore. A total of 335 square miles in the eastern portion of the Little Calumet River watershed was diverted into Lake Michigan through Burns Ditch and is not included in this drainage area. The natural divide separating the Great Lakes basin from the Mississippi River basin passes 10 miles west of the Lake Michigan shoreline at Chicago. When the canal was constructed from Chicago to Lockport, it breached the divide near Summit, Illinois. The canal is diverted at Lockport, Illinois, into the Des Plaines River, a tributary of the Illinois River. This project was essentially completed by 1900 and was acclaimed as one of the outstanding engineering projects of that time.

There are three water control points between Lake Michigan and the waterway. They are Chicago River Lock and Control Works at the mouth of the Chicago River, the O'Brien Lock and Control Works on the Calumet River and the North Shore Channel Control Works at Wilmette, Illinois.

The Chicago River Lock and the Lockport powerhouse and Control Works, are owned and operated by the Metropolitan Sanitary District of Greater Chicago (MSD). It is operated to maintain navigation requirements as directed by the Corps. One of the basic functions of (MSD) is to prevent sewage flows from discharging into Lake Michigan. During times of low flow, dilution water is taken from Lake Michigan. The O'Brien Lock and Control Works in the Calumet River is operated by our Chicago District.

The width of the Illinois Waterway varies from 400 feet near LaSalle, Illinois, to 1400 feet near the mouth at Grafton, Illinois, except through Peoria Lake where it expands to 1 mile wide. The natural drop in elevation of the Illinois Waterway in the 49 miles from the junction of the Kankakee and Des Plaines River to the head of the valley at LaSalle is about 53 feet. From LaSalle to Peoria, a distance of 61.5 miles, the fall is only 4 feet; and from Peoria to the mouth of the river at Grafton, a distance of 162.6 miles, the fall is 28 feet. A nine-foot navigation channel is maintained along the waterway by eight locks and dams and is operated by our Chicago District. The present system of locks and dams was completed in 1939.

The section from Lake Michigan to Lockport, about 36 miles long, passes through the Chicago Metropolitan area and is controlled by the lock and powerhouse at Lockport. The system utilizes the Chicago River, the south branch of the river, the Chicago Sanitary and Ship Canal and the Calumet-Sag Channel which connects the Sanitary and Ship Canal and the Calumet River and Harbor.

At Lockport, after a drop of 40 feet, the Waterway continues some 60 miles downstream via the Des Plaines and Illinois Rivers. In this reach it falls 99 feet through a series of pools which have been created by low dams. These locks and dams include Brandon Road, Dresden Island, Marseilles and Starved Rock. Through the 231 miles from Starved Rock to Grafton the waterway falls more gently, about 21 feet along this reach of river. Two wicket type dams, at Peoria and LaGrange, are in this reach.

OPERATION OF THE SYSTEM

The Illinois Waterway as we have previously described it, is composed of a series of locks and dams extending from Lockport down to the confluence with the Mississippi River. As far as operation of the system is concerned, no storage accumulation or release problem exists except at Lockport where the flow is being utlized for power and navigation purposes. These dams, because of their unsophisticated nature, present some problems. The wickets which constitute the main part of the LaGrange and Peoria dams have a limited capacity. They are lowered manually during high flows, flooding situations and for removal of accumulated debris. Considerable seepage occurs during low flows. Very little capability exists for obtaining water levels of more than a few inches above the top of the wickets at normal pool. The LaGrange Dam has a particular problem because of its proximity to the mouth of the LaMoine River. Sudden heavy rains occurring in the LaMoine River watershed have an almost immediate effect on the water level at LaGrange Dam. Unless the storm is predicted early enough, there is very little time to lower the wickets and prevent possible breaching of the dam. The operation normally takes four to six hours and is accomplished by hand from the deck of a maneuver boat which travels along the top of the dam. As a consequence, a small unused reservoir capacity must be maintained in the pool to allow time to lower the dam. In addition, during times of heavy rains, large quantities of trees, brush and debris are generally washed out of the tributary streams. If the wickets are not lowered quickly, they become covered with debris and create an extremely difficult and hazardous condition for the crew of the maneuver boat. Real-time or advanced knowledge of the meteorological conditions would alleviate such situations.

Regarding the levels and flow monitoring of the system, the information is transmitted daily by telephone to the Chicago District from the project offices located at Joliet and Peoria. There exists 19 stations along the system for which information are being transmitted daily to the District. These stations cover the distance from Joliet down to Meredosia, Illinois.

Seven of these stations are located on some tributaries to the Illinois River, such as the Sangamon River, the Fox River and the Kankakee River. At time of flooding, the information transmitted for these stations is supplemented by information from seven other stations, all located on the tributaries. Information on the headwater, the tailwater and the settings of the dams is also reported to the District on a daily basis. The latest 24-hour precipitation data at 20 stations are phoned daily to the District which uses it to estimate the average rainfall over the basin for their monitoring activities. From the information obtained, the District also computes the stage discharge relationship and transmits it daily to the St. Louis District for use in their operation of the Mississippi River.

The forecast responsibility lies with the National Weather Service (NWS). During flood conditions the District uses the Kansas City NWS to obtain their information. Presently, effort is being made, by the District, to obtain precipitation and other meteorological information from NWS for the Distirct's daily monitoring. We will address later the additional information needed to enhance the operation of the systems.

THE NEED FOR A HYDROLOGIC MODEL

The Illinois Waterway, because of its location, constitutes a hydraulically unique system bound by two constraints; the Lake Michigan and the Mississippi River. Over the years, particularly during high Lake Michigan levels, the suggestion has been repeatedly made to increase the diversion from the lake into the river as a means of alleviating the high levels. On the other hand, the downstream interests on the Illinois-Mississippi River have often attributed the worsening of the flooding problem to the diversion from the Lake Michigan. This interesting situation suggests that better control of the system is warranted to optimize its use for the maximum benefit of the river itself, the lake above it, and the Mississippi River dowstream of it. This man-made situation due to the reversal of flow from Lake Michigan into the river presents another problem. The diversion amount includes domestic pumpage, water diverted directly from the lake and the runoff from 810 square miles of highly urbanized Chicago area which formerly drained to Lake Michigan. The storm runoff from this area does, at times, aggravate flooding on the Illinois River. Flooding in the Chicago area has been a severe problem since the closing of the natural outlets at the mouth of the Chicago River in 1939 and the O'Brien Lock in 1965. Severe storms in the Chicago area produce enough runoff to require control gates to be opened at the 3 control points. Storm flows are permitted to enter the lake in order to avoid serious flood damage in the area. Such releases degrade the water quality of Lake Michigan. MSD is procedding with a deep tunnel-detention basin system to handle storm water runoff, much of it from combined sewer systems.

The situation suggests the need for a mathematical model of the river, which would serve many purposes. A model with real-time output would give possible alternatives and would aid in decision making for the coordination of flood control activities. It would predict tributary inflows and would provide a routing tool for the main stem of the Illinois River. It would eventually help alleviate flood damages by providing better information on the river and flow regime and by allowing a better control of flow release at each dam and on the lower part of the river.

In the present operation of the system, the Chicago District submits daily information on precipitation over the basin and downstream Illinois River flow to the St. Louis District. They use it in operating Lock and Dam 26 on the Mississippi River.

Using precipitation and streamflow data at some selected index stations over the entire basin, a model would provide the capability to forecast runoff, simulate gates openings all along the system; at times of flooding this simulated information could be useful for determining the level of flooding and possible pool operation to lessen the impact on the local area. The model could also assist in long-term decision making by providing the capability to determine various possible operating rules under the same type of conditions. A comparison could be made and the ideal time for gate operation could be obtained.

The model would also assist in the study and operation of the future duplicate locks projects. As you may know, the Lockport and Brandon Road Locks are nearing their full capacity. In anticipation of the need for greater capacity in the waterway, Congress in the River and Harbor Act of 1962 approved a project modification to provide for supplemental locks, 110 feet wide and 1200 feet long, at Lockport, Brandon Road, Dresden Island, Marseilles, Starved Rock, Peoria and New LaGrange. These locks will be twice the length of the existing locks. The total estimated cost to the United States is about $697 million for construction.

The modelling of the hydrology of the river will certainly be beneficial to the design, construction and operation of these projects.

As I briefly mentioned, in order to solve the interrelated water pollution and water damage problems in the Chicago area a study was undertaken by the state and local governmental agencies. Participating in the study effort were representatives from the State of Illinois, the Metropolitan Sanitary District, the Cook County, the City of Chicago and the Corps of Engineers. Two specific problems concerning the existing combined sewer system for wastewater and storm water runoff were addressed. The first concerned the need to limit the discharge of the untreated overflow from the existing sewer system into the waterway in order to meet water quality standards. The second involved preventing the excess overflow in the waterways from being released into Lake Michigan in order to avoid urban flood damages. The study culminated into a selected plan known as the Chicago Underflow Plan. The underground tunnels will accommodate the increased runoff from some 194 drainage points that make up the service area. A collector system will be used to divert the runoff into the tunnels and control spillage into the waterway and Lake Michigan during large storm events. To insure the adequacy of the system, many improvements will be required. Most of the improvements relate to the waterway and are designed to control the high river stages or improve the in-stream water quality. The routings of flow in the Chicago Sanitary and Ship Canal used in the system design were based on the assumption that an authorized Corps of Engineers project would be implemented prior to or concurrent with the tunnels and reservoirs. The project involves the reach of canal extending from the Lock and Dams at Lockport, Illinois, to the junction with the Calumet-Sag Channel. It would widen this reach from 160 to 225 feet to sustain projected growth in barge traffic on the Illinois Waterway.

The area outlet for all the watercourses except the Des Plaines River is the Lock and Dam at Lockport, Illinois. There the maximum through-flow rate is limited to 24,000 cfs. If runoff from outside the study area, together with the spillage from the mainstream and Calumet system, exceed this rate, water damage from surcharged sewers and flooding in low lying areas would be expected.

It is certain that, with the existence of a model for the Illinois River, the design and operation of the system will be more efficient and that the tunnel, by intercepting some of the overflow, will improve the situation at Lockport and downstream of the river.

THE MODEL AND THE INCREASED DIVERSION

As you may be aware, the question of increasing the Chicago Diversion is almost as old as the diversion itself and has for the past years of high Great Lakes water levels given rise to a lot of controversy. It has been the subject of a number of Congressional Bills for the past three years. Most bills suggest increasing the diversion to 10,000 cfs. The present limitation by the U. S. Supreme Court decree is an average of 3,200 cfs on a 5-year running period, with the flow in any one year not to exceed 110% of 3,200 cfs. Our Chicago District has recently completed a preliminary study to assess the impact downstream of such an increase. Needless to say, with a model of the river and real-time data, the task would have been much easier, the assessment more reliable and the decision would have been more concrete. Morever, if the diversion is authorized, the model will be used to design the operational plan for the control of the diversion increase.

NEED FOR REAL TIME OPERATION OF THE SYSTEM

As I have mentioned previously, except at Lockport, there is no storage on the Waterway and the need to improve its operation would not be for the same reason as for any other reservoir systems which contain storage. In other words there is not much storage space to manipulate along the several locks and dams on the system. However, on the basis of the several problems previously mentioned and in the context of today's advance in modelling and monitoring, there is a need to design a model of the system for use on a real-time basis. There is a need to improve the system's hydrology and hydraulics data gathering system. Additional and quicker information on precipitation are needed to better appraise the response of the system to flooding and be in a position to implement control on the lower reach of the river with more reliability. These information would include: (a) Forecast probabilities of rainfall in the area which is supplied by the National Weather Service; (b) Snowpack in the basin; (c) Ground infiltration conditions in the basin; (d) Seasonal variations in the hydrologic cycle; and, (e) Travel time from the diversion to potential flood area. The data furnished to St. Louis District for the operation on the lower part of the Mississippi River would then be improved.

The model with real-time or improved inputs could be used to forecast for any desired time period. When the inputs are based upon recently measured and forecast meteorological data, previsions can be made for several days in advance. Probabilistic inputs could be used when longer range forecasts are desired. Through a variety of alternatives, short-term and long-term information can be produced for the benefits of navigation power and other interests. The shipper who is interested in scheduling vessels is primarily concerned with the longer range forecast. It ensues that improvement in meteorological forecasting, a key to the quality of river forecasts, is also warranted. The need for a model and the need for real-time operation leads to the need to upgrade data collection and transmittal and the need to enhance the quality of the meteorological forecasts. We in the North Central are actively engaged in helping our Chicago District in its effort to model the system and operate it on a real-time basis within the very near future.

ARKANSAS RIVER SYSTEM REGULATIONS

by Dale Morrisett [1]

INTRODUCTION

The Corps of Engineers has received considerable publicity in the Midwest by constructing the McClellan-Kerr waterway system on the Arkansas River. Because of the complexity of the problems associated with operating this system, we decided that some of the real time operational problems would be of particular interest to this group. None of the problems are unique, yet we believe that the combination is different from those experienced anywhere else.

The Tulsa District portion of the Arkansas River Basin extends from the Arkansas-Oklahoma State line near Van Buren, Arkansas, upstream to Great Bend, Kansas. This portion of the basin contains about 150,000 square miles of drainage area. Average annual rainfall ranges from 17 inches in the extreme western portion of the Basin to 45 inches in the southeastern portion. The Basin is characterized by frequent drought conditions in the summer months, however, flooding can be expected at anytime of the year which may require full utilization of flood control storage. Major floods are usually produced by relatively short duration, high intensity thunderstorms.

The Tulsa District currently operates 27 projects in the Arkansas Basin (Plate 1). Five of these projects are navigation locks and dams. The locks and dams create slack pools with a minimum of 9-foot of depth for navigation and have no flood control storage. Two of the locks and dams have run-of-river hydropower plants. Five of the upstream projects are non-Corps of Engineer projects for which the Corps of Engineers is responsible only for the flood control operation. The remaining projects are multipurpose projects. All of the upstream projects, with the exception of Keystone Dam, are located on tributaries to the Arkansas River. The system of reservoirs contains about 8,125,000 acre-feet of conservation storage and 9,228,260 acre-feet of flood control storage. This flood control storage is equivalent to only 1.2 inches of runoff from the drainage area above the projects or about 40 percent of the average annual flow at Van Buren, Arkansas. Flood inflows into the projects are usually short duration with relatively high peak discharges. Since the flood control storage is small in comparison with total flood volumes, the frequency of filling the system flood storage is about once in 7 to 8 years.

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The flood control along the main stem of the Arkansas River is provided primarily by seven major reservoirs. These reservoirs are Keystone, Oologah, Fort Gibson, Pensacola, Markham Ferry, Tenkiller, and Eufaula. These projects control flows discharging directly into the Arkansas River and contain about 70 percent of the total flood storage in the system. There are about 8,000 square miles below these projects and above Van Buren, Arkansas, that are uncontrolled. The last reservoirs added to the upstream system were placed into operation in 1970. During this time, the flood control operation consisted primarily of evacuation of flood control storage as rapidly as downstream conditions would permit. The main control point for downstream flood releases is the reach of the river in the vicinity of Van Buren, Arkansas. The system was designed for an operational discharge of 150,000 c.f.s. at a 22-foot stage at Van Buren. The channel capacity for this reach has deteriorated such that only 105,000 c.f.s. can be passed at the 22-foot stage. The system operation presented little difficulty during the 1965 through 1972 period, primarily because of low flow conditions with only minor to moderate floods. During major flooding experienced in the early spring of 1973, the reduced channel capacity placed a severe constraint on the upstream system. In addition, shoaling of the navigation channel following major rises created serious navigation problems. Since that time we have attempted various operational changes in the regulation plan to better serve the various purposes. Some of the main considerations have centered around the reduced channel capacity at Van Buren, Arkansas, and the restoration of the navigation following major rises.

A discussion of some of the various operational plans is presented in the following sections of this paper.

BUFFER ZONE CONCEPT

In early 1973 buffer zones were established in the flood pool of the Corps projects (Plates 2 and 3). The higher zone called "Buffer Zone B" was to be evacuated at powerplant capacity. The lower zone called "Buffer Zone A" was to be evacuated as rapidly as scheduling for peak power would allow. Three objectives of the zone operation were: (1) to better define evacuation procedures for minor impoundments in flood control pools; (2) to increase payoff for power features; and (3) to establish tapered recession and transition procedures from high flood control releases to lower power releases. Experience gained during 2 years of trial operation under the buffer zone A and B concept at the multipurpose projects with hydropower were: (1) due to unusually high inflows during the trial periods, most of the rises were above the buffer zones: (2) problems resulted since criteria for evacuating to lower zones were not well defined; (3) even though we had record years of generation, none of the districts nor the Southwestern Power Administration were able to identify any gain in hydropower attributable to the buffer zone operation; and (4) observed reductions in downstream channel capacity had more effect on flood operations than the buffer zone levels.

The interim plan of flood control for the Tulsa District projects in the Arkansas River Basin was initiated shortly after the zone A and B concept trial period started and superceded the zone operation for the projects directly involved in the navigation regulation.

INTERIM PLAN FLOOD REGULATIONS

In the latter part of 1973 and early 1974 an interim plan of regulation for the Tulsa District multipurpose projects was developed to provide operational guidance until a more detailed plan could be developed. This was brought about primarily to help restore navigation following large rises and to establish an interim plan of flood control operation to maintain the system flood control capability for large floods, even though the channel capacity in the vicinity of Van Buren, Arkansas, had been significantly reduced.

The plan was developed using the seven major projects previously mentioned plus Wister and Hulah reservoirs (Plate 1). The projects directly control flow into the main stem of the Arkansas River and contain approximately 6,600,000 acre-feet of flood control storage, about 75 percent of the total flood control in the Arkansas River Basin above Van Buren.

The interim plan consisted of the development of: (1) guide curves in the lower portion of the flood control storage to provide flows for the production of hydropower and navigation; (2) a navigation taper following large floods to provide flows in sufficient quantity to sustain navigation while dredges are removing material in shoaled areas; and (3) a variable operating flow rate at the Van Buren gage to maintain the system flood control capability.

Individual project and system requirements were analyzed in establishing guide curves (Plates 4 through 9). Some of the main considerations in arriving at the guide curves were: (1) frequency of filling the flood storage; (2) duration of pool levels at various elevations; (3) seasonal effects of flooding; (4) damage to the pool areas caused by holding water for prolonged periods; (5) distribution of flows into the individual projects; (6) operational constraints such as channel capacities in the reaches below each project and the damage potential below each project; and (7) navigation flow requirements.

An operational guide curve (Plate 10) based on upstream flood control storage was developed for the Van Buren area. The guide curve is based on percent basin flood control storage utilized, which consists of current reservoir storage plus 10 days forecasted inflows divided by the total flood storage in the seven lake system. A variable operational discharge is provided depending on basin storage utilized and season of the year. The variable operational discharge provides for higher discharge when system storage becomes critical and permits recovery of flood control storage more rapidly. In the lower levels, storage is provided for the navigation taper. Plate 11 summarizes the estimated frequency of filling flood pools for various operational discharges at Van Buren.

Referring to Plate 11, the reduction in recent years of channel capacity at the Van Buren gage has resulted in reducing the effective flood storage in the Arkansas River System by approximately 10 percent. Withdrawal of floodwaters takes a longer period of time and as a consequence, the frequency of filling the flood pools is increased. The effective flood control storage taken from the flood control pools under the interim plan amounts to approximately 7 percent, however, by seasonally varying the operating flow at Van Buren, the frequency loss due to the interim plan is minimal.

EXAMPLE OF INTERIM PLAN FLOOD ROUTING

The Van Buren Gage in 1973 experienced the largest flood volume in the period of record (Plate 12). 1974 was the fourth largest flood volume on record. Sixty million acre-feet of water passed Van Buren in 1973 which would have filled and emptied the available flood control system almost eight times. Forty-five million acre-feet of water passed the Van Buren Gage in 1974. Two major floods were experienced in June and November of 1974. The November 1974 flood (Plate 13) had an estimated peak discharge of 765,000 c.f.s. which is second only to the June 1943 flood which had a peak discharge of 850,000 c.f.s. Shown on Plate 14 is a hydrograph of the actual operation at Van Buren for the November 1974 flood. Also shown are the releases from each of the upstream projects routed to Van Buren. Plates 15 through 20 are pie diagrams depicting the relative size of the flood storage in each lake. A perfect circle would represent a balanced system. The shaded portion indicates the storage utilized on a particular date. The dashed line represents 50 percent of the flood storage for that particular lake. For the November 1974 flood, the upstream flood control storage was empty at the end of October. Heavy rainfall was received on the 2nd and 3rd of November. Additional rainfall fell on the 10th and 11th of November and the 5th, 6th, and 11th of December. The first few days in November, only power releases were made from the upstream projects. Releases of 90,000 c.f.s. were required from Keystone on 7 November since slightly over 100 percent of its flood storage was utilized. Releases were reduced from these projects on the 9th and 10th of November to prevent additional flooding in the Van Buren reach because of additional rainfall in the uncontrolled area above Van Buren. As the uncontrolled area flows receded, releases from the projects were made to maintain 150,000 c.f.s. for the period 15 November through 27 November. Starting 27 November, the flow was gradually reduced to about 45,000 c.f.s. by 5 December. Additional rainfall resulted in nearly 70,000 c.f.s. on the 7th and 13th of December. Approximately 50,000 c.f.s. was maintained from 15 December through early January 1975. Flows of 30,000 c.f.s. were maintained through January 1975. Plate 21 illustrates the flood storage utilized in 1974. Nine tapers were attempted in 1974 and seven so far in 1975. Due to the continued wet conditions, only four have been completed over the past 2 years, the latest being in July of this year. A completed taper is a simulated natural recession from a flood flow to a steady flow of approximately 40,000 c.f.s. maintained for a minimum of 2 weeks.

SUMMARY OF REGULATION PROBLEMS

Regulation problems in the Tulsa District include extremely high flows, restrictions and reductions in channel capacities, numerous operations for the Arkansas River Navigation tapers, increased public reaction and damages resulting from high pools, and cooperation with State and other Federal agencies requesting changes in operations. Moderate floods above 70,000 c.f.s. produce shoaling in the downstream reaches of the Locks and Dams and may require 3 to 4 weeks of dredging before navigation depths are restored. Immediately after a flood and until the major shoals are removed, flows of about 40,000 c.f.s. are required at Van Buren to provide adequate depth for the dredges and navigation traffic, which can be satisfied by full power generation at the upstream projects. In order to provide this recovery time, water is stored in the lower portions of the upstream flood control pools for longer periods of time, resulting in additional damage in the lakes areas, recreation in convenience and an increase in the frequency of filling the flood control pools.

The interim plan was expected to increase the energy production. However, due to Southwestern Power Administration's increased cost of thermal purchase and the need to protect the dependable capacity because of lack of available thermal energy during critical demand periods, no significant increase in energy can be attributed to the interim plan.

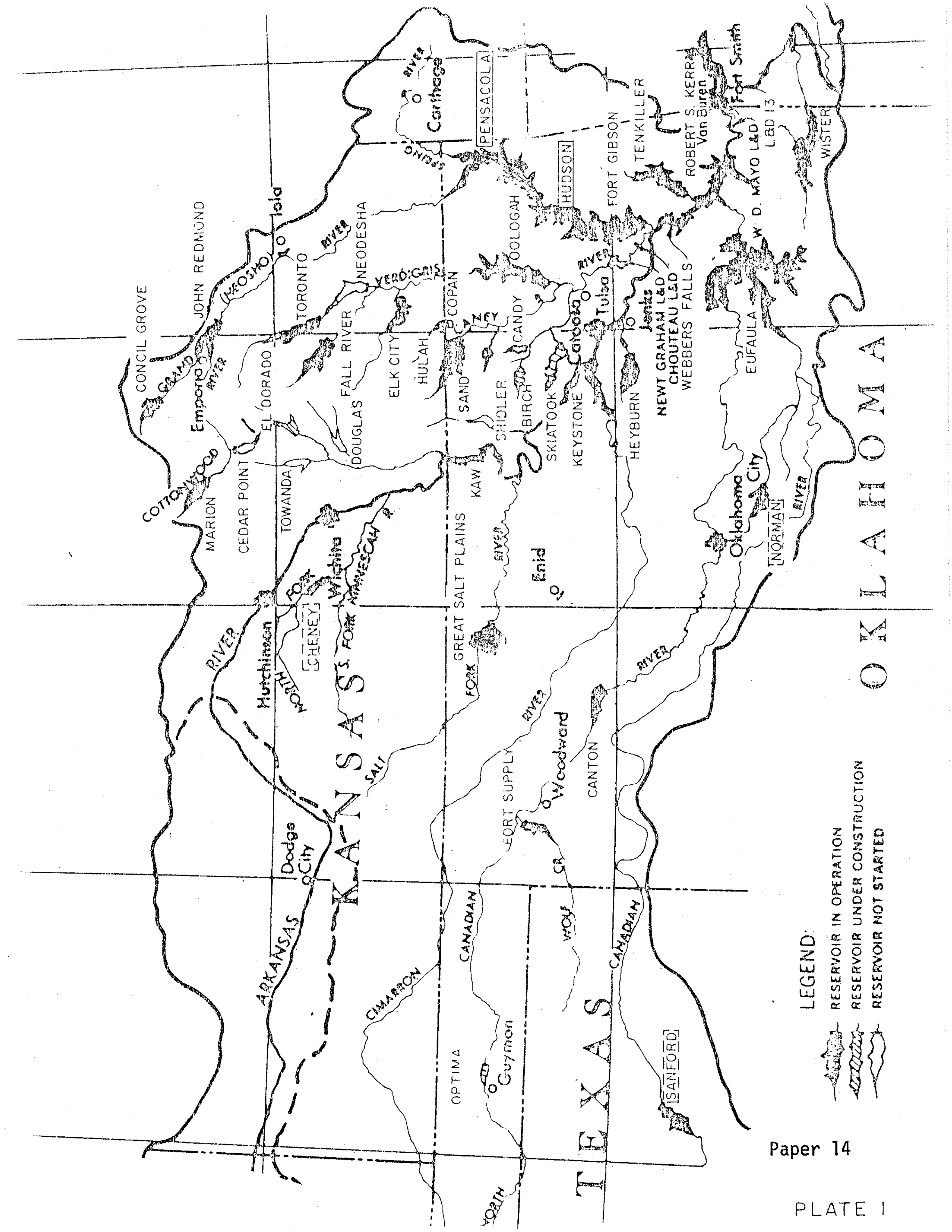
FUTURE REGULATION STUDIES

Several system analysis studies are planned for the Arkansas River System by use of the computer program "Regulation Simulation and Analysis of Simulation for a Multi-Purpose Reservoir System" developed by personnel of the Reservoir Center, SWD. Hydrologic data for the period 1940 through 1974 will be used for a hypothetical daily operation of the system. The system operation will enable the development of stage frequency, discharge frequency, flow duration, pool elevation duration, and pool elevation frequency relationships.

Studies will be made to determine whether it is economically feasible to restore the channel to a capacity of 150,000 c.f.s. or larger, and to determine the best alternate method of operation of the multipurpose system. Alternatives to be considered will include dredging, stabilization of the channel, and possible flowage easements or land acquisition.

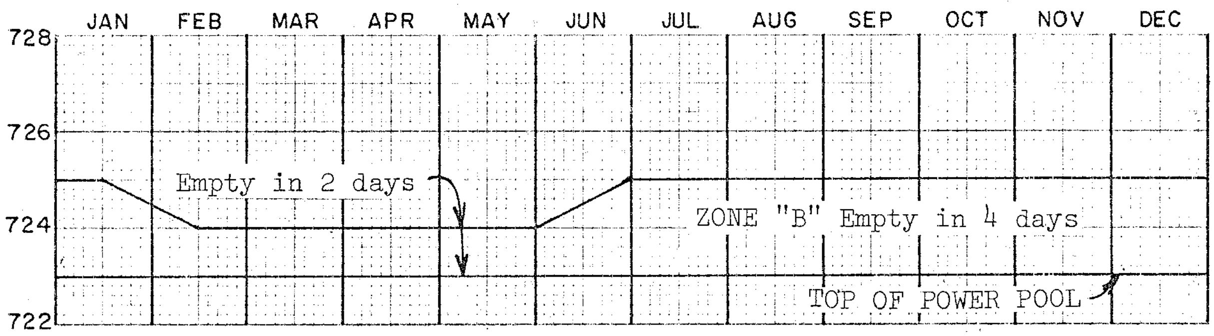
Alternative navigation taper plans will be evaluated. Consideration will be given to developing a taper plan that will account for varying problems associated with the operation, such as magnitude of the shoaling problem, location of the dredges and time required for contracting and mobilization, condition of the basin flood control capability, distribution of rainfall, baseflow conditions, and seasonal frequency of rainfall.

Workshops will be held between the Little Rock District, SWD, and the Tulsa District to analyze each simulation for success in meeting the objectives, and to determine possible modifications for future runs. After all simulation runs are complete, a plan of regulation will be selected, a report will be prepared and meetings will be held to inform interested groups such as the Arkansas River Basin Development Association and the Arkansas Waterways Commission of the results.

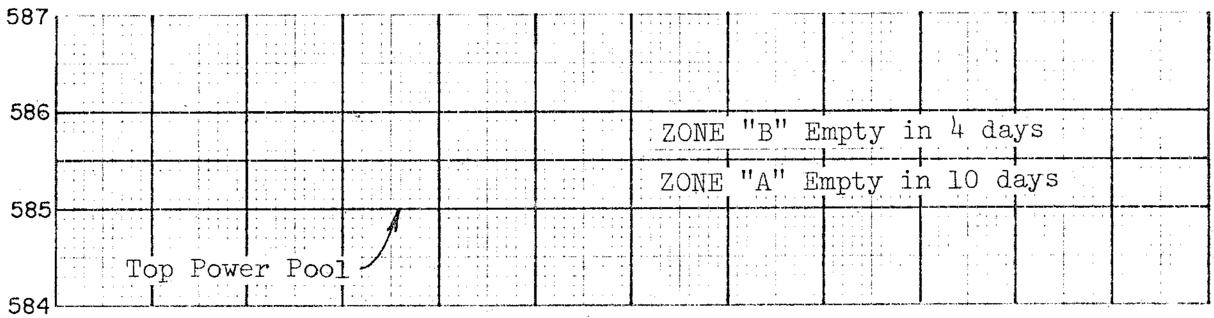


Paper 14

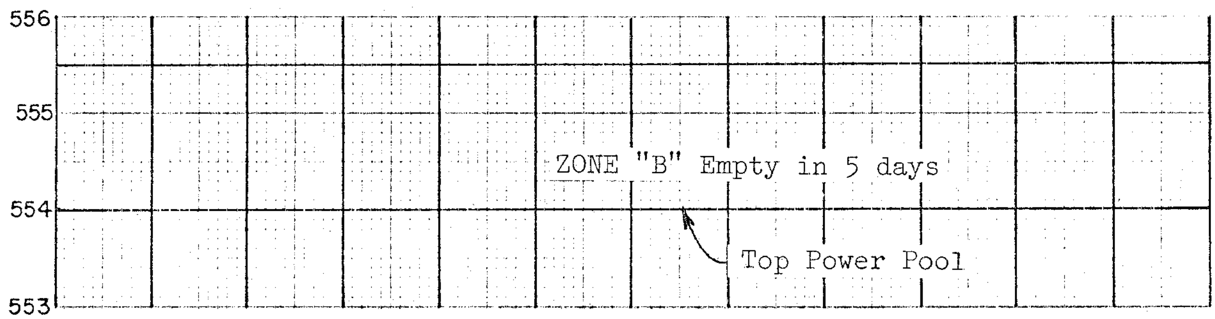
PLATE I



KEYSTONE



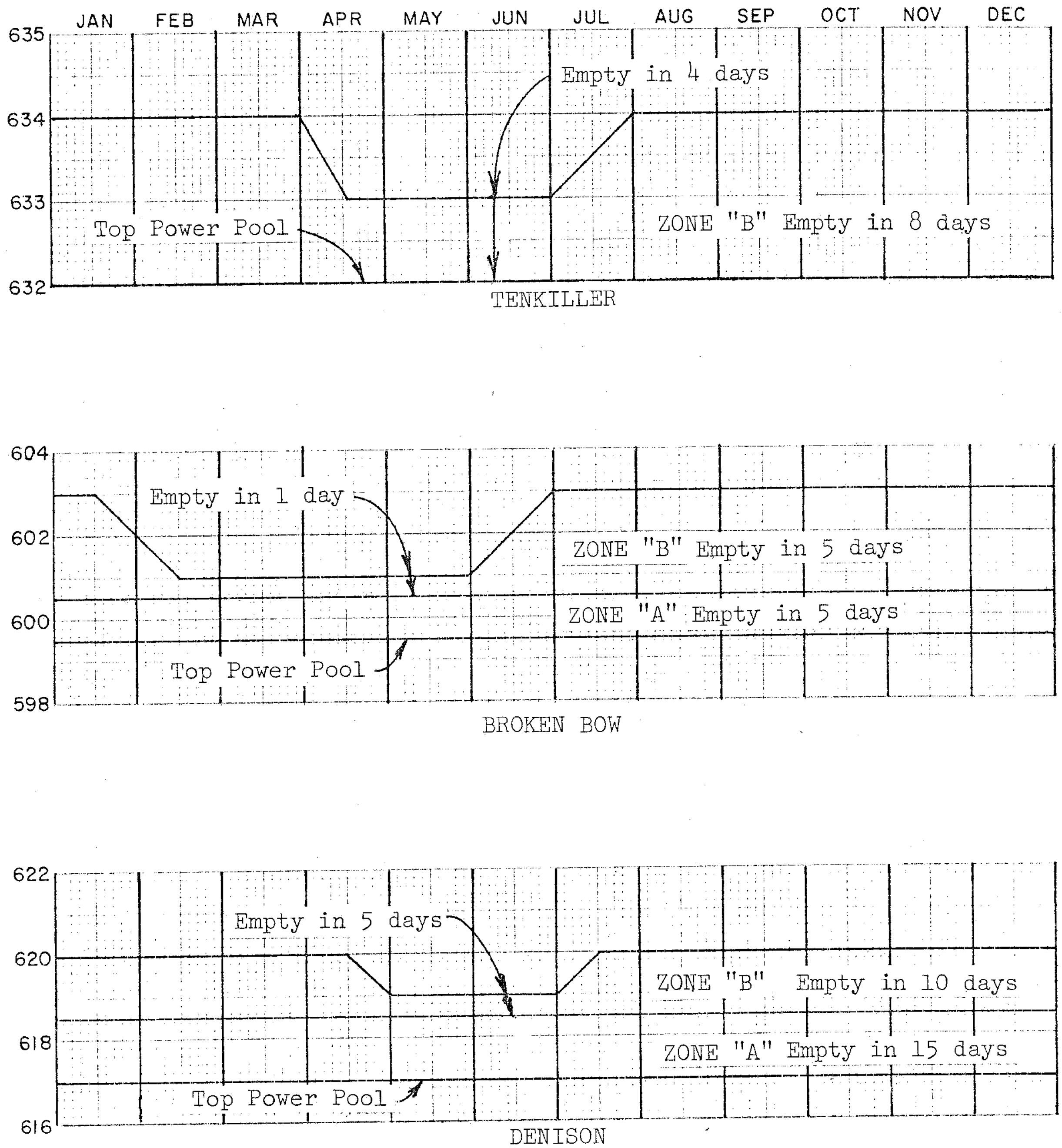
EUFAULA



FORT GIBSON

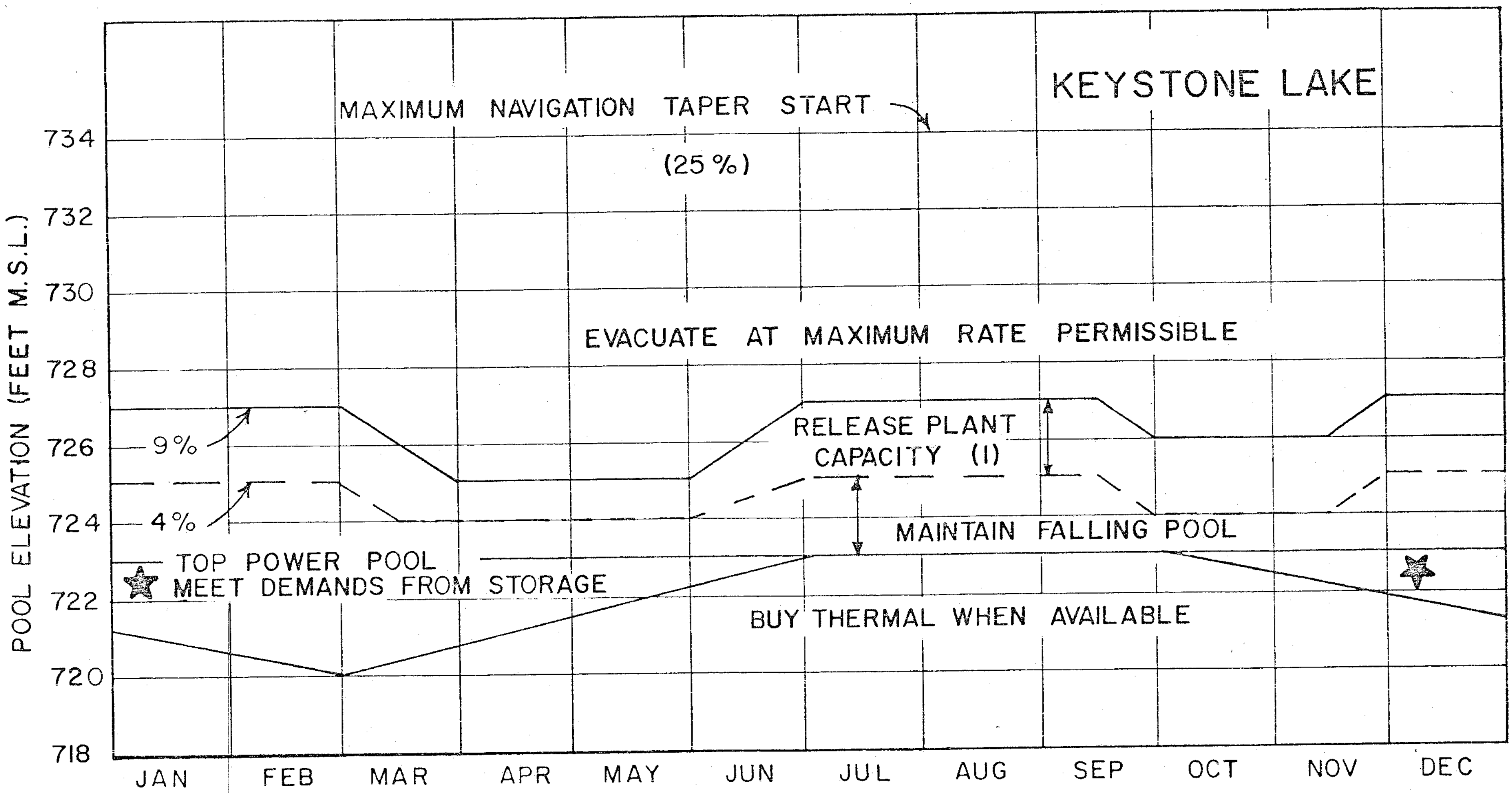
Paper 14

PLATE 2



JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
635
634
633
632
Empty in 4 days
Top Power Pool
ZONE "B" Empty in 8 days
TENKILLER
604
602
600
598
Empty in 1 day
ZONE "B" Empty in 5 days
ZONE "A" Empty in 5 days
Top Power Pool
BROKEN BOW
622
620
618
616
Empty in 5 days
ZONE "B" Empty in 10 days
ZONE "A" Empty in 15 days
Top Power Pool
DENISON

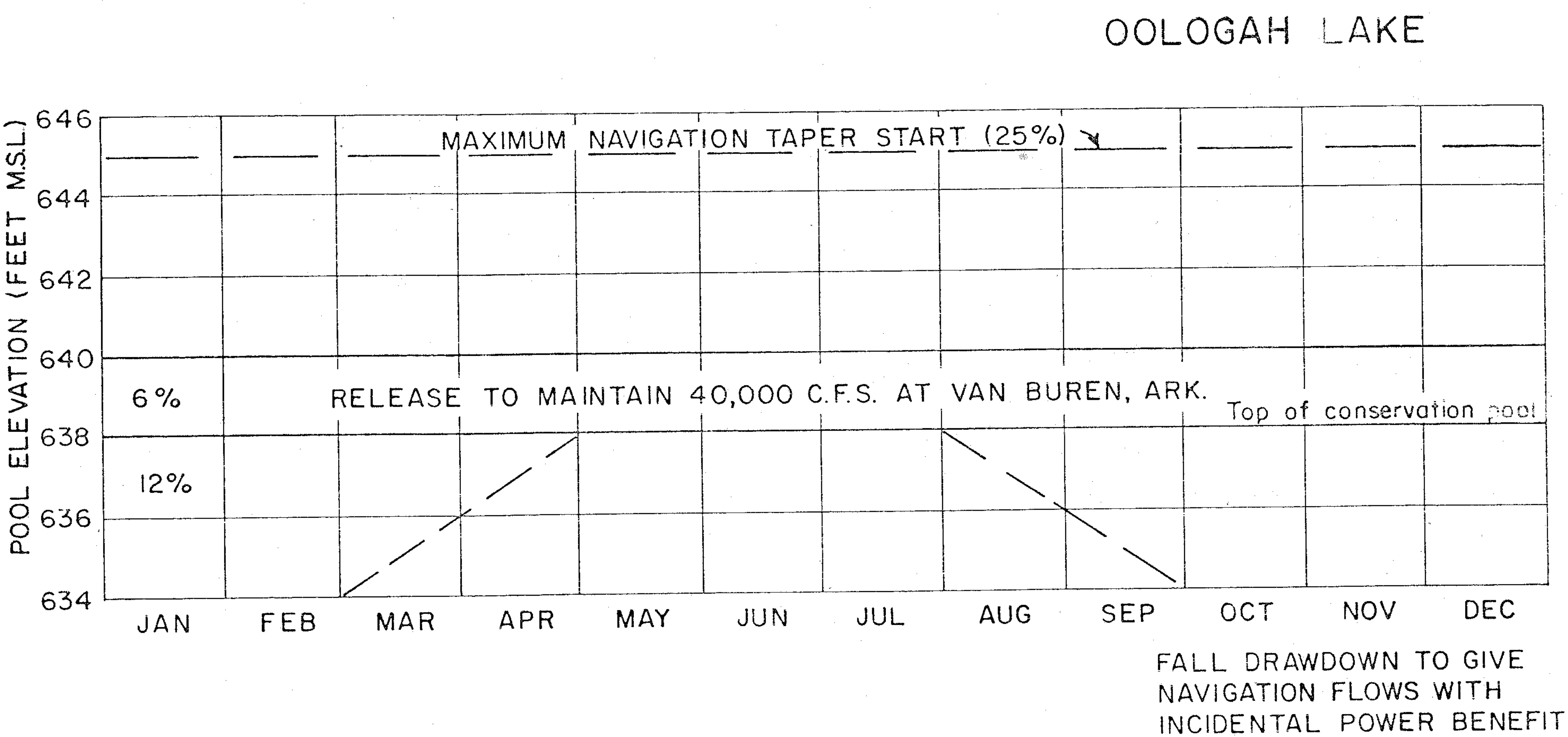
PLATE 3

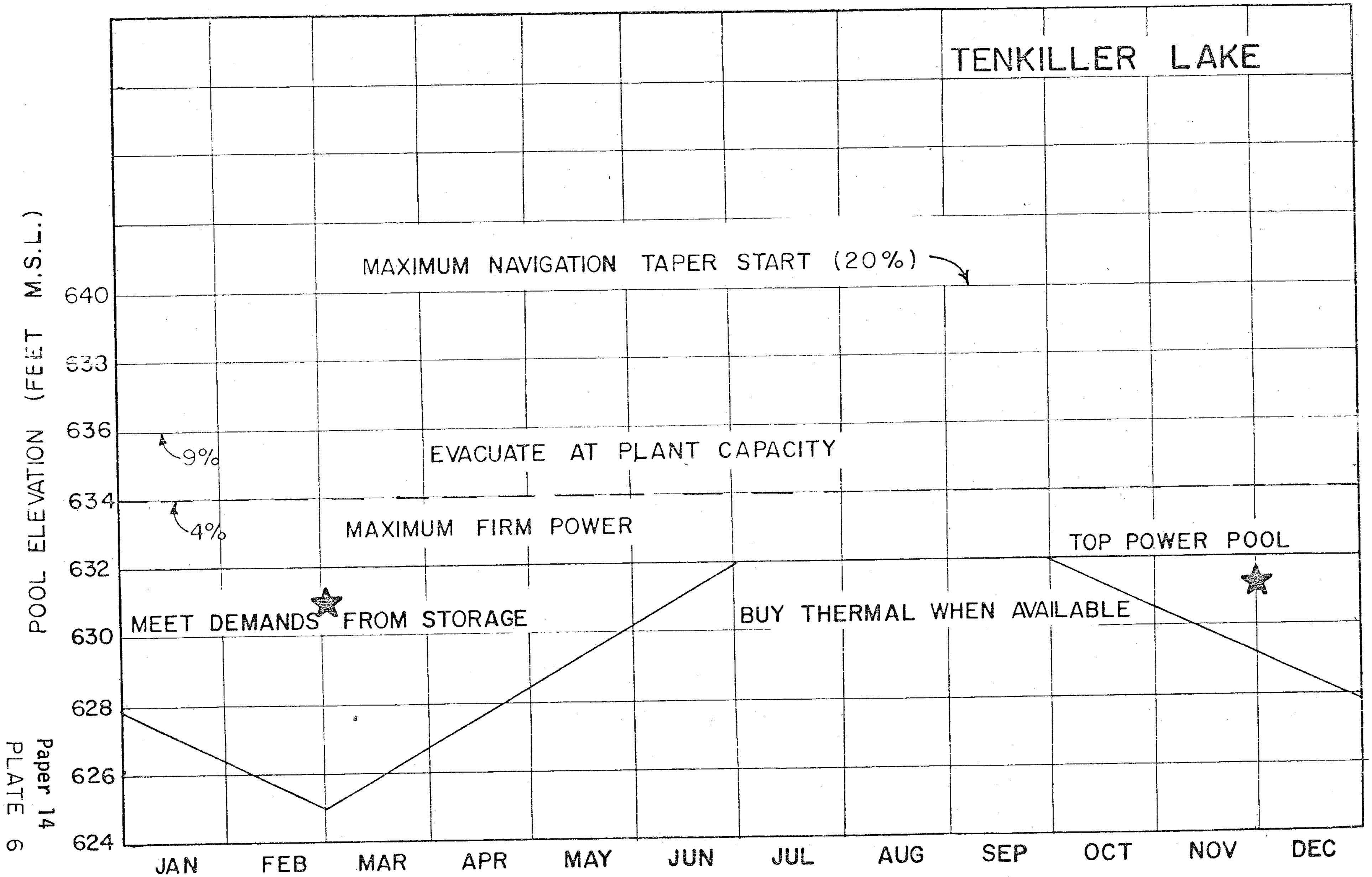


(1) A SMALLER GENERATION MAY BE REQUIRED FOR NAVIGATION

Paper 14

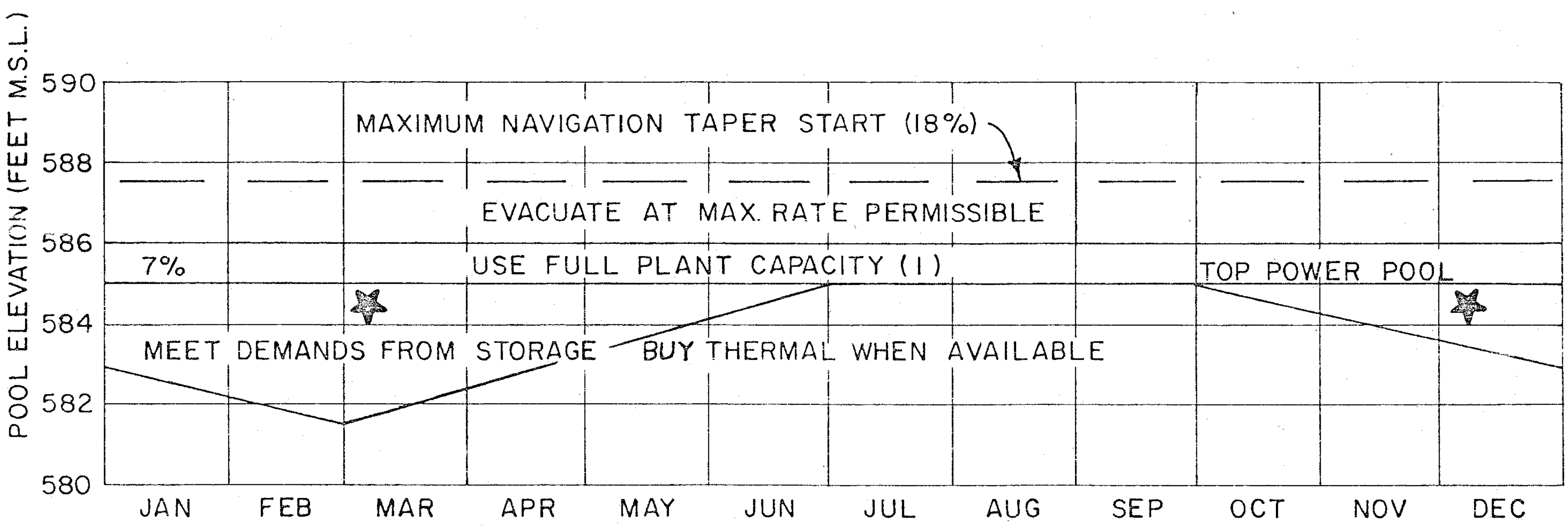
PLATE 4


OOLOGAH LAKE
MAXIMUM NAVIGATION TAPER START (25%)
RELEASE TO MAINTAIN 40,000 C.F.S. AT VAN BUREN, ARK.
Top of conservation pool
6%
12%
POOL ELEVATION (FEET M.S.L.)
646
644
642
640
638
636
634
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
FALL DRAWDOWN TO GIVE
NAVIGATION FLOWS WITH
INCIDENTAL POWER BENEFIT



Paper 14
PLATE 6

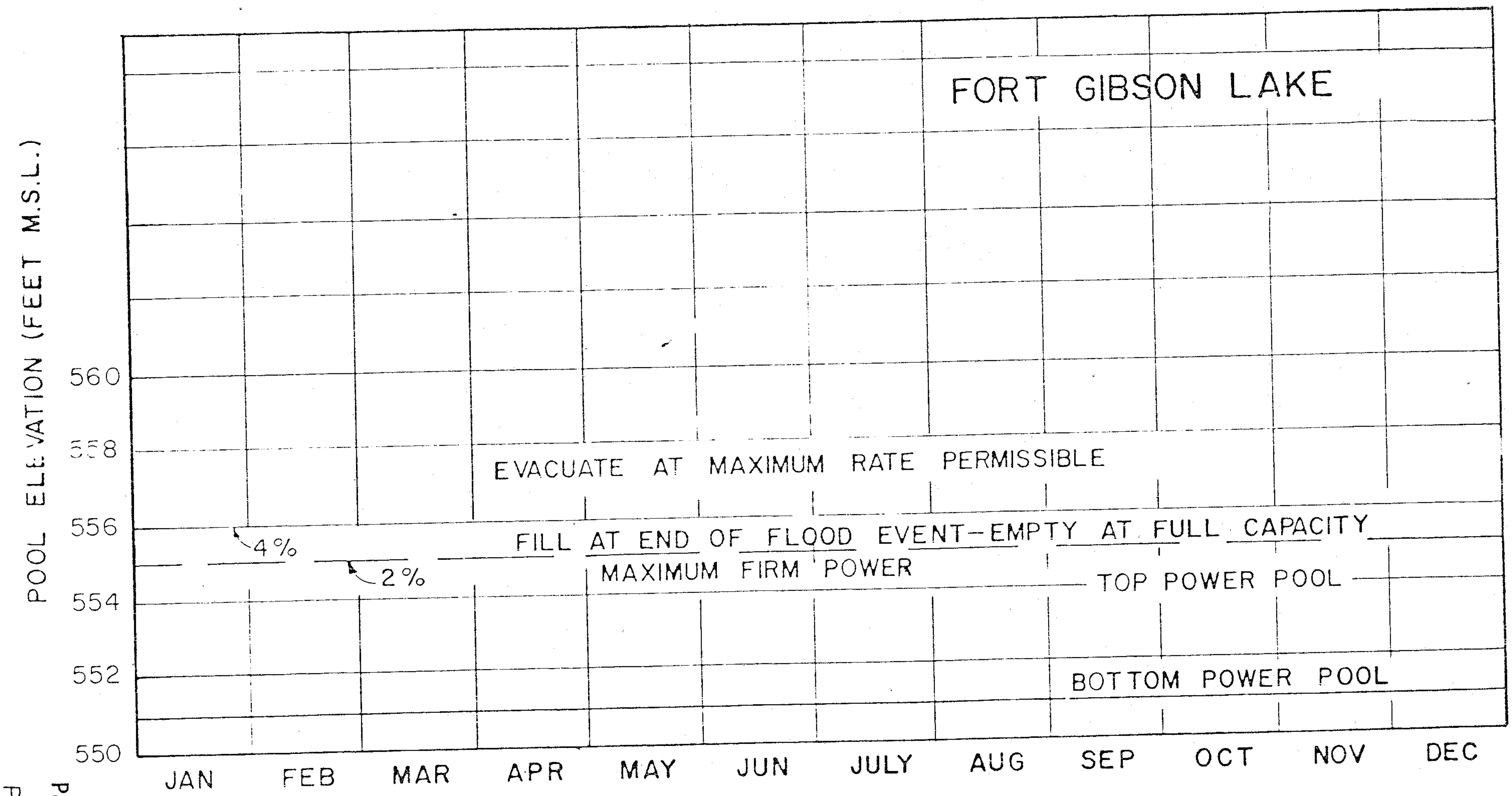
EUFAULA LAKE



(1) NAVIGATION MAY REQUIRE LESS

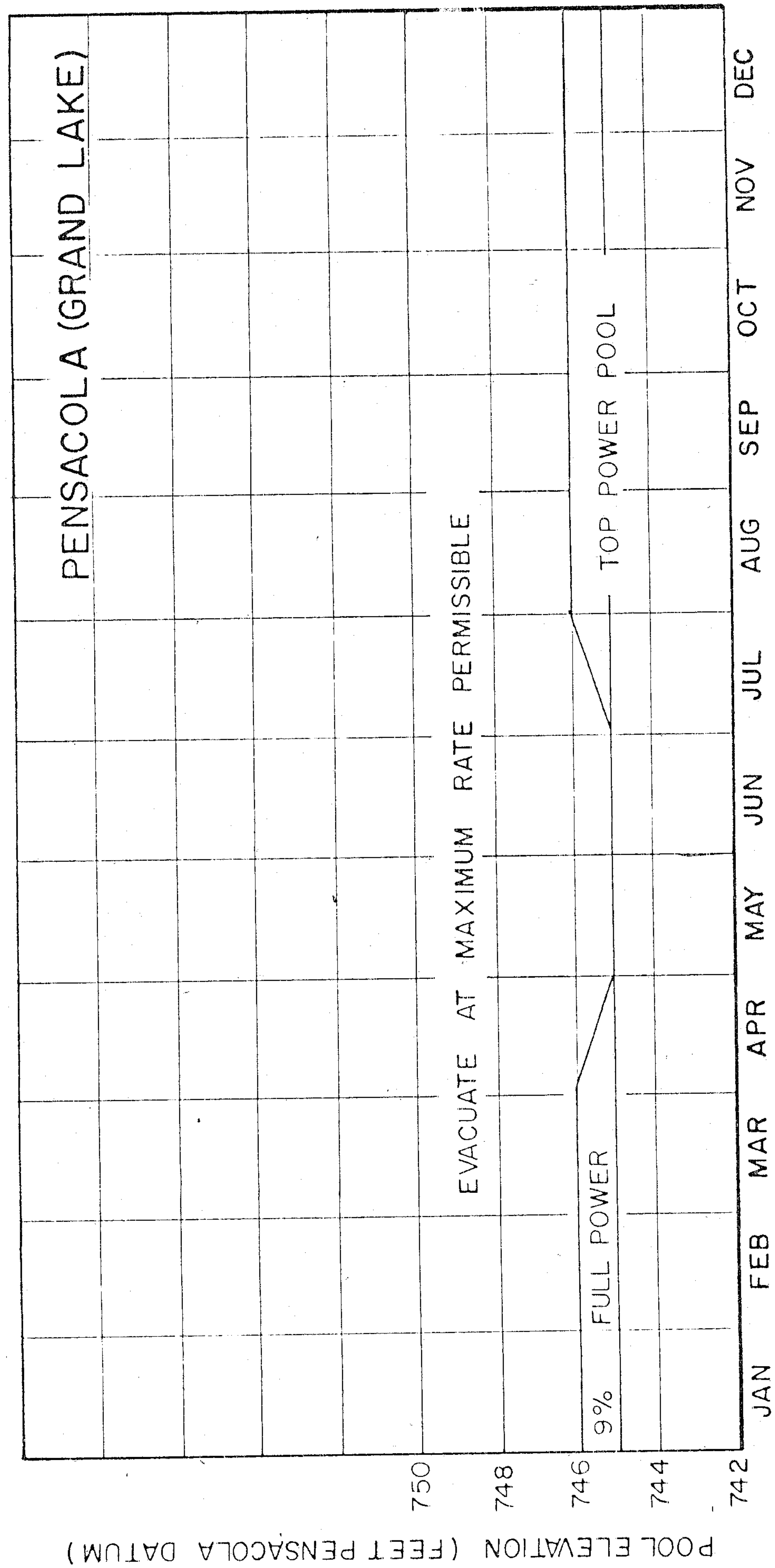
Paper 14

PLATE 7



Paper 14

PLATE 8



Paper 14

PLATE 9

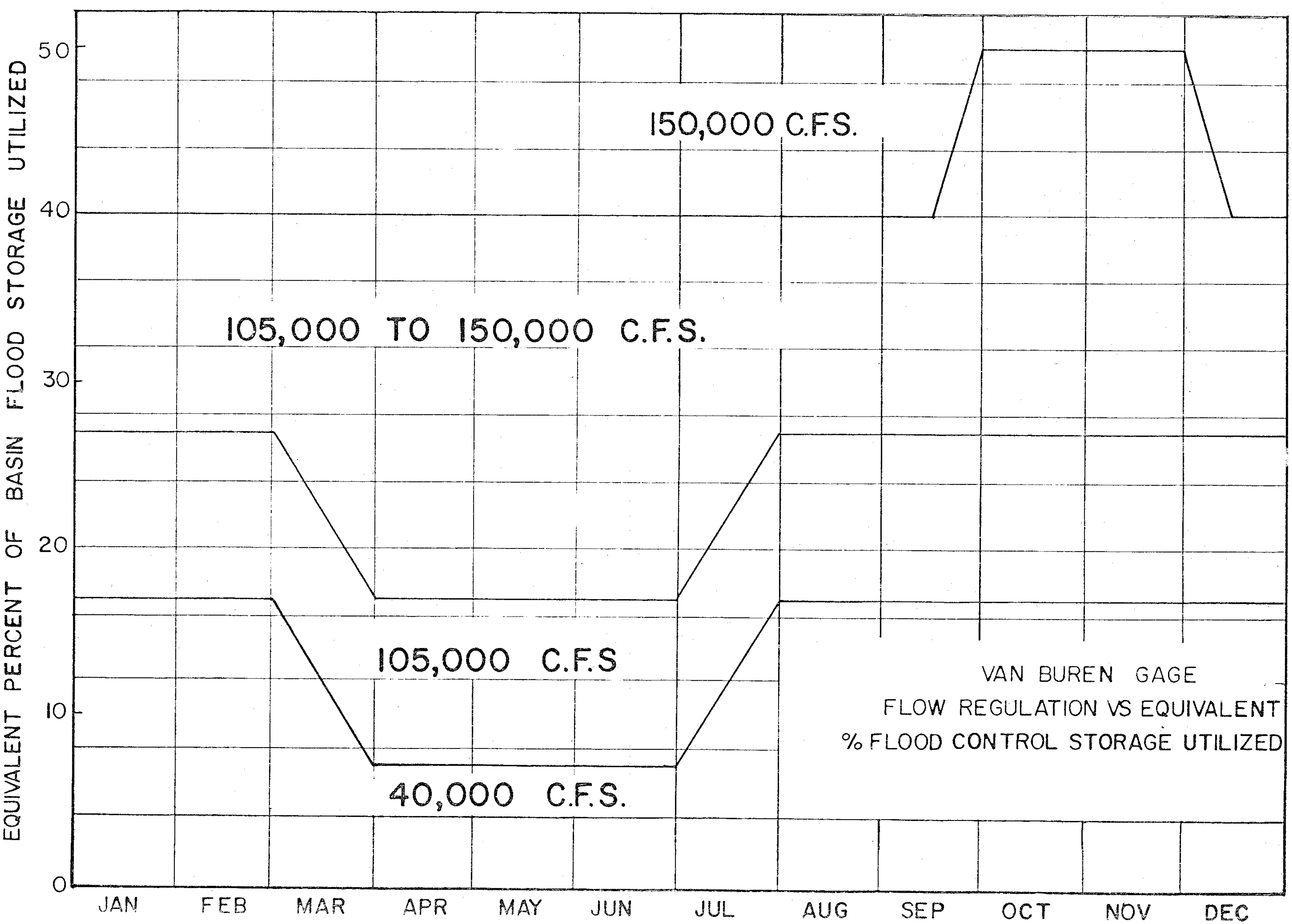
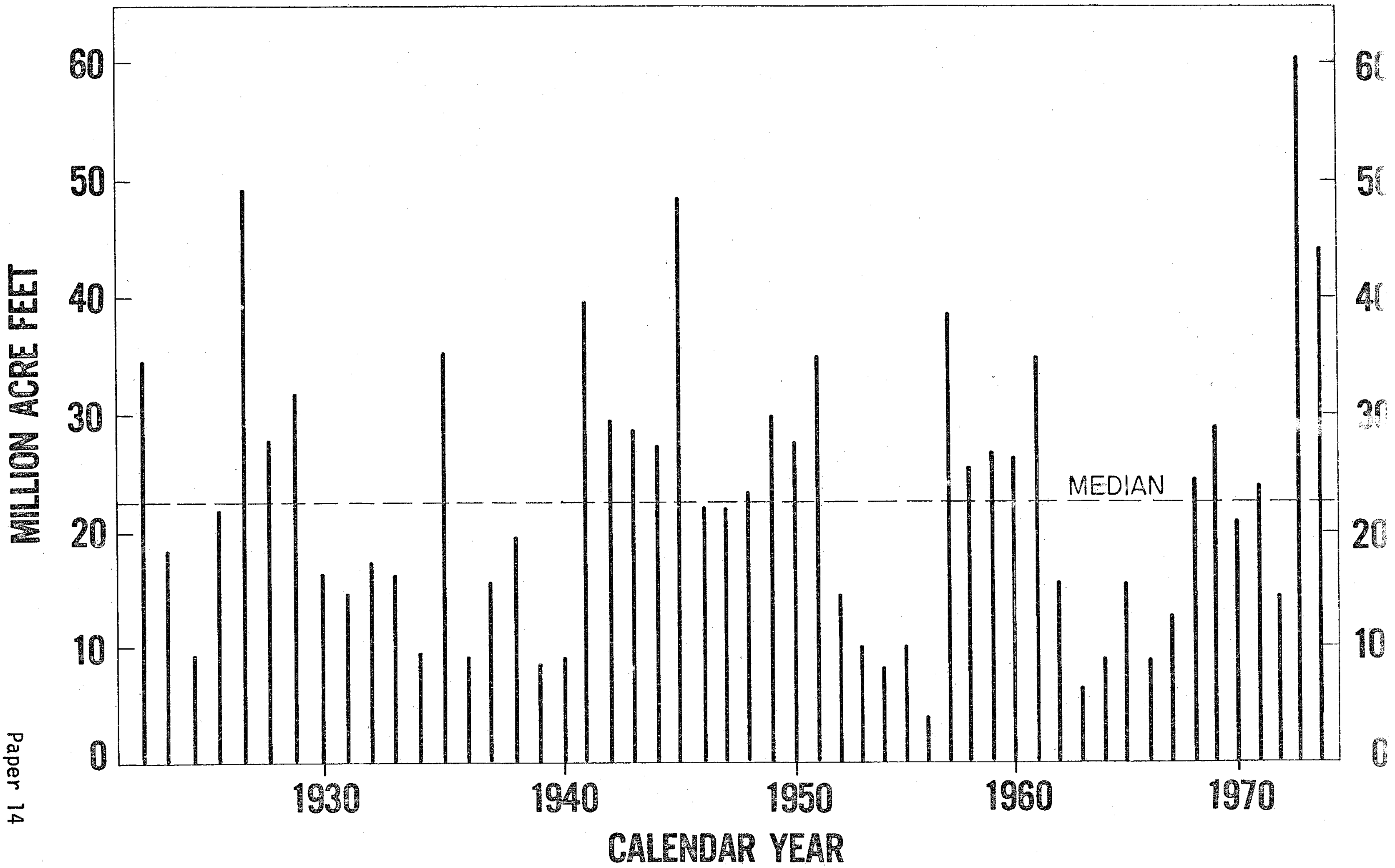


PLATE 10
Paper 14

FREQUENCY OF FILLING FLOOD POOLS

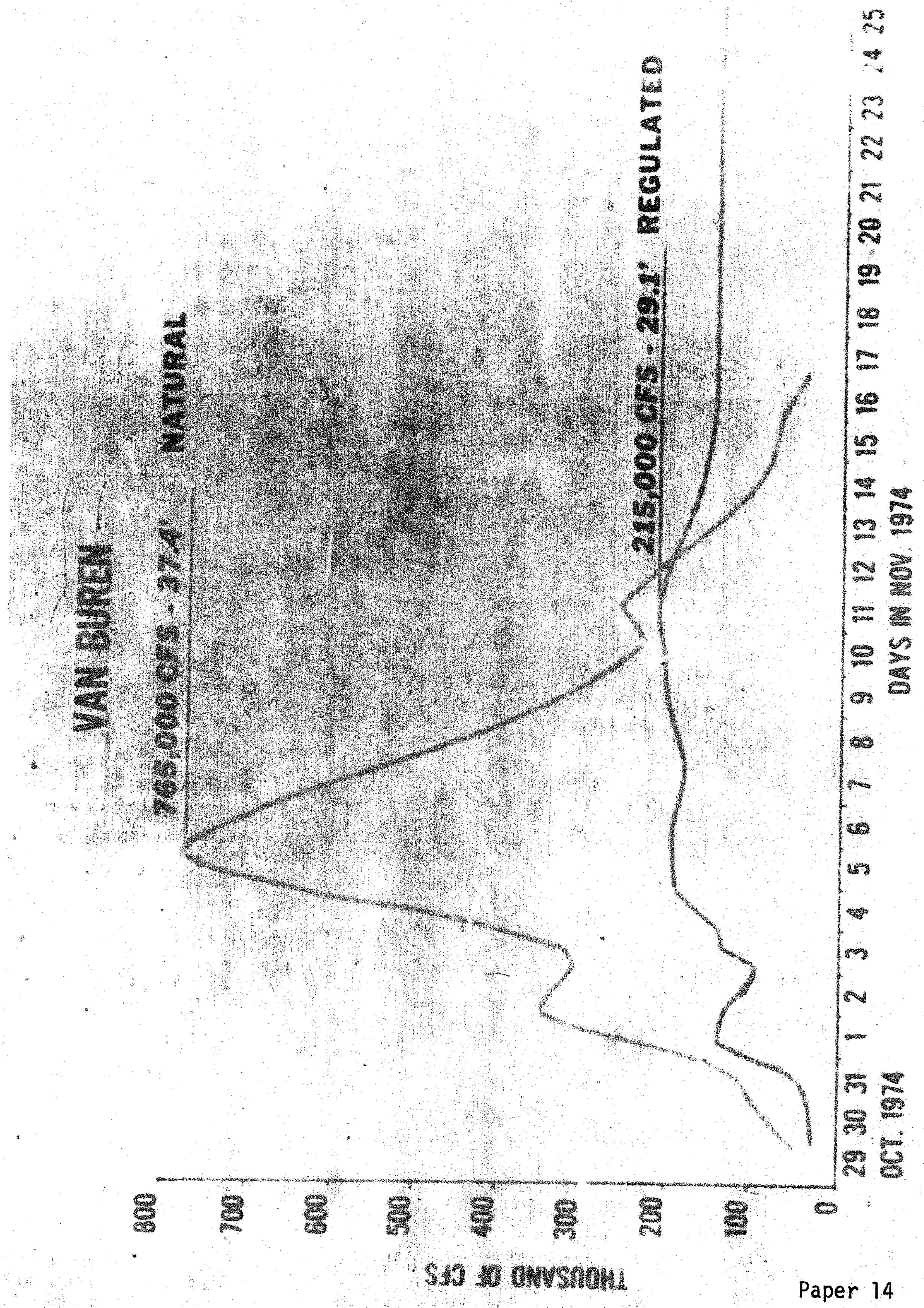
Reservoir	Original Design 150,000 cfs at Van Buren (Years)	Current Reduced Channel Capacity of 105,000 cfs at Van Buren (Years)	Proposed Interim Plan of Regulation with 105,000 cfs at Van Buren (Years)	Proposed Interim Plan of Regulation with 150,000 cfs at Van Buren (Years)
Keystone	13	5	4	12
Oologah	11	6	5	8
Pensacola	8	8	6	6
Markham Ferry	10	9	9	10
Fort Gibson	14	9	9	14
Tenkiller	9	8	6	7
Eufaula	13	10	5	9

ANNUAL RECORDED FLOW
ARKANSAS RIVER AT VAN BUREN, ARKANSAS



Paper 14

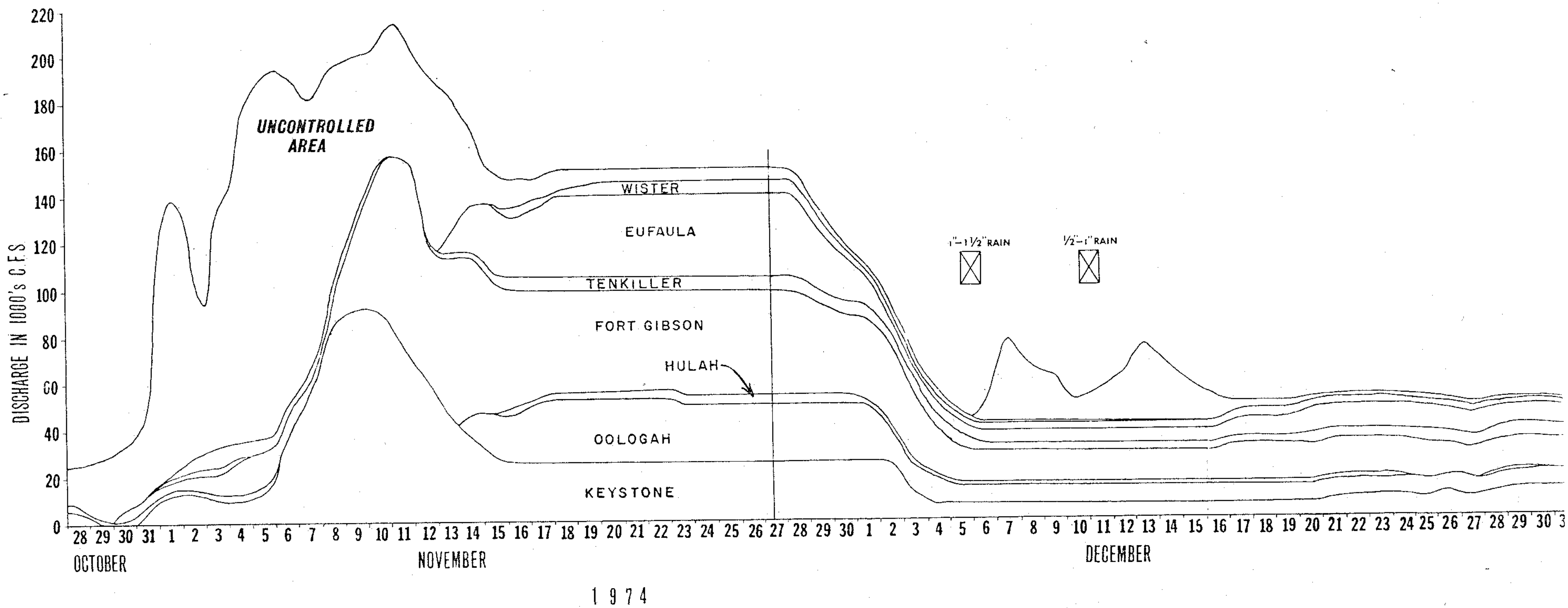
PLATE 12



Paper 14

PLATE 13

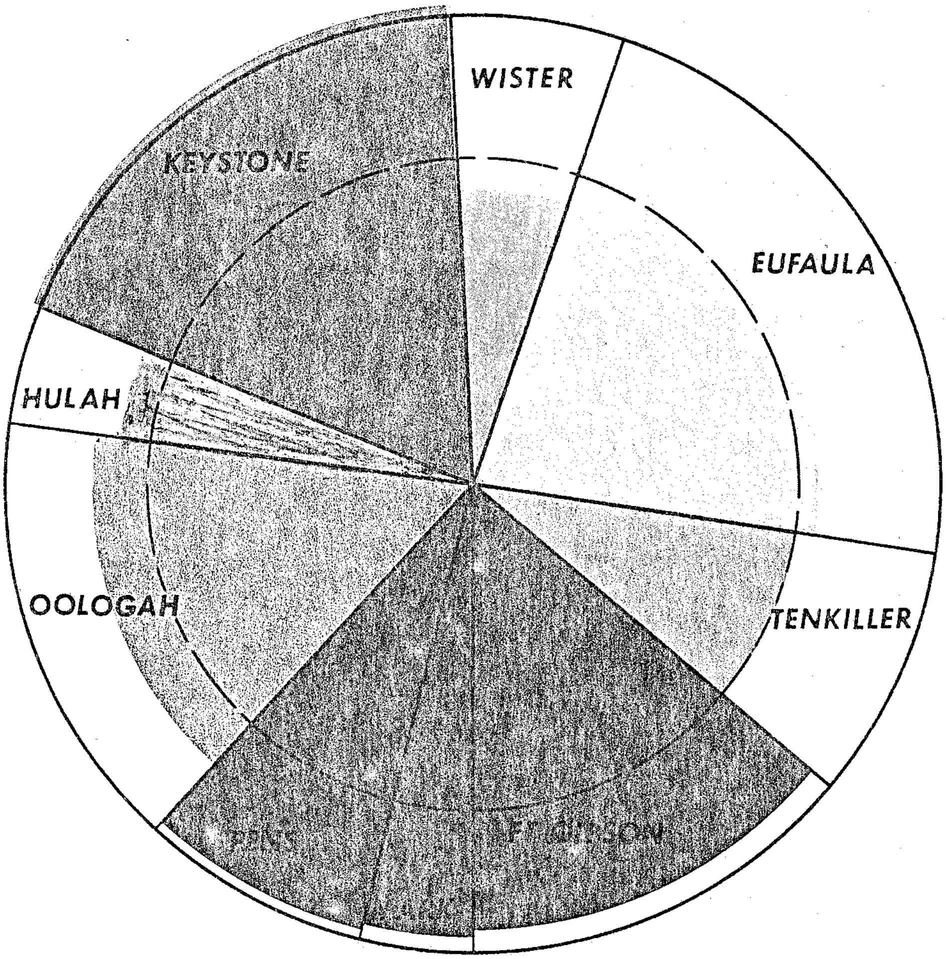
ACTUAL OPERATION
(VAN BUREN)



Paper 14

PLATE 14

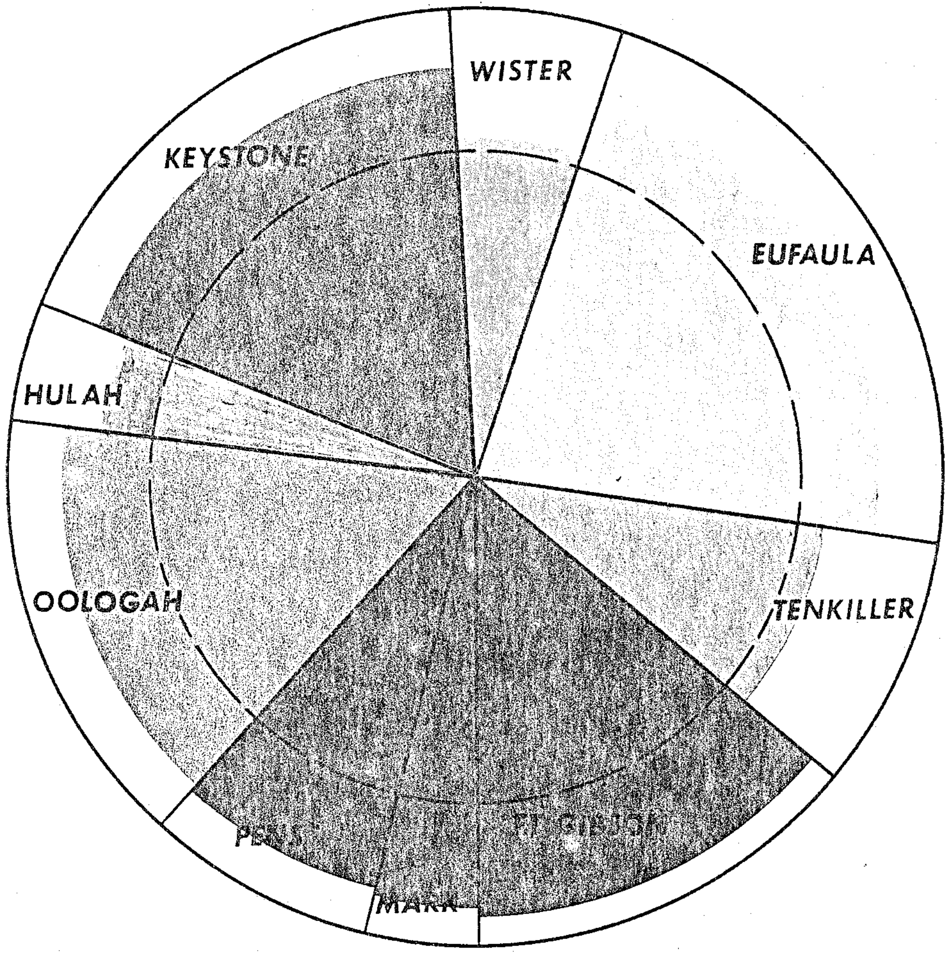
FLOOD CONTROL STORAGE STATUS



7 NOVEMBER 1974

TOTAL STORAGE 6,574,600 ACRE FEET

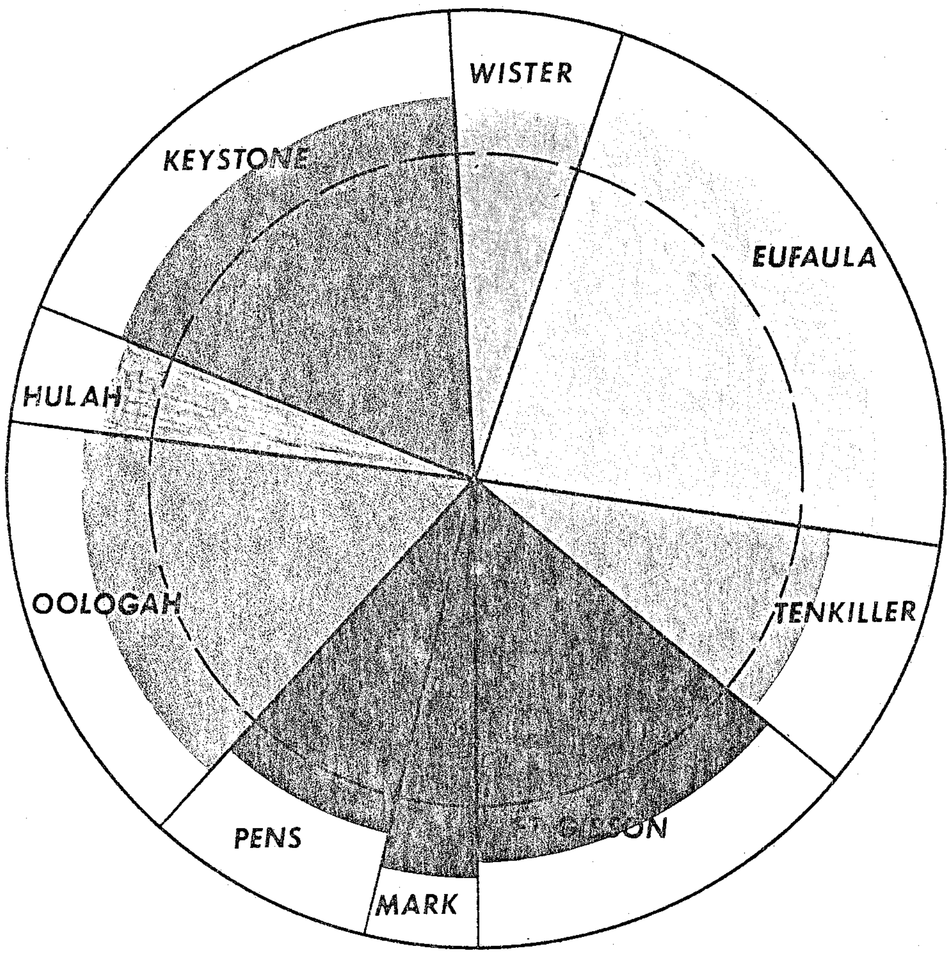
FLOOD CONTROL STORAGE STATUS



11 NOVEMBER 1974

TOTAL STORAGE 6,574,600 ACRE FEET

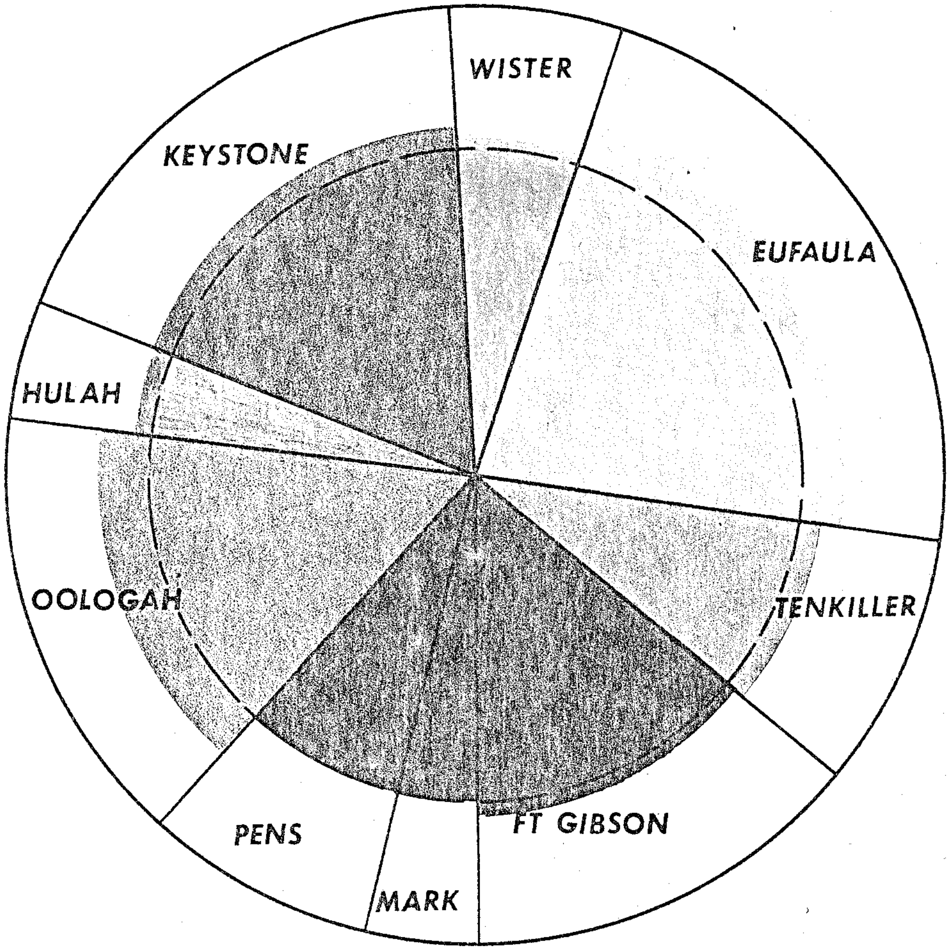
FLOOD CONTROL STORAGE STATUS



15 NOVEMBER 1974

TOTAL STORAGE 6,574,600 ACRE FEET

FLOOD CONTROL STORAGE STATUS

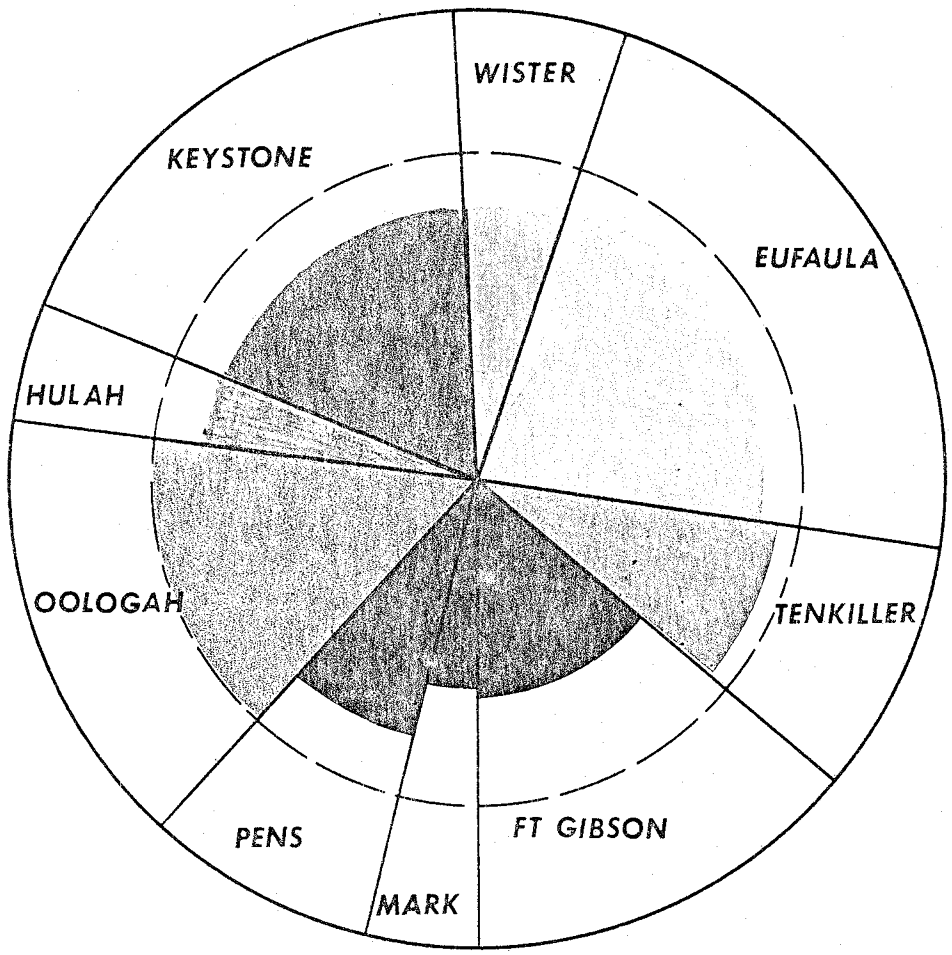


19 NOVEMBER 1974

TOTAL STORAGE 6,574,600 ACRE FEET

Paper 14

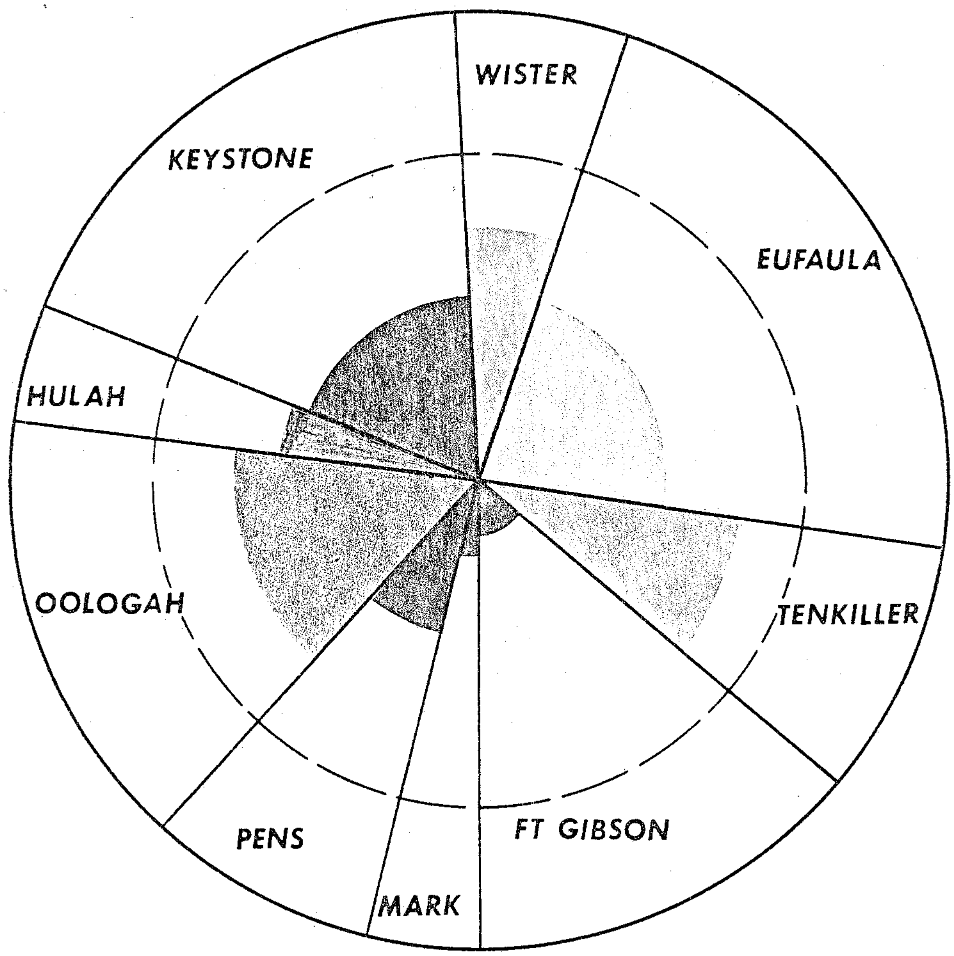
FLOOD CONTROL STORAGE STATUS



25 NOVEMBER 1974

TOTAL STORAGE 6,574,600 ACRE FEET

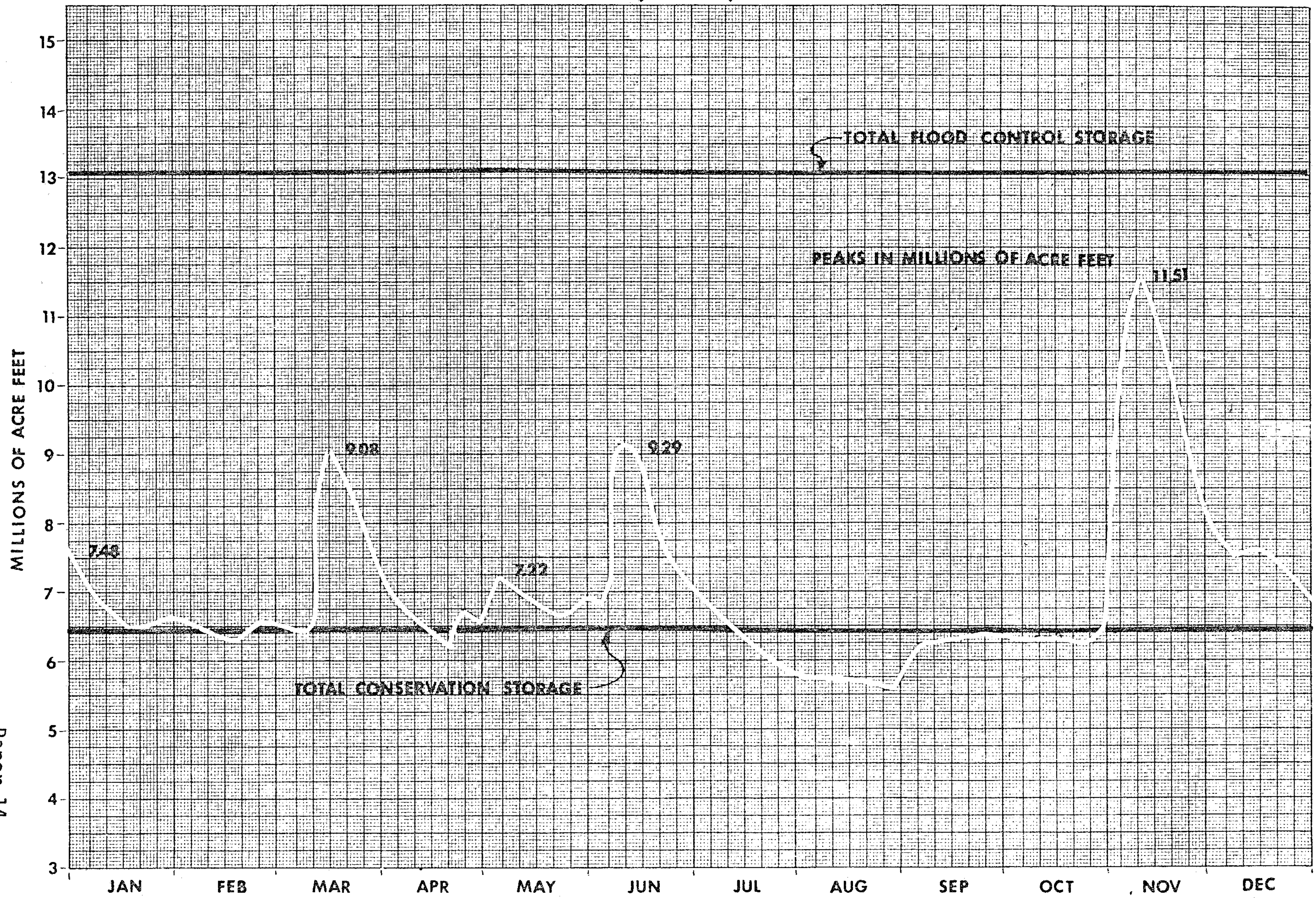
FLOOD CONTROL STORAGE STATUS



5 DECEMBER 1974

TOTAL STORAGE 6,574,600 ACRE FEET

FLOOD STORAGE ABOVE NAVIGATION SYSTEM
(1974)



Paper 14

PLATE 21

THE REGULATION PLAN FOR WATER CONTROL PROJECTS

by

James L. Butery[1], P. E.

Presented

at

THE HYDROLOGIC ENGINEERING CENTER
DAVIS, CALIFORNIA
NOVEMBER 1975

[1]Head, Reservoir Regulation Section, Hydrology and Hydraulics Branch, St. Louis District, Corps of Engineers

Paper 15

THE REGULATION PLAN FOR WATER CONTROL PROJECTS

INDEX

THE REGULATION PLAN FOR WATER CONTROL PROJECTS

Introduction

Because of the extreme complexity of problems associated with reservoir regulation, the regulation plan must be specific enough to provide firm guidelines for the regulator and establish a trend that the public can rely upon. Yet this same plan must be flexible enough to meet changing hydrological conditions. In addition to these requirements the plan of regulation should be practical, meeting special requirements of its locality and the needs of the affected public. This paper discusses the development of the regulation plan, practical considerations that should be made in regulating multi-purpose reservoirs and it provides a comparison of past and present regulating procedures at Lake Shelbyville and Carlyle Lake on the Kaskaskia River in southwestern Illinois.

Part I - Preparation of the Regulation Plan

Initial Development

The plan of regulation for a multi-purpose reservoir is necessarily developed prior to construction of the physical features of the project. During this phase of development, the engineer-regulator has no experience from this project from which he might base his decisions. His decisions must therefore be based on engineering data made available to him. Incorrect assumptions in the plan of regulation can result in costly over or under design of the project, in substantial monetary losses to the public affected by the project, and can result in the loss of credibility of the regulator-developer with the public.

Required Engineering Data

False engineering economies are often adhered to when the plan of regulation is developed. Proper engineering should include consideration as discussed in the following paragraphs:

Stream gages: Stream gages should be strategically deployed throughout the drainage basin as the first phase of development. These gages should be rated and sufficient data gathered to provide a good understanding of basin runoff conditions. Aerial photographs taken of the river channel during periods of bankful and overflow conditions, would be invaluable, combined with rated gaging stations, in determining channel capacities.

Surveying: Sufficient cross-sectioning should be provided to give an accurate channel capacity. Detailed mapping of the flood plain is also necessary and accurate economic studies should be made.

Practical Considerations

To provide the best plan of regulation, some practical considerations must be evaluated. The following paragraphs discuss a few of these as they relate to flood control.

Nature of flood protection: That which is to be protected from flooding should be identified. Permanent structures most likely would require full time protection, though an economic analysis should determine this. Lives of individuals require maximum protection though it may be more economical to displace an individual rather than add several feet of height to a reservoir for this individual's protection. Protecting agricultural lands during the dormant season may not be desirable. Overtopping of agricultural lands during the dormant season supplies nutrients in the form of sediment to the soil, it increases off-channel habitats which provide shelter and spawning areas for many species of fish and wildlife and in general increases the abundance of most biological components of the aquatic system.

Consequences of overprotection: Providing unneeded flood protection results in the expenditure of huge sums of money for lands and additional dam height to store runoff. It could result in long periods of releasing within bank flows that raise the water table, cause bank erosion, and actually cause flooding by creating a full river channel that local runoff cannot enter. In the reservoir the stored waters may interfere with recreation, result in flooding of perimeter lands, cause erosion of shoreline, and killing of shoreline vegetation.

Winter drawdown: The lowering of the reservoir surface below the summer pool level for the winter high runoff period is advantageous in several ways. First it provides additional flood storage; secondly, it may prevent flooding of perimeter lands; thirdly, it provides storage for spring runoff as the summer pool level is regained. The storage of spring runoff can be of great value if there is no rush to regain summer pool level and the refilling is accomplished from heavy storm runoff. The reason for this is that the heavy storm runoff does not have to be released thus preventing wet fields in agricultural areas during planting season.

Public Participation

Whatever the plan of regulation, the public will make requests for revisions that will better suit their individual needs. In order that their input might be constructive, it is highly desirable that they be organized, thus presenting a unified approach. The best time to organize is during the formulation of the regulation plan. If organized at this time, the plan of regulation can be coordinated with them. A realistic picture of what the project will accomplish, alone and as part of a system, should be presented. When regulation of the project actually begins the organization can prove of great value to the regulator by providing to him such information as completion of harvest dates, and other data pertaining to river flows and resultant damages.

Caution! Problems May Arise

During the initial period of regulation, information that was not available to the regulator-engineer during the designing stage now becomes available. The rate of change of the release rate that will result in bank washing or bank sloughing is one example of such data. Other examples are flow travel times, time required to deplete released waters from the river channel to prevent tributary runoff from overflowing from lack of river channel in which to flow, and other similar data. It is highly desirable to record such data as quickly as it becomes available (and to use this information) to avoid problems that might develop otherwise.

Priorities of Project Purposes

It would be desirable in the plan of regulation of multi-purpose reservoirs to spell out the priorities granted to each purpose. Project purposes conflict with one another in their demands on water levels and release rates. In periods of drought, water designated and used for water supply and navigation will result in curtailed recreation and hydro-power use. And unless special provisions are made, water released for hydro-power during the rainy season will contribute to flooding. The regulator-engineer should, therefore, consider all possibilities of conflict between project purposes, and establish priorities. It is evident that if flood protection is one of the project purposes, it should have the highest priority. Anything less and the flood protection feature will be undermined with resultant losses. On the other hand, during drought periods, water supply would be of the greatest concern. With the present energy shortage, hydro-power should probably have the next priority. As desirable a feature as recreation and fish and wildlife may be it would seem most appropriate for these to receive last consideration, making use of the reservoir under whatever conditions may exist.

Part II - Regulation Experiences within the St. Louis District

Having discussed the preparation of a plan of regulation for multi-purpose reservoirs let us take a look at a real situation as experienced in the St. Louis District. I am sure this is just one example of many similar reservoir projects.

General

The Kaskaskia River Basin in southwestern Illinois was developed by the construction of three projects in series. At the mouth of the Kaskaskia River near Chester, Illinois, a lock and dam has been constructed and 50 miles of the river has been channelized. About 100 miles upstream (mile 106.6) a multi-purpose reservoir was built (Carlyle Lake) and full joint use pool elevation gained in December 1967. At mile 221.8 a second multi-purpose reservoir (Lake Shelbyville) was built and full joint use pool gained in July 1971. River levees intended for protection of agricultural areas were not constructed, however, a levee was built to protect the town of New Athens. The general purpose of the reservoir projects are flood control, navigation on the lower 50 miles of the river, recreation, water supply, low flow augmentation and fish and wildlife enhancement. The possibilities of providing hydro-electric power are currently being studied for Carlyle Lake.

Carlyle Lake

The original plan of regulation at Carlyle Lake, the first of the three projects, called for a release rate of 10,000 c.f.s. This was a miscalculation resulting from accepting survey report data without re-study and failure to completely develop the basin. It has now been determined that 4,000 c.f.s. is the non-damaging release rate. By adopting the incorrect strategy of providing total flood protection to the downstream area the pool was forced to unnecessarily absorb basin runoff. This combined with reduced channel capacities caused pool stages to become excessively high exceeding the pool frequency curve and causing extreme lake perimeter erosion. When a pre-determined pool level was reached during the dormant season (20% of flood control storage pool utilized at elevation 450 feet m.s.l.) above bankful releases of 7,000 c.f.s. became mandatory until the accumulated storage was depleted. The storage depletion time was commonly in excess of 30 days which resulted in bank erosion to the downstream area.

Lake Shelbyville

Upon completion of Lake Shelbyville, like problems came to light. The downstream channel capacity estimated to be 4,500 c.f.s. for design purposes is now determined to be 1,800 c.f.s. for non-damaging releases.

Public Sentiment

As a result of developing an inappropriate plan of regulation in the Kaskaskia River Basin, relations with the public served became strained. Recreationists who made use of the pools were flooded out and farmers adjacent to the pools were flooded more frequently than had been determined. Downstream of the projects the farmers were kept out of their fields a good portion of the time and they suffered from excessive erosion of the stream banks. Club house owners along the river were kept out of the area for much longer periods of time than under natural conditions.

In order to present a consolidated viewpoint to the Corps, the people began to organize. Once organized, their united front began to produce positive results in developing a new plan of regulation that would better serve their varied interests. They made concessions in some areas in order to obtain maximum benefits from projects that they had come to consider as the source of most of their daily problems.

The attempts of the public to organize into united fronts was encouraged wholeheartedly by the St. Louis District, Corps of Engineers. These organizations now work closely with the pool regulators by providing input for daily decisions as well as long range planning. The pool regulator is just a telephone call away from representative spokesman relaying data as to crop conditions, local flooding, or imagined anxieties. The working relationship with these organizations is still somewhat strained as a result of past regulating experiences but good regulating henceforth should remedy the situation.

Part III - New Regulation Plans for St. Louis District Reservoirs

As pressures mounted for remedial action to be taken to improve the regulation of Lake Shelbyville and Carlyle Lake a new plan was under study. This new plan was put into effect on 1 October 1974. It differed from the previous plan in three major features:

1. Earlier and staggered drawdown to winter pool levels.

2. Flood protection limited to the growing season or major floods.

3. Later date for regaining summer pool level.

Early Winter Drawdown

To provide additional flood storage during the winter high runoff period Lake Shelbyville and Carlyle Lake in recent years planned the lowering of pool levels following waterfowl hunting season. Waterfowl season ended in early to mid-December and the drawdown attempt was accompanied by the high runoff season. To alleviate this problem, Lake Shelbyville's period of drawdown was scheduled to begin 1 October. In addition to drawing down prior to the high runoff season, Lake Shelbyville's waters could be passed through Carlyle Lake prior to attempting the drawdown of that reservoir. Carlyle Lake's drawdown begins 1 December and is accomplished in half the time required when Lake Shelbyville's releases were being passed through simultaneously.

Limited Flood Protection

Limited channel capacity made complete flood protection impossible. During winters high runoff period, waters stored from one storm could not be depleted before successive storms would occur. The pools, therefore, continued a steady rise through the winter and into the spring. To stop what the public came to know as "controlled flooding" with damages mounting daily, action had to be taken. The farming community agreed to foresake planting of winter wheat in the bottom lands in exchange for maximum protection during the growing season (May to harvest). The new regulation plan, therefore, passes winter storm runoff through the reservoirs in as natural a state as possible up to the maximum rates of release authorized. In the one year that the plan has been in effect, there have been only minor complaints from the affected public. Most of these complaints are from isolated individuals that desire a plan of regulation contrary to that of the majority. The other complaints have been received with an open mind and minor adjustments are being made in the regulation procedures to alleviate the problems. The farming community has experienced their best crops in 25 years and the recreationist both on the pools and along the river channel have enjoyed excellent conditions.

Regaining Summer Pool

In previous years of the recent past, the storing of runoff to regain the summer pool level was begun in mid-April. Storms that occurred in late May and June resulted in high pool levels. This interfered with recreational uses of the pool, caused shore line bank erosion, loss of vegetation and reduced the flood control capacity. The new plan of regulation delays the regaining of summer pool level until 1 May and if the pool level is excessively high passing of flows may continue for a brief period. This delay in regaining summer pool level reduces flood protection for early spring planting. In doing so it increases the chances of obtaining some crop from the river bottom lands by providing maximum protection for crops planted after 1 May. Those utilizing Carlyle Lake for sailboating this past year were unhappy with the delayed start they got in their sporting endeavor, however, the ideal pool level they experienced during the remainder of the season more than repaid them for their inconvenience.

A Brighter Future

The new plan of regulation has been in effect only one year, the weather was wetter than the previous two years but the precipitation was evenly distributed. The trial period is still in effect, but, if during the coming few years Lake Shelbyville and Carlyle Lake can provide the same degree of benefits as experienced this year the image projected by these projects may change, and that can only be for the better.

Part IV - Conclusions

There are three points that this paper was intended to make apparent so that future projects and perhaps those with present inadequate regulation plans, might better serve the needs of the public.

Primarily an attempt was made to focus attention on the regulation plan as the foundation on which to build a reservoir project.

Secondly, attention has been directed to the fact that overtopping of agricultural lands during the dormant crop season can be beneficial, thus, reservoirs may not need provide storage for winter runoff.

Thirdly, it should be recognized that basins developed partially will not provide the flood control benefits credited to the complete system.

Special Note: A draft ETL titled "Reporting of Hydrologic Engineering Studies for Civil Works Projects" has been issued for review and comment which provides excellent guidance to the hydraulic engineer. Use of this ETL will tend to eliminate or minimize the types of problems discussed in this paper.

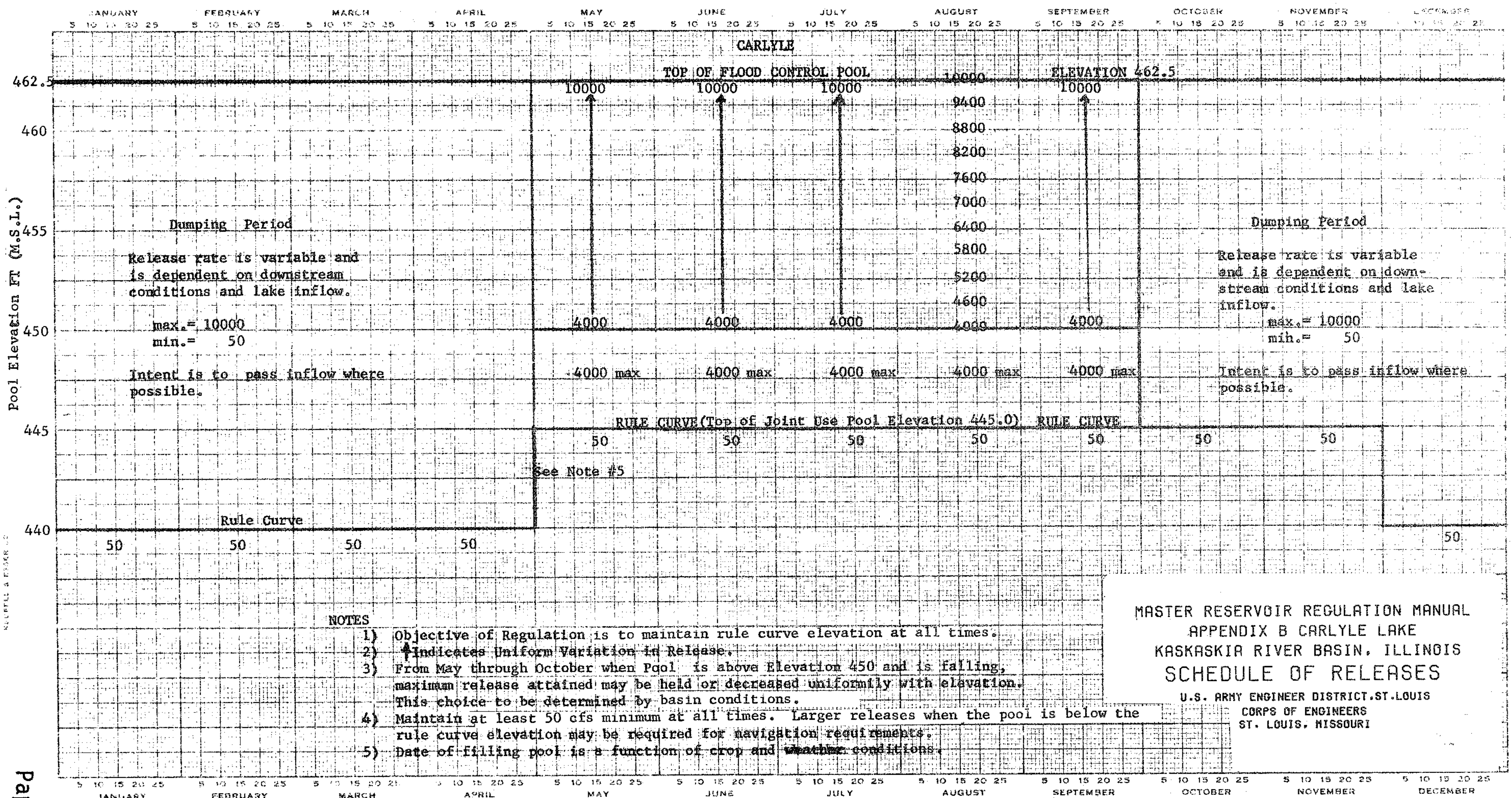


PLATE 6

Carlyle Lake Release Schedule
(Adopted 1 Oct 1974)

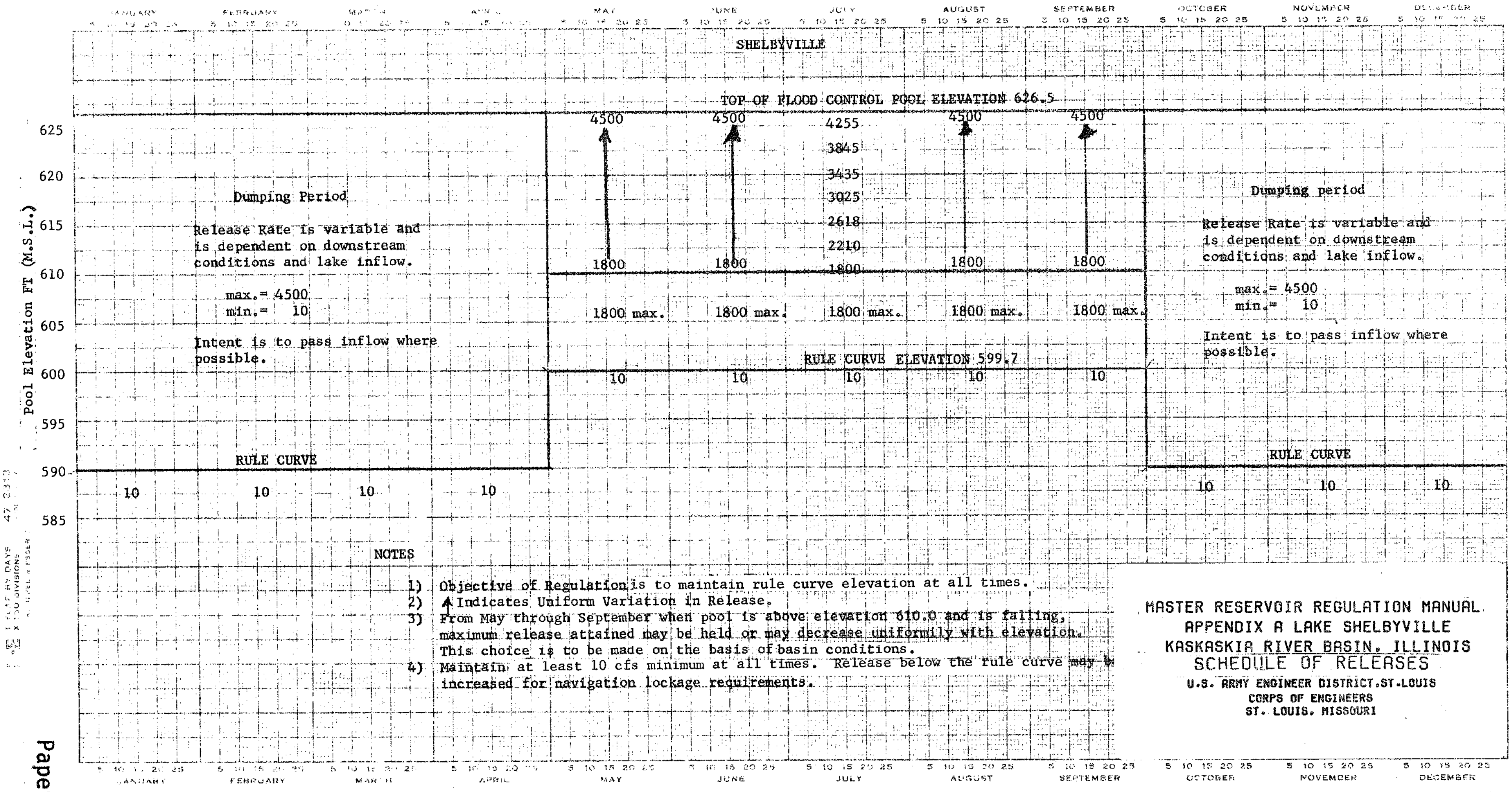


PLATE 28

Lake Shelbyville Release Schedule
(Adopted 1 Oct 1974)

The following five pages

present annual graphs 1971-75

depicting pool levels, rainfall

inflow and outflow for

CARLYLE LAKE

Nominclature

.... Max. Pool of Record
—— Pool Elevation
- - - Rule Curve

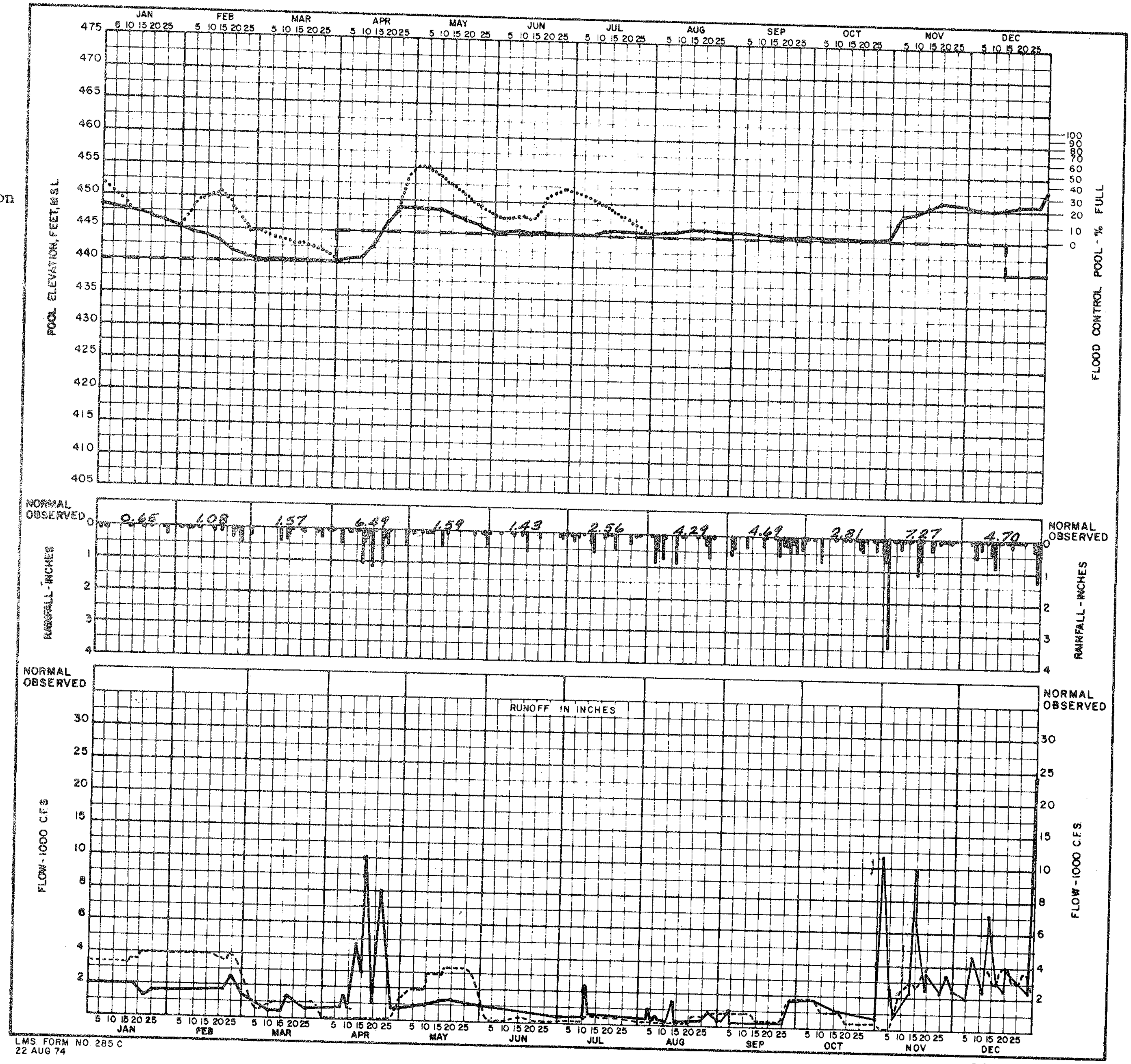
LMS FORM NO 285 C
22 AUG 74

CARLYLE LAKE
1971

—— Inflow
- - - Outflow

Nominclature

.... Max. Pool of Record
——— Pool Elevation
– – – Rule Curve

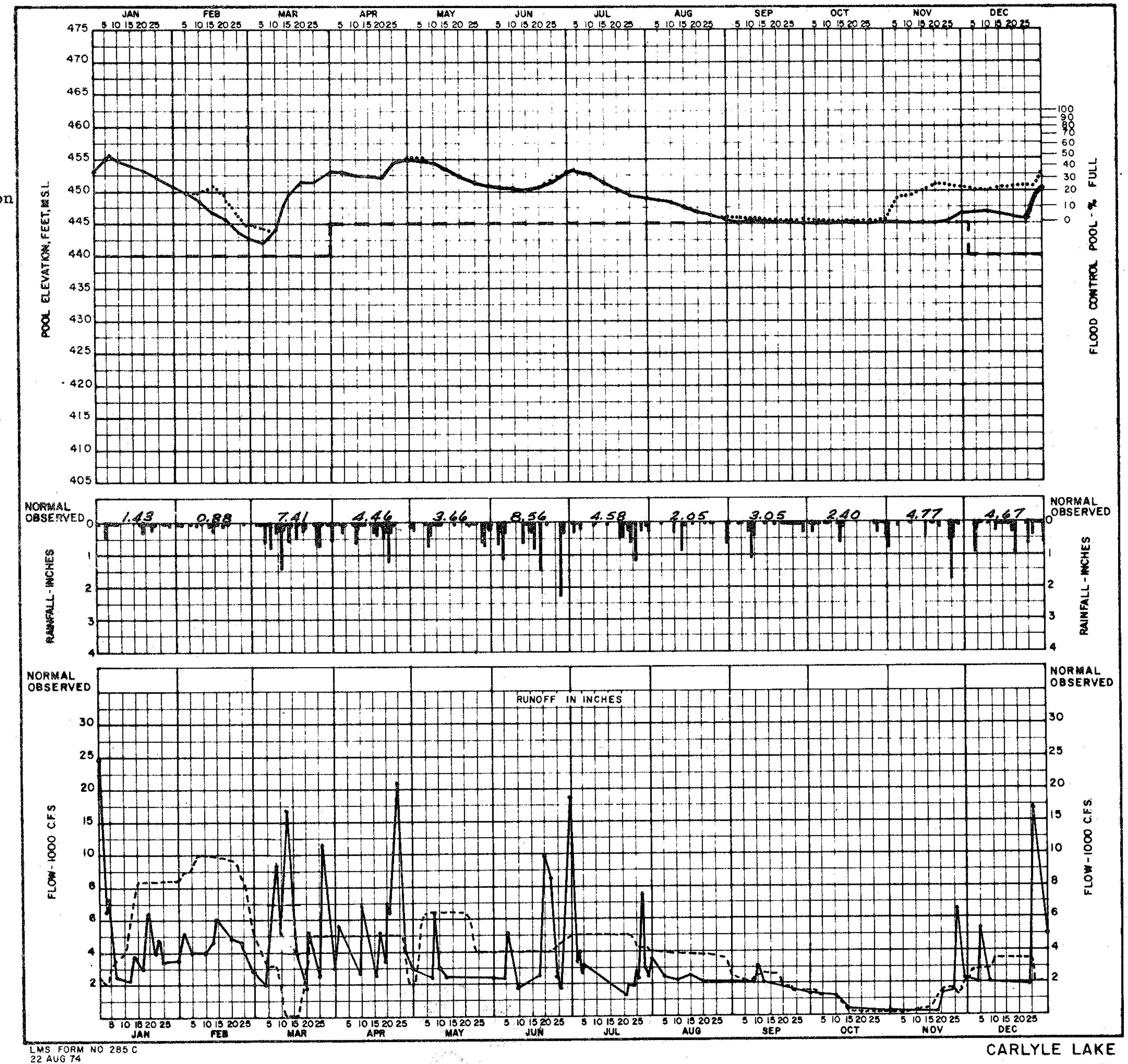


——— Inflow
– – – Outflow

CARLYLE LAKE
1972

Nominclature

...... Max. Pool of Record
—— Pool Elevation
- - - Rule Curve

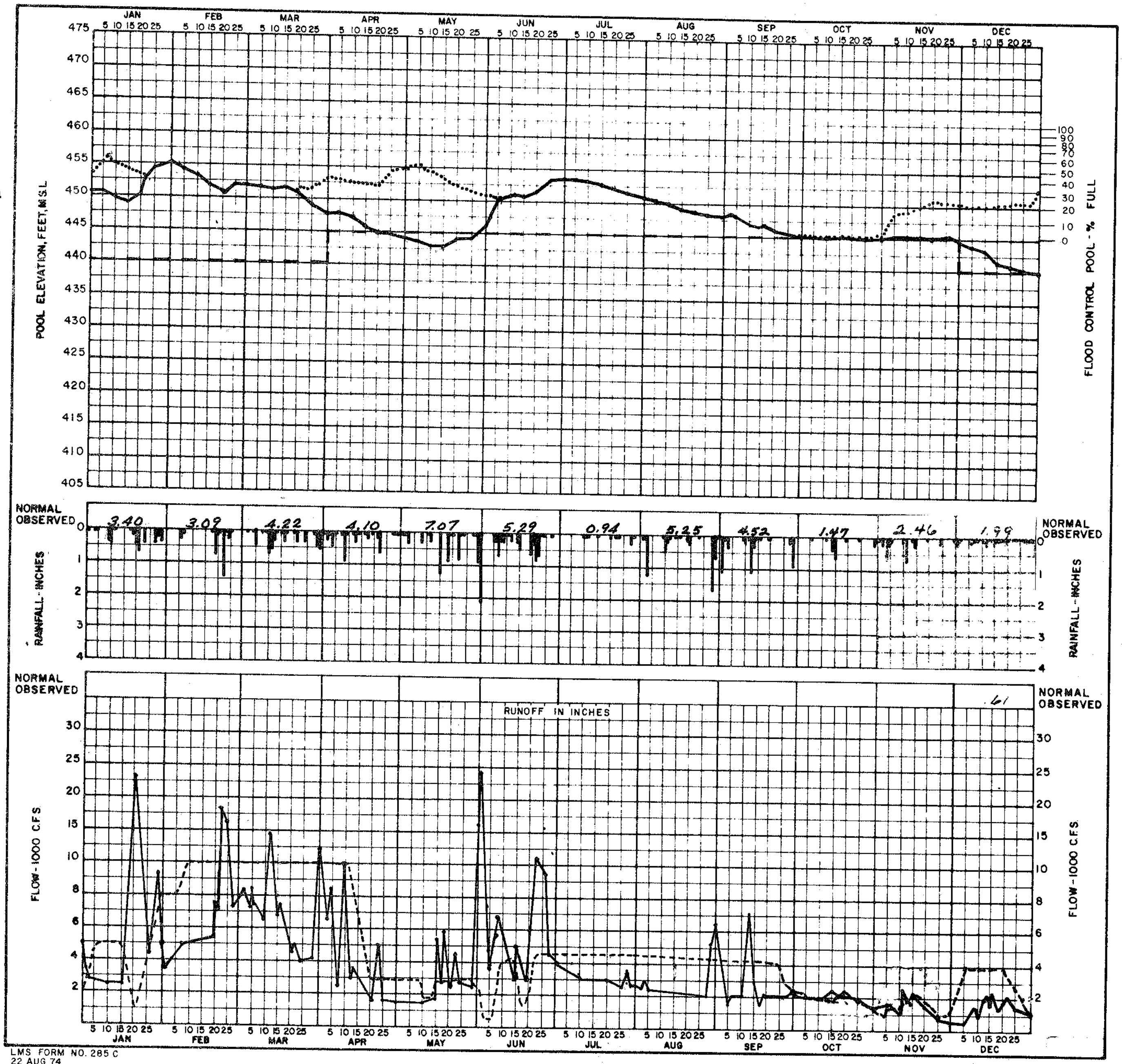


CARLYLE LAKE
1973

—— Inflow
- - - Outflow

Nominclature

...... Max. Pool of Record
——— Pool Elevation
– – – Rule Curve

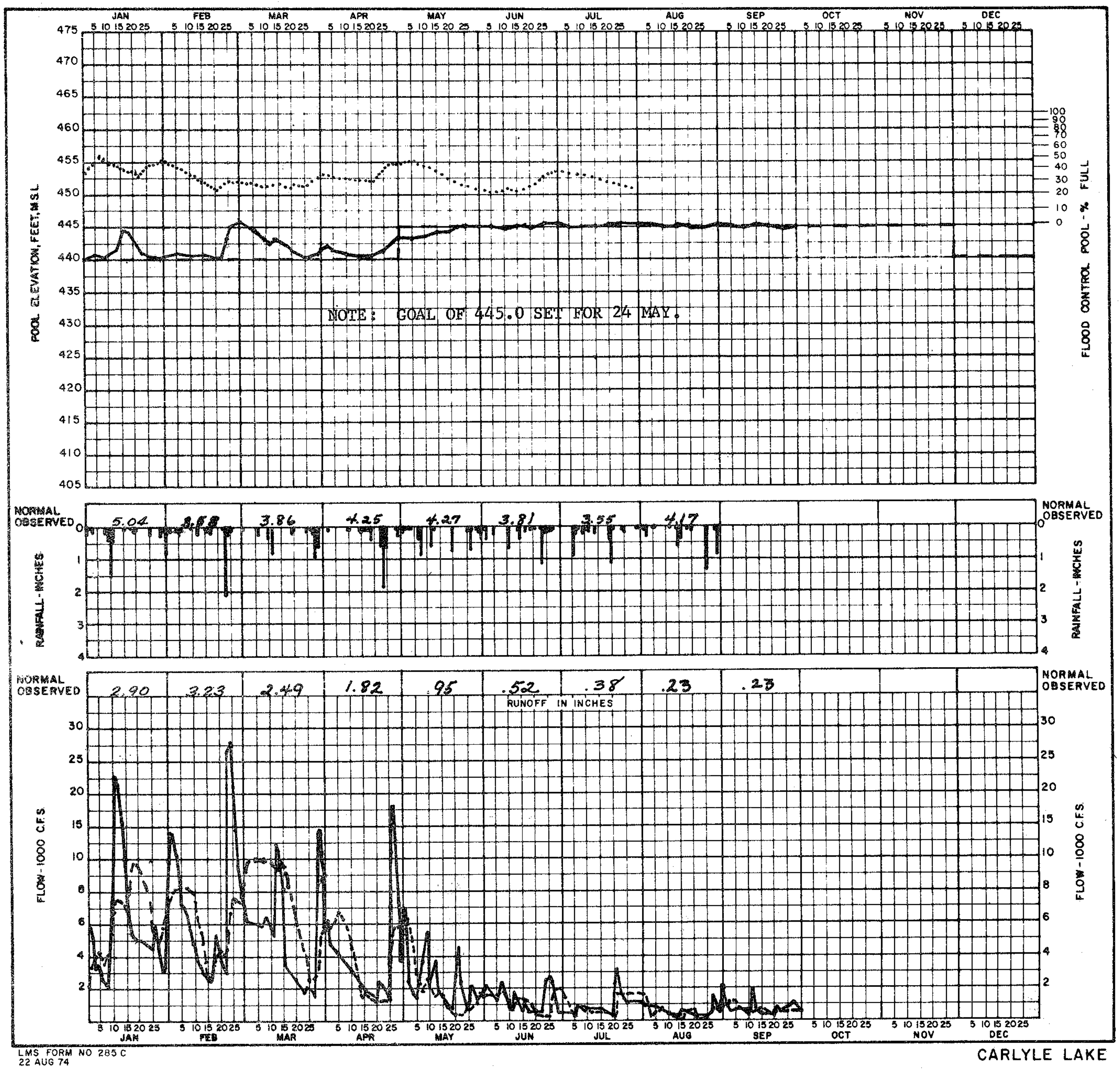


——— Inflow
– – – Outflow

CARLYLE LAKE
1974

Nominclature

....Max. Pool of Record
____Pool Elevation
- - - Rule Curve



CARLYLE LAKE 1975

____Inflow
- - -Outflow

The following five pages

present annual graphs 1971-75

depicting pool levels, rainfall

inflow and outflow for

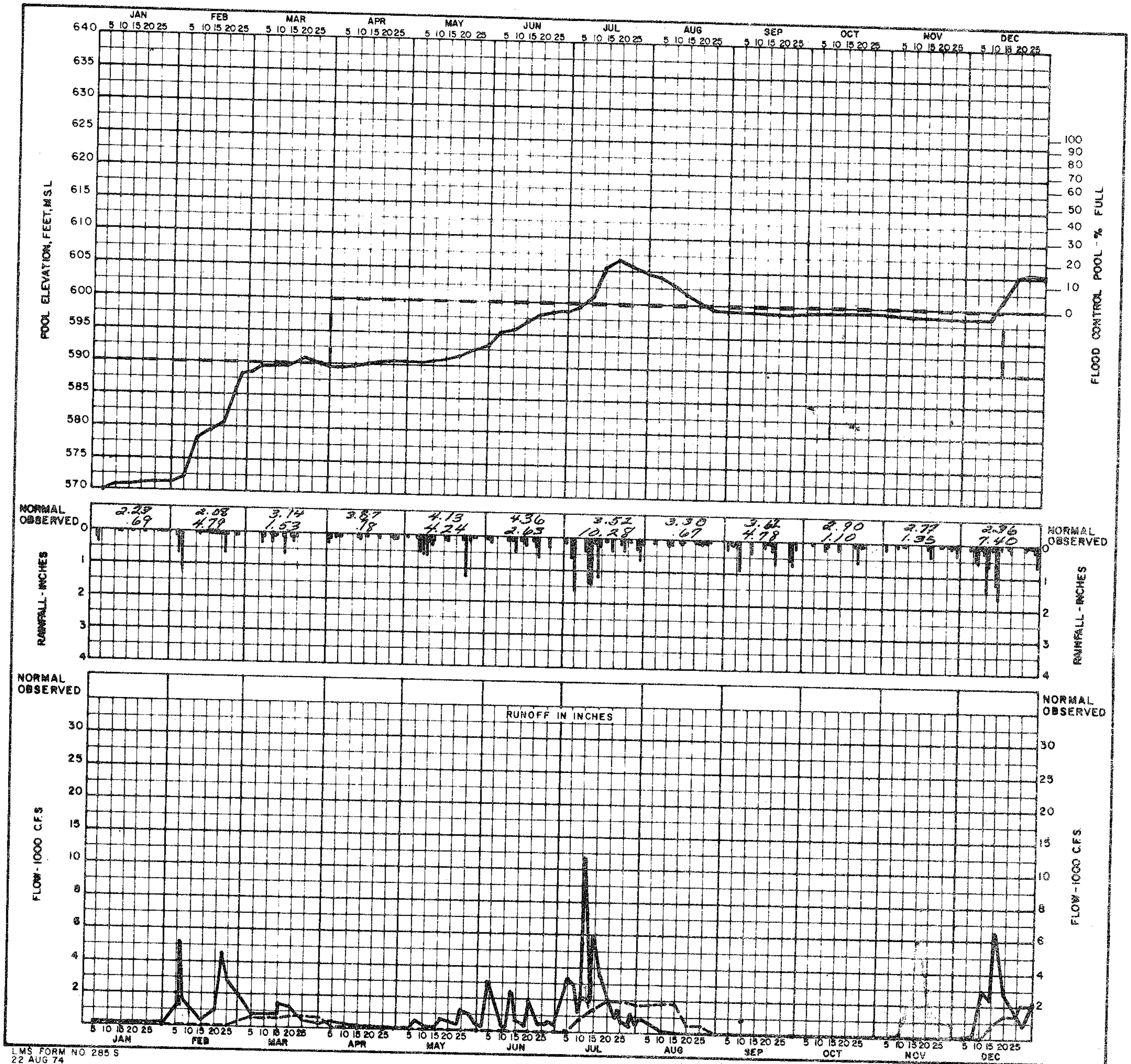
LAKE SHELBYVILLE

Diversion made 24 June 1969
Gates first closed 1 August 1970

Nominclature

..... Max. Pool of Record
____ Pool Elevation
_ _ _ Rule Curve

____ Inflow
_ _ _ Outflow

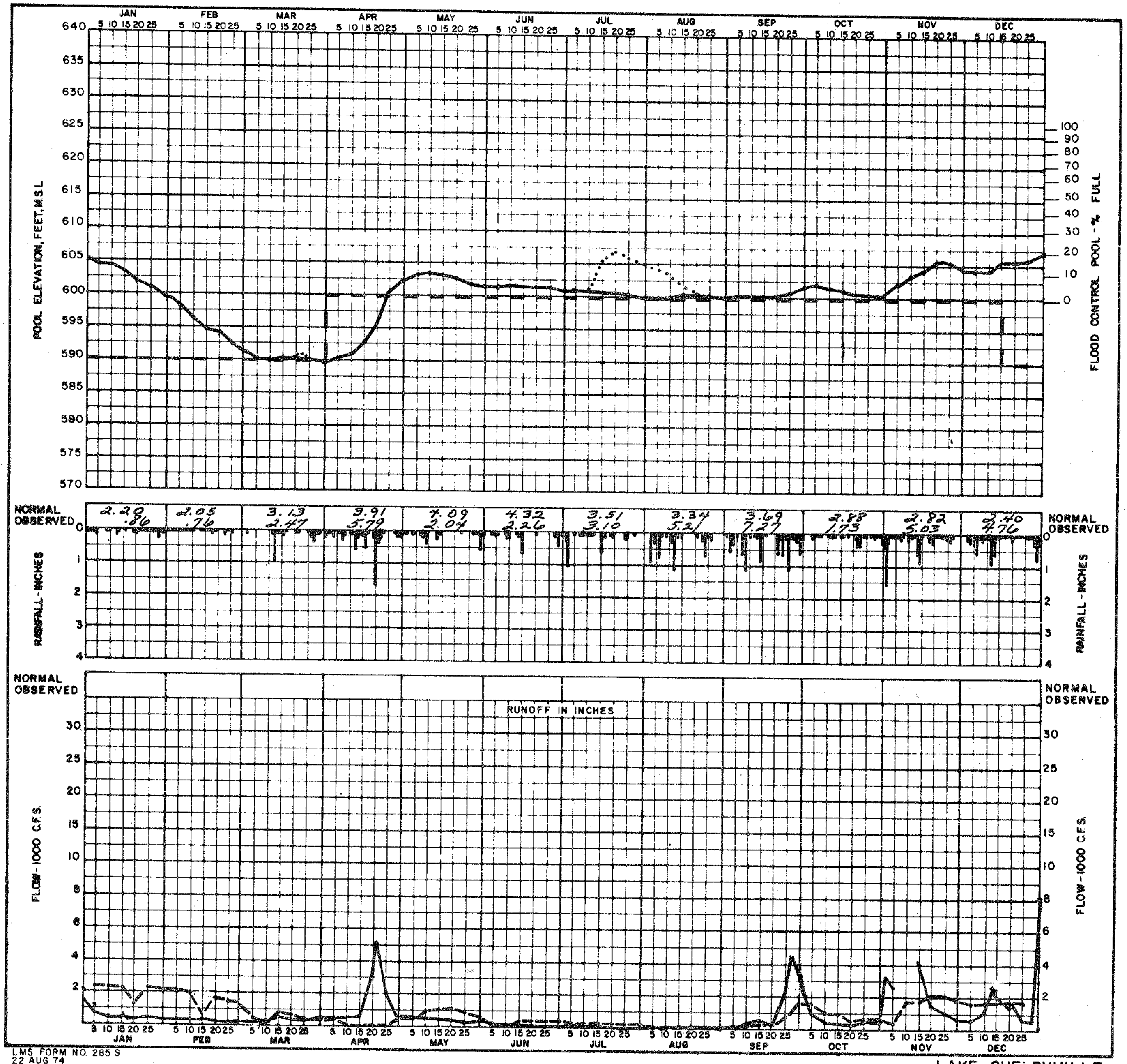


LAKE SHELBYVILLE
1971

Nominclature

......Max. Pool of Record
____ Pool Elevation
___ Rule Curve

____ Inflow
___ Outflow

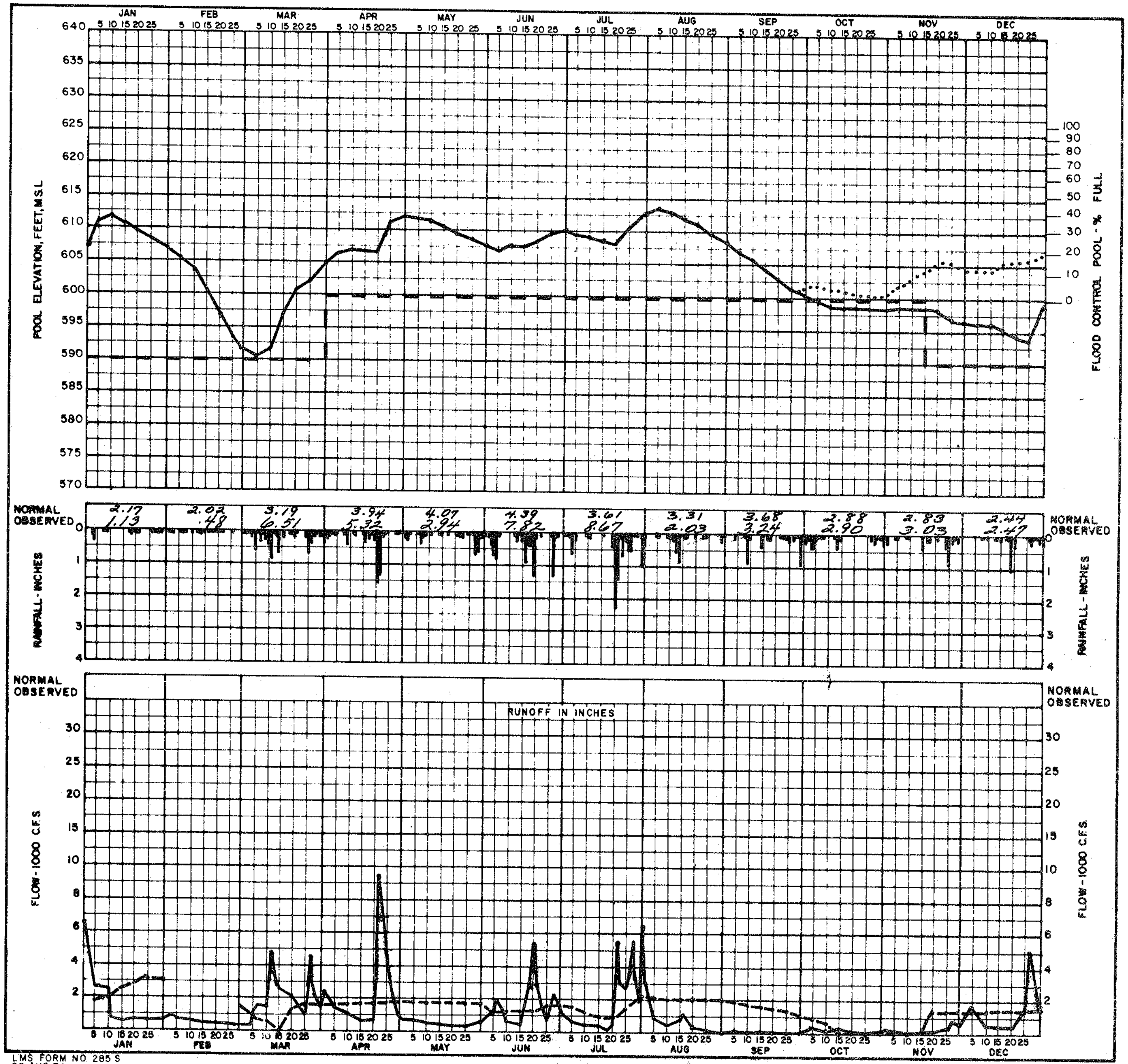


LAKE SHELBYVILLE
1972

Nominclature

...... Max. Pool of Record
___ Pool Elevation
_ _ _ Rule Curve

___ Inflow
_ _ _ Outflow

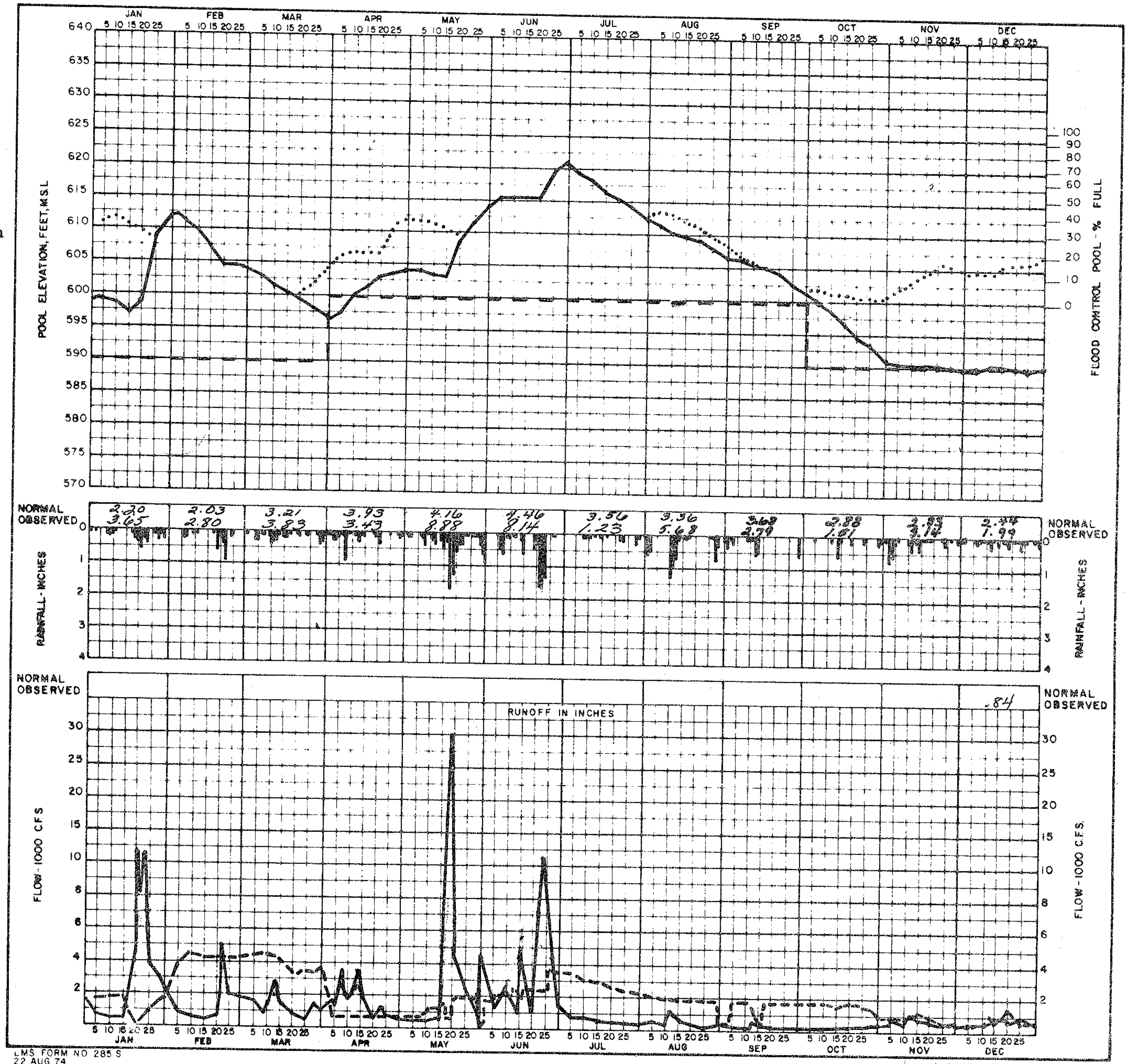


LAKE SHELBYVILLE
1973

Nominclature

..... Max. Pool of Record
——— Pool Elevation
- - - Rule Curve

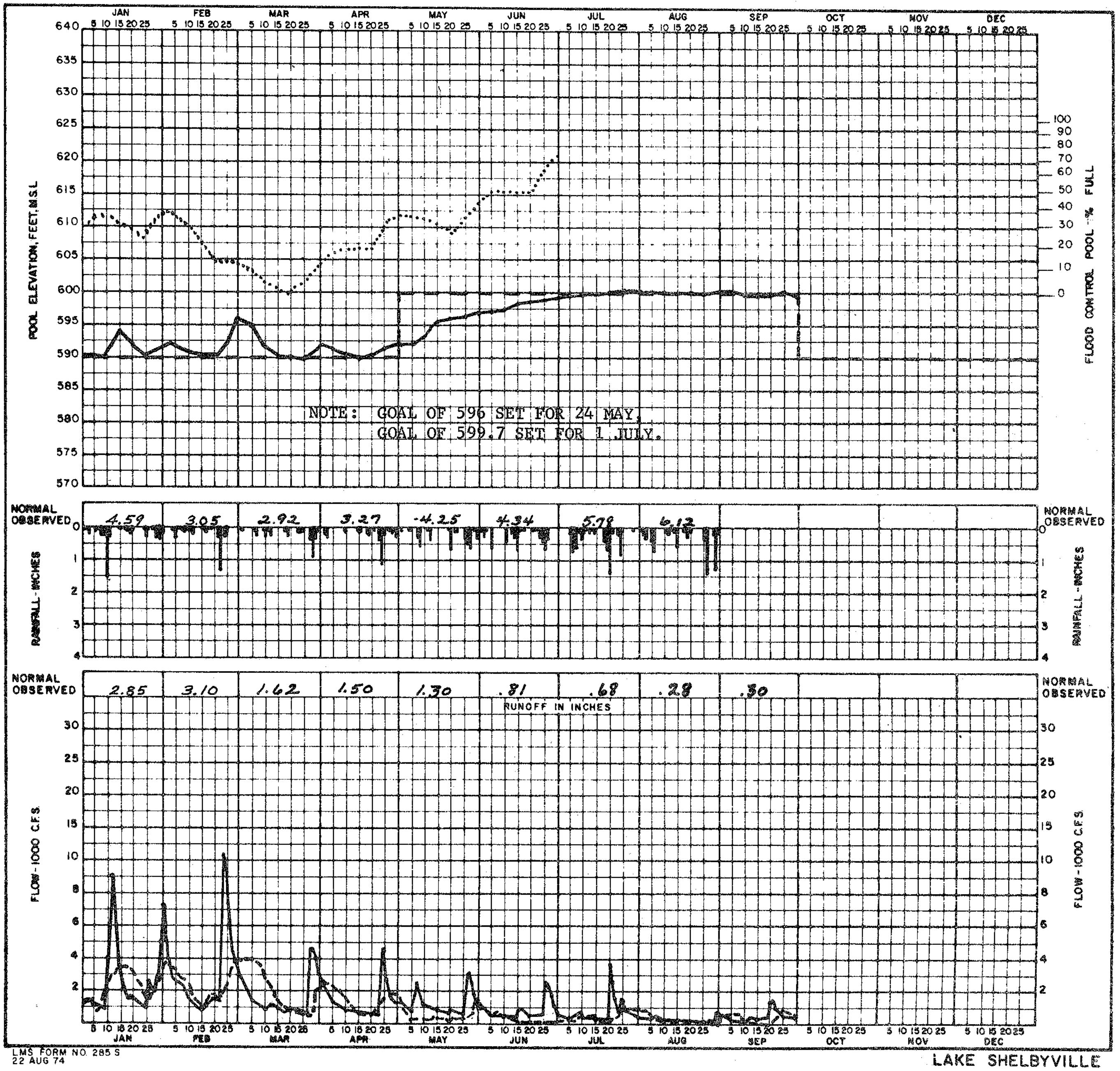
——— Inflow
- - - Outflow



LAKE SHELBYVILLE
1974

Nominclature

...., Max. Pool of Record
—— Pool Elevation
---- Rule Curve



—— Inflow
---- Outflow

1975

REAL-TIME OPERATION
OF THE CENTRAL AND SOUTHERN FLORIDA PROJECT

BY

Ormond C. White [1]

INTRODUCTION

The Jacksonville District, Corps of Engineers has acquired experience in real-time operation of the Central and Southern Florida Project (C&SF Project) since 1967, when the existing telemetry system became operational. In 1973, the U. S. Geological Survey (USGS) established an experimental ERTS (renamed LANSAT) data collection network adjacent to existing Corps gages. This paper compares performance of the two systems, justifies the need for real-time data management, and discusses future plans for a replacement system and plans for a major supplemental system under construction by the Central and Southern Florida Flood Control District (FCD).

PROJECT DESCRIPTION AND PURPOSE

The C&SF Project (figure 1) is a large multipurpose Federal project involving all or parts of 18 Florida counties. It covers an area of almost 16,000 miles of central and southern Florida from Orlando to the southern part of Dade County. It embraces Lake Okeechobee, its regulatory outlets, a large portion of the Everglades, the Upper St. Johns and Kissimmee River Basins, and the lower east coast of Florida.

The backbone of the project is the string of immense storage areas, totalling 2,300 square miles and ending at Everglades National Park (ENP). The project (figure 2) boasts 34 pumping stations with total pumping capacity of 52,000 cubic feet per second. Protective works include about 1,000 miles of levees, 1,000 miles of canals, 200 water control structures and 26 navigation locks.

Specific project purposes include flood protection and water supply to ENP, and to agricultural and urban areas. There are other benefits which accrue to navigation, to control of coastal salinity intrusion, to water-oriented recreation, to fish and wildlife, and from water quality control.

[1] Chief, Water Management Section, Jacksonville District

Basically the project provides flood protection by means of storage in Lake Okeechobee and the 3 large water conservation areas and accomodates increased runoff by improvements of existing and provision of new channels. Extensive pumping, large canals, and levees have to be provided because of the flat terrain and porous aquifer.

PROJECT OPERATIONS

FCD and the Corps share operational responsibilities. The project authorization provided that local interests would maintain and operate all the works after completion, except the major outlets from Lake Okeechobee and the conservation areas. The Federal Government controls the heart of the project and the FCD handles local works. Even so, FCD claims state ownership and riparian rights to determine where conserved water is used within the District.

Structure 12, outlet to ENP, is an example of the tremendous amount of coordination required throughout the project. Gate settings and operations of the structure are the responsibility of the Corps. Because of the great travel distance involved by Corps personnel, actual gate operations are now performed by FCD on a cost reimbursable basis. But first, release amounts are coordinated by telephone with ENP, FCD, AND USGS. Wildlife agencies are also notified by mail. Real-time data on the condition of the reservoirs is a must at all times in order to responsibly effect these acts of coordination from the Jacksonville District Office.

EXISTING REMOTE DATA ACQUISITION

FCD has plans for an elaborate remote data acquisition and control system, but for now (1975) they operate a limited microwave system from West Palm Beach to Structure 61, near Kissimmee and Orlando, Florida (figure 3). This system was licensed in July 1969 and is presently limited to 4 full-duplex voice bandwidth (3KHz) multiplex channels, one of which is dedicated to a small data acquisition network consisting of a single central microwave-VHF station with 7 remote reporting stations."[2] Performance data for this system is not available for this paper.

The Clewiston Telemetry System was constructed 8 years ago for the Jacksonville District. The system consists of six water level gaging stations, a relay station, and a collection center, as shown in figure 4. The system was installed in southern Florida to measure water stage in the Everglades. The six gaging stations are solar-powered and make use of 5-watt solid-state radio equipment. The relay

[2] Central and Southern Florida Flood Control District, "Supplementary Information, VHF License Applications for C&SFFCD Data and Control System," June 1975.

station is located at Andytown near Fort Lauderdale and operates from commercial power with a propane-powered standby generator, including automatic-transfer equipment. The collection center equipment is located at the Area Engineer Office in Clewiston, Florida.

Each gaging station utilizes a modified Shand & Jurs (S&J) keyer to convert water level to pulse information suitable for transmission by radio. Each station is equipped with 11 Ni-Cad cells for a battery voltage of 13 volts at 30-amp-hr capacity. The solar-charging panels were made from Hoffman 6-volt, 1-watt cells arranged in a series-parallel array to produce 18 volts at 2 watts. The cells are connected to the battery by way of blocking diodes to prevent loss of power during the night or during periods of little or no sunshine.

The radio equipment used in these stations is Motorola Look-out series operating at approximately 170 MHz. The telemetered information is transmitted by means of a Quindar QT-30 frequency shift-tone transmitter. This transmitter utilizes a center frequency of 2295 Hz of a total space-to-mark shift frequency of 60 Hz. Station interrogation is handled by means of four-tone decoders, designed at WES, utilizing relatively high-impedance frequency selective reed relays manufactured by JBT Instruments, Incorporated.

The relay station utilizes solid-state RF equipment capable of 120 watts output. The antenna used at the relay station is a 6-db omni-directional antenna mounted at the 200-ft. level of a 250-ft. guyed tower. A diplexer is used at the relay for simultaneous transmit-receive operation, thereby reducing antenna and feed-line requirements. Antennas at the gaging stations are ground-plane verticals in the case of nearby stations and 7.5 - to 10-db corner reflectors at the more distant stations.

The WES-made logic and control equipment located at Clewiston performs the functions of selectively calling outlying stations either manually by use of a telephone dial or automatically by means of a program-time clock. The gaging station information is stored in the Clewiston logic in Durant Unipulsers which provide a visual readout as well as electrical storage. Unipulsers are also used for the real-time clock which produces days, hours, and tenths of hours.

In operation, a gaging station will produce its digital information in the Unipulsers provided, after which time the system will scan through all Unipulsers including the clock, and print this information on a Model-32 Teletype machine. Standard 5-channel teletype tape may be made simultaneously with the page print for information storage and/or retransmission over standard teletype circuits to any other point.

Data obtained from the Clewiston Telemetry System is visually transferred to a daily hydrologic data sheet and transmitted by telecopier to the Jacksonville District Office. In emergency situations data is communicated verbally by telephone or by radio over the microwave backbone shown in Figure 5. The backbone system suffers frequent outages due to lightning strikes and local power failures at the repeater stations.

The life expectancy of the Clewiston Telemetry System was only 5 years. Reliability of the system has dropped below 50 percent, on the average. Companies which produced the S&J and Quindar units are no longer in existence, and replacement parts are no longer available. With current funding plans, the old system will have to perform for a total of 9 or 10 years, double its life expectancy. The Appendix to this paper contains a daily record of how poorly the system is functioning.

In September 1973 the U. S. Geological Survey in cooperation with NASA, and the Corps of Engineers, established an experimental prototype satellite hydrologic data reporting system. "Twenty data collection platforms (DCPs) were established in the Everglades (Figure 6) and Big Cypress Swamp to the west to transmit water level and rainfall."[3] Twelve of these stations were established along side of existing Corps stations, six of which were adjacent to the Clewiston Telemetry System.

"Before the DCP transmits the data to the satellite, it receives the data from a sensor (recorder) and then encodes and puts the data into a form for radio transmission. Although the DCP normally transmits a signal every 3 minutes, the DCP can be set to transmit every 90 seconds. When the LANSAT (formerly ERTS) satellite (Figure 7) is in mutual line of sight of a DCP and a NASA receiving station, the data transmitted from the DCP is received by the satellite and transmitted to NASA receiving stations at Goddard or at Goldstone. In southern Florida, this mutual communications occurs from 3 to 6 times per day. Each time, 1 to 4 district messages can be transmitted every 3 minutes.

"At the receiving station the data are coded and sent by NASA communication line to the Operations Control Center and then to the NASA Data Processing Facility at Goddard. At the processing facility, the data are verified and put in a form to be teletyped to Miami. The data are received by teletype as perforated tape and printout. The total elapsed time from field measurement to printout in Miami is about 45 minutes."[4]

Data which were sent to the Corps were those readings occurring most nearly to midnight. These data were reduced to the format shown in Table 1 and transmitted by telecopier to both the Jacksonville District Office and to the Clewiston Area Office. Data were received in Jacksonville usually by 1000 hours.

[3]Higer, A. L. and others, p. 15.

[4]*Ibid*.

Table I

TYPICAL WATER LEVEL AND RAINFALL REPORT
TELECOPIED TO DATA USERS

USGS MIAMI FLA.
DATA FROM LANDSAT SATELLITE
WATER SURFACE ELEVATION FT MEAN SEA LEVEL

DATE DAY TIME	32 FRI 2343	33 SAT 2348	34 SUN 2354	35 MON 2359	36 TUE 2221	37 WED 2227	38 THUR 2355
128	16.62	16.61	16.61	16.59	16.56	16.53	16.52
141	16.48	16.47	16.49	16.46	16.44	16.43	16.42
142	16.44	16.42	16.42	16.41	16.39	16.37	16.35
111	12.36	12.34	12.33	12.30	12.27	12.26	12.23
112	12.10	12.07	12.05	12.02	12.00	11.98	11.97
62	9.42	9.38	9.38	9.36	9.31	9.26	9.21
63	8.67	8.64	8.64	8.61	8.58	8.56	8.54
64	8.47	8.46	8.46	8.44	8.43	8.42	8.41
65	8.41	8.40	8.39	8.38	8.37	8.36	8.35
5	5.67	5.66	5.65	6.64	5.62	5.62	5.61
15	2.77	2.76	2.75	2.74	2.73	2.72	2.71
105	6.71	6.67	6.65	6.61	6.57	6.54	6.50

RAINFALL IN INCHES

128	0.00	0.00	0.00	0.00	0.00	0.00	0.00
141	0.00	0.00	0.25	0.00	0.00	0.00	0.00
142	0.00	0.00	0.00	0.06	0.00	0.00	0.00
111	0.00	0.00	0.19	0.00	0.00	0.00	0.00
112	0.00	0.00	0.06	0.00	0.00	0.00	0.00
62	0.00	0.00	0.06	0.00	0.00	0.00	0.00
63	0.00	0.00	0.00	0.00	0.00	0.00	0.00
64	0.00	0.00	0.00	0.00	0.00	0.00	0.00
65	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15	0.00	0.00	0.00	0.00	0.00	0.00	0.00
105	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Note: Readings will be revised as later data are received
(Table courtesy of U. S. Geological Survey)

COMPARISON OF TELEMETRY AND ERTS DATA

Summary data in the Appendix and Figure 8(a) show that the record of the first two years of reporting by the ERTS system compares only slightly below the reporting record of the first two years of operation of the Clewiston Telemetry System. Figure 8(b) compares the ERTS data with a concurrent more recent period of data from the Clewiston system. In the former case, the reporting differences may be attributed to differences in maintenance practices. For the latter case it just shows the Clewiston Telemetry System to be over-the-hill.

CURRENT PLANS - FUTURE SYSTEMS

The Corps is currently installing four C/DCSs in Lake Okeechobee (Figure 9). These will utilize the new convertible GOES/LANSAT platforms manufactured by Dorsett Electronics of Tulsa, Oklahoma. Temporarily, we plan to latch on to the tail of LMVD's plans to use the Bay St. Louis, Mississippi down link and the WES computer data bank.

Jacksonville District is currently planning a conventional microwave system for 25 additional reporting stations in the conservation areas and Fisheating Creek (Figure 10). This system has already been approved by SAD and OCE. Field personnel will soon begin rebuilding station platforms to accommodate new Leupold-Stevens Model 7000 recorders and the new radio equipment. Hopefully, funding can be provided for Fiscal Year 1977 purchase of the radio gear, microwave links, and central processing units in Clewiston and Jacksonville.

Currently, approximately 70 charts are processed on a monthly basis to provide data for monthly reports of reservoir operations. With the new system, it is anticipated that data received on a real-time basis will be frequently enough to reduce manual data processing by more than half.

C&SFFCD has initiated work on a major system (Figure 11) to collect hydrologic-meteorologic data and to remotely control project control structures and to provide voice communication. Including the central processor-control computers, the system is expected to cost between $3 and $4 million. Figure 12 shows the microwave backbone northern and southern loops. The southern loop is expected to be operational sometime within the next year. This loop contains a total of 40 Remote Data Acquisition and Control Units (RACUs) and 7 Master Concentration Units (MCUs).[5]

[5] C&SFFCD, _Op cit._

SUMMARY AND CONCLUSIONS

With the present state of the art, complete dependency on a real-time, fully automated data collection system is not practical in the Everglades. A separate backup paper record is necessary for any system that has been devised thus far. This is necessary because of the extremely difficult and time-consuming servicing conditions. A new system may work very well for 3 or 4 years, but in the later years the loss of record would be intolerable for a system in which the sensor record cannot stand alone without relying on the radio link.

REFERENCES

1. Central and Southern Florida Flood Control District. "Supplementary Information, VHF License Applications for C&SFFCD Data and Control System." June 1975.

2. A. L. Higer, A. E. Coker, E. H. Cordes, and R. H. Rogers. "Water-Management Model in Florida from ERTS-1 Data." U. S. Geological Survey, Department of the Interior, February 1975.

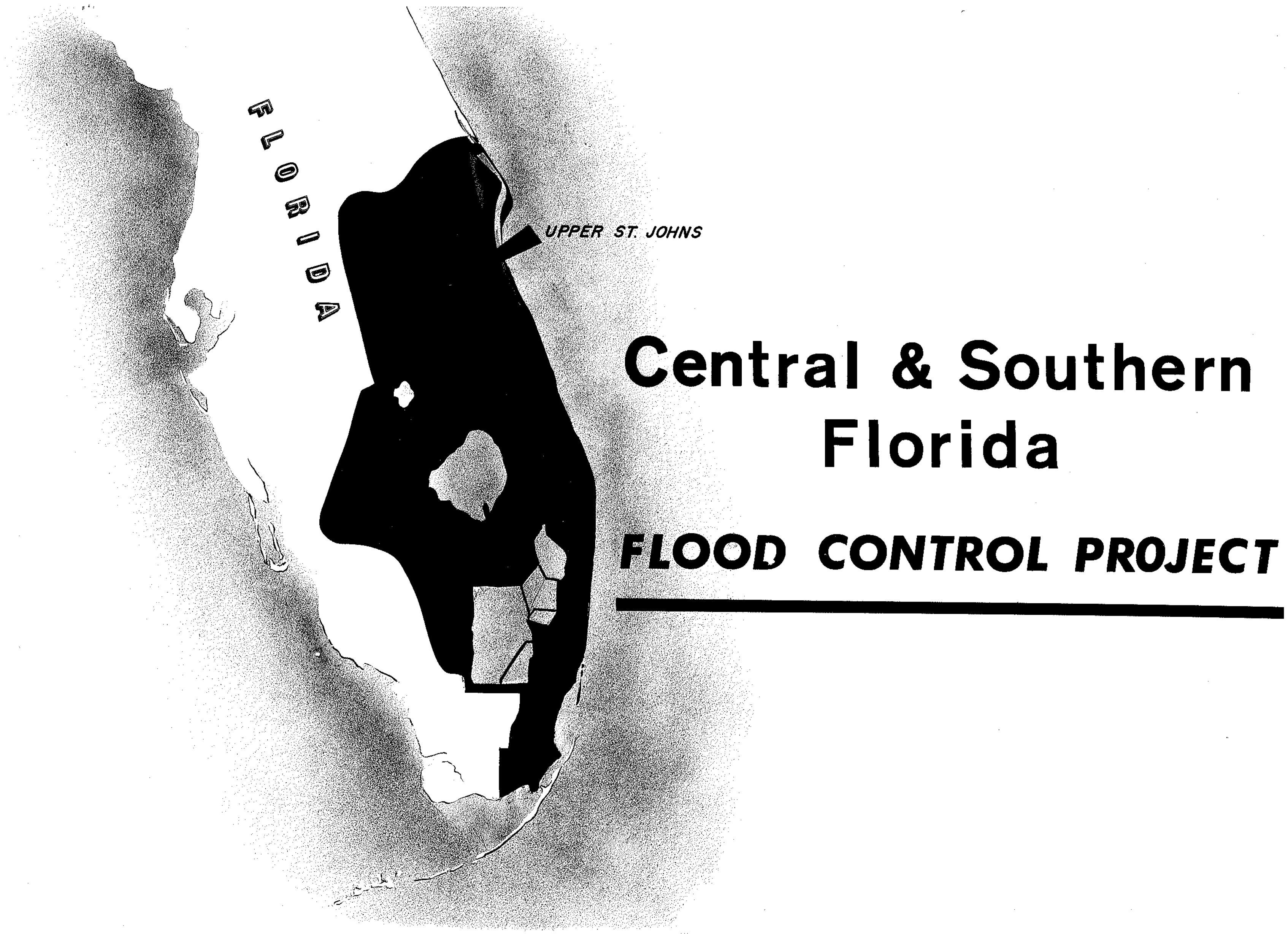


Figure 1 Key map

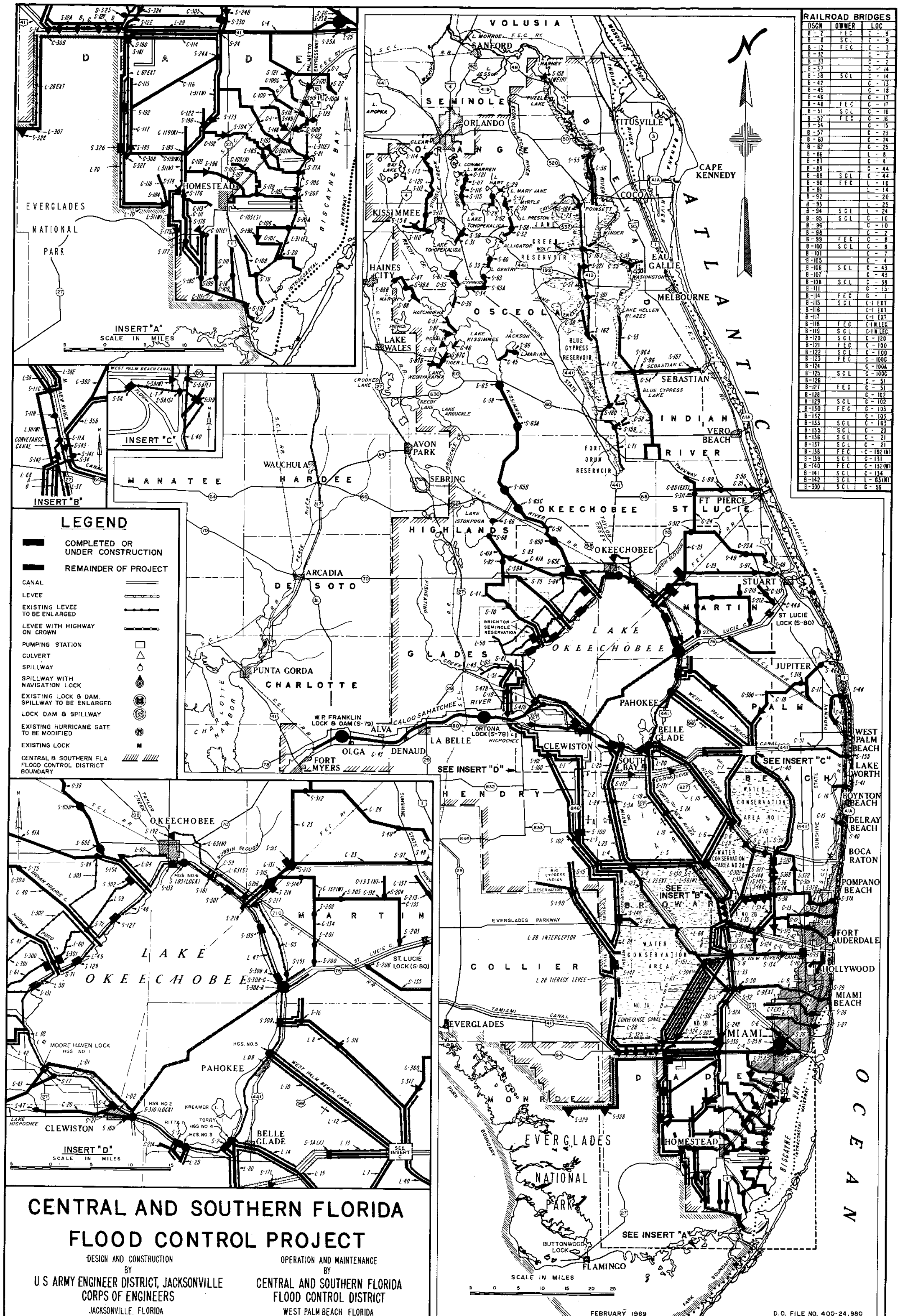


Figure 2 Project Map

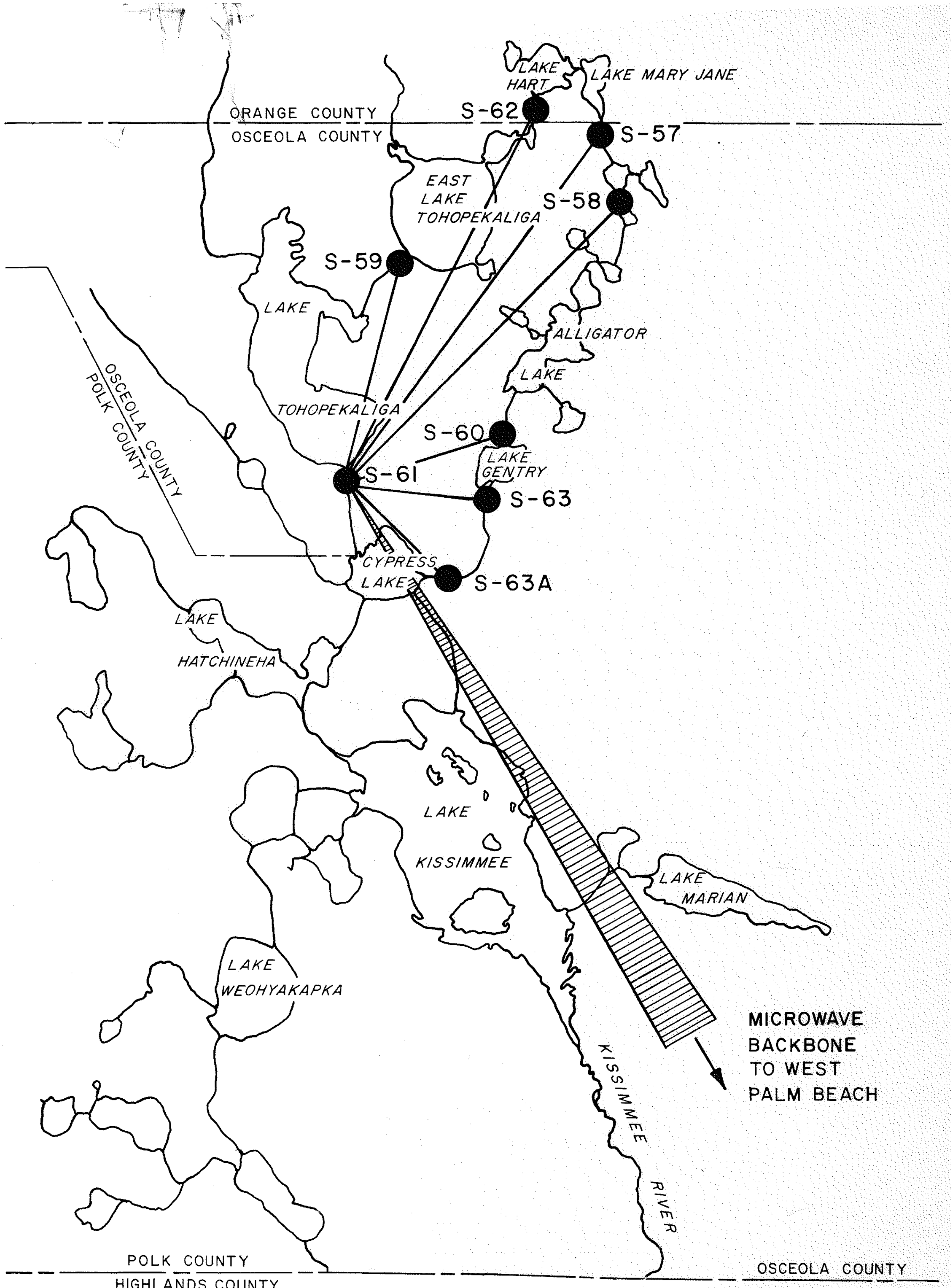


Figure 3 Existing C&SFFCD Remote Data and Control System

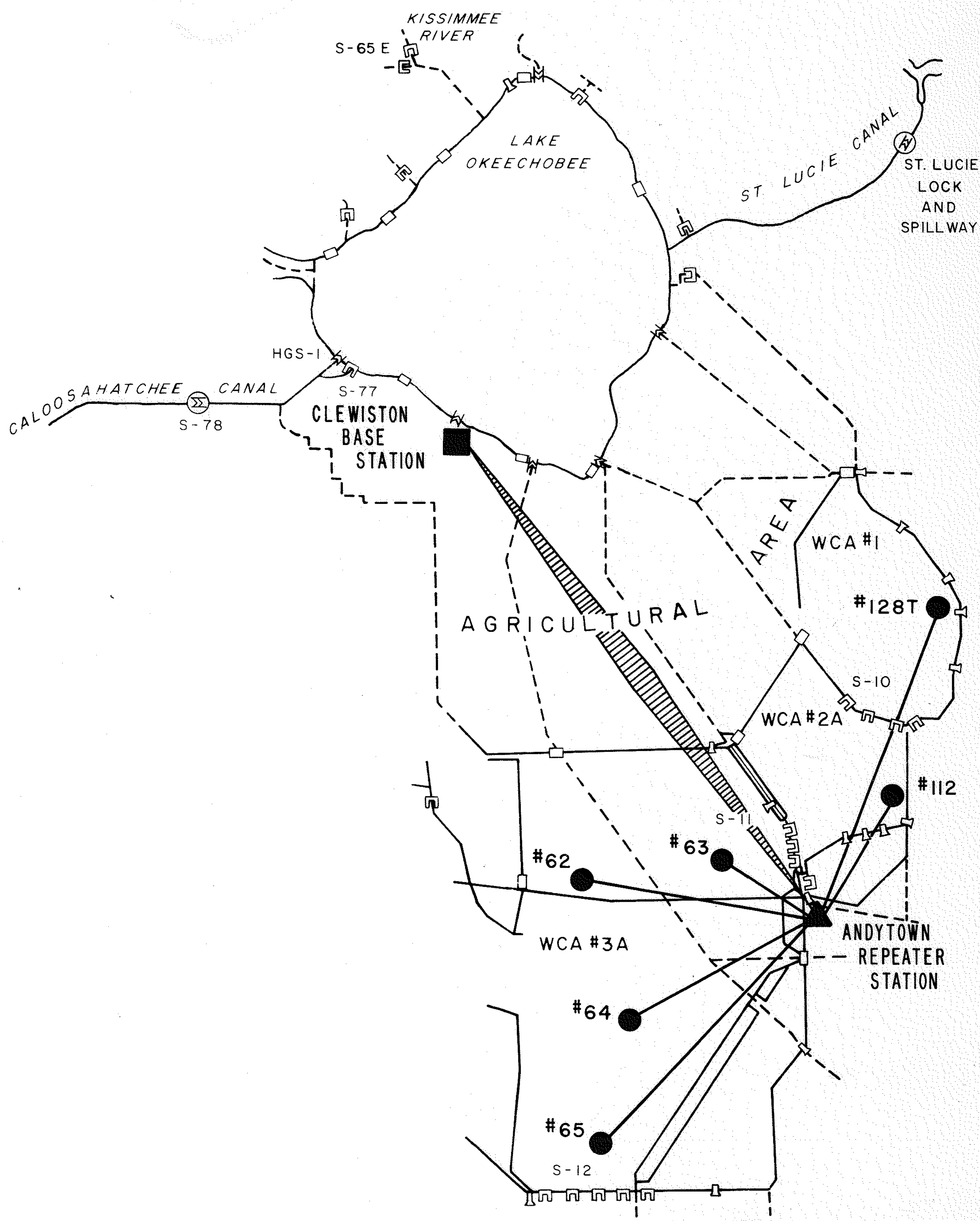


Figure 4 Clewiston Telemetry System

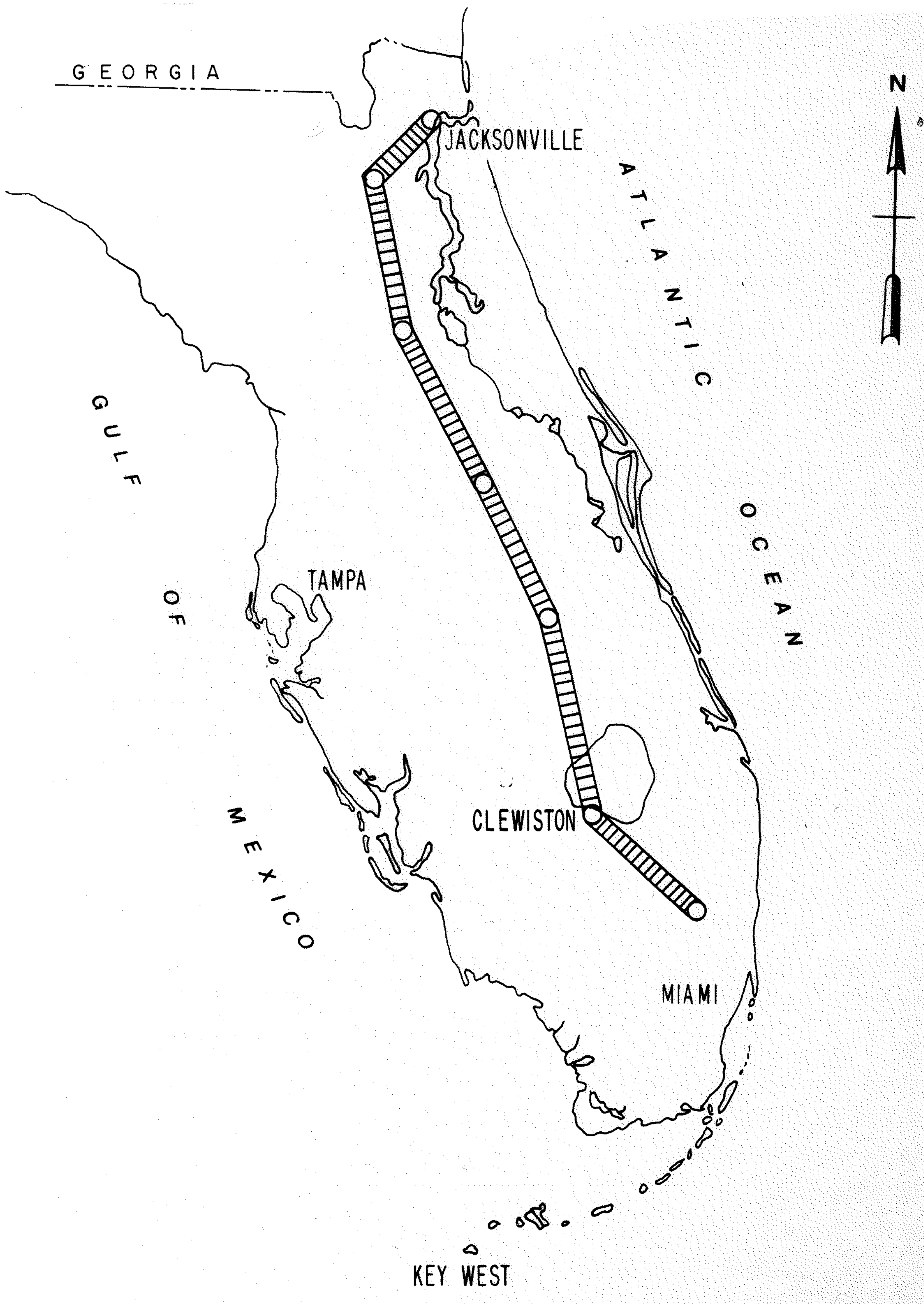


Figure 5 Microwave Backbone, Jacksonville District Communications Network

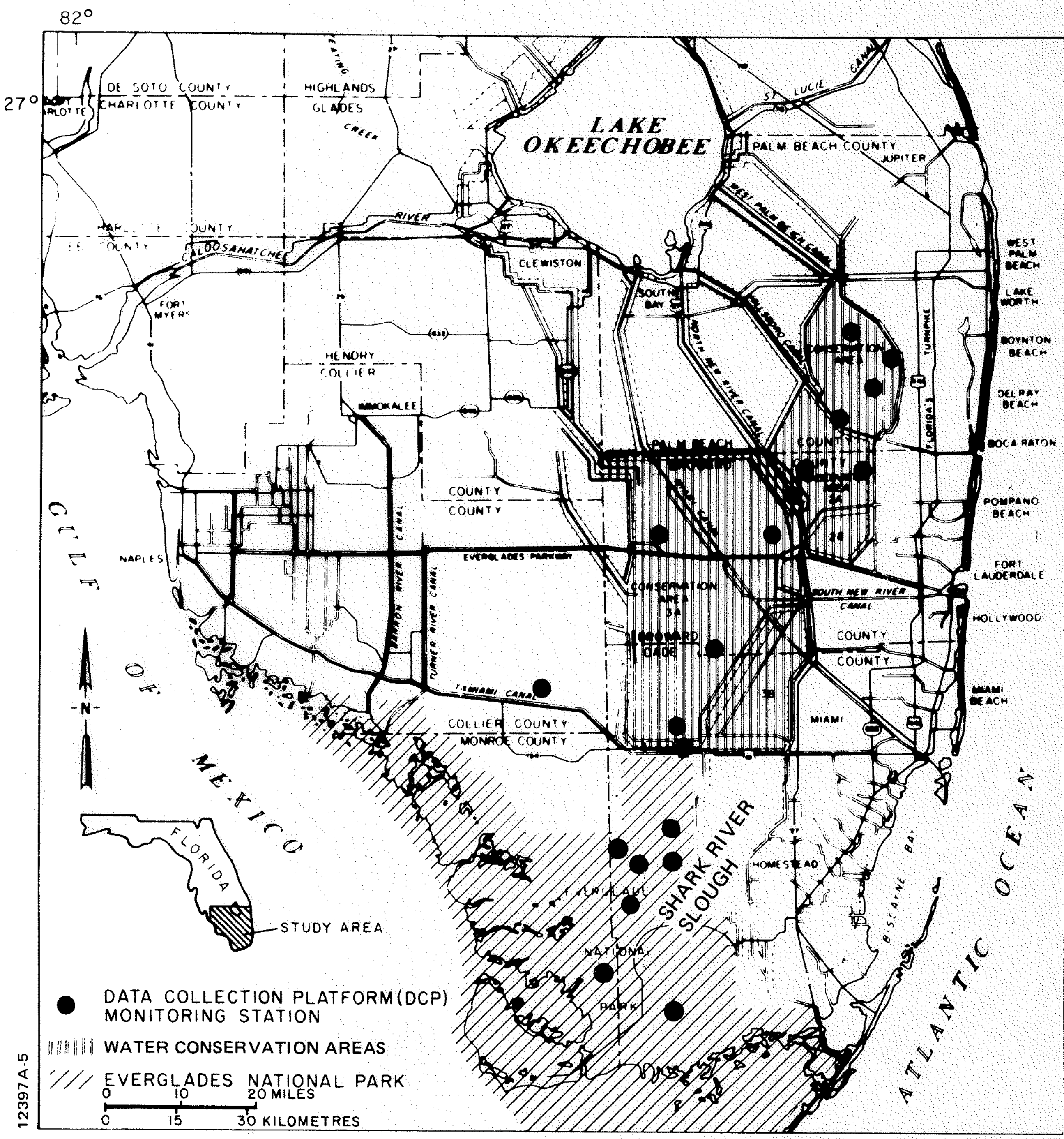


Figure 6 USGS ERTS Monitoring Stations (Figure courtesy of U. S. Geological Survey

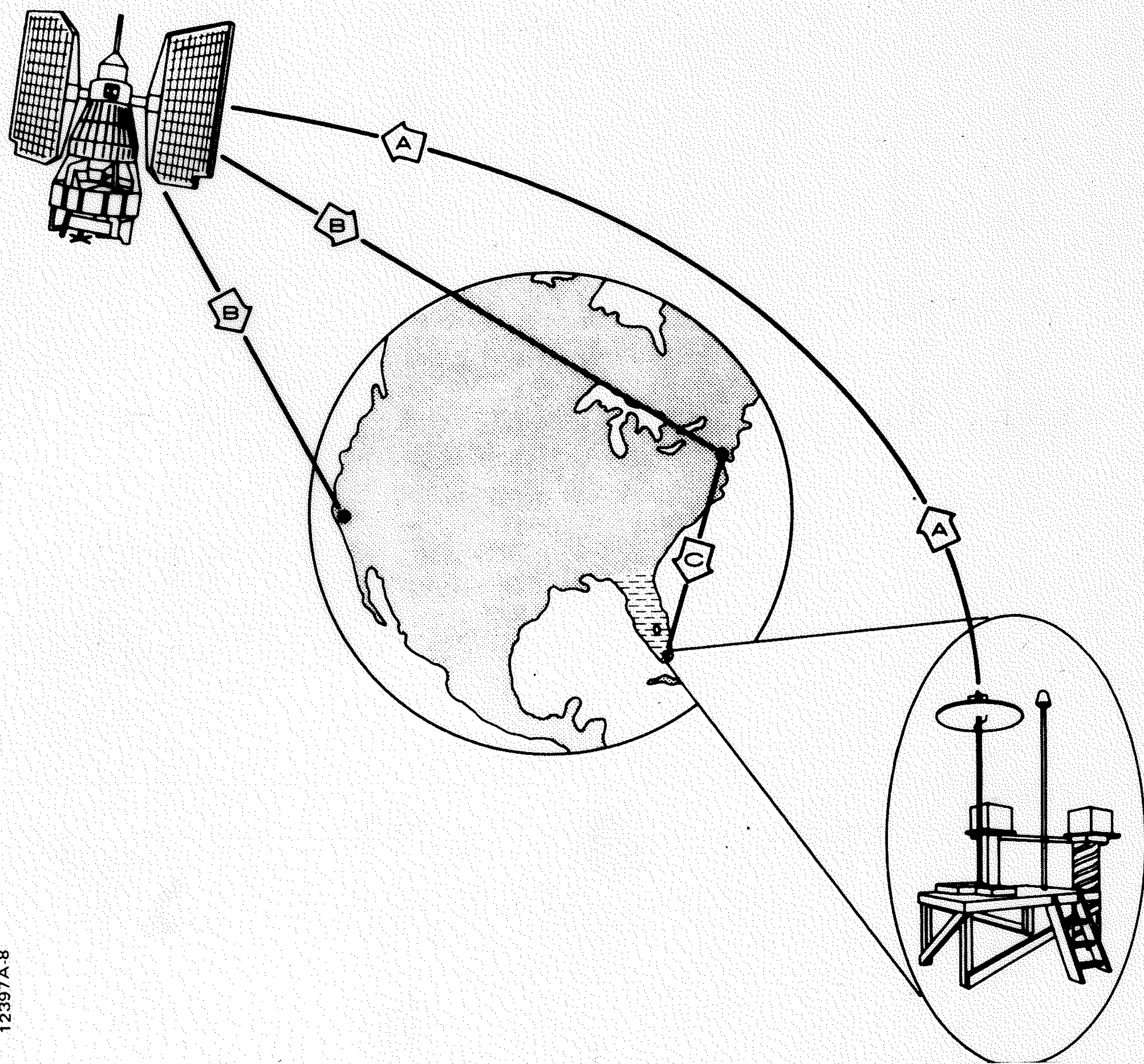


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Figure 7 Data are transmitted from the data collection platforms in the Everglades (A) via LANDSAT to NASA tracking stations at Goldstone, Calif., and GSFC, Greenbelt, Md. (B). The data are then transmitted, via NASA communications network, to the Geological Survey office in Miami. (Figure courtesy of U. S. Geological Survey).

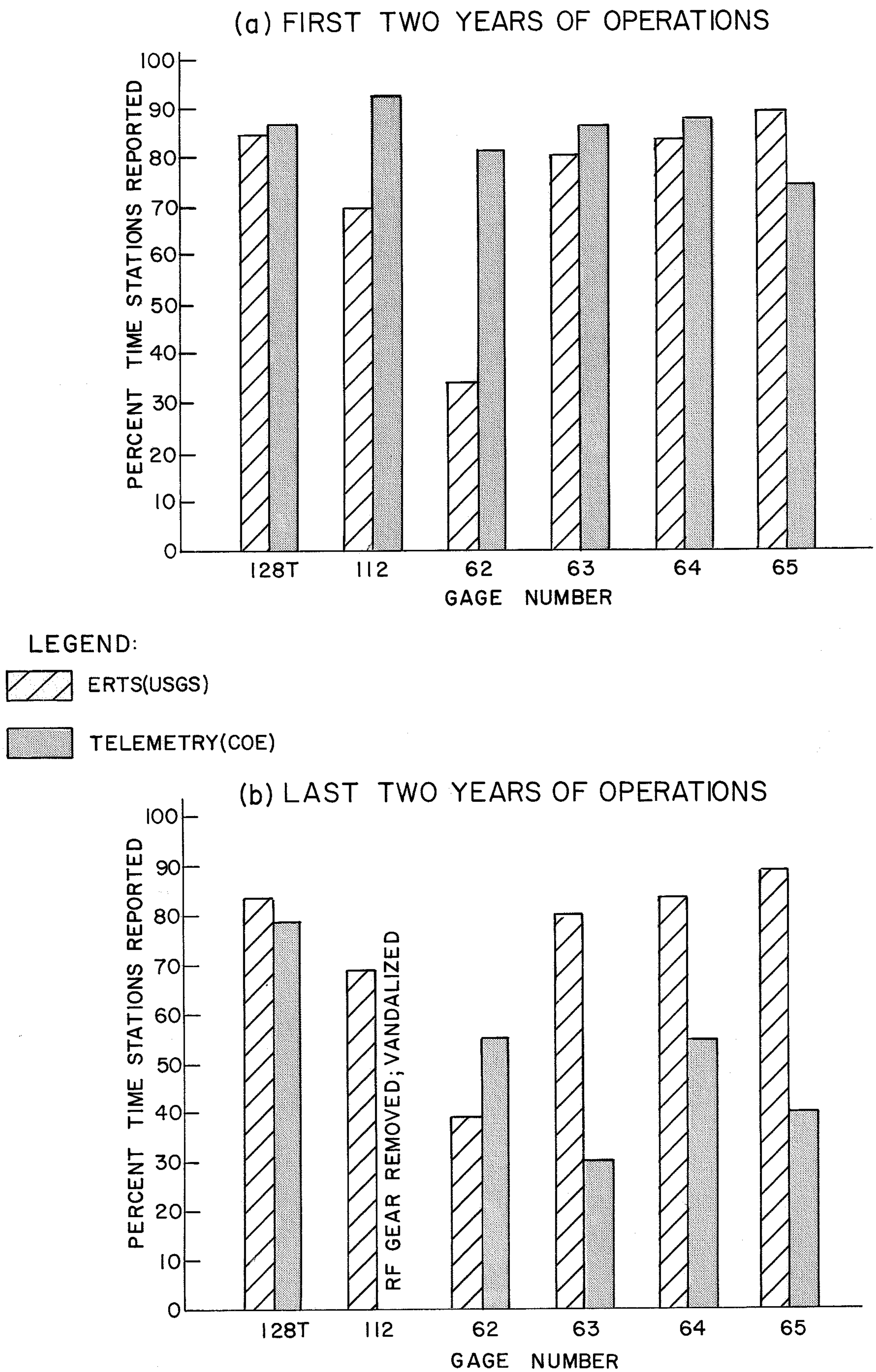


Figure 8   Performance Comparisons

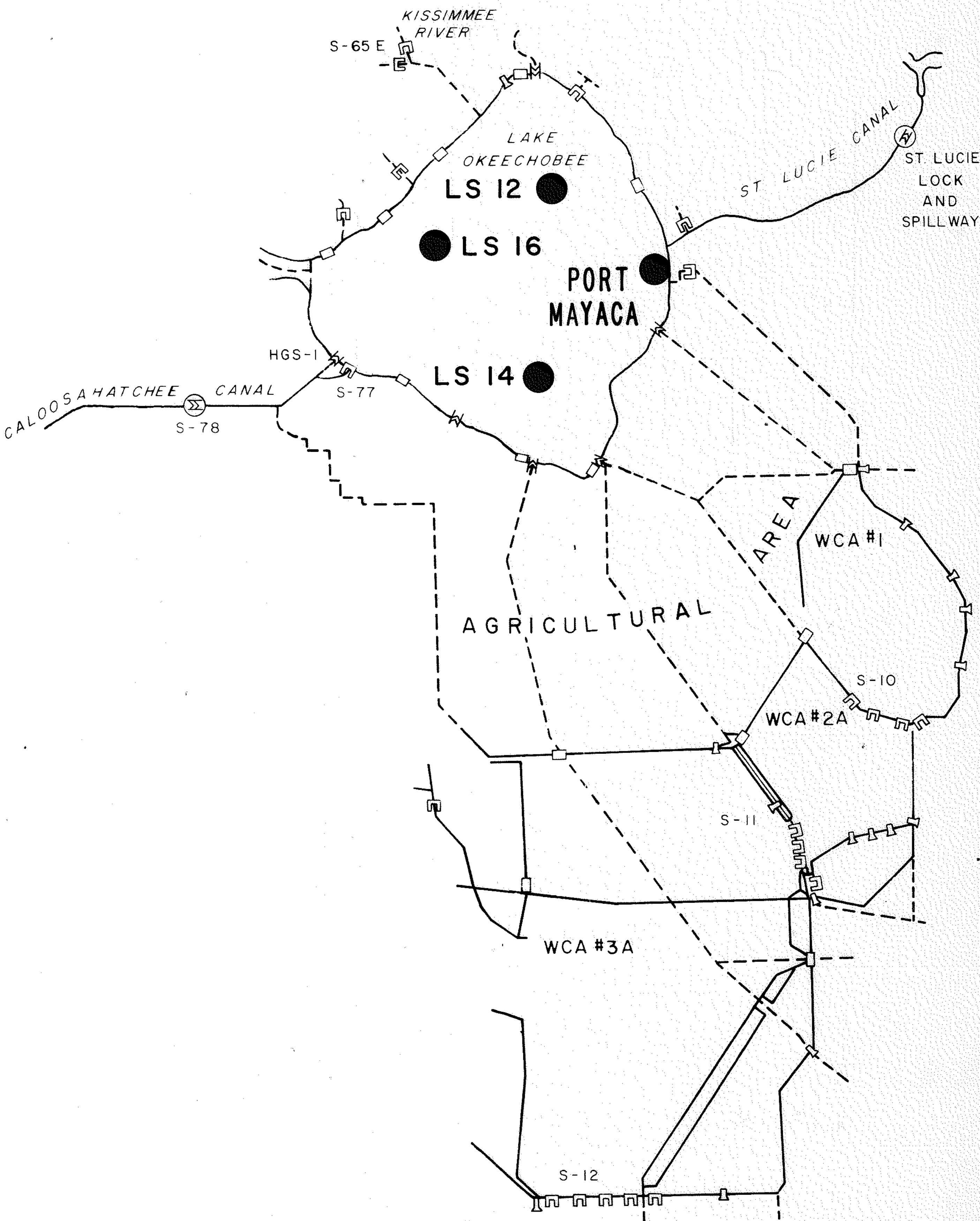


Figure 9 Lake Okeechobee LANSAT - GOES Reporting Stations.

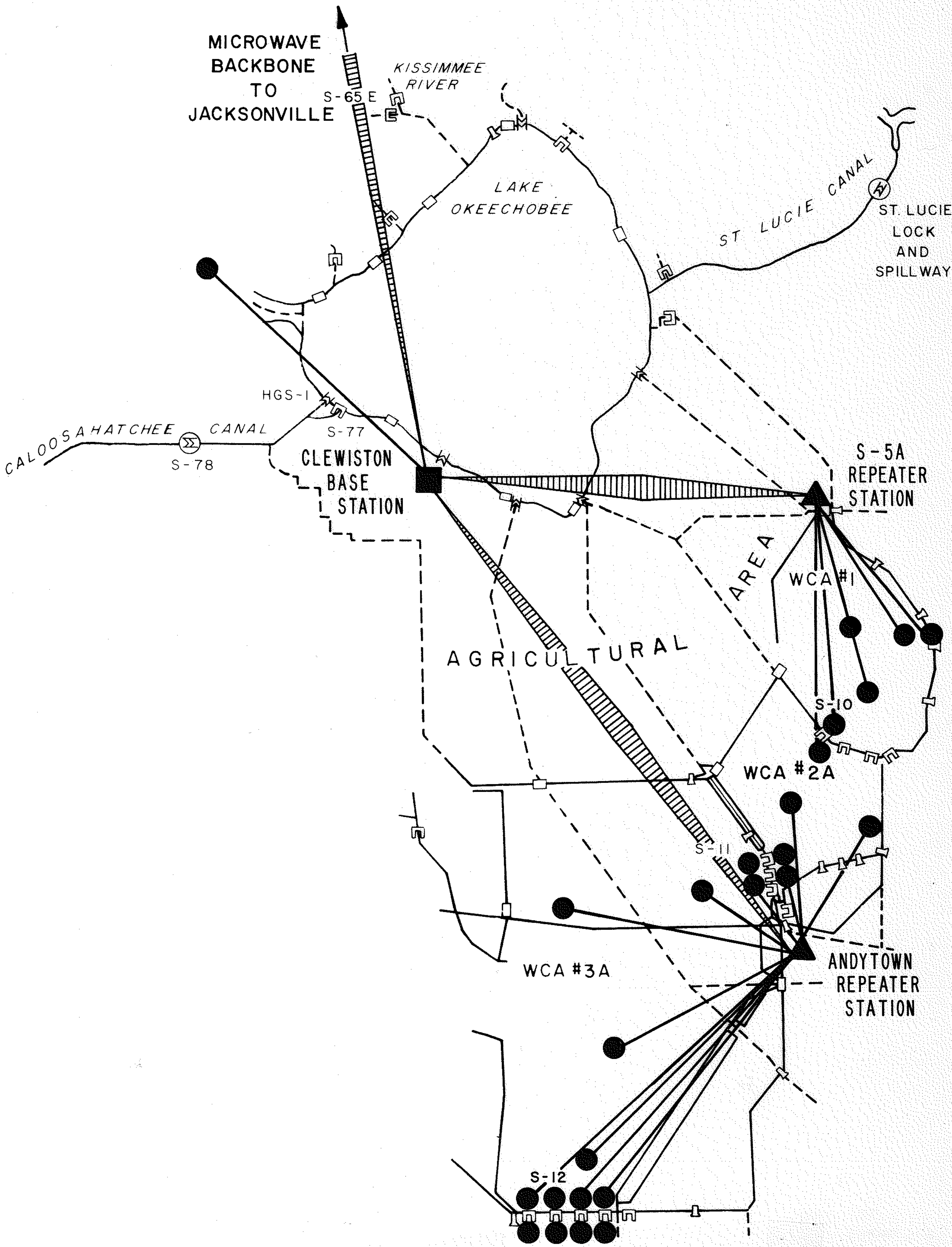


Figure 10 Proposed Clewiston Telemetry System

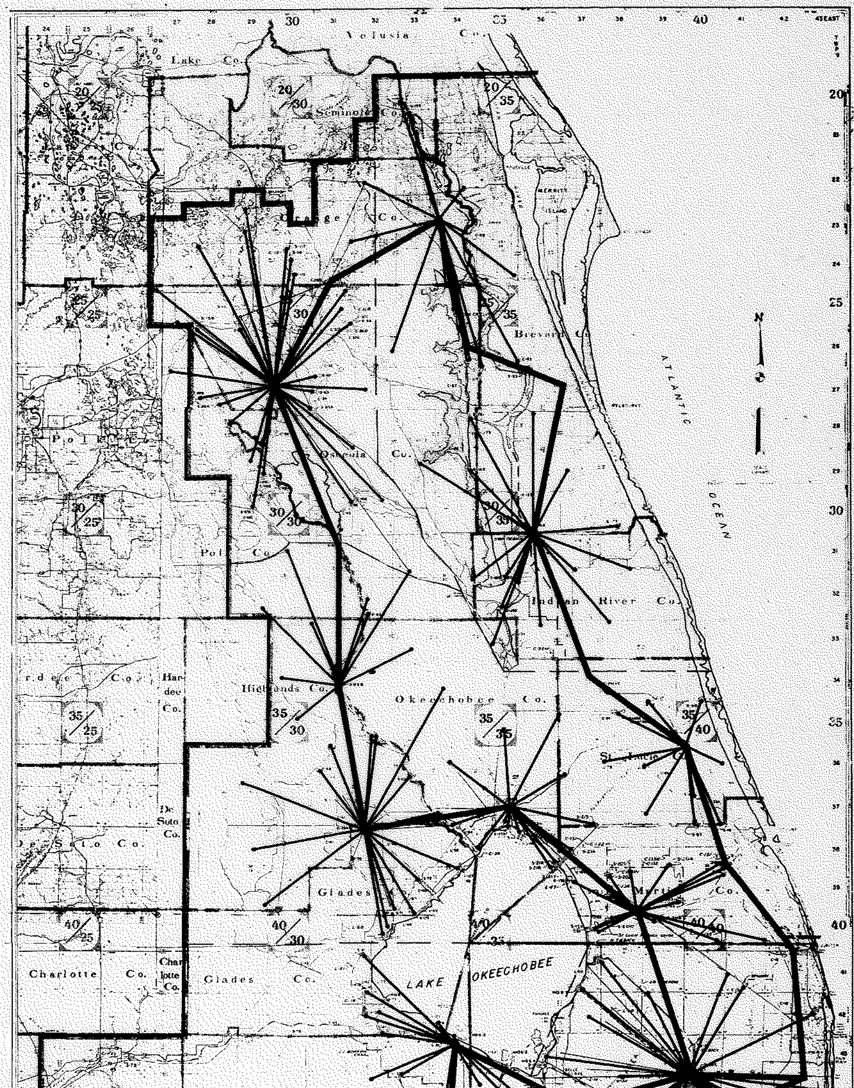


Figure 11 Proposed C&SFFCD VHF Feeder Network
(Figure courtesy of C&SF Flood Control District).

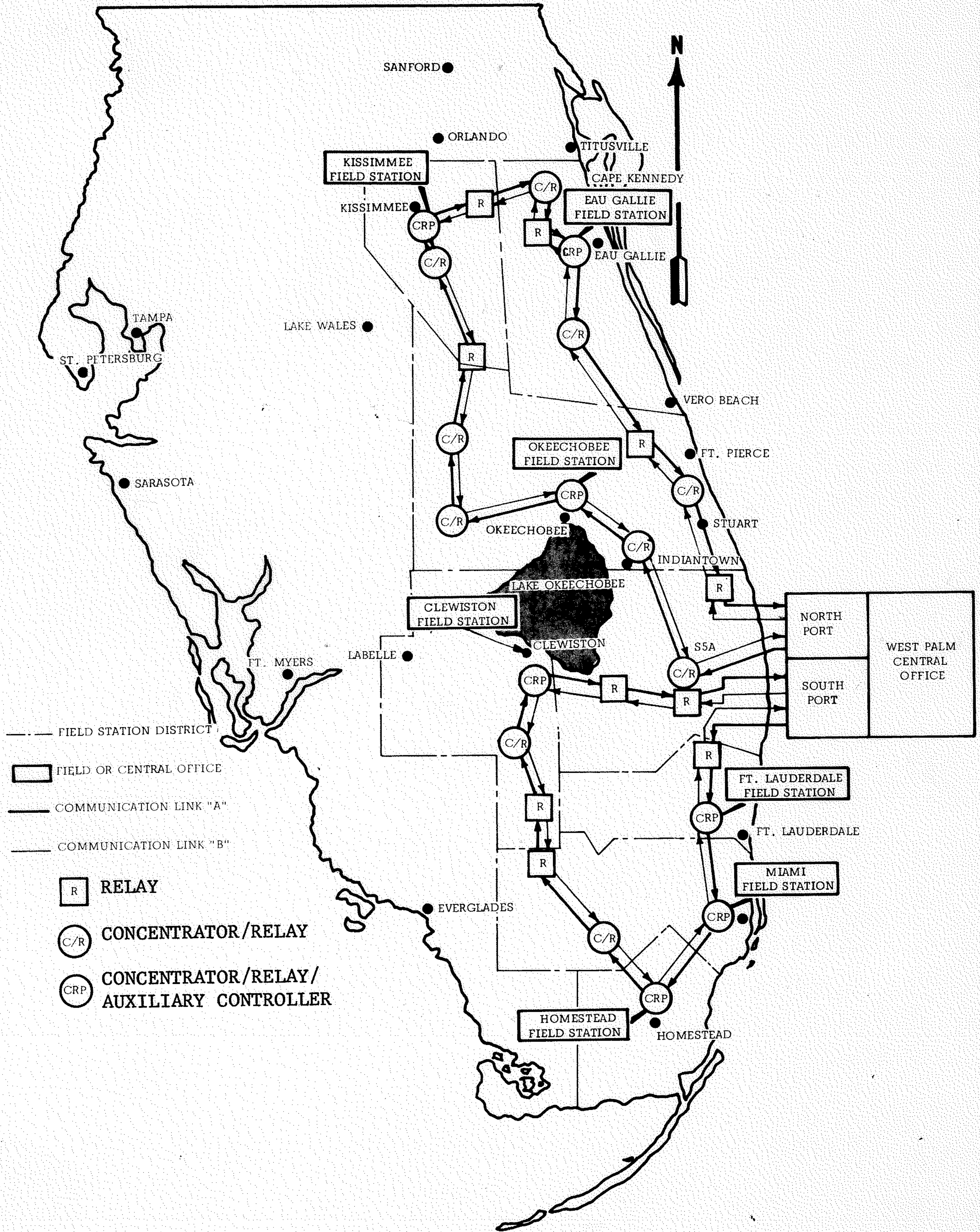
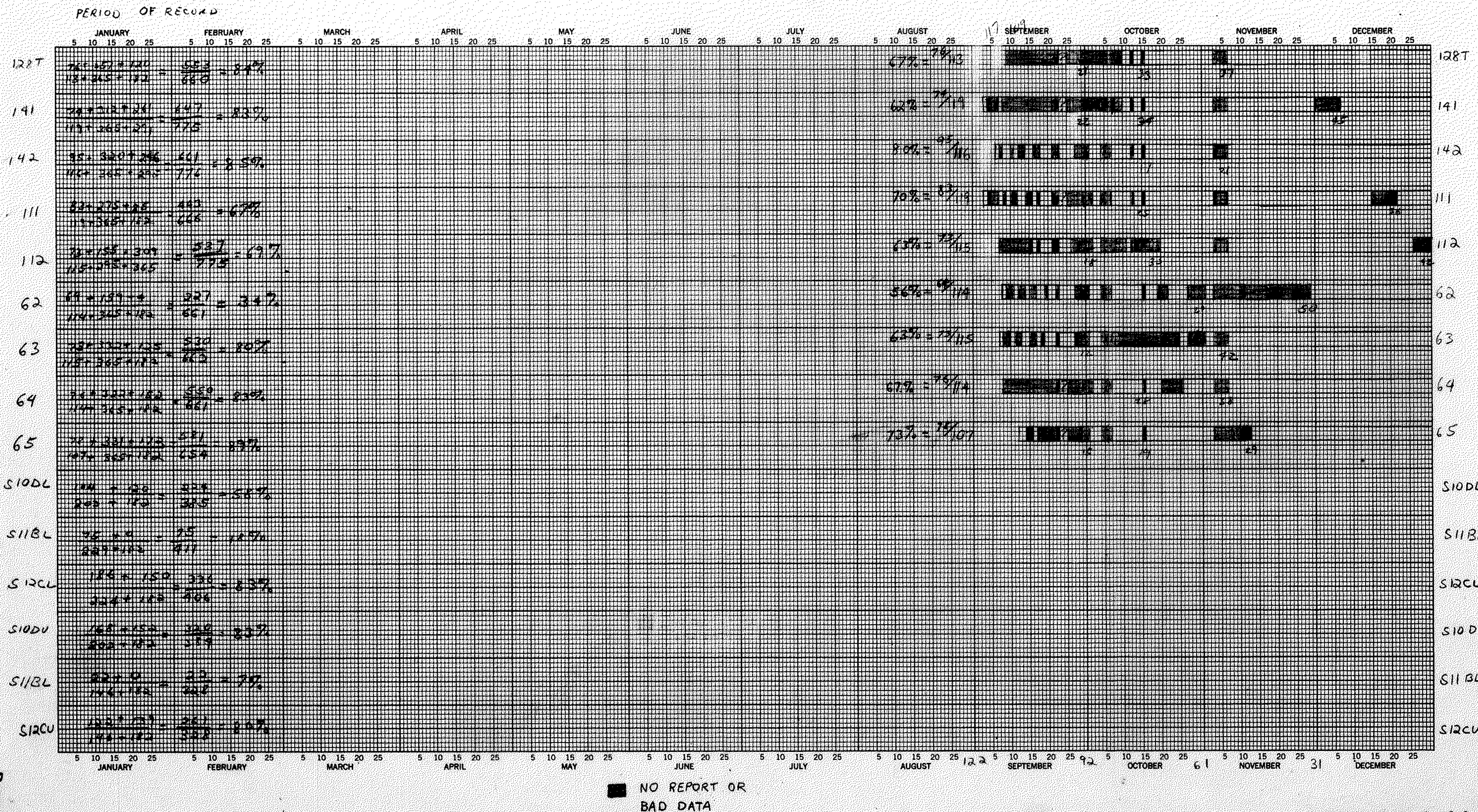


Figure 12 Proposed C&SFFCD Microwave Backbone (Figure courtesy of C&SF Flood Control District).

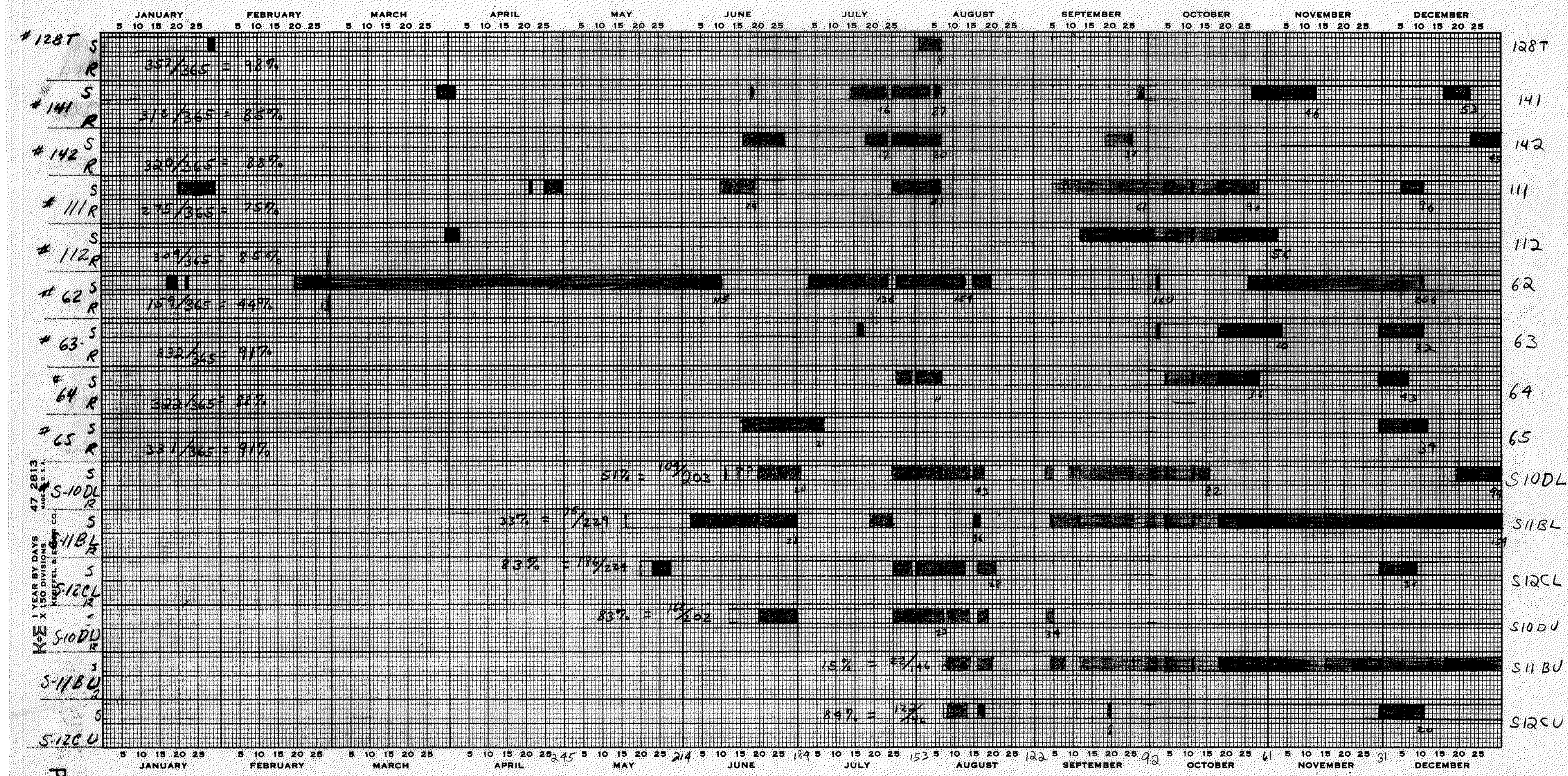
ERTS STAGE DATA

1973



APPENDIX Sheet 1 of 9

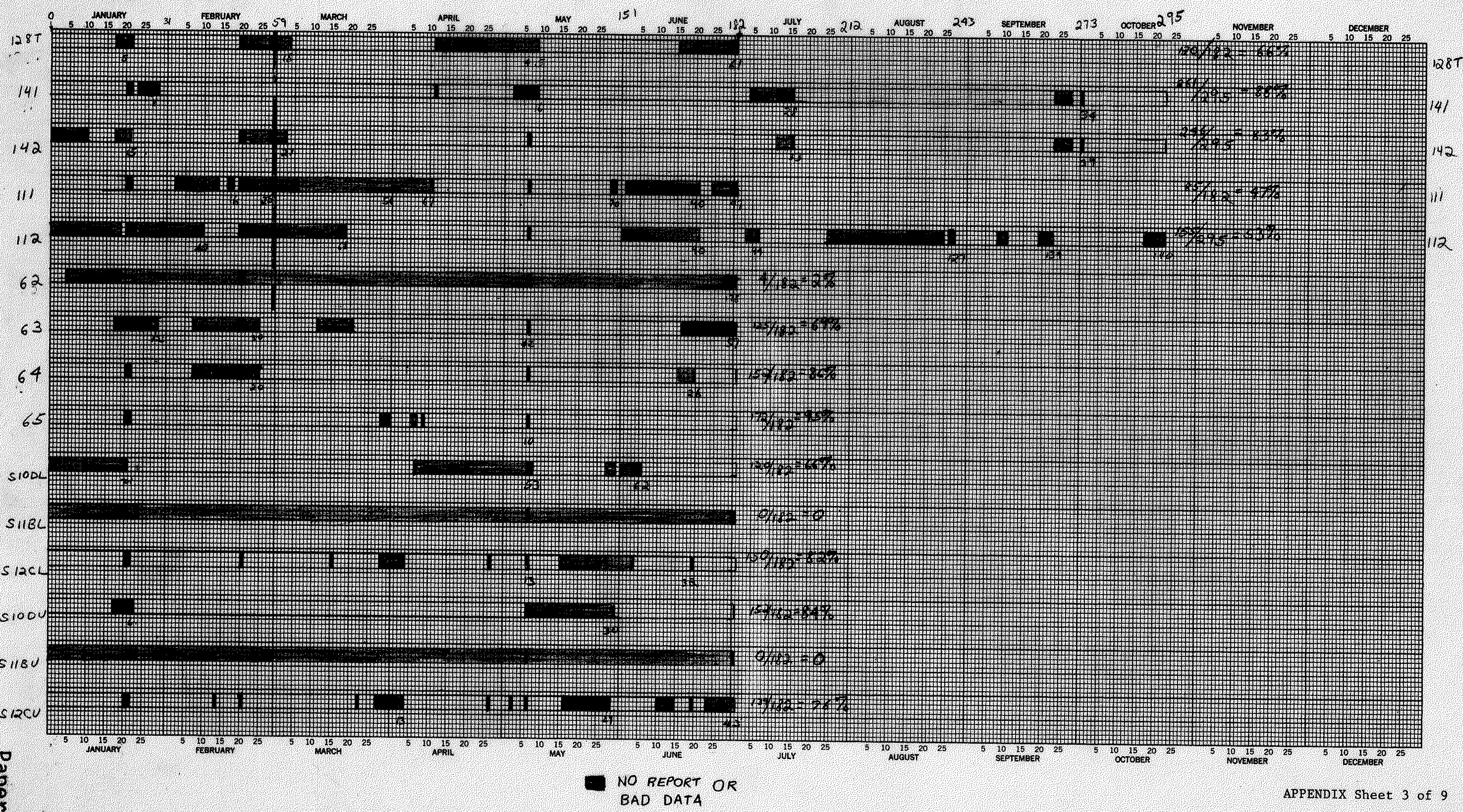
ERTS STAGE DATA
1974



NO REPORT OR
BAD DATA

APPENDIX Sheet 2 of 9

ERTS STAGE DATA
1975



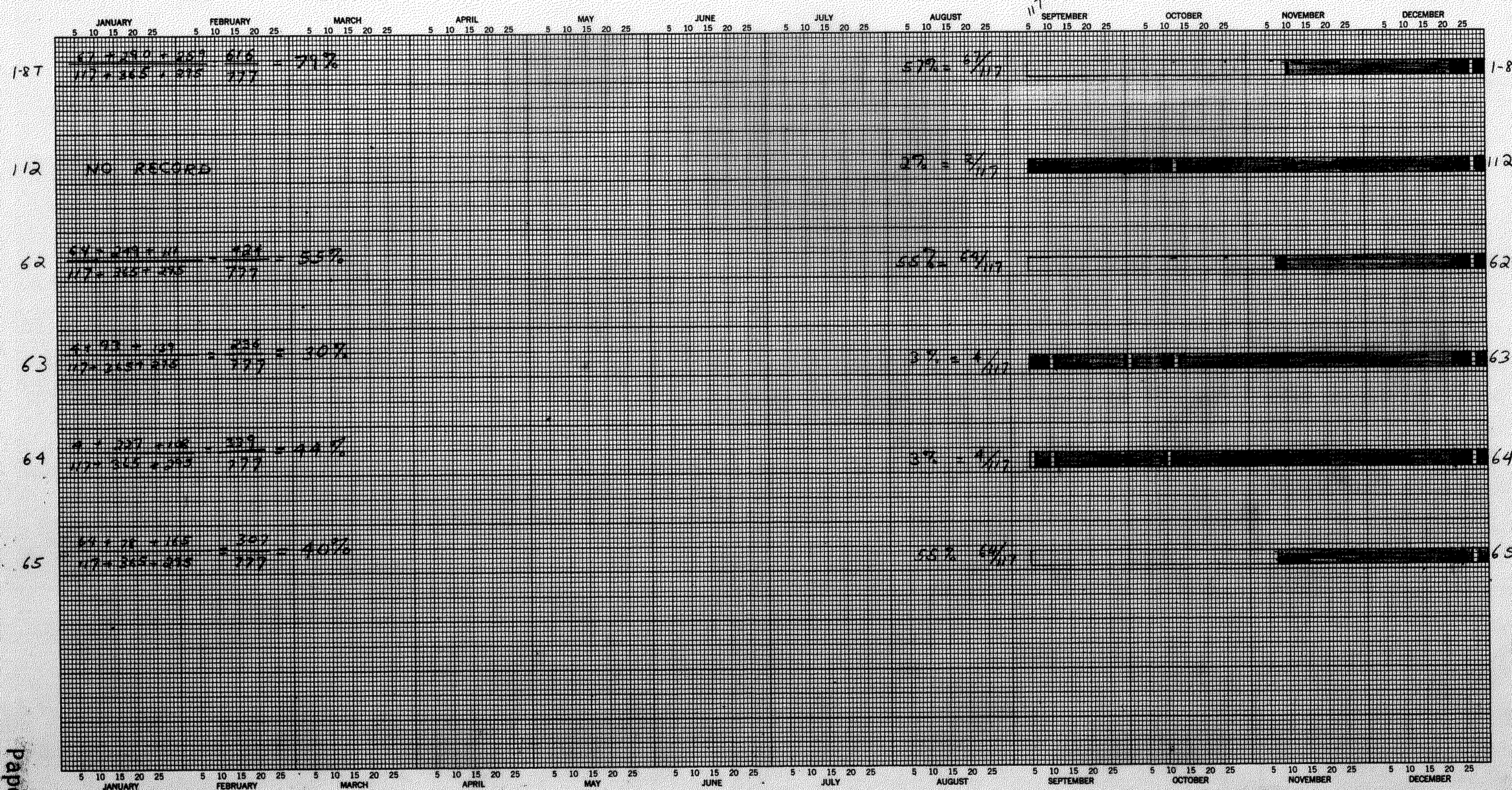
APPENDIX Sheet 3 of 9

K+E 1 YEAR BY DAYS X 150 DIVISIONS KEUFFEL & ESSER CO. MADE IN U.S.A. 47 2813

CORPS DATA

1973

STUDY PERIOD



■ NO REPORT OR BAD DATA

APPENDIX Sheet 4 of 9

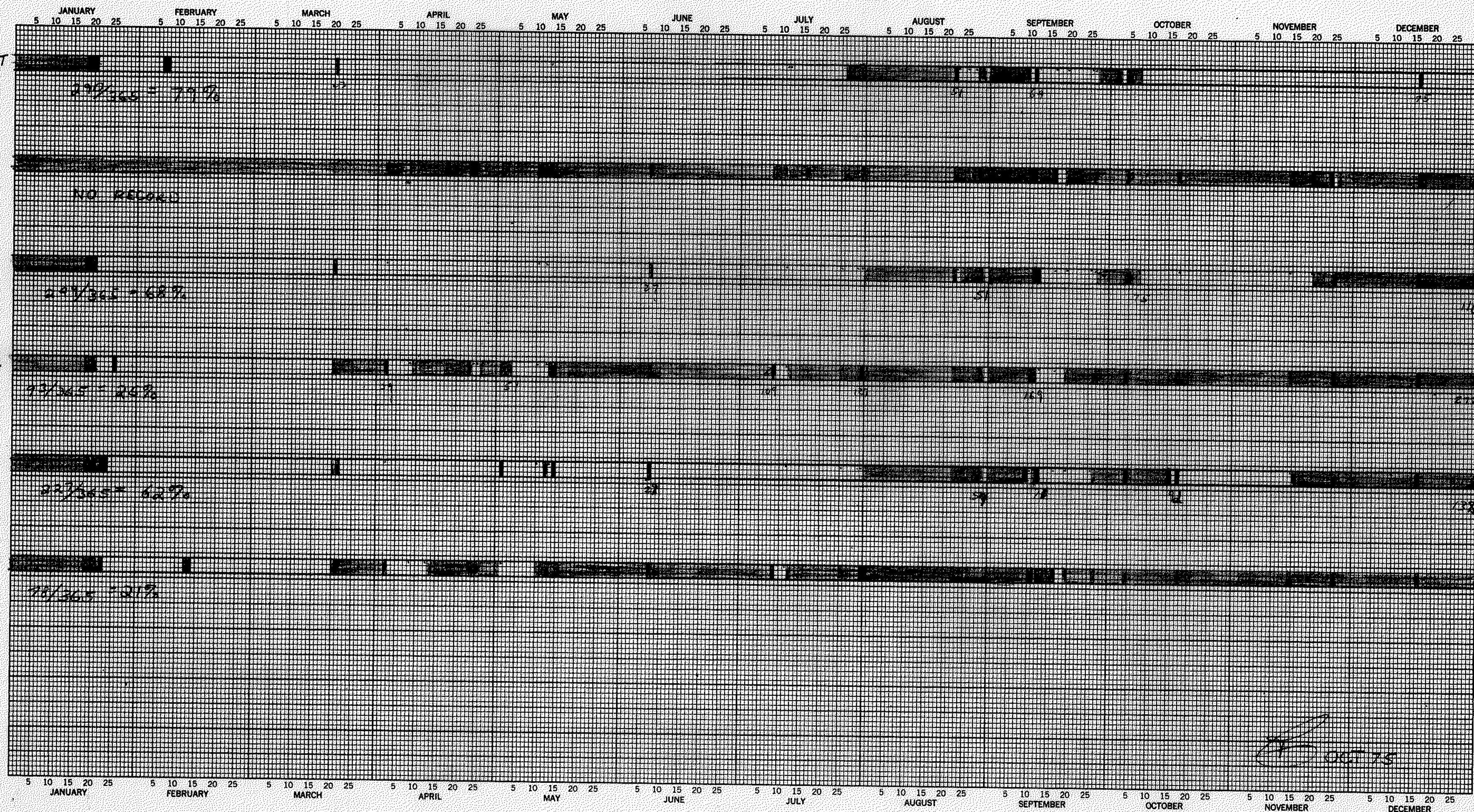
Paper 16

K+E 1 YEAR BY DAYS X 150 DIVISIONS KEUFFEL & ESSER CO. MADE IN U.S.A.

47 2813

CORPS DATA

1974



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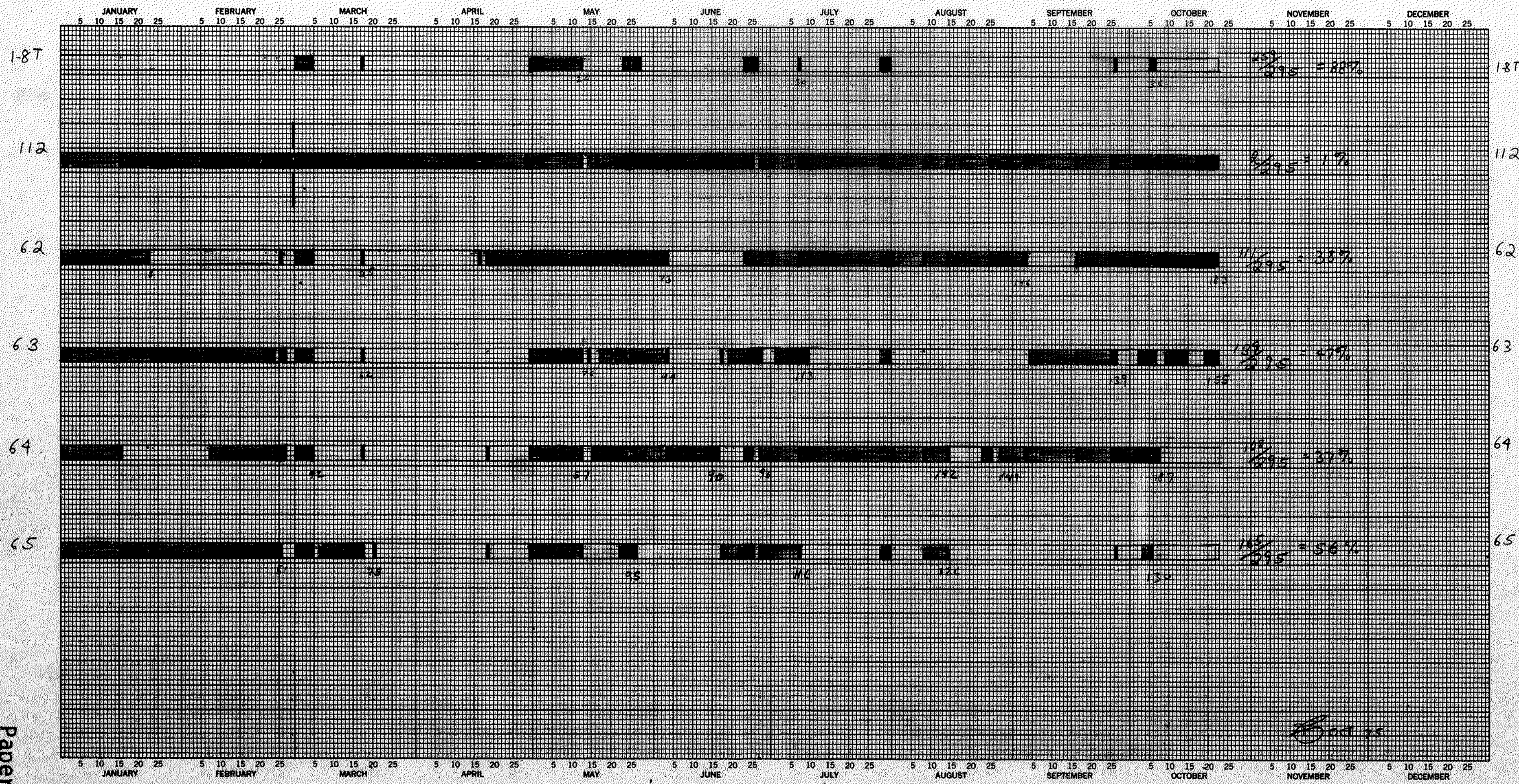
APPENDIX Sheet 5 of 9

CORPS DATA
1975



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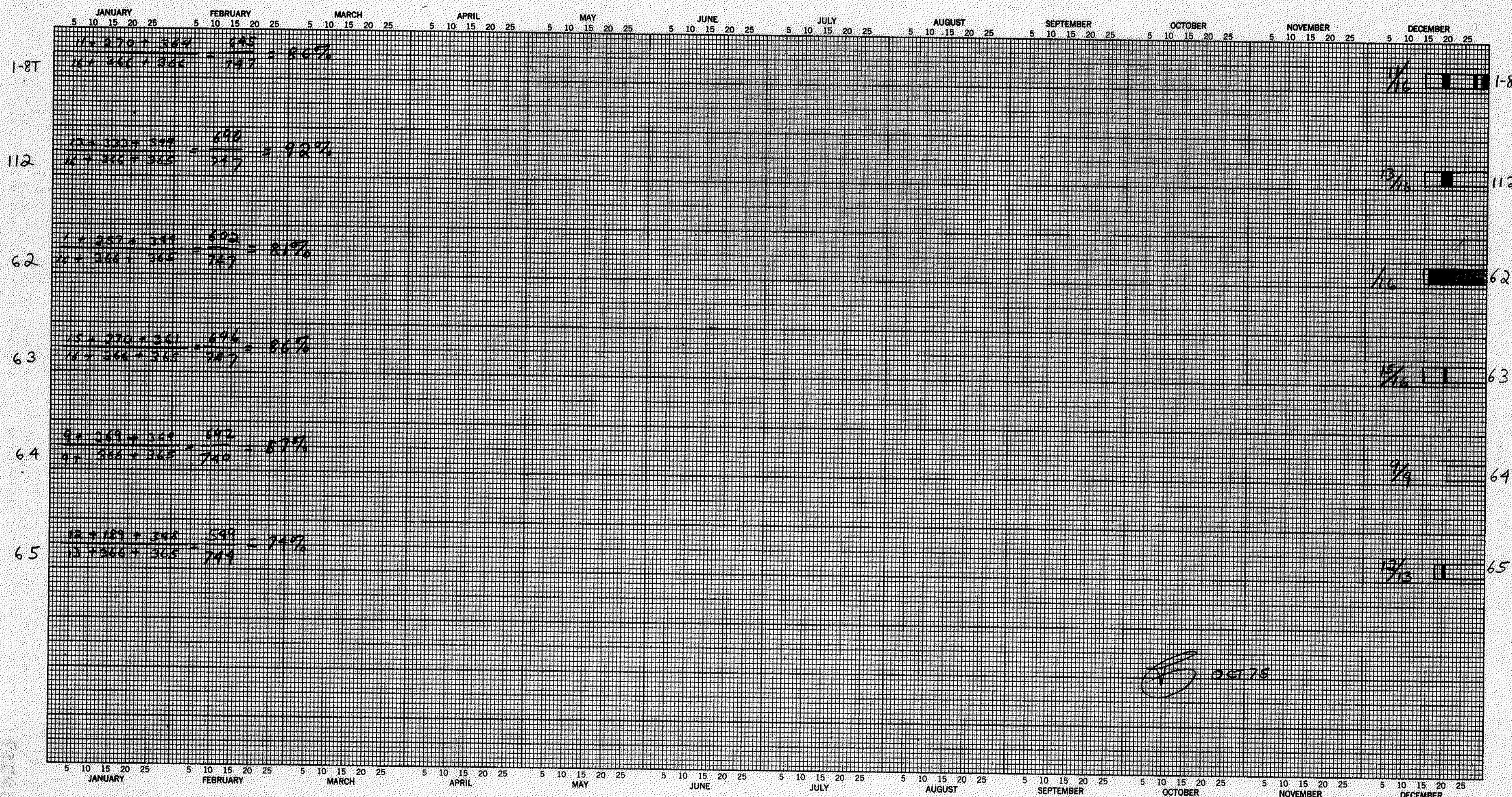
APPENDIX Sheet 6 of 9

CORPS DATA

STUDY PERIOD

1967



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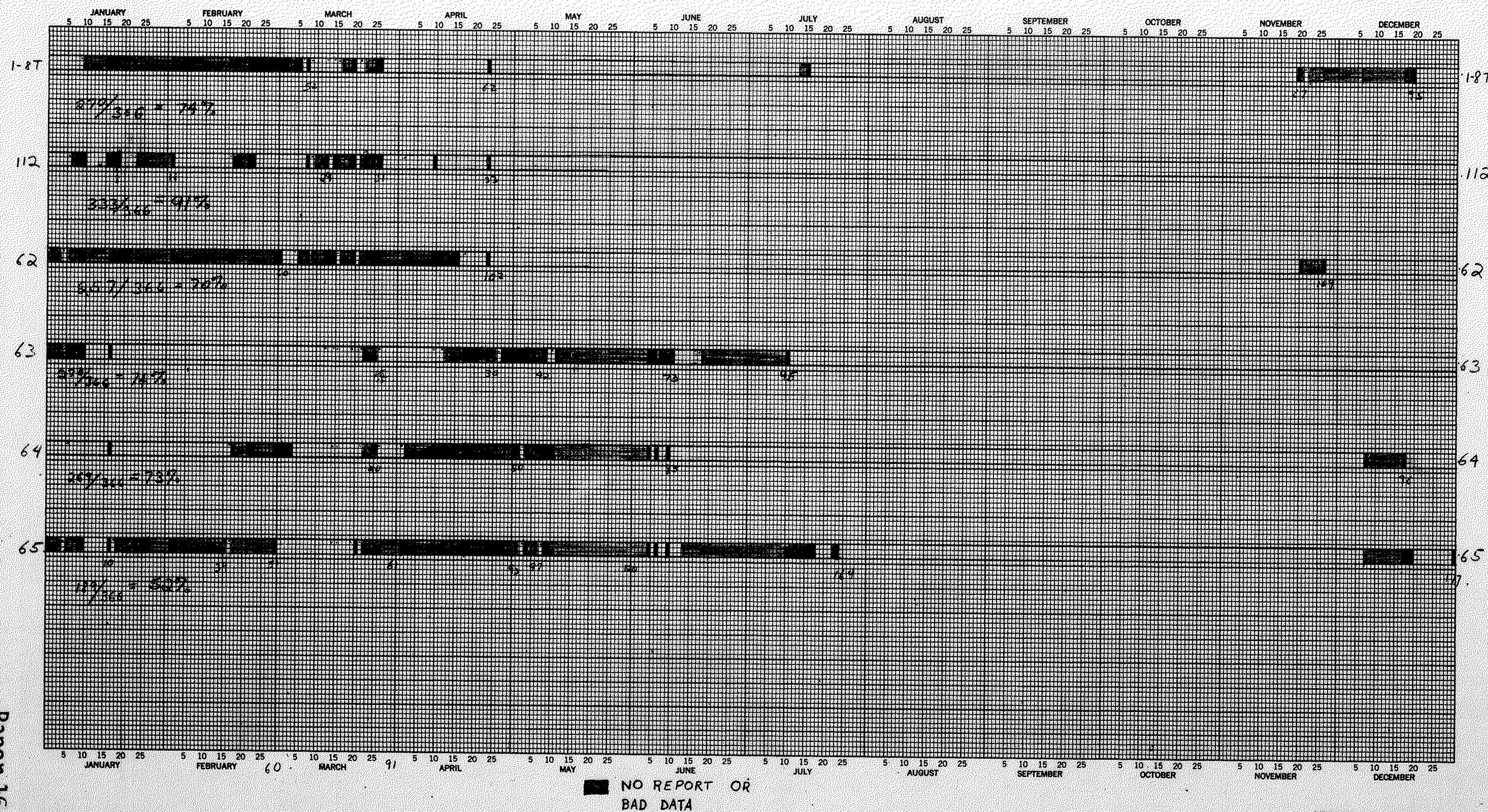
APPENDIX Sheet 7 of 9

CORPS DATA
1968



APPENDIX Sheet 8 of 9

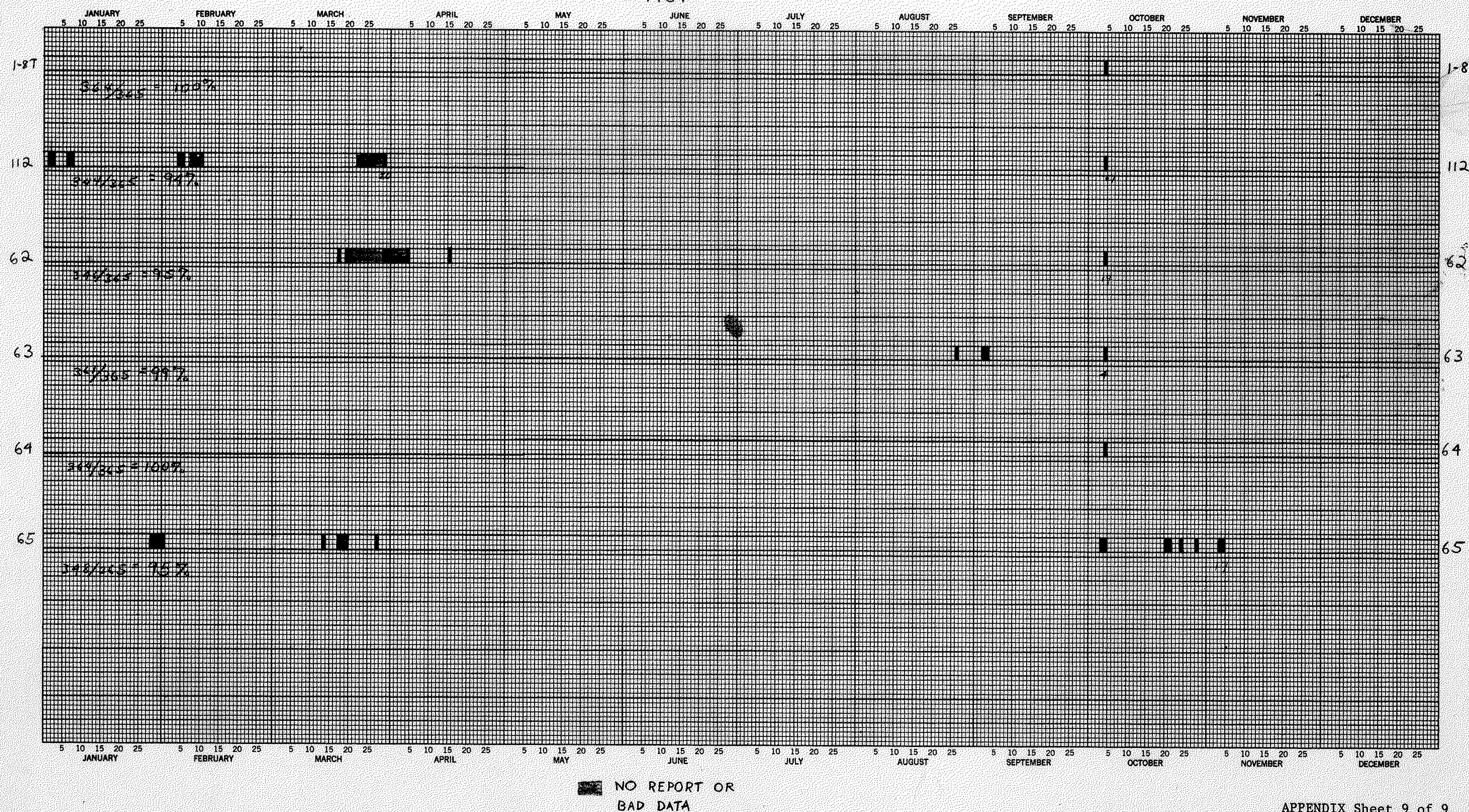
Paper 16

47 2813

K+E 1 YEAR BY DAYS X 150 DIVISIONS KEUFFEL & ESSER CO. MADE IN U.S.A.

CORPS DATA

1969



APPENDIX Sheet 9 of 9

Paper 16

344

WATER RESOURCE DATA COLLECTION SYSTEM
VIA EARTH SATELLITE
LOWER MISSISSIPPI VALLEY DIVISION

by Warren L. Sharp*

INTRODUCTION

A fully automated water resource data collection system is in the early stages of installation in the Lower Mississippi River Basin. The initial phase of the system (Figure 1), which is planned to be fully operational by September 1976, includes fifty-seven reporting stations throughout the Lower Valley. Sixteen of the stations are on the main stem Mississippi River, fifteen are at Corps reservoirs, twenty-four on principal tributaries, and two are on the Atchafalaya River outlet to the Gulf of Mexico. All of the stations can be monitored via earth satellite every four hours, at which time data recorded at the sites within the four-hour interval can also be reported. All of the stations will report water level. Rainfall will be reported from all but ten of the stations and water quality will be reported by eleven of the stations initially. The principal physical components of the system consist of: automatic measuring equipment for the parameters just mentioned; water data transmitters containing memory; earth satellites; a central receive site; and a data communications processor. Each of these components is discussed in this paper in terms of their role in the system, including mention of special features which I think will be of interest to you. In addition, comparison with other more conventional data collection systems and some uses of the data are discussed very briefly.

* Chief, River & Reservoir Control Center, Lower Mississippi Valley Division, Corps of Engineers, Vicksburg, Mississippi 39180

MEASURING EQUIPMENT

Conventional automatic recorder type rainfall and water level measurement equipment is being used in the system. Strip charts and punched paper tape may remain in use. A special study is being conducted by WES at our request to evaluate on-the-shelf water quality sensors for use in the system. The report on this study will be available about 1 July 1976 to any of you who may want it. Water quality parameters to be monitored initially are temperature, pH, DO, conductivity, turbidity and certain ions.

WATER DATA TRANSMITTERS

Water data transmitters will be installed at the recorder stations to relay data to either the Geostationary Operational Environmental Satellite (GOES), or the Earth Resources Technology Satellite (ERTS). The transmitters have several important features that will enhance the utility of the system. Some of these features are: capability to monitor four digital recorders; plus 8 analog sensors; transmission in either the GOES or ERTS modes; 832 bit memory; 16 bit digital and analog conversion to 8 bit digital word sizes; American National Standard Code for Information Interchange (ASCII); AC or DC and; essentially unaffected by weather conditions. The transmitter is contained in a relatively small drum, 11 1/2 inch diameter x 14 1/2 inches high, which will accommodate easy installation in the recorder shelters. The drum will protect against moisture and to some degree against vandalism. Separate antennas for the GOES and ERTS systems are furnished with each transmitter. Full memory data (832 bits) and station identification can be transmitted in 25.6 seconds every four hours via the GOES system. Transmission of data in sets consisting of 64 bits/set in the ERTS mode can be programmed to occur on either 90 or 180 second intervals; however, the data can only be retrieved at twelve hour (approx) intervals. Also, full memory cannot be transmitted during the ERTS orbit. Conversion from GOES to ERTS, or vice versa, is accomplished by rotating a switch on the transmitter.

Each water data transmitter will contain a time clock accurate to thirty seconds per year which is used to activate transmission. All of the variable set up data such as the GOES/ERTS mode, transmit interval, data collection between transmissions, memory usage, message formatting, and instrument configuration will be programmed into the water data transmitter through switches on a "test set" that is provided by the manufacturer. The cost of the transmitters is approximately $4,000 each, which can be reduced substantially in the future since development of the product has been established.

EARTH SATELLITES

The Geostationary Operational Environmental Satellite (GOES) and the Earth Resources Technology Satellite (ERTS) systems will be used to relay data from the recorder stations to central receive sites at Vicksburg, Mississippi and Bay St. Louis, Mississippi. The GOES system is provided by NOAA and the ERTS system by NASA. Two GOES weather satellites are stationary above the equator at an approximate altitude of 20,000 miles at longitude positions 75 degrees and 115 degrees. These two satellites contain identical channel frequencies. When a station is reporting in the GOES mode the data are relayed by both GOES satellites, therefore, the data can be received from either. There are 150 channels on each GOES satellite. Fifty of the channels will relay data from self-timed transmitters and 100 are for interrogable type transmitters. The assignment of channel frequencies is controlled by NOAA, NESS, and two self-timed channels, 35 and 55, have been assigned to the Corps for exclusive use. It will be explained later how these two channels and others when needed, can be used by any and all Corps of Engineers Districts in the United States. All transmitters in the LMVD network are self-timed and cannot be interrogated. As stated earlier, their transmission in both the GOES and ERTS modes is controlled by a self-contained clock.

The ERTS satellite orbits the earth about every 12 hours at an altitude of approximately 5,000 miles. Acquisition of water data via the ERTS system is a major improvement in terms of cost and utility over conventional land line and microwave systems. However, the infrequent and limited data relay capability of this satellite for a given hydrometeorological or water quality station, and the relatively high cost for a tracking receive station imposes a significant limitation on use of the ERTS system for real-time water control management activities. In view of these constraints, water data will normally be monitored via the GOES system and the ERTS system will be used for backup.

CENTRAL RECIEVE SITES

The principal receive site will be located at Vicksburg, Mississippi, and will be operated by the staff of the River & Reservoir Control Center. The existing interagency GOES/ERTS receive site at Bay St. Louis, Mississippi, which is operated by the National Space Technology Laboratories (NSTL), NASA, will be used as backup to the recieve site at Vicksburg. Receive sites are commonly termed ground stations or downlinks. The Corps has contributed funds for installation of the NSTL downlink.

The GOES downlink at Vicksburg will consist of a 30-ft diameter fixed type receiving antenna, which will normally be directed toward the east satellite at 75° longitude above the equator. Although termed a fixed antenna, it can be pointed toward the west satellite at 115° longitude in the event a power failure occurs in the east satellite. Also, the capability to hone in on the satellite whenever it drifts will be provided with indoor control. The 1500 lb receiving antenna will be mounted on the roof of the Division office. A receiver and two demodulation units which operate simultaneously, one for each channel frequency, will be wired to the antenna. The receiver will accommodate additional demodulation units, which will eventually be necessary to enlarge the network of stations.

DATA COMMUNICATIONS PROCESSOR

Data processing equipment for the regional ground station in the River & Reservoir Control Center consists of a mini-computer with disc storage and dial-up communications capability. The system selected is the Data General NOVA 840 which has a 32K core storage, CRT/keyboard terminal, arithmetic hardware unit, 2.5 million words of disc cartridge storage, a real-time clock, four asynchronous data communications channels, two synchronous data communications channels, and a printer. In addition, a 19 inch Tektronix Graphics Terminal will interface with the NOVA 840 and with time-share. The configuration has the advantage of a real-time, disc-operating system which allows around-the-clock unattended operation. The data communications capability provides for data programming and control information interchange with other regional ground stations in addition to NSTL. Any authorized user having a compatible terminal with dial-up phone capability will be able to access the data base. Systems and applications software developed for the NOVA 840 communications processor located at NSTL will be fully compatible, and can be used interchangeably with the Corps processor. This has resulted in a significant cost savings in the system.

The following list of equipment constitutes the data communications processor.

NOVA 840 with 32K memory	1 ea
Automatic program load	1 ea
Real-time clock	1 ea
Console CRT	1 ea
Disc System	
Disc Control	1 ea
Disc adapter and power supply	1 ea
Disc drive unit 1.25 M words	2 ea
Disc cartridge	2 ea
Printer	
Printer I/O interface	1 ea
Printer control	1 ea
Printer serial matrix 165 cps - 132 columns	1 ea
Asynchronous Communications	
I/O interface board	4 ea
Teletype I/O interface	4 ea
Voltage EIA type I/O	4 ea
Data set I/O 2 @ 110 and 2 @ 1200 baud	4 ea
Precision crystal for 110 baud	2 ea
Modem cable	4 ea

Synchronous Communications

Synchronous line adapter	2 ea
Modem Cable	2 ea

Paper Tape System

I/O interface board	1 ea
Paper tape reader control	1 ea
High speed paper tape reader	1 ea
Paper tape punch control	1 ea
Paper tape punch	1 ea

GOES Interface

General purpose I/O board	2 ea
Wire wrap pin and socket option	2 ea

Graphics Terminal

Precision Crystal Oscillator	1 ea
Teletype I/O interface	1 ea
CRT interactive Graphic's Terminal 19"	1 ea
Hard copy unit	1 ea
Data General I/O interface	1 ea
Data Communications option	1 ea

MONITORING THE NETWORK

The preferred maximum time period required to monitor the network of stations is 10 to 15 minutes, with 30 minutes considered a moderate time lapse and one hour an absolute maximum. The latter has been adopted for trial initially, with a 30 minute monitoring time period selected as a target to achieve in the not too distant future. Other constraints include: 25.6 seconds time required to transmit full memory and station identification in the GOES mode from a data collection station; 30 seconds annual drift in transmission time from a station; anticipated increase in the number of stations in the network; and the requirement by NOAA, NESS to make maximum utilization of satellite channels. Since data collected during a short time period from relatively large regions is a requirement by water control managers and because additional data from the region are seldom needed until a few hours have lapsed, the following plan was established which complies with the above constraints.

Each station on a given channel will transmit at one minute intervals, which will allow 60 stations to be monitored in one hour on each channel. The two channels can be monitored simultaneously, as well as others when added. The four hour time intervals for monitoring the network stems from making the channels available for use nationwide; i.e., the interval equals the number of time zones in the U. S. Each channel will be available to District offices in different time zones at their same local times. For example, offices in the Central time zone can use the channels from 6 to 7 a.m. local standard time (7 to 8 DST), followed by use of the channels in the Rocky Mountain time zone, again 6 to 7 a.m. local standard time . . . etc., for the Pacific and Eastern time zones. Monitoring the network automatically just prior to normal working hours will provide information on prevailing conditions for immediate use. On this basis, subsequent monitoring periods on standard time via GOES would be 10 to 11 a.m., 2 to 3 p.m., 6 to 7 p.m., 10 to 11 p.m., and 2 to 3 a.m. As mentioned earlier, data recorded during the four hour intervals can be obtained. The data can be accessed at any time by the District offices using dial-up I/O terminals. Standard grade telephone lines will be used for all communication by the processor, at least initially.

The ultimate aim is to monitor the network within 30 minutes of the allotted hour to permit more immediate use of the data. This would allow monitoring the network a second time during the allotted hour to enable checks on the data and the system.

USE OF THE DATA

Increased development of water resources projects in the basin and severe weather events during the last three years have demonstrated a need to improve water control activities in the lower Mississippi River valley and possibly the entire basin. Initial efforts have already been made toward broadening the roles of water control management and data communications. The first phase of the automated hydrometeorological data collection network via satellite is scheduled for completion in early FY 1977, and additional stations are planned for the future. Activities are underway for evaluation and adaptation of water control models to perform data handling, hydrologic forecasting, and water control decision-making tasks.

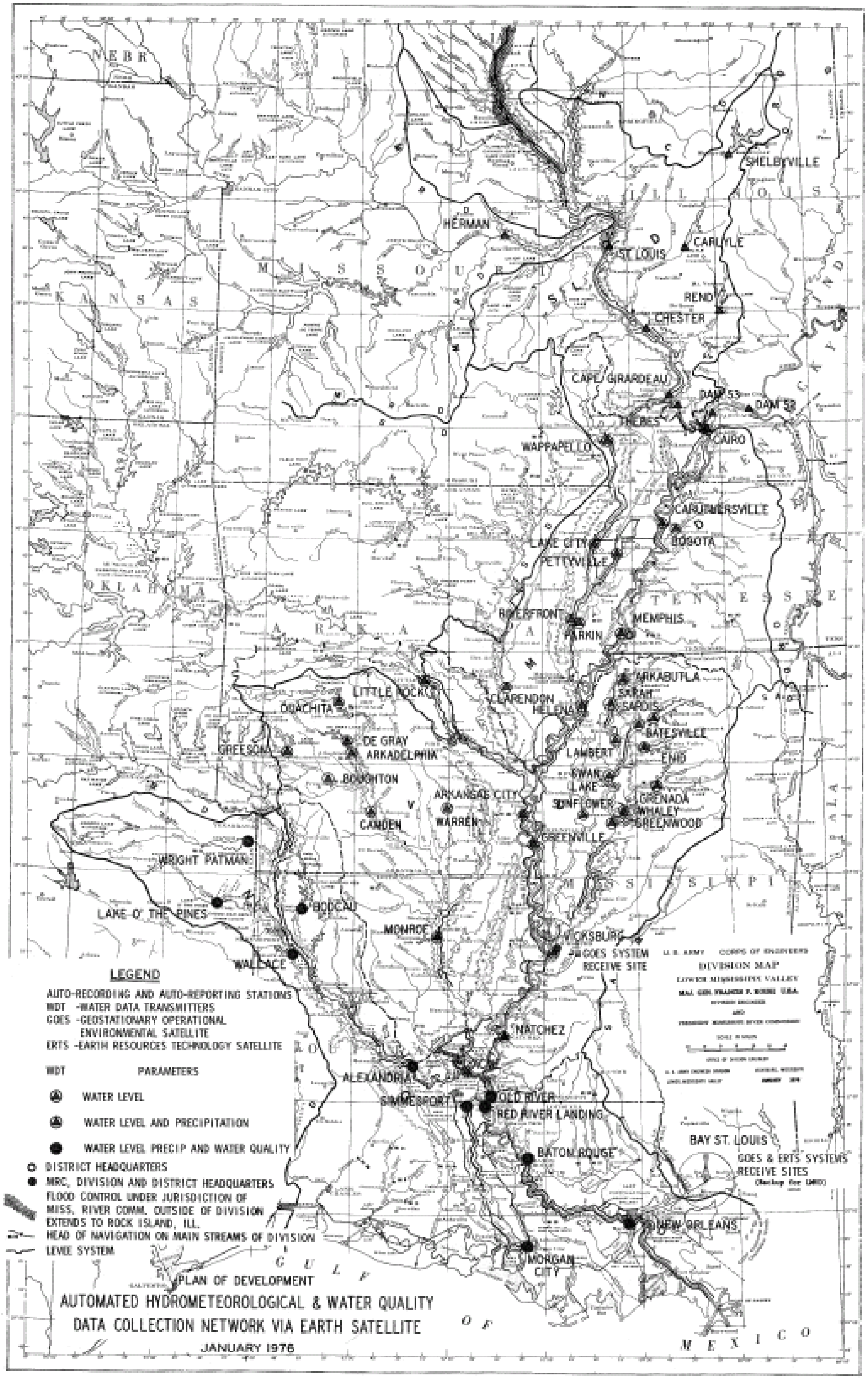
A sophisticated model for routing the Mississippi River that accounts for dynamic changes in river regimen and provides for operation of Mississippi River and Tributaries (MR&T) flood control features is in the final stages of adjustment and application. This model and others require input data in a real-time mode which will be provided by the network under installation, additional reporting stations, and the satellite data collection system. The most efficient manner for applying the models is through collecting and massaging the large quantities of data involved. The communications processor will permit more efficient utilization of INFONET and computer facilities at the Waterways Experiment Station. It is also planned to use the WES facility for a permanent data bank in a non-real-time mode to accommodate investigative type water control activities. Conversion of the form of data, verification, and frequent or continuous interaction (real-time) with the data bank are not practicable on a large diversified processing computer. This is especially true for a multi-user facility, because it would require a high-priority-interrupt for immediate access and turn-around which would conflict with other processing.

In addition to real-time water control activities, studies will be conducted in the near future to better define water control capabilities of the project under present conditions. A preliminary investigation will be made to improve the capability for water control, including consideration of physical (structural) changes in the MR&T portion of the overall Mississippi River water resource system. The communications processing system will accommodate data handling aspects of this task.

The NSTL Satellite Data Acquisition and Processing System is considered experimental in nature, and there is no mandate to maintain its current posture and provide a permanent operational service. However, the need is rapidly growing for applications of both LANDSAT and GOES systems. Even though LMVD proposes to become independent of NSTL on the GOES system, a continued interface will be maintained between LMVD and NSTL for data collection backup and for investigating the use of LANDSAT data for monitoring the extent of inundation during floods.

Developments under the direction of water control managers in LMVD are being closely coordinated with other organizational elements to ensure suitability for application elsewhere by the Corps. Appreciable savings can be realized in data collection hardware and software, satellite channel usage, and in application of water control models as a result of these developments. The LMVD satellite data collection system will increase the dependability of data acquisition and the efficiency of water control activities by providing necessary information directly to water control managers. In summary the system will:

- Permit real-time interaction between a dedicated water control processor, the data collection downlink, and water control managers
- Provide a data bank for real-time use
- Provide compatibility with the NSTL data collection system, which will ensure a major cost savings in software
- Ensure continued real-time collection of data via the GOES system
- Permit graphics interaction in the processing of data
- Provide a terminal to the Waterways Experiment Station (WES) computer or commercial computers to process large volume water control data


LEGEND
AUTO-RECORDING AND AUTO-REPORTING STATIONS
WDT -WATER DATA TRANSMITTERS
GOES -GEOSTATIONARY OPERATIONAL ENVIRONMENTAL SATELLITE
ERTS -EARTH RESOURCES TECHNOLOGY SATELLITE
WDT PARAMETERS
WATER LEVEL
WATER LEVEL AND PRECIPITATION
WATER LEVEL PRECIP AND WATER QUALITY
DISTRICT HEADQUARTERS
MRC, DIVISION AND DISTRICT HEADQUARTERS
FLOOD CONTROL UNDER JURISDICTION OF MISS. RIVER COMM. OUTSIDE OF DIVISION EXTENDS TO ROCK ISLAND, ILL.
HEAD OF NAVIGATION ON MAIN STREAMS OF DIVISION
LEVEE SYSTEM
U.S. ARMY CORPS OF ENGINEERS
DIVISION MAP
LOWER MISSISSIPPI VALLEY
NEBR
KANSAS
MISSOURI
ILLINOIS
OKLAHOMA
TENNESSEE
MISSISSIPPI
GULF OF MEXICO
SHELBYVILLE
HERMAN
ST LOUIS
CARLYLE
REND
CHESTER
CAPE GIRARDEAU
DAM 53
DAM 52
THEBES
CAIRO
WAPPAPELLO
CARUTHERSVILLE
DOBOTA
LAKE CITY
PETTYVILLE
RIVERFRONT
PARKIN
MEMPHIS
ARKABUTLA
SARAH
SARDIS
LITTLE ROCK
OUACHITA
CLARENDON
HELENA
BATESVILLE
GREESON
DE GRAY
ARKADELPHIA
LAMBERT
ENID
BOUGHTON
SWAN LAKE
GRENADA
ARKANSAS CITY
SUNFLOWER
WHALEY
GREENWOOD
CAMDEN
WARREN
GREENVILLE
WRIGHT PATMAN
LAKE O' THE PINES
BODCAU
MONROE
VICKSBURG
GOES SYSTEM RECEIVE SITE
WALLACE
NATCHEZ
ALEXANDRIA
SIMMESPORT
OLD RIVER
RED RIVER LANDING
BATON ROUGE
BAY ST. LOUIS
GOES & ERTS SYSTEMS RECEIVE SITES
NEW ORLEANS
MORGAN CITY
PLAN OF DEVELOPMENT
AUTOMATED HYDROMETEOROLOGICAL & WATER QUALITY DATA COLLECTION NETWORK VIA EARTH SATELLITE
JANUARY 1976

SEMINAR SUMMARY

by

Vernon K. Hagen[1]

This seminar in Davis, California has provided an ideal setting for exchange of technical procedures, managerial concepts and thought provoking ideas regarding real-time water control management. Discussions following each paper brought out some differences in the attitudes of managers and technical experts but largely tended to reinforce the views expressed in the formal presentations. All participants in the seminar had an opportunity to be heard and to compare water control activities in other Districts and Divisions with those prevailing in their home office. Some of the noteable conclusions reached by the majority (not necessarily the same majority for each conclusion) are as follows.

GENERAL WATER CONTROL MANAGEMENT CONCEPTS

1. Modern high-speed computers facilitate the ability to develop alternative regulation options quickly and to display the consequences of each; however, the judgment of experienced water managers must be the basis for final decisions. Otherwise, human values would be replaced by rigid rules established strictly by so-called optimizing procedures.

2. The Corps must pursue aggressive water management programs that use sophisticated models and equipment, where necessary. If not, others will demonstrate abilities to do better and may eventually take over the management of Corps projects.

3. In the real-world situation, limitations on the availability of basic data negate the value of overly complex representations of hydrologic processes.

4. Reservoir/Water Control Centers should play an unbiased leadership role among special interests groups to assure balance for all project purposes.

5. Other project purposes are having increasing impacts on acceptable flood control regulation plans. Indications are that this trend will continue and that regulation for flood control will be more difficult in the future.

6. Realistic water control plans should be developed early in project planning and design. Such plans will minimize the needs for extensive expenditures to develop functionally acceptable plans after completion of the projects.

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DATA COLLECTION AND PROCESSING

1. New facilities for data collection and processing are emerging and must be given due consideration before selecting major basin systems. Meteor burst and sattelite communication facilities appear to be offering economic and in some cases functional advantages over conventional radio and telephone facilities.

2. With the increasing demand for more data and constraints on manpower, the need for automated data collection and processing systems is becoming more urgent.

3. Enhancement of existing data facilities should be based on utilization of available facilities insofar as possible. Additional equipment should be flexible so as to permit coupling with as many modes as possible.

STREAMFLOW FORECASTING

1. Cooperation and coordination with other Federal agencies is essential, particularly with the National Weather Service.

2. Quantitative precipitation forecasts (QPF's) issued by NWS on expected future rainfall should be used to alert water managers of possible increased runoff. They should not be used as indicators of absolute future conditions.

3. Although the computer permits more sophisticated river routing programs, there is still a need to use different techniques on specific reaches of major rivers for real-time purposes. Availability of data and the need for accuracy are major factors.

REAL-TIME REGULATION

1. In many cases, time-sharing commercial computers can be successfully utilized in a real-time regulation mode. There is a need for GSA to make 24-hour, 365 day service available at reasonable cost as provided with in-house computers.

2. There is an increasing need for more accurate accounting of reservoir evaporation, particularly in those water short states with appropriated water rights.

3. Reasonable cost solutions are needed where existing reservoir projects are physically unable to meet current state-Federal water quality standards. Otherwise, there will be increasing resistance to regulation for authorized project purposes.

4. New generalized techniques and interactive computer equipment are presently available to better evaluate alternative regulation plans for systems of water control projects. Costs of such studies and equipment should be justified by potential increases in benefits, functional advantages or credibility requirements.