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This volume of the Proceedings is

dedicated to the memory of

FRANK TANNENBAUM

Founder and Director of the University Seminars Movement
1945 – 19–

Founder of the University Seminar on Pollution and Water Resources
1967 – 19–

"The complexity of our technological age necessitates the close cooperation among experts in various fields for a better insight, understanding and mutual comprehension of the problems which beset our civilization."

PREFACE

Water, essential to human life, is our most important natural resource. Although it may be available in abundance to some, others must treasure a trickle. Fortunately, it is an almost infinitely re-useable commodity. We can enjoy and employ the once polluted over and over again through improved reclamation processes. Proper water management and growing research efforts in the complex problems of water resources are essential to our efforts to preserve an adequate supply of water for man.

Much of our nation's past and, very likely, much of its future, has as its base the concentration of population upon the coastal plains, lake shores and river banks. Here is where the bulk of our people live. Here is where water is abundantly available for our needs.

The University Seminar on Pollution and Water Resources has undertaken an ambitious plan; to foster meetings, lectures and research in an attempt to focus knowledge from a variety of disciplines onto specific complex problems in water resources. The results of the first steps taken is the following summary of articles and lectures offered by the Seminar in the academic year 1967-68.

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INTRODUCTION

October, 1968

At the opening session of the International Water for Peace Conference in Washington, D. C., on May 23, 1967, President Johnson stressed the immediate need to educate and train experts who can deal with problems on the world's Water Resources and Programs. In view of the international character of the problem and the location of Columbia University to the headquarters of the United Nations, in the greatest metropolitan complex of the world, members of the University, interested in problems of Water Resources, felt that it was appropriate to launch such a program at Columbia University, where, in an international atmosphere, a training program for cooperation among nations in solving water problems could be stimulated.

On June 21, 1967—based on these principles—University Seminars in Water Resources were established with the aim that in the academic year 1967/68 besides Columbia Faculty and off-campus specialists in Anthropology, Chemistry, Economics, Engineering, Geography and Geology, experts dealing with Water Law, Legislation and Government would be invited too, to join the Seminars. Furthermore, lectures dealing with Water Resources were given on a monthly basis, which program was later extended to include problems of Pollution. Participation of advanced graduate students interested in problems of Pollution and Water Resources, was conditioned on recommendation of the members of the Seminars.

The University Seminars stress the comprehensiveness of the issues they deal with and welcomed members from different disciplines and professions involved in the area. Besides the above mentioned, the Seminars are supposed to serve the following purposes too:

a) Collect available experts in the field of specialization as an "Intelligence Pool" for eventual future enlargement of the Seminars;

b) Serve as a basis for development of a research institute;

c) Organize regular postgraduate and other courses in this discipline in accordance with the needs.

The lectures of the Seminars are handling the problems in two main categories. First, the ever-present need to bring arid areas under cultivation so that our growing populations may be fed. Second, the equally pressing need to overcome the difficulties created by the increasing complexity of urban life. Industrial development, amassing men in tightly packed communities and disgorging massive quantities of waste, places unprecedented demands on natural sources of water supply and contributes to our pollution at an alarming rate. The wider application of the various methods and technology to the problems of securing sufficient water resources for supply and of handling and treating waste waters must depend on the achievement of higher efficiencies and lower costs. That is why it is necessary to combine the expertise of industry, government research establishments and the universities. The joint effort requires that all the involved specialists participate and contribute to the understanding of the problems.

In the 1967/1968 academic year the selected and on the following pages published lectures are limited to phenomena and technology as follows: problems on hydrological data collecting for decision making in water resources; increase of precipitation by artificial weather modification; seeding of clouds; watershed management in the National Forest Area of the West; improvised desalination plant in the Caribbean; water resources planning and hydrologic risk; comprehensive approach to the problems of pollution and water resources; analysis of spit-bar development in Sandy Hook, New Jersey; relation between precipitation, flood and windbreak phenomena of the mountains—a case study from Central Europe.

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THE WATER-DATA BASE FOR WATER-MANAGEMENT DECISIONS

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February, 1968

There are in the United States about 2,000 major reservoirs; there are about 19,000 municipal supply works; there are more than 3,000 completed and more than 300 unfinished flood-control projects; there are about 1,600 hydroelectric-power structures, and about 10,000 sewage-treatment works. Many of these withdraw, store, distribute, release, use, treat, or dispose of water at rates of many millions of gallons per day. All these thousands of points of water use are also points of water decisions—gates to be opened or closed, pumps to be started or shut down, water to be stored or released, demands to be met, auxiliary supplies to be tapped, treatment processes to be maintained or modified, effluents to be held or released, and so on.

Decisions concerning the use, regulation, and disposal of water commonly have effects upon many people, some of whom may be benefited and others adversely affected. This is true of water to a greater extent than of most other natural resources, because water is widely distributed, mobile, partly renewable, and flows in orderly hydrologic systems which can be—and in many places have been—significantly modified by man. Individual decisions may have effects extending far beyond the special interests directly involved in the decision, and thus are matters of social as well as individual responsibility.

Some of the water data needed for decisions are necessarily obtained directly by the water manager or user, and are related to his specific needs. The data that are essential for appraisal of the resource and for surveillance of the changes wrought by man in the hydrologic system—data that are prerequisite for assessing current conditions and predicting future conditions—may also be collected directly by the water user or manager. But the collection of these data may be beyond the interests, the resources, or the jurisdiction of a particular water-data user. Also, these data may serve other purposes and other water users, as well as regulatory agencies. Thus, there has developed in this country a concept that the provision of the basic store of multipurpose water data is a public responsibility, and should be collected by public agencies. The Geological Survey, the public agency most heavily engaged in fulfilling this responsibility, is involved in the design of a "national water-data network," and it may be of interest to see how it—and the Federal Government generally—got involved.

Most of our forefathers, coming from Europe, an environment of relative water abundance, brought with them a heritage of taking water for granted. This shows up in our common-law doctrines of water rights, appurtenant to land ownership. If you own land along a stream you have a right to use or not use as much as you please at any time, so long as you accord similar rights to all other riparian landowners. You also have a right to as much as you want of the soil water and ground water under your land, and you can even interfere with the water supplies of your neighbor, although in some jurisdictions your use of the water must be "reasonable." These doctrines take for granted that there will be sufficient water for all reasonable "natural" needs.

The Constitution does not mention water, but it does say that all powers not expressly delegated to the Federal Government are reserved to the States and the people. So the Federal Government (except for its Navy) stayed out of water for more than three decades. Not until after the invention of the steamboat, when Robert Fulton and associates sought exclusive rights for navigation, did the Supreme Court point out that the power of Congress over commerce includes navigation. Soon thereafter, Congress passed the first River and Harbor Act (1826). Hence, the pioneer Federal hydrologic basic-data programs were related to navigation and then to flood control; the Corps of Engineers made studies of the Ohio and Mississippi Rivers prior to the Civil War. After the Civil War, in the years 1867-78, the Federal Government supported four exploratory surveys in the unknown West. Then, in 1879, Congress created the Geological Survey in the Department of the Interior, with functions of "classifying the public lands and examining the geological structures, mineral resources, and products of the national domain." Director John Wesley Powell was among the first to see the complex interrelation of land, water, and climate, and to stress the necessity for adapting, on the basis of factual information, the economic development of the West to these natural

[1] Publication authorized by Director, U. S. Geological Survey.

conditions. The Geological Survey accordingly considered water to be among the resources that should be examined (although for years some Congressmen argued that water is not a "mineral") and was collecting data on streamflow as early as 1888.

Meantime, the people of the United States wanted their West developed rapidly, and encouraged individual initiative by gifts of land: to the farmer, homesteads; to the first railroads, land grants; and, to the miner, exclusive enjoyment of any treasure he found and the land upon which he found it. Decisions concerning water and development of water supplies were necessary, and they too were largely by individual initiative. In numerous arid territories, where water is the limiting factor in man's occupancy of the land, the settlers developed new systems of water rights—based upon priority of benefical use of a specific quantity of water.

Because of the prevailing water deficiency in the West, competition and conflicts over water started early, long before creation of the Geological Survey. By 1878 the States of Colorado and California had assigned to their State Engineers the task of administering water rights, a job that required gaging of streams to determine the amounts of water available. The first stream gaging by the Geological Survey was aided by informal cooperation with several States. After funds were first appropriated for the permanent Federal stream-gaging program in 1895, Kansas became the first State to cooperate, followed closely by Colorado and Wyoming. By 1910 water data were being collected in about half the States, including all 11 Western States, under cooperative agreements whereby State funds were used to pay part of the cost of work mutually agreed upon.

By this cooperation, the Geological Survey was developing an appraisal and a continuing inventory of the natural water resources, and was collecting the quantitative data essential for research leading to understanding of the natural flow systems. The State water agencies required data for more immediate needs, including allocation of water in accordance with established rights, and assessing the potentials and limitations of the water resources for further development and use. Thus, the water data obtained in the cooperative program—which today embraces all 50 States and constitutes nearly two-thirds of the total water program of the Geologica Survey—have generally served both Federal and State requirements.

After a leisurely beginning with navigable waters and flood control in the 19th Century, the Federal Government in the past half century has invested increasing funds in projects for development, regulation, conservation, and rehabilitation of water resources. The action agencies responsible for designing and operating these programs have required both special-purpose data, of value principally to specific projects, and water data of more general public value. These agencies have transferred funds to the Geological Survey for obtaining these general basic data, and about one-sixth of its current program is supported by these other Federal agencies.

It is to be noted that Congress, in creating the Geological Survey, had insisted that its personnel should have no personal or private interest in the lands or mineral wealth of the region under survey. This has evolved into a bureauwide objective of impartiality on all matters pertaining to the natural resources. The impartial appraisal of the water resources by the Geological Survey is valued by State agencies and private interests as a basis for water allocations and management decisions, as it is by Federal agencies responsible for action programs in water resources; and pertinent basic data are accepted by the courts, commonly by stipulation of both contestants.

As one consequence of this financial support over the years by the States and by other Federal agencies, the water data generated by the Geological Survey tend to be more concentrated in areas of greatest water development, and in areas of past or current water problems, even water crises. For many years we have collected data chiefly on the "important" streams, and only recently have we given much attention to the runoff from small drainage areas or from urbanized drainage areas. In extensive areas we have very incomplete knowledge of the natural flow systems, particularly of the subterranean storage and flow of water.

Another consequence of this program is that Geological Survey personnel have worked closely with professional hydrologists, water managers, and technologists of the other agencies, with a tendency to develop a "Cabots-speak-only-with Lodges" syndrome: the raw data, analyses, and interpretive reports are suitable for professional eyes, but often are not understood by the general public. The people who are required to make only occasional decisions as to their individual water supply or disposal would probably not read government reports anyhow, but there is considerable evidence that many such decisions did not achieve a passing grade—in the numerous wells, levees, stockponds, septic tanks, canals, bridges, culverts, reservoirs, and other devices that did not live up to expectations.

The national problem of how to deal with water questions and how to provide the information base for correct decisions is further complicated by factors having roots in the past and by factors inherent in the growing social and economic interpendency characteristic of our times. For example, individual rights to the use of water are recognized as real property, protected by Federal and State constitutional guarantees which prohibit the deprivation of property without due process of law. Both the land and the water resources of the country have been developed largely by individual initiative, and today there is great variation among individuals as to rights to water: some people own land to which little or no water is appurtenant, and some own no land at all and hence have no appurtenant claim on water. In jurisdictions where priority of use of water is the determining factor, some people may have been born too late or too poor to acquire a legal right to use the available water.

What rights in water are recognized as common to all, and applicable to each member of the population? This question has come up as population has increased in arid regions, and their water requirements have grown far beyond the capabilities of the local resources to supply them; it has generally been answered by public projects to provide the needed water from *somewhere*. Although some of these projects require high financing, they have been carefully calculated as economic benefits.

People are also becoming increasingly aware of the value of water in its natural environment, whether in streams, lakes, canyons, waterfalls, wilderness, springs, whitewater, wetlands, estuaries, or oceans. Their interests may be in swimming, boating, photography, camping, hiking, fishing, hunting, painting, contemplation, or relaxation. The urban majority of the population may indulge these interests only a few days a year, or even vicariously. Have they any rights in such uses of water, or do the rights of others for consumptive use or for power generation, cooling, processing, and waste disposal take precedence? In recognizing the rights of individuals to use water for a specific purpose, our society has generally not defined clearly the degree of responsibility of the right holder for the effects of such use upon the water resources. Instead, the public rights in common have been subordinated to the specific rights of individuals.

The increasing nonconsumptive use of water is a principal contributor to the increasing pollution of the Nation's water resources. If an individual or community has been unwilling to assume responsibility for its own pollution of the resources, it is at least partly because the pollutants can move away from them—downstream or downgradient—and in many places have joined waters sufficiently abundant for ample dilution. In broader perspective, every man must use some resources and have some wastes to dispose of, but he may not have within his private property the alternatives of disposal least detrimental to society. Eventually, with increasing population, pollution becomes a social problem affecting everyone. The Water Quality Act of 1948, and amendments in 1956, 1961, and 1965, and the Clean Water Restoration Act of 1966 are among the evidences of increasing public concern and intent to correct abuses of specific environments.

The water data collected by the Geological Survey, and by other Federal and State agencies, constitute the basic water data for current decisions in the progressively enlarging public responsibilities in water management. But these water data are significantly inadequate to the needs related to the vast assortment in kinds of water data that are required to make the many new complex, as well as simple, decisions. The increasing complexity of these operations is requiring progressively increasing professional talent for efficient water management, and all the assistance that modern technology can offer, including instrumentation and requisite computer equipment. In turn, these require a continuous flow of pertinent information about the water.

A decade ago, Langbein and Hoyt [1] identified three basic deficiencies in water data:

(1) "Distribution of water information over the country is not most effective." This deficiency, recognized a decade ago, is perhaps even more obvious today. Today, programs are still born out of crises—floods, droughts, pollution—out of the need for specific project-operation information. The drought experienced in the Northeast recently focused an entirely new concern over the deficiencies in knowledge about the ground-water resources of the Northeastern United States. The inevitable result of a water-data program policy of "reaction" is uneven geographic coverage, short-term and broken hydrologic records, and certain deficiencies in data to meet new crises of the future. It is the nature of most water-data applications that data on antecedent conditions are prerequisite to effective action on a new problem.

(2) "The basic data programs have emphasized data collection to the neglect of advancing knowledge of basic principles." In the past decade this

[1] Langbein, Walter B., and Hoyt, William G., 1959, *Water facts for the Nation's future*, The Ronald Press Co., New York, p. 245-247.

deficiency has been significantly reduced. Most hydrologists, however, would argue that much remains to be done. Most agencies with water-oriented missions have developed significant programs of basic water research. The Water Resources Research Act has given impetus to an entirely new program of water-oriented research in the universities. The Geological Survey's program has been increasingly oriented toward supporting basic research. At the present time more than 10 percent of its funds are dedicated to this purpose.

(3) "Distribution of water facts among those who can use this information is inadequate." It is still true that most water data are directed toward the professional audience—those who make what are presumed to be the important water-management decisions—so, despite thousands of published pages of water data, people who have smaller-scale but nevertheless important water decisions to make still ask, "So, who knows?" Today, water problems exist everywhere and the "need to know" extends across the whole spectrum of our people.

So the need to develop and operate a single basic water-data system to supply the information needs for the water-decision process in all contexts is even more urgent than ever. Despite this recognized need, however, the problem of how to go about designing the "ideal" multipurpose system is very real and, to a great degree, unsolved. A basic problem is that in designing a data system we are "shooting at a moving target"—that is, before a system can be refined to provide basic water-data needs, the spectrum of information needs has changed. Nevertheless, much effort is now being applied in the Geological Survey directed toward a solution of this urgent problem.

There are three methods which draw first attention in consideration of how to design an adequate and efficient national water-data system.

The first method is empirical, or *ad hoc,* and has in fact been used in development of the present water-data system "in being." Essentially, this method involves the compilation of the recognized and expressed needs of all water-data users. A system developed in this manner is heavily weighted in favor of present needs and gives minimum attention to future needs. The system is uneven in distribution of effort and is always inadequate, for new requirements appear faster than new data-acquisition increments can be added to the system. Additionally, a system developed in this manner is not defensible against strictly objective criteria of adequacy and defficiency.

A second procedure in the design of a water-data program depends on the selection of data elements which are adequate to define hydrologic systems. The process is to study the terrain, the entire hydrologic system, the controls in the system, to derive the set of measurements needed to define the hydrologic system, and then to set up a method for synthesis of data at points where measurements are not made. This process of design would recognize the growing use of simulation models in water-data planning and management. However, such a process is too slow to keep up with the moving target of national needs. Additionally, the process develops credibility gaps between the system and the operator of the system. The operator cannot easily understand the relevance of the data generated by such a data system to the immediate water decisions which require his attention.

A third method for design of a national water-data system is to study the decision process itself. To implement this process of design would require analysis of ongoing and proposed developments, from policy to planning to construction and operation. The decisions that are made in each step would be identified and a determination would be made of the specific data that are pertinent to the decisions. In the final step, the required data segments would be combined into a comprehensive data system. Methods of modern systems analysis, where one may examine the effects of differing data inputs—sensitivity tests—are applicable in this method of data-systems design. This method would undoubtedly be the best possible single method to achieve a fully effective and efficient system for meeting water-data needs, but would be too slow and impractical for sole use in designing a data system.

The fact is that in the United States we have a water-data system in being, whatever its deficiencies may be, and one may not start from scratch to design the most ideally effective water-data system. Thus, we must proceed to refine the existing system by all possible methods to eliminate those components which can be shown to contribute minimal information and to identify areas of need which are not included in the system. In the process of refinement it will be necessary to assess, individually, the water-data requirements of all those who make essential water decisions. To make the system efficient, the higher-priority decision points must be examined by every practicable device to determine which components of the data program produce data of greatest relevance to the individual decision. Too, inasmuch as economic factors will never permit the collection of water data at every possible site, it will be necessary to examine hydrologic systems as systems and identify the sets of measurements that will permit simulation for use both in the synthesis of data and for the examination of hydrologic effects of alternate methods of development or modification.

Thus, the process for deriving or refining the national water-data system involves continuing effort. It is the nature of the process that the job will never be completed. We do expect, however, through strenuous effort in the immediate months, to rapidly advance reappraisal and modification of the present system so that the Nation may have the water data it needs to make intelligent decisions in the crucial years which lie ahead—years made critical by rapidly expanding water demands and by greater public sensitivity to the social values that are at stake in the way the water resources of the Nation are managed.

WEATHER MODIFICATION AND WATER RESOURCES

BERNARD A. POWER, M.S.
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February, 1968

INTRODUCTION

During the winter of 1946, the first modern seeding of clouds took place in upper New York State. The seeding, with dry ice, resulted in the production of clearly visible snow showers over the Adirondack Mountains. Today, twenty-one years later, we live in a world in which problems of water scarcity, food shortages, droughts, crop failures and devastating forest fires are almost daily newsfare.

What has happened to weather modification, or cloud seeding for water production, in these twenty odd years? What use is presently being made of its capabilities to produce water for our thirsty world?

Historical Background

While some practical attempts were made at rain making during the past hundred years, the first thoroughly scientific attempt at seeding clouds was the one made near Schenectady, N. Y. by the research team from the G. E. Research Laboratory under the direction of the late Dr. Irving Langmuir, assisted by Vincent Schaefer and Bernard Vonnegut.

The first seedings were made by dropping pellets of solid carbon dioxide (dry ice) into supercooled stratus (layer type) clouds during the winter. Substantial snow showers resulted (1).

Shortly thereafter, Vonnegut discovered (2), that a chemical called silver iodide could produce the same effect. In addition it could be dispensed much more cheaply into the atmosphere either from small ground burners or from aircraft.

Between 1946 and 1956, a major cloud seeding effort took place in North America accompanied by considerable controversy. With the aim of arriving at a definitive answer to the effectiveness of seeding, President Eisenhower set up the Advisory Committee on Weather Control (ACWC) under the direction of the late Capt. Howard Orville.

The ACWC reported in 1958 (3), that cloud seeding in hilly terrain and in upslope conditions during the winter months produced increases of from 10% to 20% in precipitation. The indicated effects of seeding in other types of terrain were inconclusive. This deservedly famous report was, however, greeted by considerable criticism from a small group of statisticians and cloud physicists.

The period from the attack on the Advisory Committee findings from 1958 up to 1966 has been referred to by some as the "lost decade", in that it marked an era of reaction and general negativism. This drastically reduced the amount of operational work carried out. The label "lost decade" is undoubtedly too severe but the period was not an easy one to live through after the initial enthusiasm and the progress which followed the discovery of the seeding process.

However, considerable seeding work, both operational and research, did continue. In January, 1966, a report (#1350) issued by the National Academy of Sciences (4), re-confirmed the general findings of the Advisory Committee eight years earlier, and extended the acceptance of the effectiveness of seeding to include nonorographic areas in eastern North America.

One year after the NAS report, at the Water for Peace Conference in Washington in May, 1967, the President of the United States announced overseas aid in the field of cloud seeding to drought stricken countries. A new era had begun.

GENERAL THEORY OF SEEDING

Nature produces rainfall from clouds in one of two ways. First, small cloud drops can be gathered by collision or coalescence to form a drop large enough to fall through the rising currents and reach the ground as a raindrop. This process operates at temperatures both above and below freezing. Cloud seeding to increase coalescence is done by introducing salt particles into the cloud to form some cloud drops larger than normal.

The second rainmaking process is the ice process. When an ice crystal forms in the upper parts of a cloud that is colder than freezing, the supercooled cloud droplets surrounding the ice crystal evaporate and the ice crystal grows rapidly. The growing crystal rapidly becomes heavy enough to fall through the cloud. In summer the snowflake eventually melts and reaches the ground as rain.

Cloud seeding using the ice process involves the artificial introduction of snow-forming nuclei into the masses of supercooled clouds which do exist in the atmosphere, to cause the precipitation process to start.

The ice process seeding, or nucleation, can be done either by dropping (a) solid carbon dioxide through the cloud, the dry ice acting because of its extremely low surface temperature (−78°C), or (b) silver iodide nuclei can be dispersed into the clouds from vaporizing units either based on the ground or carried by an aircraft.

The dry ice process is efficient but uneconomical. The silver iodide process is efficient and relatively cheap. The salt seeding process is used on clouds whose tops do not extend high enough in the atmosphere to be colder than freezing.

Conventional Seeding Methods Using the Ice Process

The silver iodide nuclei can be released either from the ground or from an aircraft.

Ground based seeding is economical, but since the efficiency of the process depends on the rate of diffusion and the complexity of the air flow, it is sometimes not easy to target the results from this type of seeding. However, the bulk of all seeding over the past twenty years has been done using the ground generating technique. A large body of special art has grown up around the operational methods used to effectively target the results; the chemical efficiency of the different generators is also important.

Airborne cloud seeding uses burners carried in an aircraft to dispense a stream of silver iodide particles directly into the clouds.

This method is considerably more expensive, but it is also more accurate. Light aircraft have been used for seeding in areas where cloud conditions are suitable and flight hazards are not high, but the requirements for seeding aircraft on major projects are demanding.

In most cases heavy duty, multi-engined aircraft, equipped with radar and the latest in navigational devices are required.

Professional cloud seeding operations, ground or airborne, require full meteorological data. In accessible areas, existing meteorological offices are used. In remote areas, field offices are set-up, which are completely and independently able to receive weather teletype data, radio facsimile weather charts and the weather satellite photographs.

New High-Yield Cloud Seeding Technique

In a ten year program of research into methods of generating extremely large numbers of silver iodide nuclei, the Weather Engineering group in conjunction with a leading Canadian explosive company, have developed a device called WEATHERCORD (R).

Weathercord is a linear detonating fuse incorporating a stabilized mixture of high explosive and silver iodide, *(Fig. 1)*. Detonation of the cord releases instantaneously, at an initial temperature of 9,000°F., enormous numbers of silver iodide crystals.

The general facts on Weathercord technology to date are as follows:

a) Weathercord is a high efficiency cloud seeding method which actually makes rain fall from suitable clouds which otherwise would not rain. (Conventional cloud seeding methods increase rain from already raining clouds.)

b) Weathercord was tested by the U. S. National Center for Atmospheric Research in 1965 (5). The tests produced showers from normally dry clouds over semi-arid California, *(Figs. 2-5)*.

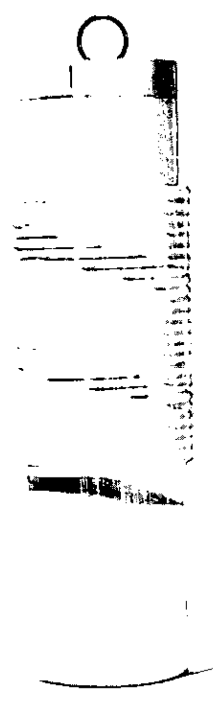

Fig. 1.
WEATHERCORD
Extremely high output silver iodide generator using explosive WEATHERCORD containing silver iodide.
The generator package is ejected from the seeding aircraft into the cloud top where it detonates, generating instantly enormous quantities of silver iodide nuclei to act as seeds for precipitation.
The package can also be detonated in a ground firing battery, allowing the silver iodide to be carried upward into the cloud base by natural vertical air currents.

Fig. 2.

Cloud before seeding with WEATHERCORD. Area, near Fresno, California. Cloud base 8000 ft. (—5° C), cloud top 12,300 ft. (—13° C). Two 25 ft. lengths of Weathercord were dropped into the clouds and detonated 1300 ft. below the top. This type of cloud is common in winter in California but does not ordinarily produce measurable rain.

Fig. 3.

Same cloud 11 minutes after seeding with WEATHERCORD. The cloud top has grown to 13,400 ft., an increase of 1,100 ft. The change in appearance of the top of the cloud is due to a change-over of the cloud from supercooled water droplets to growing ice crystals because of the seeding.

Fig. 4.

Precipitation falling under test cloud 23 minutes after WEATHERCORD drop. Precipitation was in form of snow at cloud base, melting to rain lower down.

Fig. 5.

View of the precipitation produced 27 minutes after WEATHERCORD drop. Intensity of rainfall has increased and area of precipitation has enlarged.

c) A safe reliable airborne delivery system perfected by WECOA in 1966, has been approved by the aviation authorities.

d) Weathercord was successfully used operationally in 1967 to bring heavy drenching showers to extinguish major forest fires in Labrador on two separate occasions, July 5-10th and July 22-23. The heavy rainfall was spectacular in a severe drought condition.

e) Weathercord is in current successful operational use in Iran for the Iranian Government, bringing rainfall to three watersheds in Northern Iran. Operations began in December 1967. Heavy precipitation has been received in the target areas. (See Appendix to this paper.)

What are the Results Obtained with Cloud Seeding?

It is now generally accepted by people experienced in cloud seeding work, or who have examined a sufficient amount of data, that properly designed and conducted conventional type operations produce increases of between 10% and 30%. For example:

The National Academy of Sciences found 10% increases in western United States, and 10% to 20% in the east.

The Commonwealth Scientific and Industrial Research Organization (CSIRO) reports 15% to 25% in Australian projects.

The State of Israel, after 6 years of successful experimentation, has announced a permanent program of operations every winter season for the whole of Israel north of the Negev, and expects an 18% addition to the country's rainfall as a result.

WECOA's Canadian affiliate has carried out a large seeding project in Quebec, which comprises ten years of continuous work. Exhaustive analysis of the results, using the same evaluation methods as employed by the National Academy, has established a 29.5% to 33% increase.

The above quoted projects have all employed conventional ground or airborne seeding techniques.

The introduction of the new extremely high output detonating generator called Weathercord took place in Labrador in 1967 after the Water for Peace Conference. Put into field use in July, 1967, for the Newfoundland Forest Service for massive forest fires, the device produced unprecedented increases in rainfall from weak cloud systems.

In the weak clouds characteristic of drought conditions, seeding with Weathercord detonators produced downpours sufficient to douse some 83 fires spread along a 400 mile front. The fires were under control in 5 days, and out in 10.

On a second occasion in Labrador, 23 very large fires were again extinguished in just three days of seeding with Weathercord detonators.

This device, which produces the greatest concentration of silver iodide nuclei available from any known source, has opened a new chapter in seeding technology. Because of its ability to actually make rain fall from certain clouds instead of only increasing rain from an already raining system, it allows and expansion of cloud seeding into semi-arid regions and into drought conditions for the first time.

Weathercord cloud seeding operations were begun in December, 1967, in Northern Iran for the Imperial Government of Iran, Ministry of Water and Power. The purpose is to increase precipitation on three watersheds in the Elburz Mountains for hydroelectric power generation, irrigation, and municipal water supply. Operations have resulted in very heavy precipitation occurrences, accompanied by the same visible cloud changes following Weathercord seeding as were experienced in California and in Labrador; namely rapid vertical cloud growth followed by precipitation from the seeded cloud within twenty to thirty minutes.

The Iran project will continue for three years and will include training of Iranian professional and technical personnel. (See Appendix).

Some Economic Benefits of Weather Modification

In his address to the Water for Peace Conference in May, 1967, Dr. R. B. Sen, Director General of the FAO stated that "water is of supreme importance for agricultural productivity since, of all the natural resources, it is water which provides the essential medium for all the biological phenomena on which agriculture rests." He also stressed a need for "new" fresh water.

However, the economic cost of fresh water, delivered on site, must not exceed 4¢ per thousand gallons for normal large scale agricultural operations.

Since cloud seeding with Weathercord produces water on site at a cost of not more than 1¢ per thousand gallons, it obviously provides a very attractive source of water in this critical period when an increase in the world food supply is of such importance. In Iran, for example, the cost of the water produced by cloud seeding has actually been less than 0.1¢ per thousand gallons.

It is well to recall that the benefits from increased food production spread throughout the whole economy, that better food means better health, which in turn means greater economic productivity. This in turn means greater political stability, and a major step toward the elimination or solution of some of the major problems that threaten stability and progress in the world today. Consequently, we should realize that weather modification put to use for increased water production can benefit all mankind socially, economically and politically.

Where can cloud seeding be effective? In a wider range of climates than one would at first expect. WECOA has undertaken in the past several years numerous studies of the feasibility of seeding in various parts of the world. There have been many surprises.

We have, for example, found that on the fringes of the world's greatest deserts the actual moisture content of the atmosphere is considerably higher than it is for example in eastern North America. During the rainy season in such areas the cloud moisture content, cloud temperatures, cloud structure, etc. are even more favourable for cloud seeding operations than in areas in North America, where we have successfully operated for the past many years.

To sum up, cloud seeding technology now provides a major source of "new" fresh water to water resource management engineers and administrators. Moreover, this "new" water source is in harmony with the existing ecology of the regions where it is produced. Costs of the "new" water using the high-yield Weathercord seeding process is currently less than a cent per thousand gallons.

Cloud seeding does not involve drastic ecological disruption, nor very large capital expenditures. This "new" water source makes existing irrigation and hydroelectric installations more productive; it makes marginal projects, not previously undertaken, now possible. Used where needed and when needed, weather modification benefits all of us.

APPENDIX

Since the above report was delivered results have come in from the first complete season of operations in Iran using the Weathercord technique and these data have been completely analysed (6).

The analysis of this first season (December 1967-April 1968) shows very large increases in precepitation caused by the seeding—in excess of 100%.

Standard evaluation tests used were used in the analysis—i.e. target-to-control regression correlations, and non-parametric target comparisons. The regression results are statistically significant at over the 99% level, and non-parametric tests were significant at the 95% level, that is, the seeding increase can be taken as established.

The percentage increases in precipitation caused by the seeding ranged from 177% with one set of controls to 111% with a second target-to-control relationship. A figure of 80% has been calculated as the minimum, over-all, long term average seeding increase.

It is interesting to note that shortly after these Iran results were established, Todd and Schertz of the U. S. Navy Weather Research Facility at Norfolk, Virginia, reported (7), that their computer predictions showed very large increases due to seeding were apparently possible. Some of the computer predictions of the increases which complete seeding of a cumulus cloud would produce are over 150%. Such high predicted increases, which would only have cast doubt on the suitability of the computer model a few months ago, are now verified in the Weathercord operations in Iran.

Operations for the second season in Iran began on November 1st, 1968 and are continuing. Very large precipitation increases have again been recorded in the target areas.

REFERENCES

1. First Quarterly Progress Report on Meteorological Research, General Electric Research Laboratory, Schenectady, N. Y., July 15, 1947.
2. Vonnegut, Bernard. The Nucleation of Ice Formation by Silver Iodide. J. App. Phys. 18 (7): 593-595, July, 1947.
3. Final Report of the Advisory Committee on Weather Control. (2 Vols.) U. S. Superintendent of Documents, Washington, D. C., 1958.
4. Weather and Climate Modification: Problems and Prospects. (2 Vols.) Pub. No. 1350, National Academy of Sciences—National Research Council, Washington, D. C., 1966.
5. Goyer, Guy G., L. O. Grant, and T. J. Henderson. The Laboratory and Field Evaluation of Weathercord, a high output cloud seeding device. J. App. Meteor., 5: 211-216, April, 1966.
6. Annual Report, Project IR-1: Cloud Seeding for Water Production in Iran. Weather Engineering Corporation of America, Washington, D. C., August, 1968.
7. Todd, Clement J., and D. C. Schertz. Climatology of the potential for modifying convective clouds with ice-phase seeding. Proc. First National Conference on Weather Modification, Albany, N. Y. Amer. Meteor. Soc., April. 1968, 280-286.

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Figures 2, 3, 4, 5—Courtesy NCAR/Atmospherics Inc.

MANAGING FOREST LANDS FOR WATER PROTECTION

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INTRODUCTION

This paper outlines a concept of watershed management—and how it fits into the total water resource complex—and describes in some detail that part of watershed management which deals with increasing water yields from upland watershed areas.

Conceptually, watershed management, as it is understood and applied on the 187 million acre National Forest System, is concerned with management of the ecosystem-climate, vegetation, soil, and underlying geology—for a water resource purpose. It cannot be considered as a discrete resource to be dealt with independently of other resource values—forage, timber, scenic, and recreation opportunities—available from the watershed. It is also irrevocably related to such alterations of the landscape as highways and mining operations, and is directly concerned with, and contributes to, the water resource development programs of construction agencies—such as engineering works needed to harness water yield to better serve the Nation's socioeconomic needs.

In short, watershed management is concerned with each activity in the watershed which can alter its hydrologic behavior and with the delivery of that water to the engineering works.

WATERSHED PROGRAM

For ease of understanding and to facilitate the allocation of resources to specific parts of the watershed program, the program is divided into four parts. These are watershed protection, watershed rehabilitation, water yield improvement, and water resource development.

Watershed Protection

Watershed protection is the most important single water resource purpose in management of mountainous watershed lands. Watershed protection consists of preventing soil compaction or erosion, water quality deterioration, and, most important, maintaining the hydrologic performance of the watershed under a program of resource use and development. Watershed protection also entails analyzing the consequences of timber harvesting, mining, recreation, and road construction activities, the use of insecticides and herbicides, and similarly modifying influences, and prescribing the preventive and post-operative treatment measures to be incorporated in the project design.

Basically, watershed protection aims to distribute runoff substantially in accord with preoperational conditions, and through prompt revegetation of disturbed areas to maintain infiltration capacity of the watershed.

Watershed Rehabilitation

Watershed rehabilitation is the reclamation of land damaged beyond the point of natural recovery within the time limits of economic resource, and social demands. The primary purpose is to restore hydrologic capability and to halt soil erosion. In so doing, the productive capacity of the area for timber and forage is also restored, as is

Figure 1. Single tree selection harvesting resulting in 16% increased water yield.

the aesthetic value of the land. Reestablishing a suitable cover of vegetation is an essential part of, and commonly the only treatment measure use, in the rehabilitation job. However, when deterioration has reached the stage of severe gullying, reinforcement measures such as contour terracing, gully plugs, and debris dams, are needed. In the case of eroding streambanks or lakeshores, structural measures such as revetments, jetties, and flow retarding *devices*, become a part of the rehabilitation program.

Water Yield Improvement

Water yield improvement involves the preplanning of land treatment practices—specifically, to increase the quantity or to improve the timing of runoff. In most of the National Forest System, this is accomplished by changing the pattern of vegetative cover. In the Alpine Zone, structural measures are employed—primarily, snow fencing. This part of the watershed program will be discussed in more detail later in the paper.

Water Resource Development

That part of the National Forest Watershed Program which deals with water resource development programs includes the above three elements—protection, rehabilitation, and yield improvement. In addition, it analyzes the interrelationship of the engineering works of improvement and the overall management of tributary lands. The purpose of this work is threefold:

1. It determines whether the use of the land for the project is compatible with National Forest purposes.
2. It analyses the requirements of project design and operation which, from a land and associated resource point of view, must be incorporated to optimize public values.
3. It identifies and prescribes the resource development and management programs on tributary lands which must be adjusted in furtherance of the multi-purpose objective of the development.

The end product of the analysis—the impact—is a report which supplements the engineering report of the construction agency and becomes a part of the overall project proposal. This report also serves as the basis for liaison between the construction agency and the Forest Service during the construction period, and it becomes the broad blueprint for resource development and management on tributary lands during the postconstruction life of the project.

Fitting Watershed Programs into the Total Water Resource Complex

Design of a sound watershed program must include two basic coordinating steps. The first of these is a design in harmony with other values obtainable from the watershed. This is an essential part of ecosystem management. The second is a design which harmonizes with the broad and specific social and economic demands made on the water resource.

In the first of these—"Multiple Use Coordination Requirements" is the Forest Service term—each action which can have a significant or lasting effect or other resource or development programs is reviewed from the standpoint of its contribution to the optimum product mix for the particular area being considered. A proposal to install a recreation area is typical. The recreation planner establishes the fact that public use has reached the point where additional facilities are needed. He has selected the most suitable site—from a recreation point of view—and has developed the supporting material to justify the proposal. What are some of the interrelationships which must be considered? Will the transportation system safely and conveniently accommodate this added traffic load and still permit other traffic, such as logging trucks? Will concentrated public use at this area substantially increase the risk of man-caused fires? Will the area need to be fenced to avoid unacceptable conflict between livestock and people? Are the soils such as to support the heavy trampling without danger of compaction and resultant killing of trees? Is the area suitable for waste disposal without contamination of soil and water resources? Will the increased fishing pressure deplete the natural fishing to the point where artificial stocking is required? To what extent will timber harvest practices need to be modified and what effect will this have on timber supplies for the dependent timber industry? This analytical process considers both the physical impact on other activities and the socio-economic impact on the productive capacity of the area.

It is only after this type of analysis is completed that a decision is reached on the location, the size, and the type of recreation area to be installed, is made.

The second part of the coordination process begins with the setting of water resource objectives for the specific watershed being considered for treatment. These forested watersheds usually occupy headwaters areas. They are a major source of the Nation's streamflow—about half originating in forest lands. In the Western United States about 90 percent of the usable water yield originates on the forested and alpine watersheds.

Completion of the Oroville Dam—a key feature of the California water project—will illustrate the important interrelationship of upland watershed management and downstream engineering works. Most of the two million acres of mountainous land tributary to the 3½ million acre-foot Oroville Dam is National Forest land. For more than 50 years the watershed objective on the land has been to hold peak runoff at the lowest possible level to help reduce the periodic severe flooding of valley lands and cities. This objective required priority attention to maintenance of a complete vegetation cover and to the rapid revegetation of areas disturbed by timber harvest and fire. Holding sediment and yield at a low production level was a corollary objective and the result of cover maintenance.

The completion of the Oroville Dam in combination with other water-control structures immediately changes the objective of watershed management. The California Water Plan states "The objectives of watershed management in relation to the California Water Plan would be (1) to reduce the silt deposition, or sedimentation, and (2) to increase watershed yield by improving the regimen and characteristic of runoff."

Forest Service management plans are being adjusted towards this new objective.

This situation is not unique. A large percentage of the major reservoirs in the Western U. S. have been installed, or planned for installation, within or adjacent to the Western National Forests. Many of these are hydroelectric projects in the upper mainstream tributaries. These are frequently supplemented by larger multi-purpose reservoirs installed near where the rivers leave the mountains and enter the arid valleys. These reservoirs typically have irrigation, municipal and industrial water supply, and flood control as their primary water-resource purposes. They also provide a base for an extremely important and growing recreation activity. Regardless of purpose, they usually provide sufficient regulation of annual runoff to make increased water yields a major water-resource objective of tributary land management. This is particularly true where such projects serve the arid lowlands—a situation common in the Southwestern United States. It is this situation which prompted the Forest Service, in 1961, to give major emphasis to a program leading to widespread application of measures which will increase water yields from National Forest System lands.

The Basis for A Program to Increase Water Yield

In the 11 Western States not including Hawaii and Alaska, the National Forests occupy 21 percent of the area, receive 32 percent of the precipitation, and produce more than 50 percent of the total runoff. The average annual runoff from the National Forests is 14 inches, as compared to 3 and a third inches from areas outside the forest boundaries.

These humid islands surrounded by semiarid-to-arid lowlands are in effect enormous water factories. Water factories are provided by nature—but are still awaiting the creative cooperation of the watershed scientist to set their productive capacity into motion.

The Forest Service began its research program at Wagon Wheel Gap more than 50 years ago. Here it was conclusively proven that it was possible to increase runoff through management of the vegetation.

Since then, research aimed at increasing water production from forested lands has continued and expanded to provide basic scientific know-how in each of the major hydrologic provinces of the United States.

A projection of research findings indicates that in the west 9 million acre-feet of increased water yield can be obtained from forest, brush and alpine watershed treatments for an average annual cost of 13 million dollars. This additional water has an estimated primary delivered value, weighted according to agricultural, industrial, and domestic uses, of almost one half billion dollars annually.

With a foundation of more than half a century of solid research results, a program of scientific management to increase usable water supply has gone beyond the point of speculation—it is a program which can be put in motion, and one which is presently underway at a modest level.

Converting Research Findings to Scientific Management

As a means of bridging the gap between research findings on small watersheds and operational water yield improvement programs on larger watersheds, the Forest Service uses a system of "barometer watersheds." These are basic management tools, not research facilities.

These watersheds range in size from 50 thousand to 150 thousand acres—the same size we use in planning and applying the operational program of water yield improvement. These watersheds are used to develop detailed treatment systems, parameter values, and prediction equations for use throughout the hydrologic province they represent.

When the watershed is adequately characterized hydrologically, and the present values for each parameter determined, the contribution to water yield due to the alteration or modification of each parameter can be evaluated.

The barometer watersheds are instrumented to measure all important factors affecting hydrologic behavior.

The climatic station measures absorbed radiation, wind movement, temperature, dewpoint and precipitation. A snow pillow measures the water content of snow. Reliable streamgaging stations measure total water yield. Weirs are installed to measure the distribution of runoff throughout the year, and the response to individual climatic events and to land treatment measures.

This system of barometer watersheds is also used to develop basic data for economic evaluation of proposed treatment programs.

Water Yield Improvement Technique

One of the basic watershed treatments is snow fencing in alpine areas.

Figure 2. Snow fence on continental divide, note induced drift.

The purpose of snow fencing is to induce snow to accumulate in deep drifts. This is accomplished by causing air turbulance on the leeward side of the fence, bringing the snow into a sheltered air pocket where it is deposited.

This same snow, if not deposited, would continue airborne until evaporated, or until intercepted by trees at lower elevation, where a large part is evaporated without reaching the ground. We have found both in research studies and in operational application that a mile of fence will accumulate about 50 acre-feet of additional water.

We have one alpine zone project underway. This is in the Lake Creek watershed in Colorado. This 42 thousand acre watershed on the San Isabel National Forest flows directly into the Twin Lakes reservoir—a unit of the Fryingpan-Arkansas project.

Our plan of management calls for 13 miles of snow fence.

Figure 3. Snow fence Lake Creek Barometer Watershed San Isabel N.F., Colorado

Two miles of fence have been installed. Our plan of management includes several practices.

One is induced avalanching to break off cornices in order to deposit the snow in the glacial cirques—the equivalent of reservoir storage.

The second basic watershed treatment is applicable to the commercial timber zone. Here managed water production results from two effects. One is the reduction of evapotranspiration with a corresponding increase in water yield. The second effect is induced snow accumulation—through changing the pattern of forest cover. The principle involved is the same as in snow fencing.

The reduction of evapotranspiration in the commercial timber zone comes as a corollary consequence of the timber harvesting program. The yield increase is in proportion to the amount of vegetation removed.

It is in the commercial timber zone that the best opportunities for managed harvest of the National Forest water crop are found. It is the largest water production area of the western National Forests, and some 500 thousand acres are being treated each year in watersheds tributary to the water-short western United States.

Research results from a study conducted in the high elevation commercial forests of the Sierra Nevadas in California show the importance of combining water production and timber harvest in the treatment prescription.

In this area there was an 8.6 inch increase from strip cutting as compared to a 3.4 inch increase when a tree selection method was used—a difference of almost one half acre-foot per acre treated.

Conversion of deep-rooted stands of brush, non-commercial timber and riparian vegetation is the third major treatment program the Forest Service uses to increase water production.

Most of the non-commercial timber lands are located in a rather broad belt between the alpine lands and the commercial timber zone.

Figure 4. Partially completed type conversion program—Mendocino N.F., Calif.

These areas of high precipitation and high water yield frequently support a dense stand of aspen. Research conducted in Utah and in western Colorado show that on soils deeper than 4 feet, an increased yield of from 4 to 9 inches can be expected by converting from aspen to grass.

Another large body of forest land which lends itself to improved water harvesting techniques is the millions of acres of brushland below the commercial timberlands.

The primary value of this brush cover is to stabilize the soil and to reduce the threat of flood runoff. There are, however, numerous places where type conversion to increase water yield can be done without accelerating erosion or peak runoff.

Research conducted in the brushland zone in Arizona and in Southern California showed that conversion from dense brush to grass can produce as much as one half acre-foot of water per acre treated.

One of the three brushland management projects under way is in the Big Creek watershed on the Sierra National Forest in central California. Within this watershed there are some 11 thousand acres of treatable brushland and 12 thousand acres of treatable timberland.

To date, 15 hundred acres of dense brush—only part of which is on deep soils—have been converted to a permanent grass cover with a computed 2 hundred acre-foot increase in water production.

A second brushland water harvesting project, also in California, is the Santa Ynez watershed on the Los Padres National Forest. This 138 thousand acre watershed flows into Gibraltar reservoir, a primary source of domestic water for the city of Santa Barbara, and, through the Bureau of Reclamation's Cachuma reservoir, provides irrigation water for some of the valuable agricultural lands in the Santa Ynez valley. There are 57 thousand acres of treatable brushland and 54 hundred acres of treatable phreatophyte and other riparian vegetation. When completed, these treatment programs will produce an additional 18 thousand acre-feet of water in years of average precipitation.

To date, 33 hundred acres of dense chaparral have been converted to perennial grass cover. This initial treatment program is producing 15 hundred acre-feet of water per year above natural yields.

The first of the water harvesting projects to be undertaken by the Forest Service—and the largest one—is in Arizona. This project contemplates a water production treatment program for the 4½ million acres of National Forest lands tributary to the Salt and Verde Rivers. In addition to being the first and the largest of our ongoing projects, it is the only jointly sponsored and jointly financed project under way. Here the Salt River Valley Water Users Association is both a planning and financial partner.

So far, the land treatment program has been confined to the lower elevation brushland areas. Twelve thousand acres have been treated, with a computed yield increase of 55 hundred acre-feet.

As a part of this project we recently completed a hydrologic analysis of 350 thousand acres of brushland. Average annual precipitation ranges from 16 to 25 inches within the area. This analysis—based on a reconnaissance level survey—identified 99 thousand acres of treatable land, which, if intensively managed for water production, will produce an additional 43 thousand acre-feet.

Looking to the Future

Building on the experience gained from the Salt-Verde project, the Forest Service is making a reconnaissance level survey of the 40 million acres of National Forest lands tributary to the Colorado River, the Central Valley of California, and the coastal area of southern California.

This survey, to be completed in late 1968, will give us for the first time some idea of what a large scale water harvesting program might produce. The survey is designed to:

1. Identify and map areas suitable for water production management and to develop an estimate of the potential yield increase.
2. Identify and map areas of accelerated erosion.
3. Provide a basis for establishing a priority of treatment programs.

Based on the detailed hydrologic analysis of the 12 barometer watersheds associated with the survey area, it appears that water yield can be increased by 4 million acre-feet annually. This first approximation will be refined through comprehensive hydrologic surveys and prescriptions of individual watersheds. This phase of the job will be carried forward on priority areas identified in the reconnaissance survey.

Figure 5. Designed small openings to trap and shade snow.

ESTABLISHING PROGRAM PRIORITIES

Under current and foreseeable financial restraints, selection of watersheds for priority consideration becomes a vital step. In general terms, priority should be given to the arid West. Also in general terms, as has been done in the case of the Salt-Verde project, high priority should be given to areas of current over-utilization of available supplies where —unless additional water is made available—a major adjustment or a decline in the economy is the inevitable consequence. Such considerations as these may call for priority attention to watersheds where the acre-foot yield per dollar of investment is not the most favorable.

Another important factor in determining priorities is the presence of an installed delivery system— the structural works of improvement and the processing plant which converts raw water to a useful commodity.

These are the hydroelectric plants, the reclamation projects, the municipal water supply plants and the irrigation projects.

It is apparent that there are many priority places where a water harvesting program would enhance the output values of installed facilities and the people they serve.

The fact that these structural works were installed means that water supply needs have been established, justified and amortized on the basis of natural runoff. Under these circumstances, the incremental value of additional water through a designed water harvesting program reaches unusually high levels—an important factor in establishing priorities.

CONCLUSION

The National Forest water production program can be accelerated, and will be, when public interest in this overlooked field of water resource development expresses itself, in a manner which will cause the financial resources needed for the job to be provided.

DESALINIZATION PROJECT IN THE GUANTANAMO NAVAL BASE, CUBA

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April, 1968

Background

The lecture is going to be more on the practical side than the theoretical side of desalinization, and is based on our experience in building a Desalinization Plant and a Power Plant for the Navy at Guantanamo Base in 1964 and 1965.

First, I would like to give you the historical background of the Guantanamo Naval Base Desalinization Plant erection.

On February 6, 1964, the Cuban Government announced that water to Guantanamo Base in Cuba would be cut off unless 36 Cuban fishermen held by the United States were released immediately. The men were seized fishing in Florida waters. I think everybody knows that we didn't retreat, and the water was cut off at 1:58 P.M. on February 6, 1964.

At that time the base had fifteen million gallons of water on hand. The maximum water consumption was set at 900,000 gallons per day for the following months.

To insure that quantity of water, the Navy ordered first barges, then ships, to bring water from Jamaica and Florida to the base. During the construction of the plant, the tankers brought more than 293 million gallons of Florida water to Guantanamo.

On February 10, 1964 a survey team from Washington arrived at the base to survey the personnel situation and the possibility of converting sea water to fresh water by installation of a local Desalinization Plant. The Navy had a Distilling Ship on the base, which was reactivated and could produce 120,000 gallons per day of fresh water; but was put on stand-by status because the whole base was depending on Florida water only shipped by fresh water tankers.

On February 15, 1964, the Navy gave a contract to Westinghouse Corp. to build a 2.25 million gallon per day capacity Desalinization Plant on Guantanamo Base. The operating Desalinization Plant at San Diego, California, capacity 700,000 gallons per day, was ordered dismantled, transported and to be reassembled at Guantanamo.

The Desalinization Plant, First Phase, was scheduled to operate in July, 1964 and this was planned by operating the assembled Point Loma Plant from San Diego, California.

Westinghouse, as prime contractor, negotiated a subcontract with Burns and Roe Construction Company for designing and building the plant. Westinghouse supplied all the desalinization equipment, boilers, generators and electrical materials.

Burns and Roe's first personnel showed up at the base on March 20, 1964. Construction work started March 28, 1964 from sketches and drawings which arrived twice a week by air. The working crew, from the States, was never over seventy men.

Methods of Desalinization

Efforts to desalt water go back many centuries. In 49 B. C., Julius Ceasar obtained fresh water for his troops during the siege of Alexandria. He used primitive solar evaporators to separate salt from sea water but the technology practically stood still thereafter.

There is no great trick involved in turning salt water into fresh water. It can be done by simply boiling the water in a tea kettle and catching the steam in a cool container where it condenses into fresh water.

The problem is to do the job economically. In the beginning, it cost about \$5.00 to produce 1,000 gallons of fresh water from sea water—more than fifteen times the average cost of 30¢ per 1,000 gallons for conventional supply in the United States. The different methods of desalinization are as follows:

1. *Solar Distillation*—Very uneconomical. It takes an acre size still to produce 6,500 gallons per day in a very sunny climate. The cost for this distillation runs from \$1.50 to \$4.00 per 1,000 gallons.
2. *Freezing System*—The sea water is cooled below its freezing point. The ice crystals produced are pure water in solid form but they are tiny and irregularly shaped. It is very expensive to free the trapped salt from the crystals by washing with large amounts of fresh water.

3. *Electrodialysis Technique*—The electric current separates salt from water and the salt takes the form of positively and negatively charged particles. These particles are forced through a special membrane by the current, leaving pure water behind. The problem is that it consumes electrical energy roughly in proportion to the salt that must be removed.
4. *Multistage Flash Distillation*—The heated salt water evaporates in steam heated chambers under reduced pressure. The steam goes through cooling tubes and condenses to fresh water. This is the most reliable and satisfactory process for producing fresh water from sea water. This type of Desalinization Plant was built at Guantanamo (Gitmo) Base.

There is an article in the New York Times March 10, 1968 issue concerning the study of nuclear powered agro-industrial complexes. In this complex, nuclear reactors generate electricity; cheap power desalts water for irrigation, makes fertilizer, makes possible year round harvests, etc.

Another interesting article is in the April 5, 1968 issue of *Time* Magazine. Alcoa Aluminum Company announced development of an aluminum based desalinization system. No details are available, but they want to utilize the waste hot water that is discharged by chemical processing plants.

The best solution seems to be a "Dual Purpose Plant", a huge "water kilowatt factory" where we can distribute capital investment and daily operating costs over both products. In a power plant, heat makes steam to drive a turbine which produces current, but only 35 per cent of heat is actually converted into power and the extra heat can be used to distill sea water. Low operating and maintenance costs would suggest using nuclear power for electrical generation and multistage flash distillation for fresh water production.

The Guantanamo Plant

At Guantanamo we built a power house with three boilers, two 7,500 KW steam turbo-generators and a desalinization plant with three 750,000 gallons per day flash type evaporator units. Power required by the Desalinization Plant amounts to 3,500 KW; the remaining 11,500 KW is available for the base. Two boilers can supply all of the steam needed; the third is a stand-by.

The first unit from Point Loma, San Diego, California, was a 36-stage plant designed for 1,000,000 gallons per day, but at Guantanamo was down-rated to 750,000 gallons per day. During installation all deteriorated equipment was repaired or replaced. Second and third units were new 15 stage flash evaporators, with the same design characteristics as those in Point Loma unit. Fresh water produced contains less than 50 parts per million total dissolved solids. The fresh water flow passes through a lime bed and it is pumped into storage tanks.

Each unit consists of a brine heater, a multiple stage flash evaporator, pumps and associated equipment.

All three units depend on one common evaporator acid feed system, one common circulating sea water system, and one product water treatment system. All these of course, depend upon the common power plant for steam, fresh water and electricity.

Information about the boilers, evaporators and other equipment follows:

1. *Boilers*—The plant has three boilers each, each with 120,000 lb/hr steam generating capacity at 630 PSIG and 830° F. They are oil fired package type boilers designed for operation on Navy specification fuel oil, supplied by Wickes Boiler Company.

   The boilers take feedwater from the feed water header and convert it into steam. The water lost from the steam condensate cycle, as a result of relief valve operation, and from continuous blowdown is made up with chemically treated fresh water via a chemical feed system. Water for total makeup is supplied by the unlimited evaporator distillate which comes up as steam from the lime bed.

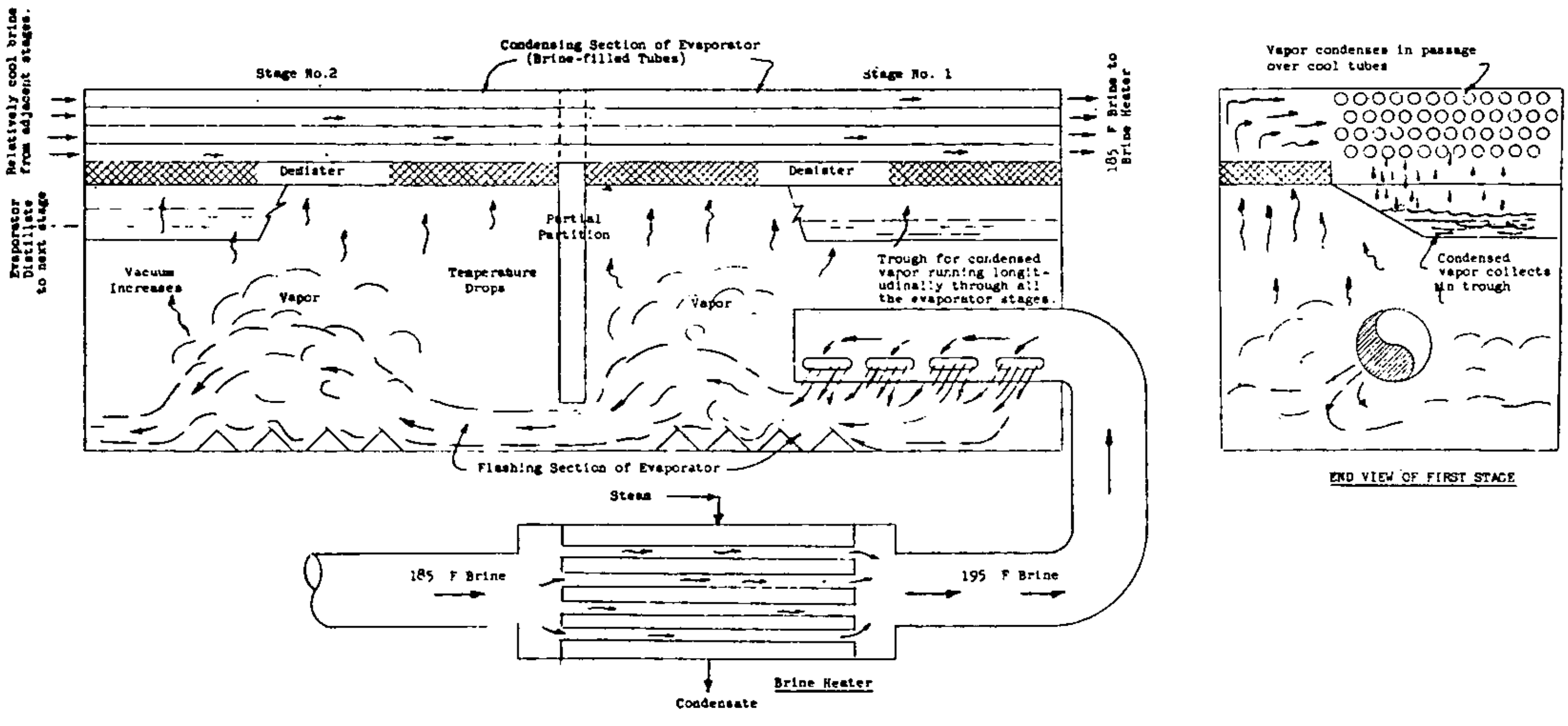
2. *Power Generating Facilities*—Two each steam turbine driven generators. The turbines take steam from the main steam supply header. Expansion of the steam in the turbine drives the turbine and the generator coupled to it. Each turbine has three openings in the shell for steam extraction. The extracted steam supplies the heat for the brine heater.

3. *Condenser System*—Unextracted steam passes to the condenser and releases the heat to the condensers circulating sea water inside the cooling system tubes. Condensed steam flows into the hot well and the feed water system. Air and gas are removed from the condenser by air ejectors.

   The vacuum created in the air ejector is greater than that in the condenser, and this causes the air and non-condensable gases to flow to the ejectors, where they are entrained in the air ejector steam. The air and steam

FIG. 1

INTERNAL SCHEMATIC OF TWO ADJACENT STAGES
OF A GITMO UNIT EVAPORATOR



LONGITUDINAL SECTION THROUGH FIRST & SECOND STAGES

END VIEW OF FIRST STAGE

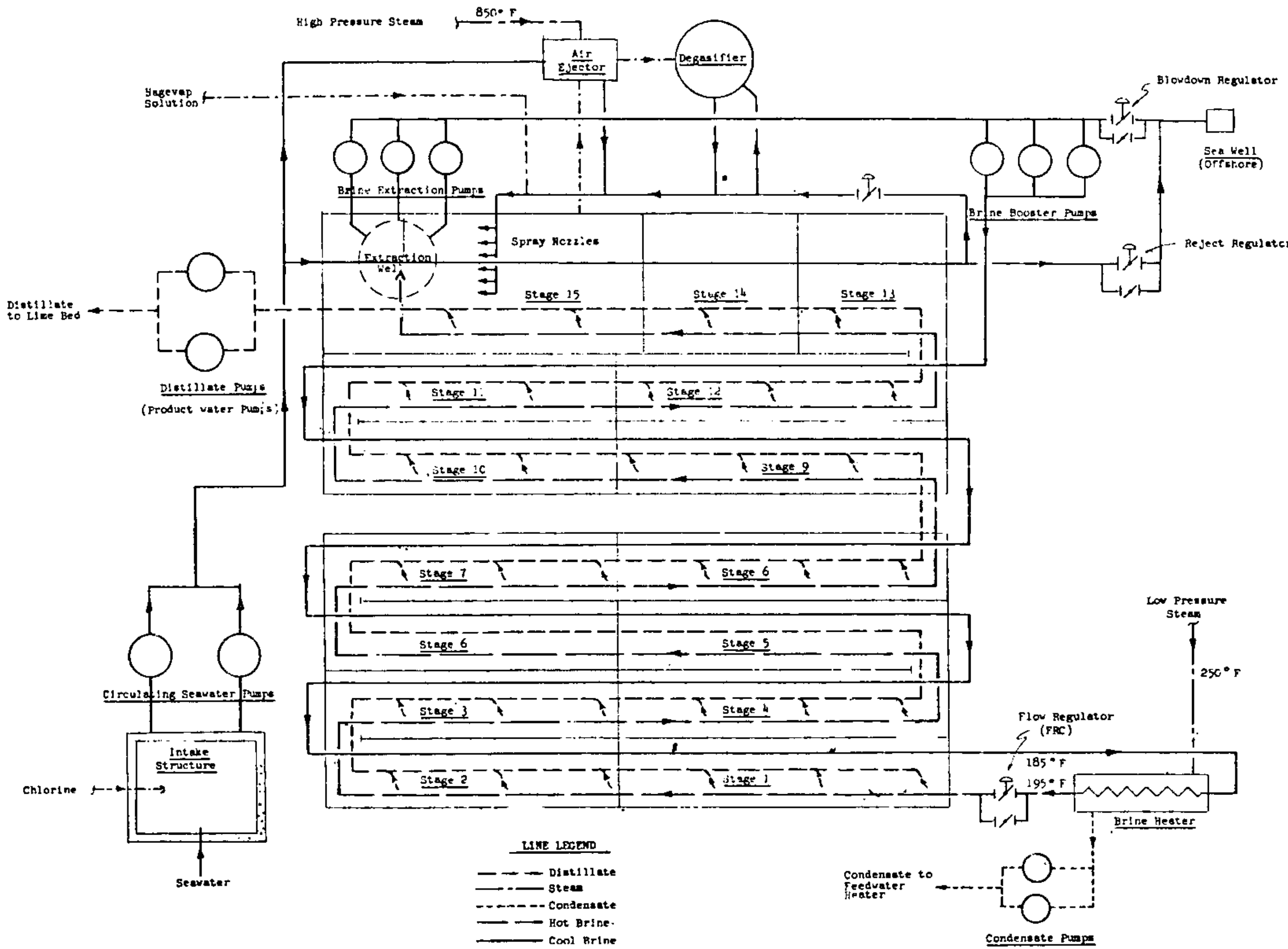


FIG. 2

ARRANGEMENT OF GITMO UNIT EVAPORATOR

mixture enters the air ejector condensers, where the steam is condensed and returned to the condensate system while the air is vented to the atmosphere.

The condensate system collects all steam line drains and the boiler drain to the condensate surge tank. From the condensate surge tank the condensate booster pump, pumps to the boiler with short term imbalances between the total condensate supply and the feed water demand.

4. *Evaporators*—Each Guantanamo unit has one 15 stage flash evaporator. Each stage has one flashing and one condensing chamber. In the series-connected flashing chambers, hot flowing brine evaporates a little at a time, stage by stage, at successively lower sub-atmospheric pressures.

   Vacuum is maintained by use of steam jet air ejectors. In the condensing part of the stages, the flashed steam condenses to fresh water as it passes over.

5. *Brine Heater*—The brine which served as cooling water in the evaporators reaches the brine heater at approximately 185° F. and the outgoing brine temperature is 195° F. In this way the condensing parts of the evaporator serve as both condenser and heater, with the brine being pumped to the brine heater serving as cooling water to condense the flashed steam. The heat for the brine heater comes from the turbine as extracted steam. To prevent foaming of the brine and accumulation of scale in the brine heater, the sea water make-up is treated by Hagevap solution (Polyphosphatic). Formed sludge in the brine heater must be cleaned periodically by pumping sulphuric acid into the makeup water until the sludge dissolves in the tubes through which brine is being pumped to the brine heater.

Interacting Cycles

The equipment description above indicates that there are three interacting liquid-vapor cycles, which transfer heat to achieve various end results. The basic processes involved concern a steam loop, a seawater brine loop, and the end product or fresh water loop. The steam loop, although the least ingenious, is probably the most important cycle. The steam is used to:

1. Drive turbines and generate electricity.
2. Heat the brine to a temperature near the flash point.
3. Provide the high-pressure steam jets to create a vacuum in the flash evaporators.
4. Provide the means to return the concentrated brine to the ocean.
5. Provide water for the feed water heater for the steam boilers.

Steam is extracted from the turbines at three points. The high pressure steam is used for the air ejectors; while the low pressure steam, at about 250° F. is used to raise the incoming sea water, which has been previously heated in the condensers from about 185° F. to 195° F. in the brine heater. Thus, in the steam cycle the steam is used at lower and lower temperatures and lower and lower pressures to do the necessary work.

The sea water cycle is nearly the opposite from the steam cycle in that the cold sea water is first used to condense the flashing brine (the fresh water being manufactured). At the end of this process the sea water is further heated in the brine heaters and then as it passes through each stage of the evaporator, due to the vacuum being maintained by the air injector high-pressure steam jets, it flashes into steam or in part remains as a steadily concentrating brine which finally reaches the extraction pumps, which then recirculate the brine through condensers to the heater, where the cycle is repeated. The sludge from the concentrated brine is removed through brine extraction pumps from the extraction well. The introduction of the Hagevap solution is essential for the efficiency of this process.

After use in the cooling process to condense the water vapors, the sea water is partially discharged and partially flows to the flashing section as make-up water. The seawater from the flashing sections of the evaporator is recirculated through the brine heater and back to the flashing section for boiling and making product water. Since only the water is flashed in the flash chamber, the salt is left behind, increasing the concentration in the remaining brine. Therefore, to maintain the brine concentration at the desired level, part of the concentrated brine is removed from the last stage and discharged to the Discharge System, and replaced by make-up water from the cooling sea water.

The potable water, or third loop, starts with the flashing vapor in the various stages of the evaporator units. This is cooled by the incoming sea water brine in the condenser units and eventually is pumped into a distillate lime bed for discharge into the fresh water system. Thus, in the manufacture of fresh water the cooled sea water is used as a condensate, which is absorbing heat along the way until it approaches the temperature of the hot brine, from which the fresh water vapor will flash. Low pressure steam provides the increment of extra heat, while the high pressure steam provides the necessary reduction in pressure to cause flashing.

Fresh Water Available to Gitmo Base

Prior to 1949, when the Cuban government nationalized the water facilities, the Navy had been piping water four miles from the Yateras River, to the base to supply 10,500 military and civilian personnel—about 2 million gallons of water per day. In 1949, Navy began contingency planning for a self-contained water supply. The four years before the Cubans cut off the base supply in 1964 were utilized to install emergency water supply tanks to increase the on-site storage. They also constructed the necessary piping so that the base could be supplied with water from barges and tankers. A water ship, with storage capacity of 4.5 million gallons and a distillation capacity of 120,000 gallons per day was also moored at the base.

In order to refute the charges that the Navy was illegally tapping the water piped from the river, Rear Admiral John D. Durkley cut the pipe and put the base on a 900,000 gallons per day water rate. At the same time, he was authorized by President Johnson to go ahead with construction of the facility described herein. The completion of the first unit in about 6 months made the base self-sufficient at the rationed rate. With the completion of the third water unit, Guantanamo Base can more than supply the pre-1964 average needs of the base population by the on site desalinization of sea water. In addition to solving the water problem, the plant has improved the electrical generating capacity of the base and has eliminated the need for a number of smaller diesel generating stations.

Conclusions

The construction of the combined desalinization and electrical generating plant demonstrates not only that we now have the technical capability to economically manufacture fresh water from sea water with a by-product of a significant quantities of electrical power, but also that it is possible to combine standard units of equipment such as ship boilers and turbines with flash evaporators and condensers from a desalination plant into an efficiently operation facility. The construction of the plant was an unusual engineering achievement with respect to the speed with which the facility was constructed and the small number of men required to assemble and install the component parts. The Guantanamo emergency also demonstrates that continued research and development in the desalinization of water to make the process more efficient and to bring down the cost of the fresh water is as important as contingency planning.

WATER RESOURCES PLANNING AND HYDROLOGIC RISK

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May, 1968

INTRODUCTION

Uncertainty and Risk

The rational use of water resources requires some estimate of the economic, social, technological, and hydrological conditions that will exist in the future. Managing our water resources to satisfy only our immediate demands might indeed maximize the immediate utility derived from these resources; but the present value of the total derived utility over the long run may be considerably reduced.[1] Yet estimating future events is at best a hazardous occupation. This is especially true with respect to uncertain and risky events.

An uncertain event is one in which even the probabilities of various possible outcomes are unknown. Who can accurately predict what municipal or industrial wastewater treatment facilities will cost 20 to 50 years from now, and what new unit processes will be incorporated into these treatment facilities for removing wastes that are not yet even produced? Who knows what interest rates will be considered relevant a decade from now, or what the Federal policy will be with regard to the evaluation and quantification of stream quality benefits associated with various river basin management policies? Uncertainties such as these make any dynamic planning task most difficult and one that requires a constant updating of previous projections and forecasts.[2]

In spite of the difficulties associated with planning for an uncertain future, planning must be done if management policies are ever to be improved. Presented in this report are some deterministic and stochastic models that have been structured for defining and selecting alternative policies for managing (designing and operating) river basin systems. The stochastic models explicitly consider hydrologic risk. Their solutions can be compared to the solutions obtained from deterministic models to determine the influence of risk on water resources planning.

Unlike uncertain events, risky events are those in which the probabilities of various possible outcomes are known or at least can be estimated from past outcomes. Hydrologic events are risky events. One cannot predict tomorrow's rainfall or streamflow with certainty but on the basis of past performance one can assign probabilities to the various possible amounts of rainfall or streamflow that might occur.

When the probability of any particular event is dependent on other events, the sequence of events is said to be correlated. Conditional probability distributions are used to describe both serial and cross correlated events. If the probability of the current streamflow is dependent on past streamflows, the streamflows are serially correlated, i.e., correlated in time. On the other hand if the probability of the current streamflow is dependent on the current streamflow in adjacent watersheds, the streamflows are cross correlated, i.e. correlated in distance. Usually, events such as streamflows are to some extent both cross and serially correlated. The models that are presented in this report incorporate discrete probability distribution that describe the extent of the risk and correlation that are present in any given river basin. Discrete probabilities are used to approximate the continuous probability distributions of streamflow, reservoir volumes, and other hydrologic events, and discrete intervals of time are used to describe the continuous variable time. These discrete approximations are made in order to simplify the mathematical manipulations required for model solution.

Discretizing continuous distributions and variables is not the only compromise made to preserve mathematical tractability. Many other approximations are necessary when modeling the essence and not the overwhelming detail of an actual water resource system. Each of the assumptions and approximations will be made explicit during the development of each model.

[1] Unless of course our social discount rate increases to the point where the present value of future benefits is essentially zero, a situation not likely to happen in spite of the current upward trend of both private and public interest rates. For an elementary discussion of social discount rates see Arrow, K. J., "Criteria for Social Investment," *Water Resources Research*, Vol. 1, No. 1, pp. 1-8, 1965.

[2] For some insight into methods of planning for an uncertain future see Marglin, S. A., *Approaches to Dynamic Investment Planning*, North Holland, 1963 and Raiffa, H., *Decision Analysis*, Addison Westley, 1968.

Preliminary Screening

It is important to realize that these models are merely preliminary planning tools. We repeat: *these models are developed solely for the preliminary screening of alternatives.* They are to be used before the costly collection of additional information, needed for a more sophisticated and detailed analysis, essentially precludes from consideration many of the possible alternatives. These models can be used to help narrow down the number of alternatives to be further analyzed by digital simulation techniques.

The preliminary screening models to be presented are static models. They are models for suggesting what the resource management policies should be, given some estimate of future economic, social and technological conditions. They do not define the dynamic aspects of achieving these future policies. For example, the solution of a model of a particular river basin may specify the location and amount of additional reservoir capacity needed some given number of years from now, but the solution will not indicate when this increased capacity should be installed. But by asking the question "What is the best system in the next 5, 10, 15 etc. years?" one may get some inkling of when system changes should be made. The static approach is one of taking snapshots of future periods, looking at what the system ought to be in these periods, without precisely determining the best way to establish such a system.

This snapshot approach for defining management policies for various future periods sidesteps some very difficult and important issues. In spite of this limitation the models presented herein have been applied with success to some actual water resource problems and have proved to be useful aids for the preliminary formulation of water resource management policies. Yet they can undoubtedly be improved. It is hoped that some of the ideas presented in this report will inspire others to develop and apply even better methodologies for water resources planning.

DISCRETE PROBABILITIES AND MARKOV CHAINS

Conditional Probabilities and Correlation

Before discussing any of the optimization models for defining management policies, it might be useful to review some of the probability theory that forms the core of many of the stochastic models. Those readers already familar with terms such as discrete, joint, marginal, conditional and transition probabilities, and with the fundamentals of Markov processes, will find this section extremely elementary. However, if the concepts outlined in this section are well understood, one should have little difficulty in extending and applying them to the problems that follow.[1]

The future states or outcome of any stochastic process, e.g. the annual flow in a particular stream, cannot be predicted with certainty. However based on past performance one may be able to estimate the probability associated with any particular outcome. Such is the case with streamflows. Consider a stream in which the annual flow varies from 60 to 100 thousand acre feet (KAF). Dividing this annual flow into four discrete intervals of 10 KAF each it is possible to determine the percent of time the past annual flows were within each of the four intervals. Denote p_i as the percent of time the annual flow was in the interval i where $i=1,2,3,4$. Then

$$p_i = \frac{\text{No. of years flow was within interval i}}{\text{Total number of years of record}}$$

The probability of having a streamflow within interval i is precisely p_i, *if* one assumes that the probability distribution of annual streamflows is not significantly changing over time.[2] Assume that the actual probabilities associated with the annual flows in our hypothetical stream are as shown in the histogram in Figure 1. Thus the unconditional probability of an annual streamflow from 60 to 70 KAF is 0.15, from 70 to 80 is 0.31, and so on. These streamflow intervals can be thought of as discrete states of the streamflow system. If all possible states are defined by these four intervals then,

$$\sum_{i=1}^{4} p_i = 1$$

A state probability vector is nothing but a vector, in this case, of streamflow probabilities.

$$\begin{aligned} p &= (p_1 \quad p_2 \quad p_3 \quad p_4) \\ &= (0.15 \quad 0.31 \quad 0.32 \quad 0.22) \end{aligned}$$

[1] There are statistical methods for testing this assumption, and there is recent evidence to suggest that hydrologic distributions are in fact not stationary over time. (See Mandelbrot, B. B. and Wallis, J. R., "Some Long-Run Properties of Geophysical Records," *Water Resources Research,* Vol. 6, 1969 (forthcoming)). However it has not been established that there exists any significance in this possible nonstationarity with respect to river basin planning of up to 50 years into the future. In this report we assume as have others that past records do provide a reasonable basis for defining hydrologic probability distributions.

[2] A more thorough treatment of probability theory and its applications may be found in numerous texts in this area, including those of Parzen, E., *Modern Probability Theory and Its Applications,* John Wiley & Sons, 1960, and Feller, W., *An Introduction to Probability Theory and Its Applications,* John Wiley & Sons, Vol. 1, 1957 and Vol. II, 1966. An excellent discussion of discrete Markov processes is also found in Kemeny, J. G. and Snell, J. L., *Finite Markov Chains,* D. Van Nostrand Co., 1960.

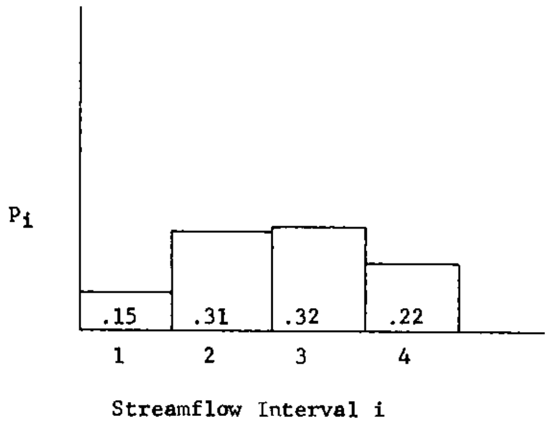

Figure 1. Unconditional Probabilities of Annual Streamflows

Serial correlation of annual streamflows may exist and as a measure of this correlation discrete conditional probability distributions of various lags may be defined. Using our hypothetical stream, define P_{ij} as the conditional probability of the current annual streamflow being within interval j given that last year's streamflow was within interval i.

$$P_{ij} = \frac{\text{Number of streamflows within interval } j \text{ following those within interval } i}{\text{Total number of streamflows within interval } i}$$

Then[1] $\sum_{j=1}^{4} P_{ij} = 1$ for all intervals i

$$\sum_{j} P_{ij} = 1 \quad \forall i$$

Figure 2 illustrates the discrete distributions that might have been derived from the past flow records of our hypothetical stream.

One could go on and define conditional probability distributions based on two, three, and more past annual streamflows. We will assume here that the conditional probability distributions based on the previous two annual flows are essentially the same as the distributions based on only the immediate past annual flow, and these distributions are as shown in Figure 2. Thus the lag-one con-

[1] Some symbolism can be defined to simplify further notation. Summing over all subscripts i will be written $\sum_{i}$ with the upper limit of i understood to be its maximum value. The symbol $\forall$i will be represent for all subscripts i. Thus the last equation can be expressed:

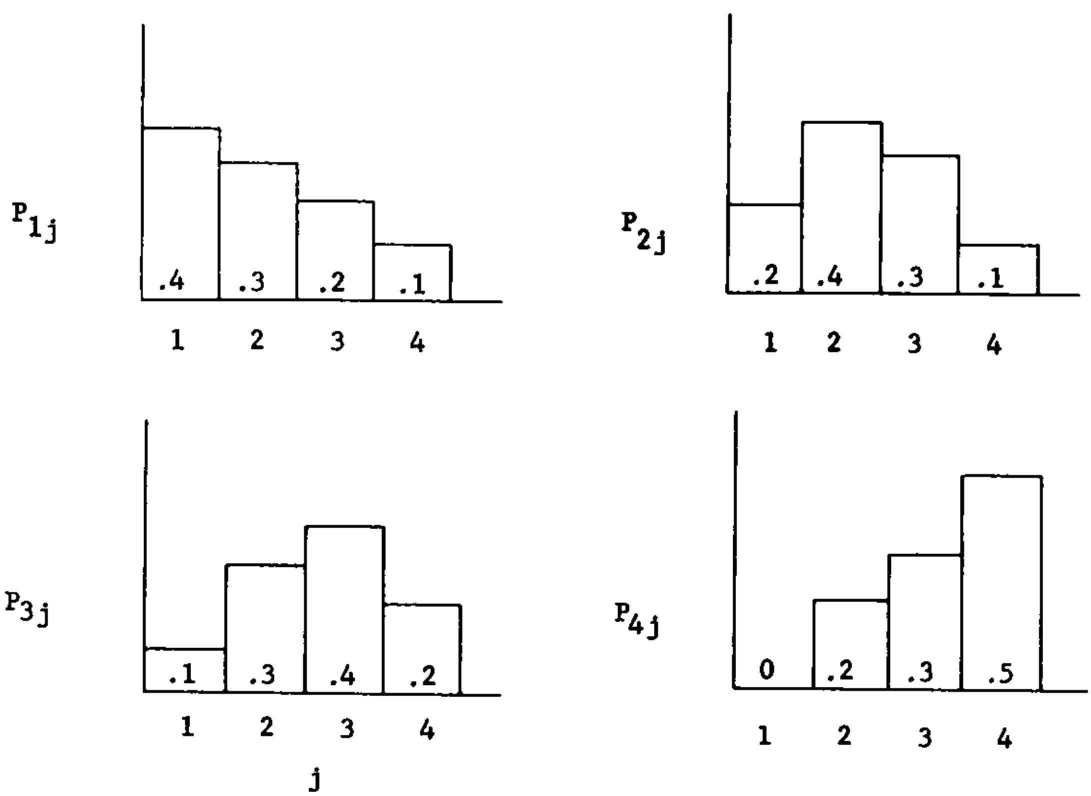

Figure 2. Conditional Probabilities of Annual Streamflows

ditional probabilities, being significantly different from each other and from the unconditional probabilities of Figure 1, are a measure of the serial correlation or the dependency of each annual flow on the immediate past year's flow. In this case the correlation is positive, i.e. high flows are more likely to follow high flows and low flows are more likely to follow low flows.

The information given in the histograms of Figure 2 can be expressed by a matrix of conditional probabilities as shown in Figure 3. Each row of the matrix is the conditional probability vector corresponding to an initial streamflow interval i. Each element P_{ij} in the matrix is the probability of a transition from streamflow i in one year to streamflow j the next year. These conditional probabilities are called transition probabilities. If the transition from one annual streamflow to the next can be described by a matrix of transition probabilities such as that shown in Figure 3, then this stochastic streamflow process is called a discrete Markov process. The matrix in Figure 3 is called a first order Markov chain.

		Streamflow state j year y+1			
		1	2	3	4
	1	0.4	0.3	0.2	0.1
Streamflow state i	2	0.2	0.4	0.3	0.1
year y	3	0.1	0.3	0.4	0.2
	4	0.0	0.2	0.3	0.5

Figure 3. Matrix of Streamflow Transition Probabilities

Markov Chains

Stochastic processes that can be described by first order (lag one) Markov chains are those in which the probability of a future state is dependent only on the present state and not on any of the past states. The Markov chain in Figure 3 is a stochastic matrix because the elements in each of its rows sum to one. Markov chains have many other properties. Perhaps among the most important properties of those chains describing such ergodic processes as streamflows is that there exists what is called a stationery or steady-state probability distribution which is independent of the initial state or streamflow. What this means is that although next year's streamflow probabilities are determined by this year's streamflow, the distribution of streamflows expected—say, 10 years from now—is, quite reasonably, independent of this year's flow. This property can be illustrated using the Markov chain shown in Figure 3.

Assume that in year y the streamflow measured 74 KAF. Since this is in the interval between 70 and 80 KAF, we can say that the state of the "streamflow" system in that year was 2. The initial probability vector $p^{(y)}$ for year y is (0 1 0 0). Knowing $p^{(y)}$, the probabilities $p_j^{(y+1)}$ of each of the four possible streamflow states j that could occur in year y+1 can be determined. From Figure 2 the probability vector $p^{(y+1)}$ is obviously (.2 .4 .3 .1). This vector can be calculated by realizing that in year y+1 the probability of being in state j is equal to the sum of the probabilities of being in each state i in year y times the probability of a transition from state i in year y to state j in year y+1. For example, the probability of being in state 1 in year y+1 equals the probability of a being in state 1 in year y times the probability of a transition from 1 to 1 (0.0 x 0.4), plus the probability of being in state 2 times the probability of a transition from 2 to 1 (1.0 x 0.2), plus the probability of being in state 3 times the probability of a transition from 3 to 1 (0.0 x 0.1), plus the probability of being in state 4 times the probability of a transition from 4 to 1 (0.0 x 0.0). Thus the probability of being in state 1 in year y+1 equals 0.2. Similar operations can be performed to compute the probabilities of being in states 2, 3 and 4 in year y+1. Letting $p_j^{(y)}$ represent the unconditional probability of being in state i in year y, this operation can be written:

$$p_j^{(y+1)} = p_1^{(y)}P_{1j} + p_2^{(y)}P_{2j} + p_3^{(y)}P_{3j} + p_4^{(y)}P_{4j} = \sum_i p_j^{(y)}P_{ij} \quad \forall j$$

where P_{ij} is again the probability of a transition from state i to state j and is here assumed independent of year y. Denoting the matrix of P_{ij}'s by P and the probability vector for year y by $p^{(y)}$ the above set of equations can be written:

$$p^{(y+1)} = p^{(y)} \quad P$$

To calculate the probabilities of each streamflow state in year y+2 we simply solve the following equations using $p^{(y+1)}$.

$$p^{(y+2)} = p^{(y+1)} \quad P$$

Continuing in this manner it is possible to compute the probabilities of each possible streamflow state for years y+1, y+2, y+3, . . . and so into the future. The probability vectors for the first y+10 years are listed in Table 1. Notice that, as y increases, the probabilities tend toward a limiting value, namely that of y+9 or y+10. These are the unconditional steady state probabilities of having any one of the possible streamflow states. These are the same probabilities that appear in the histogram of Figure 1.

Year y	Streamflow State Probabilities $p_1^{(y)}$	$p_2^{(y)}$	$p_3^{(y)}$	$p_4^{(y)}$
y	0.000	1.000	0.000	0.000
y+1	0.200	0.400	0.300	0.100
y+2	0.190	0.330	0.310	0.170
y+3	0.173	0.316	0.312	0.199
y+4	0.164	0.311	0.314	0.211
y+5	0.159	0.310	0.315	0.216
y+6	0.157	0.309	0.316	0.218
y+7	0.156	0.309	0.316	0.219
y+8	0.154	0.309	0.317	0.220
y+9	0.15	0.31	0.32	0.22
y+10	0.15	0.31	0.32	0.22

Table 1. Successive Streamflow Probability Vectors

Define π_i as the steady state probability of state i and π as the steady state probability vector containing elements π_i. Then it should be clear from Table 1 that

$$\pi_j = \sum_i \pi_i P_{ij} \qquad \forall j$$

or in vector notation

$$\pi = \pi P$$

which in our example equals

$$(.15 \quad .31 \quad .32 \quad .22)$$

The steady state probabilities of any discrete Markov process can be derived simply by solving the above set of simultaneous equations for all but one of the states j together with the equation

$$\sum_i \pi_i = 1$$

Most likely the annual streamflow correlation will not be as pronounced as that indicated in this example. However, monthly, weekly and especially daily streamflows will generally show the increased serial correlation that would be expected. Assuming that the unconditional steady state probability distributions for, say, monthly streamflows are not changing from one year to the next, a Markov chain could be defined for each month's streamflow. Since there are 12 months in a year there would be 12 Markov chains, the elements of which could be denoted as $P_{ij}^{(t)}$, the probability of a streamflow state j in month t+1 given a streamflow state i in month t. Each month's stationary probability vector having elements $\pi_i^{(t)}$ can be found by essentially the same process as previously described, only now all 12 Markov chains are used to define simultaneously all 12 steady state probability vectors

$$\pi_j^{(t+1)} = \sum_i \pi_i^{(t)} P_{ij}^{(t)} \qquad \forall j,t$$

and

$$\sum_i \pi_i = 1 \qquad \forall t$$

THE INFLUENCE OF RISK: AN EXAMPLE

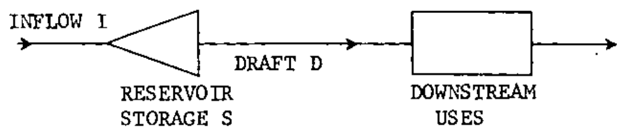

Figure 4. Hypothetical River Basin System

The Problem and a Deterministic Model

The effect of hydrologic risk on project design and operating criteria can be illustrated by means of a numerical example of a simple multiple use problem. The problem is illustrated in Figure 4. Uncontrolled streamflow enters the reservoir, and this flow plus the initial reservoir storage volume is available for release to downstream uses. The problem is to determine reservoir capacity, reservoir storage and draft targets, and the reservoir operating policy that maximizes the expected net benefits. Targets are the planned or expected quantities of water allocated to each use. Deviations from these targets may result in losses. For example, consider reservoir recreation facilities, e.g. docks, beaches, campgrounds and sanitary facilities. These facilities may be constructed at a particular elevation based on a certain planned or target reservoir storage level. The actual levels may vary from this planned or target level, depending on the variability of the reservoir inflow and release. If the variation in storage is too great either some facilities may be inundated or mud may replace the water that normally borders the docks or beaches. This results in losses. To be determined are those targets and operating policies which minimize these losses. In this example we assume that the reservoir has not been built and therefore the reservoir capacity, the reservoir storage target and the release or draft target are all unknown variables.

Functions that quantify the benefits and losses associated with both the reservoir storage and draft uses are shown in Figure 5. The loss functions in this example are assumed to remain the same within a reasonable range of possible targets. This assumption is not necessary but is made here to keep the modeling fairly simple. Whatever the target value, the short-run loss function is tangent to the long-run benefit function at the target and the slopes of the loss function are as indicated by the numbers adjacent to the linear segments of that loss function. Also shown in Figure 5 is the reservoir cost function. All benefits, losses, and costs are in thousands of dollars.

In this example we assume that there are only two, not necessarily equal, seasons per year, winter (t=1) and summer (t=2), and that benefits from reservoir storage are derived only in the summer season. The total draft target, T_D is assumed to be divided into two portions, the winter portion, $0.3T_D$, and the summer portion, $0.7T_D$.

Figure 6 specifies the four assumed discrete inflows each season and their transition probabilities. Using methods described in the previous section, the steady state unconditional probability for each inflow in each season is 0.25. The sum of these probabilities times the four discrete inflows each season defines the mean winter inflow of 15 KAF and the mean summer inflow of 5 KAF. These mean inflows can be used in a deterministic linear programming model for finding the reservoir capacity and the storage and draft targets that maximize the total annual benefits less the annual costs of reservoir construction and operation and the losses from deviations of storages and releases from their respective targets. It is assumed that the reader is familiar with the fundamentals of

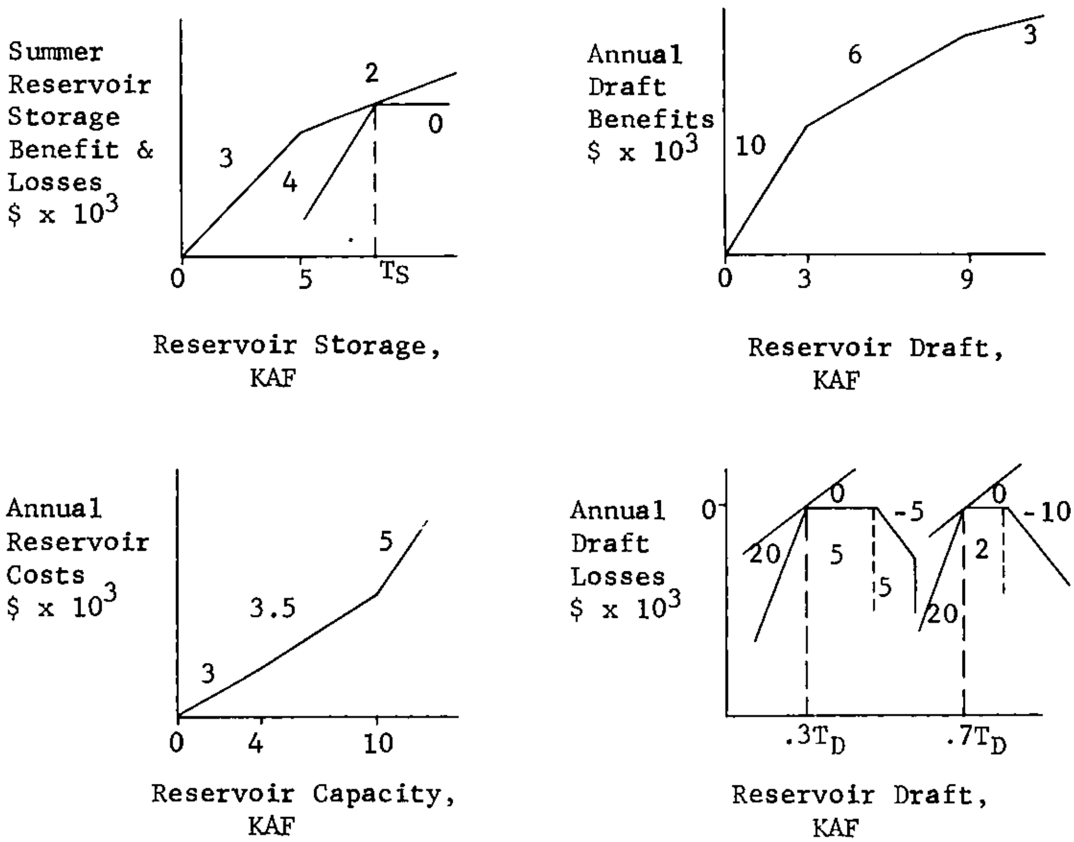

Figure 5. Benefit-Loss and Cost Data for Example Problem

		Summer Streamflow, KAF			
		0.40	3.73	6.27	9.60
Winter	6.95	0.7	0.3	0.0	0.0
Streamflow	12.77	0.3	0.6	0.1	0.0
KAF	17.23	0.0	0.1	0.7	0.2
	23.05	0.0	0.0	0.2	0.8

		Winter Streamflow, KAF			
		6.95	12.77	17.23	23.05
Summer	0.40	0.6	0.4	0.0	0.0
Streamflow	3.73	0.4	0.4	0.2	0.0
KAF	6.27	0.0	0.2	0.5	0.3
	9.60	0.0	0.0	0.3	0.7

Figure 6. Transition Probabilities for Season Inflows

convex linear programming, the details of which will not be presented here.

The two long-run benefit functions shown in Figure 5 indicate the gross annual benefits associated with any particular target quantity of water. For example if the reservoir storage target T_S were 7 KAF and the storage level equaled this target level, the annual benefits from storage would be (\$3/KAF) (5KAF) + (\$2/KAF) (7-5 KAF) = \$19. Similarly if the draft and the target draft, T_D, equaled 8 KAF, the annual benefits from the reservoir draft would equal (\$10/KAF) (3KAF) + (\$6/KAF) (8-3 KAF) = \$60.

The gross benefits associated with each target can be computed by dividing each target into a number of portions, the exact number equaling the number of linear segments of the respective benefit function. By limiting each target portion to the amounts of water included within the respective segment of the benefit function, the total target can be determined by adding up those amounts allocated to each target portion.[1] Let

$T_S = T_{S1} + T_{S2}$ where $T_{S1} \leq 5$, and $T_D = T_{D1} + T_{D2} + T_{D3}$ where $T_{D1} \leq 3$, $T_{D2} \leq 9\text{-}3 = 6$, and let the reservoir capacity $S_M = S_{M1} + S_{M2} + S_{M3}$ where $S_{M1} - 4$ and $S_{M2} - 10\text{–}4 = 6$. Using the marginal benefits and costs shown on Figure 5, the long-run portion of the objective function associated with target benefits and reservoir costs can be written:

$$\text{MAXIMIZE:} \quad 3T_{S1} + 2T_{S2} + 10T_{D1} + 6T_{D2} + 3T_{D3} - 3S_{M1} - 3.5S_{M2} - 5S_{M3}$$

Subtracted from this partial objective function must be the losses due to average summer storage deficits or surpluses and draft deficits or surpluses. Deficits occur when the storage volumes or drafts are less than their respective targets; surpluses occur when these volumes or drafts exceed the targets. Letting S_t equal the initial storage volume in period t, the storage deficit V_t and surplus W_t can be defined as the difference between the storage S_t and the storage target T_S.

$$S_t = T_S - V_t + W_t$$

Similarly for the draft deficit X_t or surplus Y_t, Z_t. Denoting D_t as the draft in period t,

$$D_1 = .3T_D - X_1 + Y_1 + Z_1$$
$$D_2 = .7T_D - X_2 + Y_2 + Z_2$$

where from Figure 5 it is clear that

$$Y^1 \leq 5$$
$$Y^2 \leq 2$$
$$Z^1 \leq 5$$

Obtaining from Figure 5 the marginal losses associated with each segment of each loss function, the overall objective function for maximizing net annual benefits becomes:

$$\text{MAXIMIZE:} \quad 3T_{S1} + 2T_{S2} + 10T_{D1} + 6T_{D2} + 3T_{D3} - 3S_{M1} - 3.5S_{M2} - 5S_{M3}$$
$$- 4\,\frac{(V_1+V_2)}{2} + 0\,\frac{(W_1+W_2)}{2} - 20\,(X_1 + X_2)$$
$$+ 0\,(Y_1 + Y_2) - 5Z_1 - 10Z_2$$

The constraints that must be satisfied while maximizing the above objective function are simply:

(1) The continuity constraints specifying that initial reservoir storage S_t plus inflow I_t less the reservoir release D_t equals the final reservoir storage volume which is identical to the initial reservoir volume S_{t+1} at the beginning of the following period. (If t=2, t+1=1.)

$$S_t + I_t - D_t = S_{t+1} \quad \forall t$$

(2) The constraints defining deficits and surpluses and their upper bounds if any.

$$S_t = T_S - V_t + W_t \quad \forall t$$
$$D_1 = .3T_R - X_1 + Y_1 + Z_1$$
$$D_2 = .7T_R - X_2 + Y_2 + Z_2$$
$$Y_1 \leq 5$$
$$Y_2 \leq 2$$
$$Z_1 \leq 5$$

(3) The reservoir storage volume cannot exceed reservoir capacity.

$$S_t - S_M \leq 0 \quad \forall t$$

[1] Of course this technique can only be used when maximizing a concave function, e.g. benefit functions having successively decreasing slopes, or minimizing a convex function, e.g. loss or cost functions having successively increasing slopes. If these conditions do not hold the value of the total benefits, losses or costs may be incorrectly computed.

(4) The constraints defining the partial targets and reservoir capacities and their upper bounds if any.

$$\left.\begin{aligned} T_D &= T_{D1} + T_{D2} + T_{D3} \\ T_{D1} &\leq 3 \\ T_{D2} &\leq 6 \end{aligned}\right\} \text{draft targets}$$

$$\left.\begin{aligned} T_S &= T_{S1} + T_{S2} \\ T_{S1} &\leq 5 \end{aligned}\right\} \text{storage targets}$$

$$\left.\begin{aligned} S_M &= S_{M1} + S_{M2} + S_{M3} \\ S_{M1} &\leq 4 \\ S_{M2} &\leq 6 \end{aligned}\right\} \text{reservoir capacity}$$

The deterministic model just structured can be simplified somewhat but for clarity it will remain in its present form. In this form there are 27 variables; only two, namely the inflows I_1 and I_2, are known. Setting the inflows I_t equal to their means and solving this model yields the solution given in Table 2.

Annual Net Benefits:		$74.5 x 10^3$	
Reservoir Capacity	S_M:	9.0	KAF
Storage Target	T_S:	5.0	KAF
Draft Target	T_D:	20.0	KAF
Initial Storage			
Winter	S_1:	0.0	KAF
Summer	S_2:	9.0	KAF

Table 2. Deterministic Solution Using Mean Inflows

A Stochastic Design Model

This problem can be structured as a stochastic linear programming problem by including more than one inflow and more than one possible initial storage level and reservoir draft in each season. The more discrete inflows, storage levels, and drafts included in the model the more precise will be the solution. However the larger the programming model the more costly is its solution.

Sufficient for the purposes of illustrating the effect of hydrologic risk on the planning of water resource systems, the deterministic model just presented will be expanded to include only the four inflows defined for each season as shown in Figure 6, three possible storage levels at the beginning of each season, and 12 possible reservoir releases during each season. Let the subscripts i and j indicate the particular discrete inflows in seasons t and t+1, and the subscript k and e indicate the particular discrete reservoir storage volumes in seasons t and t+1. The variable S_t becomes S_{kt}, the k[th] initial storage volume at the beginning of season t; the inflow I_t becomes I_{it}, the i[th] inflow during season t; and the reservoir release D_t becomes D_{kit}, the reservoir release given the k[th] initial storage volume and the i[th] inflow in season t.

Assuming a pure operating policy, i.e. that the reservoir release is unique for a particular k and i, the continuity constraints can be written:

$$S_{kt} + I_{it} - D_{kit} = S_{e,t+1} \qquad \forall_{k,i,t}$$

where e is uniquely determined by k and i. The functional relationship between k, i and e will be called the operating policy. Since more than one possible inflow, storage volume and draft are defined it is necessary to calculate the probabilities associated with these discrete storage levels and drafts. This can be done even though the actual storage volume, S_{kt} and reservoir releases, D_{kit}, remain continuous unknown variables. What is required is a knowledge of the operating policy, the particular final level e given the initial level k and inflow i. This is itself an optimization problem. For the moment a non-optimal but reasonable policy will be defined to illustrate the calculation of PS_{kt}, the probability of the k[th] initial storage level in season t. To define a reasonable policy assume that as the subscript k increases the associated initial storage volumes do not decrease. This condition can be assured without loss of generality by the constraints:

$$S_{kt} \leq S_{k+1,t} \qquad \forall_{k,t}$$

Likewise since the inflows I_{it} are known they can be ordered such that

$$I_{it} < I_{i+1,t} \qquad \forall_{i,t}$$

It seems reasonable to suppose that the final reservoir storage volume resulting from a low initial storage and low inflow might be lower than a final volume resulting from a high initial storage and high inflow. Letting e equal the integer portion of (i+k) /2 results in such a policy. Knowing the transition probability of each pair of discrete inflows $P_{ij}^{(t)}$, the probability of each reservoir storage level, PS_{kt}, and reservoir draft, PD_{kit}, can be computed for this arbitrary operating policy by solving the following simultaneous set of equations:

$$PD_{e,j,t+1} = \sum_{k,i} PD_{kit} \; P_{ij}^{(t)} \quad \forall_{e,j,t} \text{(less one for each t)}$$

$$e = \frac{i+k}{2}$$

$$\sum_{i} PD_{kit} = PS_{kt} \qquad \forall_{k,t}$$

$$\sum_{k} PS_{kt} = 1 \qquad \forall_{t}$$

where the known inflow transition probabilities, $P_{ij}^{(t)}$, are those given in Figure 6.

Note the similarity of the above set of equations to those for finding the steady state streamflows. Instead of only the streamflow interval, a state is now defined to be the initial reservoir volume and streamflow interval. Instead of summing over all streamflows i, the sum is over all streamflows i and volumes k that result in a final reservoir volume e as defined by any particular operating policy, in this case the integer portion of $(i+k)/2$.

Having defined the probabilities for storage volumes PS_{kt} and drafts PD_{kit}, it is possible to write objective functions for maximizing the expected value of the annual net benefits. The constraints correspond to those of the deterministic model except the appropriate subscripts k, i and e have been added where necessary.

$$\text{MAXIMIZE:} \quad 3T_{S1} + 2T_{S2} + 10T_{D1} + 6T_{D2} + 3T_{D3} - 3S_{M1} - 3.5S_{M2} - 5S_{M3}$$

$$- \frac{4}{2} \sum_{t} \sum_{k} PS_{kt}V_{kt} - 20 \sum_{k} \sum_{i} \sum_{t} PD_{kit} X_{kit}$$

$$- 5\sum_{k} \sum_{i} PD_{ki1} Z_{ki1} - 10 \sum_{k} \sum_{i} PD_{ki2} Z_{ki2}$$

Subject to:

1) $S_{kt} + I_{it} - D_{kit} = S_{e,t+1} \qquad \forall_{k,i,t}$

$$e = \frac{i+k}{2}$$

2) $S_{kt} = T_S - V_{kt} + W_{kt} \qquad \forall_{k,t}$

$D_{ki1} = .3T_D - X_{ki1} + Y_{ki1} + Z_{ki1} \qquad \forall_{k,i}$

$D_{ki2} = .7T_D - X_{ki2} + Y_{ki2} + Z_{ki2} \qquad \forall_{k,i}$

$Y_{ki1} \leq 5 \qquad \forall_{k,i}$

$Y_{ki2} \leq 2 \qquad \forall_{k,i}$

$Z_{ki1} \leq 5 \qquad \forall_{k,i}$

3) $S_{kt} - S_{k+1,t} \leq 0 \qquad \forall_{k,t}$

$S_{3t} - S_M \leq 0 \qquad \forall_t$

4) $T_D = T_{D1} + T_{D2} + T_{D3}$

$T_{D1} \leq 3$

$T_{D2} \leq 6$

$T_S = T_{S1} + T_{S2}$

$T_{S1} \leq 5$

$S_M = S_{M1} + S_{M2} + S_{M3}$

$S_{M1} \leq 4$

$S_{M2} \leq 6$

The stochastic model just structured can be simplified in the same manner that the deterministic model could have been simplified, but again for clarity it will remain in its present form. In this form the unknown variables have expanded from 25 to 125. Using the eight discrete inflows, I_{it} as defined in Figure 6, the stochastic model was solved and its solution is presented in Table 3. For ease in comparing the different solutions, portions of the deterministic solution are also presented.

A Stochastic Operating Policy Model

Before discussing the differences between the deterministic and stochastic solutions it is reasonable to ask if some improvement can be made in the operating policy, i.e. the functional relationship between initial storage level k and inflow i and final storage level. In other words can some improvement in the objective function be obtained by holding the design variables constant (e.g. T_S, T_D, S_M, V, W, X, Y, Z.) and maximizing the net benefits with respect to the probabilities PS_{kt} and PD_{kit}?

To answer this question a new model can be structured for defining optimal operating policies for reservoir systems whose design variables are already known. This new model will be called the "operating policy" model to distinguish it from the "design" model just presented. Together these two models form the complete stochastic programming model of this simple hypothetical multiple-use water resource system.

		Model			
		Deterministic	Stochastic		
Annual Expected Net Benefits	$\$x10^3$	74.5			−1.51
Reservoir Capacity, KAF	S_M	9.0			8.8
Storage target, KAF	T_S	5.0			5.0
Draft target, KAF	T_D	20.0			17.3
Initial Reservoir storage	k		1	2	3
Winter Storage, KAF	S_{k1}	0.0	0.0	0.3	0.9
Probability	PS_{k1}	1.0	.43	.16	.41
Summer Storage, KAF	S_{k2}	9.0	7.0	7.9	8.8
Probability	PS_{k2}	1.0	.43	.17	.40

Table 3. Stochastic Solution Using Assumed Operating Policy

The objective function of the operating policy model is simply an expansion of the relevant portion of the objective function of the design model, i.e. that portion containing the probabilities of various storage volumes, PS_{kt}, and drafts, PD_{kit}. This portion is expanded to include all feasible combinations of drafts resulting from the discrete inflows, I_{it}, and storage volumes, S_{kt}, defined in the solution of the design model. Thus

$$D_{kiet} = S_{kt} + I_{it} - S_{e,t+1} \qquad \forall\, k,i,t$$

and for all e such that $D_{kiet} \leq 0$. Knowing the draft targets, $.3T_D$ and $.7T_D$, and the upper bounds on each of the segments of each loss function, the values of the deficit variables, X_{kiet}, and surplus variables, Y_{kiet} and Z_{kiet}, associated with each feasible draft D_{kiet} can be determined:

$$X_{kie1} = .3T_D - D_{kie1} \qquad \forall\, k,i,e,1 \text{ if } X_{kie1} \leq 0$$

$$X_{kie2} = .7T_D - D_{kie2} \qquad \forall\, k,i,e,2 \text{ if } X_{kie2} \leq 0$$

For all X_{kiet} that would be less than zero,

$$Y_{kie1} + Z_{kie1} = D_{kie1} - .3T_D$$

$$Y_{kie2} + Z_{kie2} = D_{kie2} - .7T_D$$

where

$$Y_{kie1} = 5 \text{ if } Z_{kie1} > 0$$

$$Y_{kie2} = 2 \text{ if } Z_{kie2} > 0$$

and $Z_{kie1} \leq 5$

If $Z_{kie1} > 5$, the particular draft D_{kie1} is infeasible even though it may be positive. For all infeasible D_{kiet}, $P_{kiet} = 0$.

Having calculated all feasible drafts and their associated deficits or surpluses, all that remains is to determine the probabilities of releasing these drafts if the operating policy were optimal, i.e. one which maximized the annual expected net benefits. Recall that in order to determine an optimal set of design parameters (reservoir capacity, targets, storage volumes and drafts) an operating policy was arbitrarily assumed, and once assumed the storage and draft probabilities could be calculated and included in the objective function of the design model as known parameters. Now the process is reversed. Known are the design parameters. The probabilities associated with these design parameters are unknown. These unknowns include the probabilities of each storage volume, PS_{kt}, and of each feasible draft, P_{kiet}. The draft probabilities are the joint probabilities of having an initial reservoir volume S_{kt}, and inflow I_{it}, and releasing an amount D_{kiet} in season t so as to have an initial volume $S_{e,t+1}$ in season t+1.

The objective function of the operating policy model corresponds to the probabilistic portion of the objective function of the design model with the addition of the subscript e where appropriate

$$\text{MAXIMIZE } -\frac{4}{2} \sum_{t} \sum_{k} V_{kt} \quad (PS_{kt})$$

$$- \sum_{k} \sum_{i} \sum_{e} [20X_{kie1} + 5Z_{kie1}] \quad (P_{kie1})$$

$$- \sum_{k} \sum_{i} \sum_{e} [20X_{kie2} + 10Z_{kie2}] \quad (P_{kie2})$$

The constraints of the operating policy model are merely an expansion of the simultaneous equations previously used to solve for PS_{kt} and PD_{kit} given an arbitrary functional relationship between k, i and e.

(1) The joint probability of a final volume e at the end of period t times the probability of an inflow j in period t+1 equals the joint probability of an initial volume e and an inflow j in period t+1.

$$\sum_{k} \sum_{i} P_{kiet} \, P_{ij}^{(t)} = \sum_{m} P_{e,j,m,t+1} \qquad \forall_{e,j,t}$$

less one for each t

(2) Summing over the appropriate subscripts defines the probabilities PS_{tk} and PD_{kit}.

$$\sum_{e} P_{kiet} = PD_{kit} \qquad \forall_{k,i,t}$$

$$\sum_{i} PD_{kit} = PS_{kt} \qquad \forall_{k,t}$$

(3) Finally the sum of all probabilities in each period t must equal one.

$$\sum_{k} PS_{kt} = 1 \qquad \forall_{t}$$

The solution of the operating policy model does not directly specify the operating policy, only the steady state values of the probabilities P_{kiet}, PD_{kit} and PS_{kt} if an optimal policy is followed. However knowing the joint probabilities P_{kiet} enables one to calculate the conditional probabilities of a final volume e given an initial volume k and inflow i.

$$\text{Prob. } (e|k,i,t) = P_{kiet} / \sum_{e} P_{kiet} \qquad \forall_{k,i,e,t}$$

Although not constrained to do so, these policy models will usually yield 0,1 conditional probabilities. Policies defined by these 0,1 conditional probabilities are called pure policies, policies in which only one e is specified for a given k and i. If more than one final volume e were optimal for a particular initial volume k and inflow i, it would be necessary to define the drafts in the design model as a function of k, i and e, i.e. D_{kiet} rather than D_{kit}. The fact that optimal policies are usually pure policies justifies the omission of the subscript e from the draft variables in the design model.

The solution of any operating policy model is of course only optimal for the particular values of the design parameters used in its objective function. These design parameter values are of course either assumed or determined by solving the design model in which case they are optimal only for a previously computed or assumed operating policy. Unless the policy derived from the current solution of the policy model is identical to the previously computed or assumed operating policy, it is reasonable to conclude again that there may be an improved set of design parameter values, i.e. targets, capacities, storage volumes and drafts, associated with the improved operating policy. This can be determined by resolving the design model after incorporating into the objective function the revised probabilities, PS_{kt} and PD_{kit}, and after defining the new operating policy in the continuity constraint set. If an improved design results, i.e. a set of targets, reservoir capacity, storage volumes and drafts, that yields a higher expected annual net benefit, it is again reasonable to assume that there may be an improved operating policy associated with this improved design. Again this can be determined by resolving the operating policy model after incorporating the new values of the design parameters in the policy model's objective function.

This two-stage iterative process can be continued until no further improvement can be made, either in the design or operating portion of the solution. There is no guarantee that a global optimum will always be reached using this procedure yet for this example three quite different starts all eventually converged to the same optimal design and policy. Further improvement could not be obtained.

The optimal solution obtained from the two-stage process just described is presented in Table 4. Again it is compared with the deterministic solution in which only the mean inflows in each season were considered. The stochastic model not only considered four discrete inflows each season but also the serial correlation of those inflows as defined by the transition probabilities presented in Figure 6. The optimal operating policy is presented in Figure 7. It can be seen from Figure 7 that even though considered "reasonable," the originally defined policy, namely e equals the integer portion of (k+i) /2, is far from optimal.

		Model			
		Deterministic	Stochastic		
Annual Expected Net Benefits	$\$x10^3$	74.5		3.96	
Reservoir Capacity, KAF	S_M	9.0		8.1	
Storage target, KAF	T_S	5.0		6.9	
Draft target, KAF	T_D	20.0		16.5	
Initial Reservoir storage	k		1	2	3
Winter Storage, KAF	S_{k1}	0.0	0.0		
Probability	PS_{k1}		1.0		
Summer Storage, KAF	S_{k2}	9.0	6.9	7.8	8.1
Probability	PS_{k2}		0.50	0.25	0.25

Table 4. Optimal Solutions for Example Problem

Winter Inflow i

		1	2	3	4
Initial Winter Storage level k	1	1	2	1	3

Initial Summer Storage Level *e*

Summer Inflow i

		1	2	3	4
Initial Summer	1	1	1	1	1
Storage level k	2	1	1	1	1*
	3	1*	1*	1	1

Initial Winter Storage Level *e*

* = Combination of k and i that will not occur in the steady state operation of the reservoir system.

Figure 7. Optimal Operating Policy for Example Problem

DISCUSSION AND CONCLUSION

There are several points worth emphasizing in this concluding section. The first is that experience with several examples similar to that just presented indicate that the design variables, i.e. the reservoir capacities, targets, storage levels, drafts, etc., are not too sensitive to changes in the assumed or computed operating policy. Compare, for example, the design solutions presented in Table 3 with those presented in Table 4. The operating policy originally assumed for the solution in Table 3 is quite different from the optimal policy given in Figure 7. However, for the purposes of preliminary planning, the design variables presented in Tables 3 and 4 are not significantly different. One of the reasons for this is that for any fixed operating policy, the actual targets, volumes and drafts remain continuous unknown variables, and even though their probabilities are fixed they are free to take on those feasible values which maximize the total expected net benefits.

The second point of interest is that, although the example presented in this paper may not be the best that could have been chosen, it does illustrate the fact that the incorporation of hydrologic risk into any analysis of reservoir systems does not always lead to increased reservoir capacities as one might expect. The optimal solution of the deterministic model specified a reservoir capacity that was 11% larger than that specified in the optimal solution of the stochastic model. The deterministic model considered only the mean inflows each season whereas the stochastic model explicitly considered four discrete serially correlated inflows each season.

Finally, in spite of increased reservoir capacity and therefore cost, the deterministic solutions of the example problem is clearly overly optimistic with respect to the net benefits expected each year. While the deterministic model predicted a maximum annual net benefit of $74.5 thousand, the stochastic solution suggests only $3.96 thousand can be expected, a significant reduction indeed! Part of the reason for the magnitude of this decrease is the high variance of the inflow, both within each season and between seasons, and the relatively high losses for deviations about the draft target. Nevertheless, this example does illustrate the optimism inherent in many deterministic solutions of stochastic systems. The introduction of any uncertainty or risk imposes a cost, and this additional cost is indeed substantial in this relatively simple example.

The separate stochastic design and operating models can be combined into one having both linear and quadratic constraints and a quadratic objective function. Solutions to this non-linear model have not been attempted. It seems possible that policy improvement, especially in multi-reservoir problems where the policy portion of the model would become too large, might be accomplished through examination of the dual variables of the continuity constraints. These remaining questions and many more may eventually be answered, but they are presented here only as food for thought or as possible research topics.

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A COMPREHENSIVE APPROACH TO THE PROBLEMS OF POLLUTION AND WATER RESOURCES

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The distinguished delegate representing the Secretary of the Interior at our first Seminar meeting in January, 1968, outlined as the official policy of the Federal Government the prospect that "comprehensive" studies would be carried out, in contrast to the engineering-type of approach formerly characteristic of pollution studies.

This apparent change in official attitudes is certainly welcome not only to economists, but to all pollution experts. In the past, the best solutions for comprehensive pollution problems were often referred to as "complex", implying, in effect, that the approaches commonly used were inadequate, and did not cover the complexities of the problems.

The interrelated aspects of pollution and water resource problems demand that many factors be investigated and given balanced consideration, so that the final analyses result in a comprehensive solution. This realization by the Federal Government is definitely encouraging, certainly a more realistic approach.

The once free supply of clean water and clean air have become scarce in our everyday life. It is time that we realized that the economic principles directing the pricing of goods on the basis of cost and scarcity should be applied in the pollution context. Such an approach is natural for economists, yet we have been slow to shape appropriate procedures. This is all the more surprising since our every-day statistics show that today about 70 per cent of our 200 million people are urban dwellers and urban water usage goes as high as 1,600 gallons per capita daily in the larger cities. In addition, today there are roughly 12,000 communities having sewer systems as compared to 950 communities in 1900. Urbanization, combined with the increase in industral development, has inevitably placed the outfall of sewer systems closer and closer to the water intake for industries and communities. The volume of water used and concurrently and necessarily returned to the stream has been multiplied many times.

At this juncture, it is very clear that if we do not move rapidly and, more important, if we do not take into consideration all the factors involved in pollution problems, including the many-sided conservationist approach, the result very easily may be a curtailment of our economic progress. (Not to mention recurrent crises—such as the water shortage that occured in the New York City megalopolis during 1966, and periods of acute smog.)

The crises that have already developed because of short-sighted pollution policies have warned us of inconveniences to the public and hazards to public health. The future is all the more threatening because there are a growing number of industrial areas where we can no longer afford to proceed on the old haphazard basis, where we ought to evolve a carefully planned program. It is clear that such a program must be long-range and comprehensive; besides quantity considerations, quality is becoming increasingly important. All of this presents a complex problem of so-called socio-economic development. It is difficult to avoid a critical tone in reviewing what appear to be instances of inefficient management of pollution problems and water resources by political authorities. It almost appears that the agenda for economists should have centered upon devising institutions whereby imperfect administrators may be forced to learn from past errors. The decision makers seem to go on committing the same mistakes, despite a torrent of past advice from economists (Eckstein, Krutilla, McKean, Maass, Renshaw, Kneese, Haveman, Bain, Caves, Margolis, Schmid, Hirschleifer, Milliman, Ciriacy-Wentrop and others).

Rational reallocation of existing water supplies, coupled with pricing in response to scarcity, is almost never considered as an alternative even in cases of new construction. There are several reasons, but just to mention a few: (1) administered prices are inflexible, and difficult to charge equitably. (2) Political rigidity usually makes it impossible for one water jurisdiction to sell title or rghts to another. Thus, construction has to exploit the supplies within the local boundaries. (3) Errors in basic economic reasoning frequently occur, e. g., double counting of benefits, ignorance of the marginal principle, the inappropriate establishment of low discount rates. The implicit belief is that the general need for water supply is the domineering factor, or, in economic terms, that demands are absolutely inelastic in this field. Until today, planning, in general, has been based on this official policy.

Before getting into details of socio-economic concepts as the criteria for any comprehensive solution for pollution and water resource problems, let us sort out certain concepts in a more systematic manner. The pricing of intangible goods that is used in practice by the federal agencies introduces the same concept which is applicable to new "environmental" products. The crux of the problem is the relative pricing of goods like foodstuffs, chemicals, etc. versus factors such as health and recreation. The idea is that people should be able to get prices set on the basis of which their decisions can be made (consumption model) and thus define the product-mix they desire.

There is an implicit exchange ratio or price for every act for which an alternative exists. Hence, any administrative standard that defines the pricing of public goods will result in a common denominator for an exchange between goods obtained and goods foregone.

Intangibility is sometimes used in the sense that the good has a value closer to infinite. In practice, this good has no alternatives and no further analysis is needed. Another variation of this concept is the idea that the value of some goods cannot be measured. This leads to confusion in pollution and water resource decision making.

The real problem is "quality"; the actual attempt to quantify quality changes in goods which have been regarded as non-measurable because of their form. The problem of quality is subdivided in many theoretical works into economic and non-economic parts; quantitative and non-quantitative, or in other words, measurable and non-measurable.

In order to clarify many of the generalizations which have been made about quality, it is desirable to consider the possible solutions suggested under the heading of quality change. One approach used in this respect takes into account all specifications of a measurable (specific) nature which affect the value of the commodity. The methods worked out in this manner actually are based on the welfare concept of how to define "real values". Richard Stone, for instance, uses some multiple regression techniques to consider the effect on price when changes in quality in each type of specification take place.

In general, it is apparent that no directly applicable (one step) practical solution is readily available. The "conventional method" is to translate quality into quantity by reference to market price. The "alternative method" contends that quality can be judged directly from the study of consumer response to previously preferred goods as compared with consumer evaluation of currently preferred goods. However, this procedure also is dependent on market data.

Far more promising are proposals to measure quality change by establishing extraneous but reasonable standards for specific purposes.

If the truth be told, the concept of the "intangible" reveals more of our own ignorance than of the character of the good itself. To sum up, the concepts of intangible and non-measurable products do not offer us a usable analytic tool for predicting market performance.

Economists today refer to the term "public goods" in the application of comprehensive policy decisions. There are two extreme components of the concept of public goods, discussed in the economic literature: the marginal cost of water is zero for some groups, namely, consumption of water by one user does not reduce utility to another. Second, assuming that since water as a product is a public good, potential users cannot be excluded from its use.

The individual always knows that if others provide the public goods, he will benefit whether he pays or not. Thus, he has no economic incentive since the water is retained at a standard quality and is available to all users at no extra cost. In addition, nobody can be excluded from the use of this public good as a product.

If pure water is maintained in a river, the cost of obtaining this product is not affected by the number of consumers. Even if this product (pure water) must be produced by treatment of a waste discharge, the marginal cost of another beneficiary is zero. This creates problems for bilateral market bargaining, especially if the group is so large that an individual does not affect the outcome. One hypothetical alternative in that case is that the product will not be produced at all, despite the fact that there is a group of buyers willing to share the cost; on the other hand, for potential users' needs, this product has to be provided.

Some economists envisage two bargaining government agencies dealing on the market for the right to water. The power to buy as a group in a more or less political process is a valuable right. The problem is, however, that environmental products have rights established usually by the first user, as a result of our historical development. The question arises, will we presently face a fight for water similar to the 19th century cattlemen-versus-sheepmen controversy? Today's problem is analogous, though the parties involved have changed. Billions of dollars are spent in two opposing directions, due to the lack of any comprehensive guidelines for water resource use.

True, there are many efforts taken in the right direction, that is, to find solutions in a more-or-less balancing manner. The central issue seems to be how to secure, for the present and future, unspoiled natural environments. Yet, a basis for decisions is lacking since there is no market from which adequate decisions can be evolved. Thus, the complex problem of how to organize a market for public goods arises.

There are *static considerations* at best, in existence, and they tend to have their real significance in the analysis of the effects of parametric shifts in tastes and technology. Our real problem is, however, akin to a *dynamic* programming model, which requires a present action to be applied in the attainment of future considerations. True, this approach may violate the rules of conventional cost-benefit criteria. Obviously, a great deal of research in the area of so-called socio-economic considerations is involved. We have much to learn concerning the determinants of outdoor recreation versus the quantitative significance of producing industrial goods.

Our urgent task is to definitely prevent potentially adverse consequences for future human welfare. Outdoor recreational opportunities, in addition, will be provided largely by public bodies and attempts have to be made to homogenize the recreation commodity—in other words, to arrive at a common denominator for the analysis of public goods.

To sum up, the valuation of the variables necessary for the application of a dynamic model is the real task for economists and government planners for the sake of any comprehensive solution of pollution problems.

The practical question is, what is a "comprehensive pollution control plan?", the phrase so often used in the Federal Water Pollution Control Act. It is certainly a systematic plan for the implementation of all the means and measures needed not only to control pollution but also to prevent pollutants from affecting the quality of water. Thus, a comprehensive water pollution control program must include: (1) complete hydrologic data and material from analyses, (2) the determination of river, lake, and estuarine biological characteristics, (3) sedimentation analyses in case of lake and estuarine studies, (4) population forecasts and industrialization projections for definite areas, (5) determination of future waste uses and returns of waste, (6) waste treatment and waste removal projections, (7) cost estimates, including added costs for reducing concentrations which cannot be treated adequately because of the augmented quantities involved, (8) the implementation of a general control and planning program diversified at the regional (planning) level.

Finally, a comprehensive pollution control plan involves systemized guidelines for the cooperation of all the federal agencies, such as the Army Corps of Engineers, the Bureau of the Department of Agriculture, and other Federal agencies.

The Federal Water Pollution Control Act thus authorizes eventually two superagencies, such as the Secretary of the Department of Health, Education and Welfare to advise these agencies, and the Secretary of the Department of Interior as the coordinating authority in these matters.

One practical implication is that multi-agency review is now required for federal water and pollution investments. The real problem is that too many agencies' time is spent on reviewing other agencies' projects. Yet a broad program is demanded by the government and by the public as well, under which the various agencies not only review existing problems on small projects but are directed by broad and comprehensive criteria.

In any event, a bold new approach is needed on the basis of an empirical analysis of the impact of all of the "investments".

Cost-benefit issues and all the other considerations of maximizing national welfare emerge from today's pollution problems. At the same time, it must be borne in mind that tastes neither stand still nor are necessarily communicated clearly.

In historical perspective, Keynes taught that bilateral bargaining could bring equilibrium at less than full employment. This means that everybody takes the opportunities offered and maximizes benefit within his field of opportunity. In sequence, the field becomes worse until total performance is satisfactory to comparatively few. To be able to get a different performance requires a group action to redistribute and restructure the power of decision, with the result that costs and benefits decrease. Thus, investment that was unsound for the individual under a given set of market rules became sound when coordinated differently.

More recently, Boulding and Singh argued that bilateral bargaining may bring an equilibrium of prices and supplies that does not maximize growth. Hence, to get the desired performance, you need group action. Acting individually and affected by a market, people may not be able to achieve the price set consistent with the performance they want. But modification of property rights or direct government administration may produce the desired price set in this case. Clearly, economic analysis needs to present data to help people choose the game they want to play and understand the broad range of opportunities available, as well as how to play a given game to their advantage. To sum up, regional planning and consideration of the so-called socio-economic problems are the guides for any "comprehensive" solution of pollution problems. The tools and capabilities for developing such a program are available, and they are essential for the sake of a rational solution and a solution primarily in the public interest.

ANALYSIS OF SPIT-BAR DEVELOPMENT AT SANDY HOOK, NEW JERSEY

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ABSTRACT

A foreshore spit-bar is a seaward-convex ridge of non-cohesive, clastic debris attached to land at its proximal terminus and having its distal terminus in deep water. The spit-bar crest, lying within the foreshore beach zone, decreases in elevation toward the distal terminus. The *lower-foreshore spit-bar* has its crest below mean-high-water, an up-convex seaward face as seen in transverse profile, and a steep landward face representing a depositional slope of repose between landward bar edge and adjacent trough.

Detailed transverse profiles (109) allowed transverse geometry of lower-foreshore spit-bars located at Horseshoe Cove, Sandy Hook, New Jersey to be investigated by linear correlation, regression analysis, and analysis of covariance. Sequence of positions occupied by crest, edge, and trough of the landward migrating spit-bar show statistical parallelism (at 5% level) of upward-trending regression slopes in eight of the twelve profile groups tested. Therefore, geometric similarity of form is maintained during landward migration.

An annual cycle of spit-bar formation is recognized as the result of an annual cycle of prevailing wind blowing over Sandy Hook Bay. In fall, winter, and spring large, steep waves from the northwest erode the Arrowsmith feeder beach and deposit this sediment along the adjacent Horseshoe Cove spit-bar shoreline. In summer, small, low waves from the south push this newly deposited sediment into a succession of landward migrating lower-foreshore spit-bars that merge and usually form a new upper-foreshore spit-bar by the end of each summer.

The sequence of spit-bar development at Horseshoe Cove is considered analogous to the landward migration of foreshore bars on ocean-facing beaches. Such foreshore bars have a similar transverse profile to the spit-bars described and are important because they contribute most of the sediment that allows seaward growth of the summer berm.

INTRODUCTION

Sandy Hook is a barrier spit six miles long, which lies at the northermost extremity of the New Jersey portion of the Atlantic Coastal Plain (Figure 1). At the Highlands Bridge Sandy Hook connects with a barrier bar that joins the mainland 3½ miles to the south at Monmouth Beach. East of Sandy Hook lies the Atlantic Ocean; on the west is Sandy Hook Bay. Sandy Hook Bay is connected to the Navesink and Shrewsbury tidal estuaries through a dredged channel between Sandy Hook and Highlands.

Sandy Hook Bay is subject to a semi-diurnal tide of 5.6-foot mean spring range. Mean tide level is 2.3 feet above mean-low-water datum established for Sandy Hook Bay by the U. S. Coast and Geodetic Survey (1962, p. 217). The lowest water ever recorded at Sandy Hook tide gauge occurred on 24 January 1936, when tide fell to 6.0 feet below mean sea level. The highest recorded water of 8.3 feet above mean sea level occurred during passage of Hurricane Donna on 12 September 1960.

Caldwell (1967) has reported new estimates of the littoral drift of sand along the New Jersey coastline. From a nodal point at Dover Township, 35 miles south of Sandy Hook, there is a net northward movement of sediment toward Sandy Hook. During the 50-year period, 1885-1935, the average rate of sand accretion at Sandy Hook was 493,000 cubic yards per year.

Until 1962 most of Sandy Hook lay within the Fort Hancock Military Reservation. Protected thus from public access and commercial activity it represented one of the most extensive metropolitan beaches on the Atlantic Coast of the United States free from continual man-made disturbance. In 1962 the southern 400 acres of Sandy Hook were transferred to the State of New Jersey for use as a park.

Recent discussion of a plan to include Sandy Hook as part of a National Seashore complex in New York Harbor may allow greater scientific study and public utilization of this area.

Johnson (1919, p. 290) classified Sandy Hook as a compound and complex spit: compound, because of successive beach ridges marking stages in northward advance of Sandy Hook; complex, because sediment eroded from the inner side of Sandy Hook by local waves in Sandy Hook Bay has been formed into small secondary spits unrelated to curvature of the Atlantic Ocean shoreline. Because the western shore of Sandy Hook rarely, if ever, is subjected to significant refracted or diffracted Atlantic Ocean wave trains, it is an ideal location for study of beach forms shaped by local wind-generated waves in the

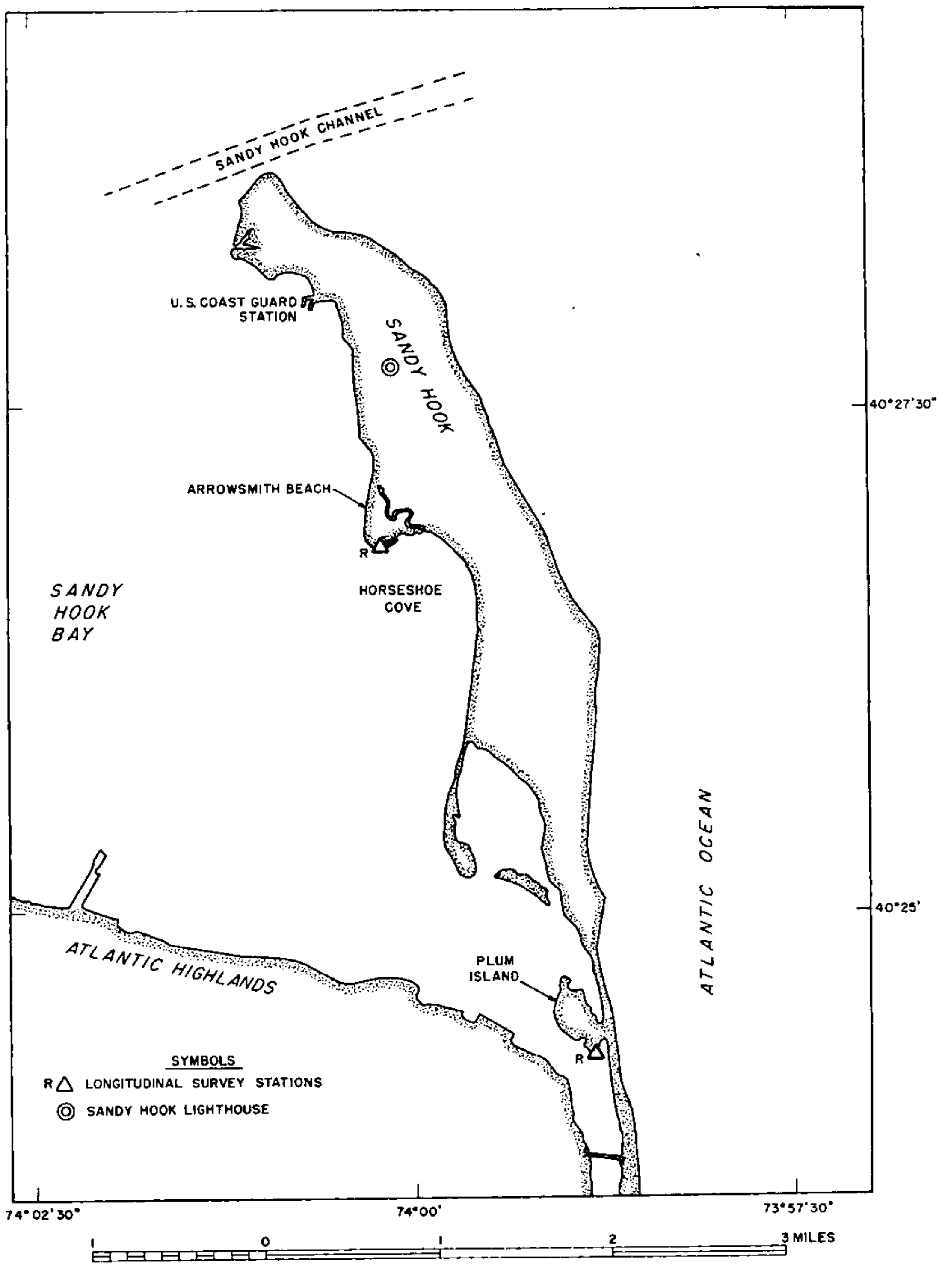


Figure 1. Location map, Sandy Hook, New Jersey

relatively shallow water of Sandy Hook Bay under a regime of moderately large semi-diurnal tides. Another inherent advantage of the study area is its modest size and well-defined geomorphic boundaries. Steers (1946, p. 62) has suggested that many of the mistakes in generalizations about coastal features are probably caused by neglect of observations along small segments of coastline. The Horseshoe Cove spit-bars, although comprising one element of the Sandy Hook barrier spit, are themselves typical of compound-recurved-spit formation on a smaller scale (Figure 3). Of great significance is the relatively deep water immediately to the south and east of the foreshore spit bars at Horseshoe Cove, where water depth is approximately 20 feet at mean low water. A record of dredging activity in New York harbor since 1905, maintained by the New York District, Corps of Engineers, fails to reveal any dredging activity at or near Horseshoe Cove. It may therefore be assumed that bottom topography and sediments in the study area are of natural origin.

Two spit-bar shorelines and associated feeder beaches for which detailed growth records exist are Blakeney Point and Scolt Head Island. Both are located on the Channel Coast of England and, as areas held under protection of the British National Trust, have been intensively studied using records dating to 1585. Sketch maps showing development of Blakeney Point during the period 1913-1926 are presented by Oliver (1926, p. 4-5). Steers (1946, p. 348-A) includes plane table maps, on a scale of approximately 1:18,100, for the 1919 and 1926 shorelines. Steers and Grove (1960) also show shoreline changes at Scolt Head Island based on plane table and transverse-profile surveys.

It is to be noted that both Blakeney Point and Scolt Head Island have gently sloping offshore bottoms that allow exchange of sediment between foreshore and offshore: both have large, migrating tidal channels at distal termini of their spit-bars.

Perhaps because of the more limited area involved, it has been possible to study foreshore spit-bars at Horseshoe Cove in greater detail than appears to have been attempted in any similar study with which the writer is familiar. Because of the relatively deep water immediately seaward of the foreshore spit-bars, Horseshoe Cove presents a simplified physical setting for study of classic spit-bar growth in the sense that there is no chance of an exchange of foreshore and offshore sediment as might occur at Blakeney Point and Scolt Head Island. Prior to inception of this present series of studies, there had never been a published field study of beach morphology at Sandy Hook, New Jersey.

The present paper deals primarily wtih an analysis of transverse beach profiles surveyed in 1960 and 1961. During 1968 a resurvey of remaining triangulation and station markers in the Horseshoe Cove area was begun in anticipation of a new field program that will attempt to relate continued spit-bar growth, as described in this report, to wave and tide parameters.

Studies of foreshore bars having transverse profiles similar to the spit-bars at Horseshoe Cove will also be initiated on ocean-facing beaches on Sandy Hook.

The original report of investigations at Horseshoe Cove was presented as a technical report with limited distribution to coastal geomorphologists and oceanographers (Yasso, 1964). A re-evaluation of these data is presented in the present paper for the value they may have in generalizing about coastal processes along similar coasts.

CLASSIFICATION AND NOMENCLATURE OF BEACH FORMS

Beach zones

A beach subjected to water-level oscillations caused by astronomical tides is divided into zones that parallel the trend of topographic contours in the shore area (Figure 2). King (1959, p. 48-49) follows general usage in dividing the beach into three zones: (1) an *offshore zone* extending from the seaward limit of sediment transport by wave action landward to the contour line of mean low water; (2) a *foreshore zone* extending from the mean-low-water contour to the upper limit of swash at mean high water; and, (3) a *backshore zone* extending landward from the upper limit of swash at mean high water to the upper limit of storm swash during times of exceptionally high water levels. The backshore zone includes the wave-cut cliff of a steep coast. On a low coast the backshore zone may include the dunes and mature salt marsh.

Following coastal engineering usage, Krumbein (1954, p. 7) refers to the landward portion of the offshore zone as *nearshore bottom*. He splits the foreshore zone into upper and lower segments without giving a basis for such division.

The writer has followed terminology discussed above. In addition, the tidal lagoon situated landward of the foreshore spit-bars at Horseshoe Cove is designated the *lagoonal subzone* of the backshore zone. The *upper-foreshore zone* is here considered to be a zone lying between the mean-high-water contour and the upper limit of swash at mean high water. By the latter extension of terminology a zone is defined in which beach form change and sediment transport are usually accomplished only by the swash-backwash mechanism of wave action.

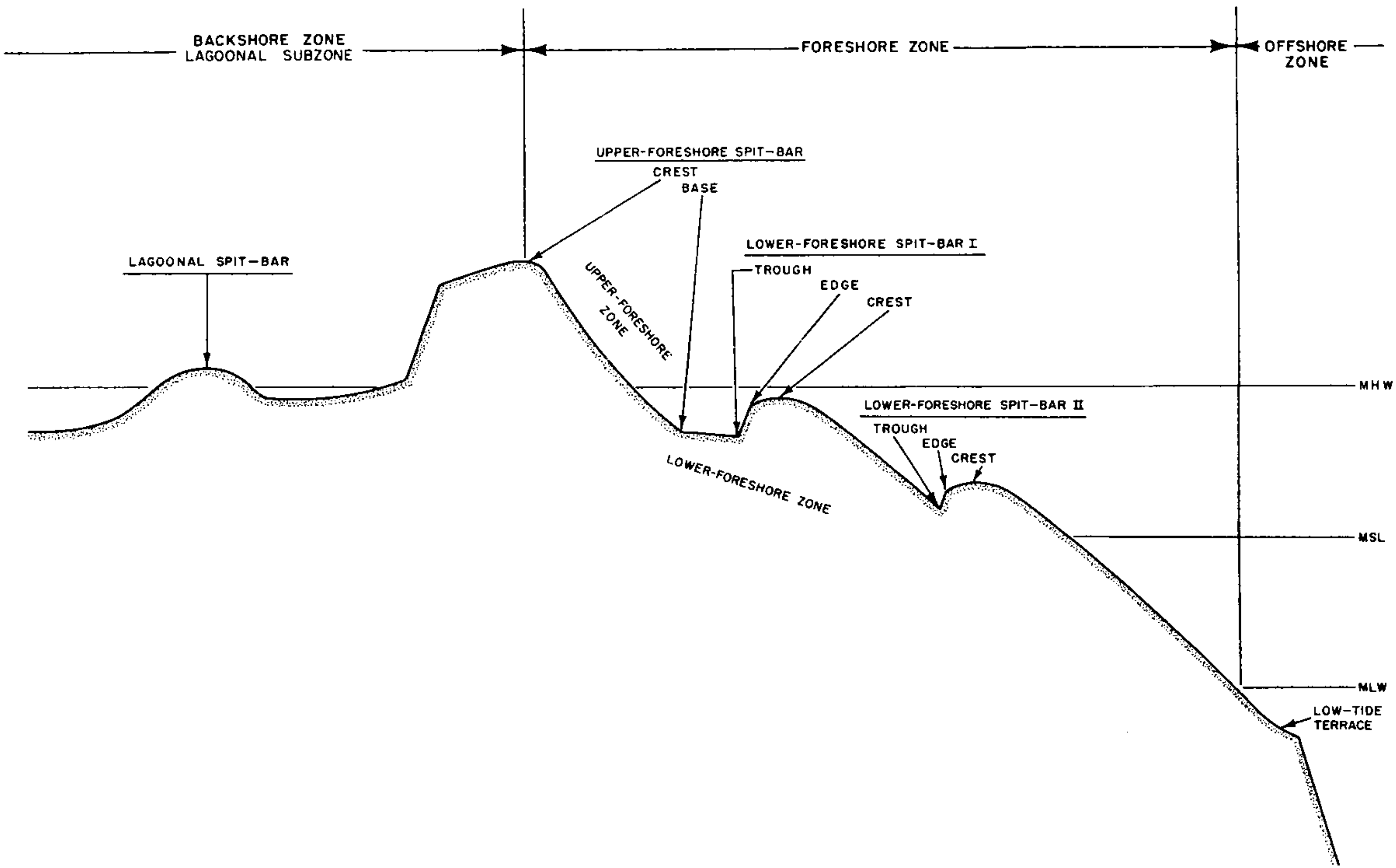


Figure 2. Beach profile showing terminology used

The *lower-foreshore zone* is here considered to be a zone lying between mean-low-water and mean-high-water contours.

Foreshore topography at Horseshoe Cove

The literature shows no uniformity in terminology applied to the foreshore zone. Positive and negative relief features created from beach sediment, by wave action in the foreshore zone, have been referred to respectively as: ridges and runnels (Guilcher, 1958, p. 83); fulls and lows (Gresswell, 1957, p. 124); bars and runnels (Williams, 1960, p. 115); and, ridges and furrows (Cornish, 1898, p. 538). Shepard (1952, p. 1904) suggests that the term *bar* be restricted to sand shoals that are permanently under water. In this report, the term *bar* is taken to mean any relatively long, narrow ridge or enbankment of non-cohesive sediment formed in the zone of breakers and surf. Various types of bars may be distinguished according to zone of occurence and mechanism of formation.

Following a generally uniform usage found in the literature, a *spit-bar* is defined as a wave-formed ridge of non-cohesive, clastic debris whose crest stands above low-tide level, whose *proximal terminus* is attached to a headland beach or barrier bar, and which extends with convex-to-seaward curvature plan to a *distal terminus* in relatively deep water. This definition of the term *spit-bar* is a logical extension of definitions and descriptions of spits given by Gulliver (1899, p. 241), Johnson (1919, p. 287-290), Evans (1942, p. 846), King (1959, p. 255), and Gierloff-Emden (1961, p. 82-85). The hyphenated term *spit-bar* used in the present report has been adopted because a spit is basically a form of bar having a free end. As defined above, a *spit-bar* would be considered by most writers to be merely a transitional depositional feature that eventually becomes a barrier-bar or baymouth bar that closes off some coastal embayment such as a bay or estuary (Shepard, 1937, p. 624; Evans, 1942, p. 846).

Characteristic planimetric curvature of bars and intervening troughs (ridges and runnels, or other similar forms) has not been specified in the literature. King (1959, p. 337-341) mentions that four sets of ridges and runnels are usually present in the foreshore zone: the lowermost set having greatest topographic relief, the uppermost set having smallest relief. No statement is to be found concerning changes in position or orientation of ridges and runnels except that ridges are cited as being discontinuous in plan as a result of ponded tidal water cutting cross-channels in rip-current type of erosion. Published descriptions of ridge and runnel foreshore topography fail to apply to foreshore spit-bar topography at Horseshoe Cove.

In an effort to resolve conflicting terminology and avoid genetic implications that other writers have attached to use of particular nomenclature, spit-bars of the foreshore-zone at Horseshoe Cove are herein referred to as *foreshore spit-bars.* A generalized transverse shoreline profile taken at right angles to surface contours is given in Figure 2. Two spit-bars are shown in the lower-foreshore zone: uppermost of these is termed *lower-foreshore spit-bar I,* lowermost is termed *lower-foreshore spit-bar II.* Usually we find only one lower-foreshore spit-bar along a given transverse profile, making use of Roman numeral designations unnecessary. A narrow, gently seaward-sloping platform is sometimes present at about low-tide-level (Figure 2); it is called the *low-tide terrace* (Kuenen, 1950, p. 268) and is discussed in a later section of this report.

Well developed lower-foreshore spit-bars are characteristically asymmetrical in transverse profile: a smoothly up-convex surface, with landward decreasing radius of curvature, terminates at the top of a steep, with landward-facing *edge-trough slope,* lying between *spit-bar edge* and *spit-bar trough.* Inclination of this straight, depositional slope ranges between 8.5 and 33.2 degrees as measured by Abney level at various times and places. The term edge thus denotes the abrupt discontinuity between *spit-bar* crest and depositional slope. Highest point on the up-convex profile seaward of the edge is called the *spit-bar crest.*

A single spit-bar is found in the upper foreshore zone and is termed the *upper-foreshore spit-bar.* The seaward face of the upper-foreshore spit-bar is generally of sigmoidal profile, up-concavity near its base changing to up-convexity near its *crest.* A gently landward-sloping surface between crest and edge *(crest-to-edge slope)* leads to a steeply landward-sloping surface between edge and trough, as in the lower-foreshore spit-bars. Sigmoidal profile of upper-fore-shore spit-bar seaward face develops only when the spit-bar crest has been built to the limit of swash at high tide. Prior to that time, transverse form of the upper-foreshore spit-bar may be quite similar to that of the lower-foreshore spit-bars. Landward of the upper-foreshore spit-bar there may be found one or more fairly symmetrical, up-convex embankments of low relief which are here called *lagoonal spit-bars.* They are erosional remnants of upper-foreshore spit-bars marking stages in seaward progradation of the Horseshoe Cove shoreline.

The beach ridge at Arrowsmith Beach

A *beach ridge* extends northward from station C along the upper foreshore at Arrowsmith Beach (Plate I and Figure 3). Johnson (1919, p. 404) refers to any wave-built ridge on a prograding shoreline as a beach ridge. King (1959, p. 49) prefers to use the term only for shingle (gravel) ridges. The beach ridge at Arrowsmith Beach is an embankment of unconsolidated sand and gravel, essentially analogous to the upper-foreshore spit-bar at Horseshoe Cove, but it is formed on a retrograding stretch of shoreline bounded by head-lands at both ends. Where the beach ridge is covered by wind-deposited sediment, it is generally referred to as a dune ridge. Davies (1957) has summarized previous studies of origin and development of beach ridges. Importance of vegetation in the growth and stabilization of dune ridges is suggested by McKenzie (1958).

HORSESHOE COVE-ARROWSMITH BEACH SHORELINE

Topographic map of the Horseshoe Cove area

Data for a large-scale topographic map of Horseshoe Cove and the southern portion of Arrowsmith Beach were obtained by plane table and alidade survey during summer of 1959. Based on 744 topographic stadia sightings from survey stations, the map shown in Figure 3 was prepared on a scale of one inch to sixty feet (1:720) with one-foot contour interval.

Relatively gentle seaward slope of the foreshore at Horseshoe Cove steepens abruptly just below low-water line and descends to an organic-mud bottom at about minus 20 foot elevation as measured by SCUBA depth gauge. The organic-mud

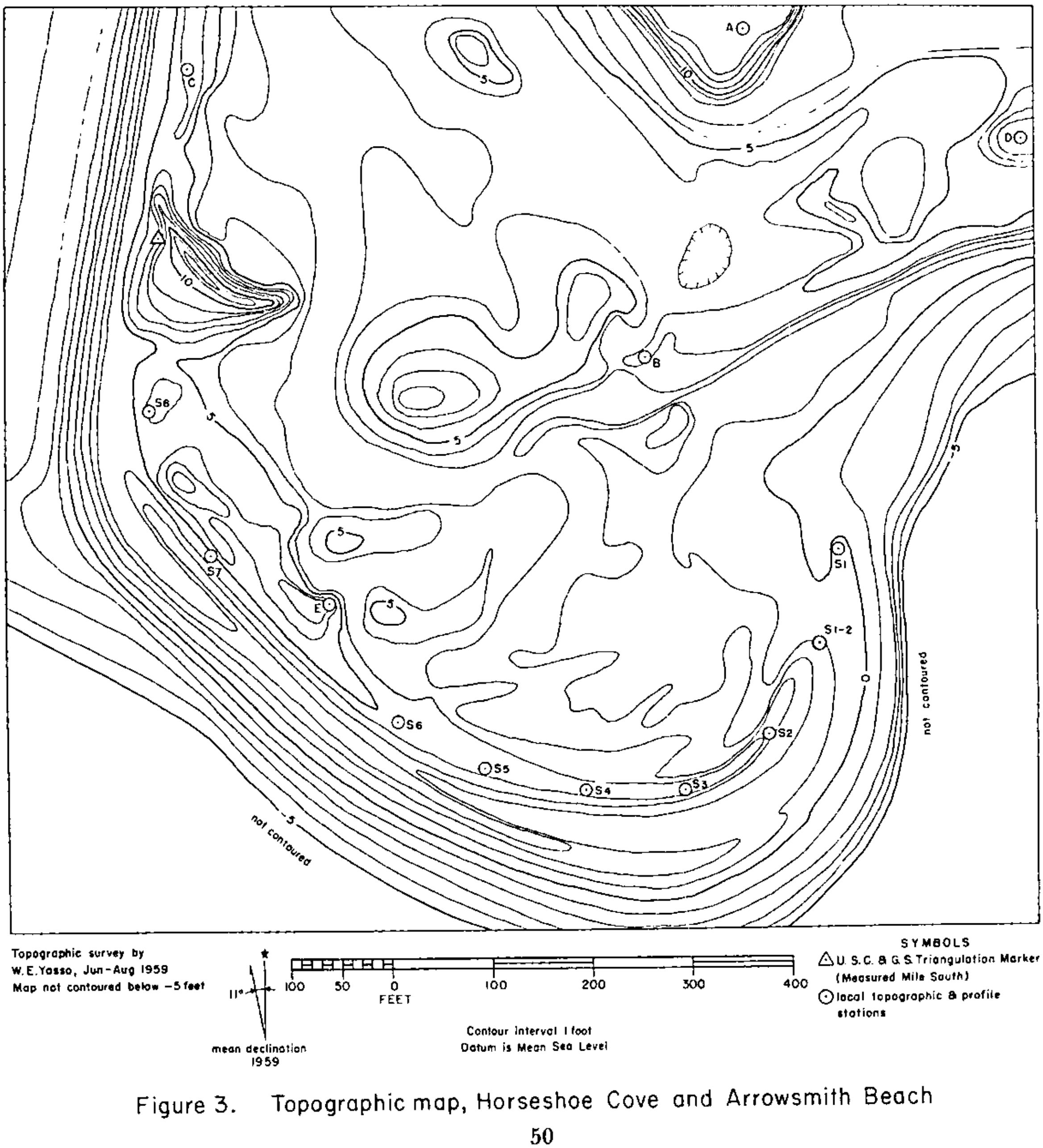


Figure 3. Topographic map, Horseshoe Cove and Arrowsmith Beach

TRUE NORTH

1000 0 1000 2000 FEET

Vertical aerial photograph of Arrowsmith Beach and Horseshoe Cove, 5 Oct 1952

YASSO, PLATE I

was probed to a depth of ten feet without finding a firm bottom. In contrast, the more steeply sloping Arrowsmith Beach foreshore descends to a sand and gravel nearshore bottom having a relatively gentle seaward slope starting at about minus 4 foot elevation. This broad, shallow, nearshore bottom extends from Arrowsmith Beach northward to the U. S. Coast Guard dock (Figure 1). A beach ridge, with crest elevation ranging from five to six and one-half feet above mean sea level, parallels Arrowsmith Beach contours between stations S8 and C.

Both upper- and lower-foreshore spit-bars are shown in Figure 3. Proximal terminus of the upper-foreshore spit-bar is continuous with the stabilized beach ridge that extends from station E northwest to station S8. The upper-foreshore spit-bar crest drops from five-foot elevation near station E to approximately mean-high-water contour (2.3 foot elevation) at its distal terminus between stations S1-2 and S2. Seaward-convex curvature of the upper-foreshore spit-bar can be seen in the trend of topographic contours and in the position of station markers placed along its crest in June, 1959.

The lower-foreshore spit-bar has a roughly rectilinear segment between station S6 and its proximal terminus at station S7. Seaward-convex planimetric curvature is present between station S6 and the distal terminus, but does not exactly parallel the curvature of the upper-foreshore spit-bar. Asymmetry of the lower-foreshore spit-bars and decreasing width of the trough between upper- and lower-foreshore spit-bars as proximal terminus is approached are also well portrayed in Figure 3. Three distinct landward (northerly) deflections of the two-foot contour in the tidal lagoon between stations S2 and S6 represent northeast-trending lagoonal spit-bars. Aeolian removal of fine sediment has left lag-gravel-covered embankments of low relief to mark positions of former upper-foreshore spit-bars. As shown by topographic map (Figure 2) and aerial photograph (Plate 1) of Horseshoe Cove, the shoreline plan is characteristic of that found elsewhere for compound recurved spit-bars.

Tide-created currents are credited with maximum geomorphic effect at inlets and mouths of estuaries. Because of relatively small discharge and relatively large channel width, tidal currents were not observed to have an effect on sediment transport or spit-bar planimetric curvature at Horseshoe Cove. In this respect Horseshoe Cove may represent a somewhat unique study area. It is generally agreed that tidal currents cannot in any case increase crest elevations of beach ridges; such building can only be accomplished by wave action (Gierloff-Emden, 1961, p. 83; Kidson, 1961, p. 17). But even if not important in itself as an agent of erosion and deposition the weak tidal current may have an additive effect on wave-induced longshore current acting in the same direction. Bagnold (1940, p. 28) calls attention to possible importance of the phenomenon. One of the first attempts at quantification of sediment transport relationships in such a combined stress field is reported from a model study conducted by Inman and Bowen (1963). Significance of the study to natural beach processes is not yet known.

The beach ridge north of station C (Arrowsmith Beach) is sparsely populated with grass *(Spartina)*, but no vegetation is found on spit-bars at Horseshoe Cove. In contrast, the ridge between stations E and S8 is heavily populated with *Spartina*, as is all of the low-lying area north of a line joining stations E, B, and S6. A thick growth of poison ivy *(Rhus radicans)* covers the double hillock just west of station B, the barchan-dune-like ridge midway between stations S8 and C, and the small hill midway between stations A and C. These hills appear to be of aeolian origin as indicated by cross-bedded, fine sands of which they are composed. Poison Ivy is a faculative halophyte that does not usually play such an important role in beach stabilization as appears to be the case at Sandy Hook (Davis, 1957, p. 10, A-9). Holly *(Ilex opaca)* and eastern red cedar *(Juniperus virginiana)* comprise the dominant tree population of older beach ridges at Sandy Hook.

Observation of spit-bar landward migration

An upper-foreshore spit-bar and one or two lower-foreshore spit-bars are usually found coexisting at any given time at Horeshoe Cove. (Complete nomenclature applied to topographic forms, as previously defined, is summarized in Figure 2). The steep, edge-trough slope of a spit-bar is primarily a deposit of sediment carried over the spit-bar crest by swash of waves breaking against seaward face of the spit-bar. At times when the height of tide is such that the swash can overtop the crest of either the upper- or lower-foreshore spit-bar, the swash meets standing water in lagoon and trough respectively. Sediment is quickly deposited as swash velocity is reduced by contact with the standing water. The depositional slope between edge and trough is thus created and extended landward by continued application of this swash mechanism. McKee and Sterrett (1961, p. 24-27) report similar growth process for bars formed in a wave tank and for natural bars at North Bimini Island, Bahamas. Landward migration of foreshore bars at Sapelo Island, Georgia, results from an identical sedimentation process (Hoyt, 1963, p. 311).

Magnitude of landward accretion on edge-trough slope of spit-bars at Horseshoe Cove can be seen by comparison of successive transverse profiles. A one-foot landward migration of the lower foreshoe spit-bar by slope accretion in a single tidal cycle is not uncommon. A two-foot landward migration of the lower-foreshore spit-bar within a single tidal cycle, measured at S5 profile line on 15 August 1961, was caused by waves of 1.0-foot maximum breaker height, 2.0-second period, and wave crest direction at 74° to the shoreline at S5. Flourescent tracer-particle experiment run on the S5 profile line at the same time (Yasso, 1962, p. 26-27, 44-45) showed that sediment deposited on the edge-trough slope was derived from the entire seaward face of the lower-foreshore spit-bar.

Sediment travels up the seaward face of the lower-foreshore spit-bar on every tidal cycle for which there is some component of wave energy directed normal to the shoreline. Usually this sediment will be smoothly distributed over the crest portion of the spit-bar or added to the edge-trough slope. But occasionally the sediment will accumulate as a distinct ridge (designated as lower-foreshore spit-bar II) on the foreshore somewhere below the crest of the pre-existing lower-foreshore spit-bar (designated by Roman I). Lower-foreshore spit-bar II may subsequently merge with its predecessor or may develop independently. In the latter case, lower-foreshore spit-bar I, if not driven landward to merge with the upper-foreshore spit-bar, will certainly be overriden when lower-foreshore spit-bar II has attained greater volume and relief.

Beach sediment along western shore of Sandy Hook

Foreshore sediment along the west-facing Sandy Hook Bay shoreline consists primarily of sand-size quartz particles. Fine-sand fractions represent a smaller percentage of this sediment compared with sediment of the Atlantic shoreline of Sandy Hook. Because heavy minerals are concentrated in the fine-sand size range, the small percentage of fines along the Sandy Hook Bay shoreline makes use of mineralogic layering as an indicator of erosion and accretion less practical than it would be on the Atlantic shoreline. Perhaps as a result of the continuous dredging in Sandy Hook channel, there is a lack of nourishment for beaches along the western shore of Sandy Hook. Absence of depositional topographic forms southwest of Sandy Hook channel and coarseness of beach sediment are lines of evidence indicating that little if any sand is carried around the north end of Sandy Hook.

Source and characteristics of spit-bar sediment

Arrowsmith Beach appears to be the sole source of sediment (feeder beach) for nourishment of foreshore spit-bars at Horseshoe Cove. The steep offshore slope and great depth to offshore bottom (with respect to depth of wave effects) seaward of spit-bars at Horseshoe Cove suggest that new sediment must move toward distal termini of these spit-bars exclusively by foreshore-zone transport processes. Strict limitations of sediment source to foreshore-zone transport from a well-defined feeder beach create an uniquely simple situation of great value for interpretation of spit-bar growth. Much of the transported sediment eventually comes to rest on the offshore slope at Horseshoe Cove, thus causing the persistent seaward progradation of the shoreline.

It is apparent from visual examination of surface materials that beach sediment generally decreases in coarseness toward distal termini of all foreshore spit-bars at Horseshoe Cove. This decrease might be expected, because intensity of wave action—and hence competence for transport—generally decreases in the distal direction. To investigate the apparent size-distance relationship, a sediment sampling program was begun during the 1961 field season.

Mechanical analyses of lower-foreshoe spit-bar crest sediment confirmed the visual estimate of decrease of coarseness in the distal direction (Yasso, 1964, p. 64). As an example of the size trend, geometric mean diameter was calculated for samples of sediment from the crest of the lower-foreshore spit-bar, taken along transverse profiles S3, S4, and S5 on 25, 29, and 30 August, and 7 September, 1961. Mean of the geometric mean diameters for S5 samples was 0.880 mm, for S4 samples it was 0.757 mm, and for S3 samples it was 0.697 mm.

SPIT-BAR TRANSVERSE PROFILES

Laboratory studies of beach profiles

Extensive studies of wave-formed beach profiles have been performed by U. S. Army, Beach Erosion Board. However, mathematical description of phenomena associated with shoaling and breaking waves is not sufficient for prediction of a resulting transverse profile even in non-tidal wave-tank experiments that direct uniform wave trains against the model beach (Eagleson *et al.*, 1961, p. 40-43). In a model study of transverse profiles created by waves of uniform height and period, Bagnold, (1940, p. 3, 38-39) found height of beach crest to coincide with height of swash. He was able to show that height of beach crest above a datum defined as elevation of wave trough in advance of the breaker bears a linear relation to deep-water wave-crest height above that datum for differing sediment sizes and wave-crest heights up to 30 cm. Tidal action was not introduced in the model. Foreshore of the model beach was consistently up-concave throughout the tests.

The only extensive laboratory study yet to come to the writer's attention, dealing with tidal effects on wave-formed beach profiles, was performed by Watts and Dearduff (1954). Profiles were developed in four model beaches having different sand sizes. Each beach was subjected to two different sets of waves acting over tidal ranges of 0.24 and 0.50 feet with one-hour and four-hour tidal periods. Form of transverse profile developed in the tidal environment was not materially different from that developed prior to introduction of tidal action. Rector (1954) produced a similar transverse profile form in a non-tidal study that used the same sand sizes and wave conditions employed by Watts and Dearduff. In no case is the characteristic Horseshoe Cove spit-bar profile form found in the model studies discussed above.

Profiling procedures

Local survey stations used for topographic mapping (Figure 3) and transverse profiling were marked by seven-foot, galvanized steel, channel fence posts driven six feet into the sand. All leveling to these stations was run by U. S. Geological Survey three-wire method using the Kern NK 3-C Precise Engineer's Level. Vertical closure error for level lines is well within the limits of second-order accuracy.

Almost all transverse profiles taken seaward from survey stations on the beach were made using K & E Engineer's Transit and stadia rod. Prior to Hurricane Donna (12 September 1960) transverse profiles were run with approximately five-foot horizontal spacing between rod stations. This distance was reduced to a three-foot horizontal spacing when it was realized that much more detail was needed for acurate drawing of topographic profiles on a scale of 1:120 horizontally and 1:12 vertically. Rod station interval of less than one foot horizontal distance was frequently used near crest portions of spit-bar profiles. Topographic profile data are within ± 0.003 foot accuracy.

Transverse Profile groupings

Well-formed lower-foreshore spit-bars (those possessing distinct crest, edge, and trough) show a systematic landward migration along the transverse profile line. Landward migration of lower-foreshore spit-bars results in formation of new upper-foreshore spit-bars which mark episodes in the seaward progradation of the Horseshoe Cove Shoreline.

Of fundamental importance to the study of shoreline progradation by spit-bar growth is an analysis of the way in which lower foreshore spit-bar growth influences and contributes to growth of new upper-foreshore spit-bars.

Best development of the typical spit-bar transverse profile is found between stations S3 and S6 at Horseshoe Cove. Of 92 detailed profiles run by transit survey in 1960, 41 were taken at stations S3 and S4. The few transverse profiles taken at station S5 in 1960 are shown in Figure 16. On the basis of the spit-bar growth pattern during 1960, revealed by transverse profiles, an additional 68 profiles were surveyed from stations S3A, S4A, S5, and S6 during 1961 field season (stations S3A, and S4A were established in 1961 thirty feet seaward of stations S3 and S4 respectively). Amount of time required by 0.5- to 3.0-foot spacing of rod stations along profile lines in 1961 seriously limited the number of profiles that could be obtained.

Groups of transverse profiles were selected from among the total of 109 profiles taken at the survey stations listed above. These groups were formed in order to investigate quantitatively certain aspects of spit-bar geometry resulting from landward migration of spit-bars. Each group of profiles contains all transverse profiles from a given survey station that were taken during landward migration of a particular lower-foreshore spit-bar. The earliest transverse profile in each group shows initial position of the lower-foreshore spit-bar. The most landward position reached by an individual lower-foreshore spit-bar is shown by the latest transverse profile in each group. Therefore, profile groups represent individual time sequences of lower-foreshore spit-bar development. As elsewhere throughout this report, transverse profiles are plotted with ten-times exaggeration of vertical scale. Where a group is composed of more than five transverse profiles these have been split between upper and lower portions of each figure. As each transverse profile was plotted from an average of 45 closely spaced rod stations, it was felt that plotting for maximum clarity would be justified even if the time sequence of a group had to be split between upper and lower portions of a figure. The left-hand border of Fgures 4, 5, 6, 7, 9, 11, and 12 represents zero horizontal distance which marks the survey station location.

Groups of transverse profiles were selected to show time sequences of lower-foreshore spit-bar development, viewed from each survey station, as follows: S3 group 1 and S4 group 1 profiles of 1960 (Figures 4 and 6) show spit-bar development prior to Hurricane Donna (12 September 1960), whereas corresponding group 2 profiles (S3 group 2 profiles are shown in Figure 5) represent post-hurricane development; S3A group 1 profiles of 1961 (Figure 7) portray development of lower-foreshore spit-bar I, while corresponding group 2 profiles (Figure 9) portray development of lower-foreshore spit-bar II, both before and after spit-bar I merged with the upper-foreshore spit-bar; exactly the same type of lower-foreshore spit-bar development found in S3A

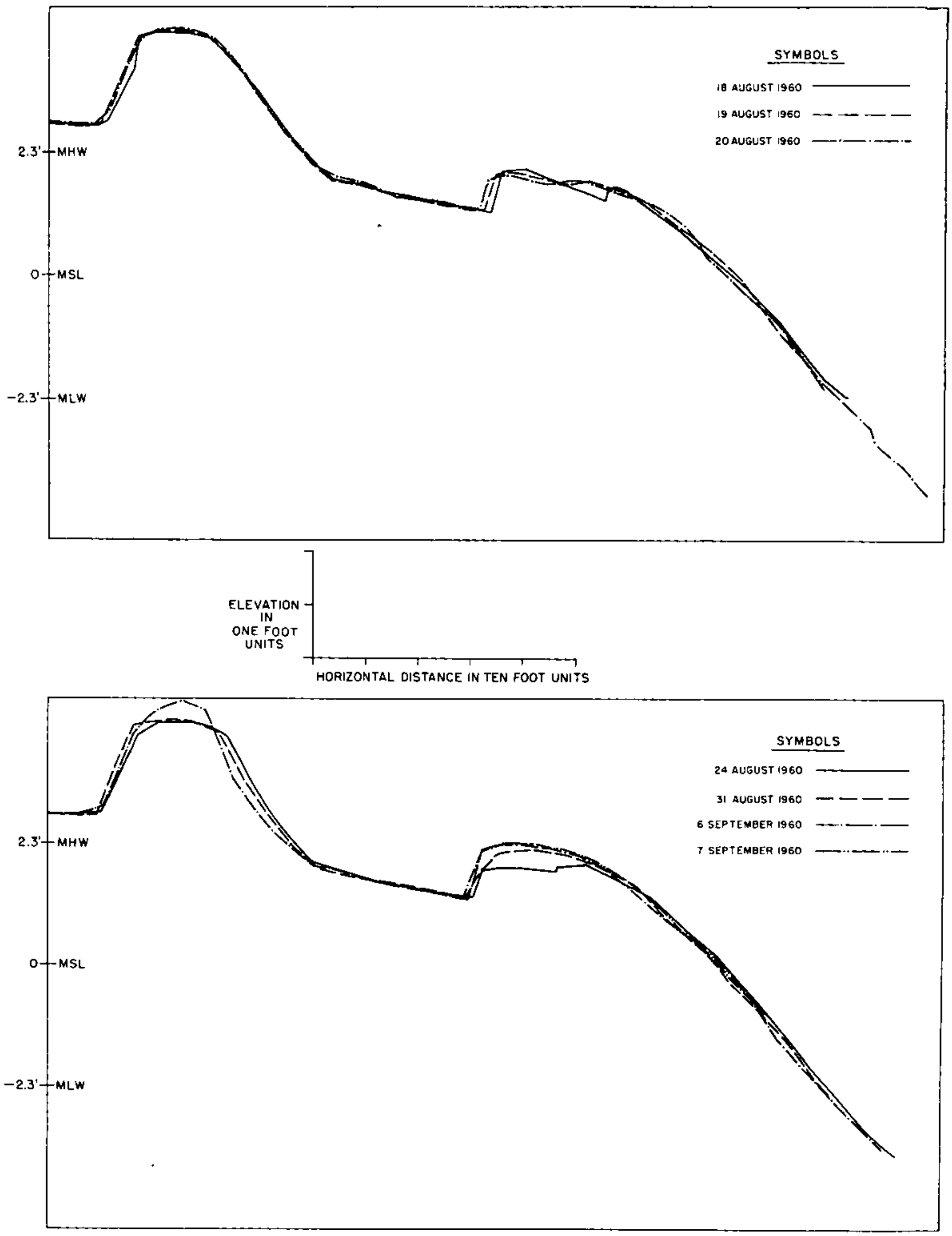
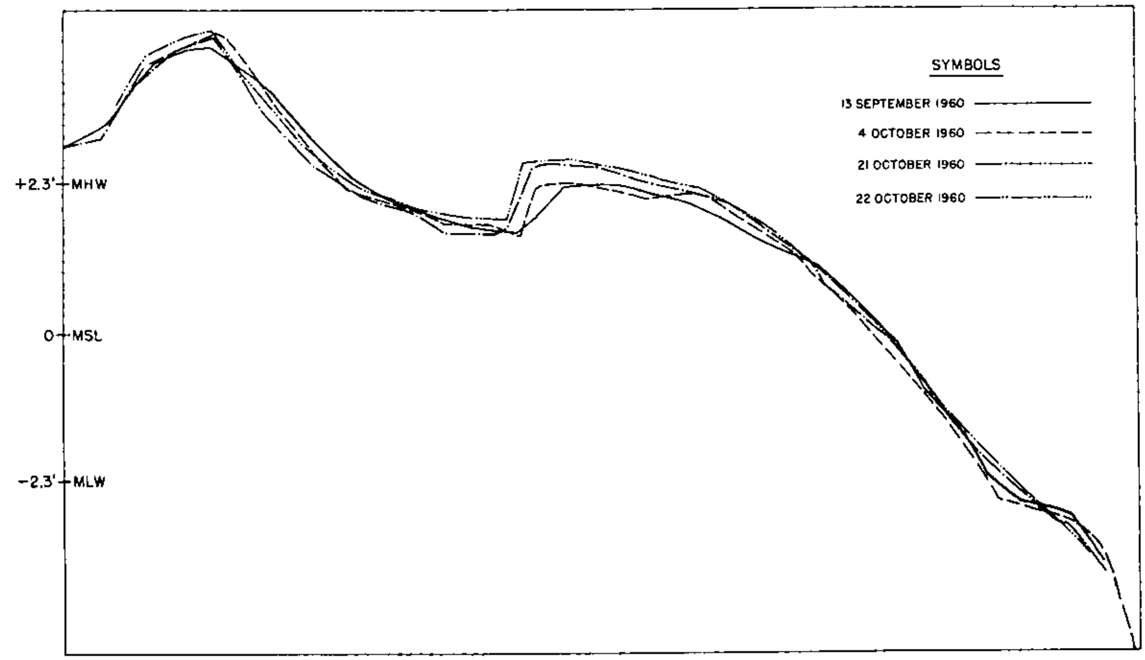
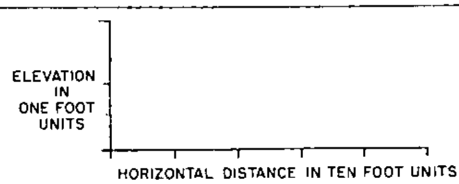


Figure 4 Profiles from station S3, 1960, group I





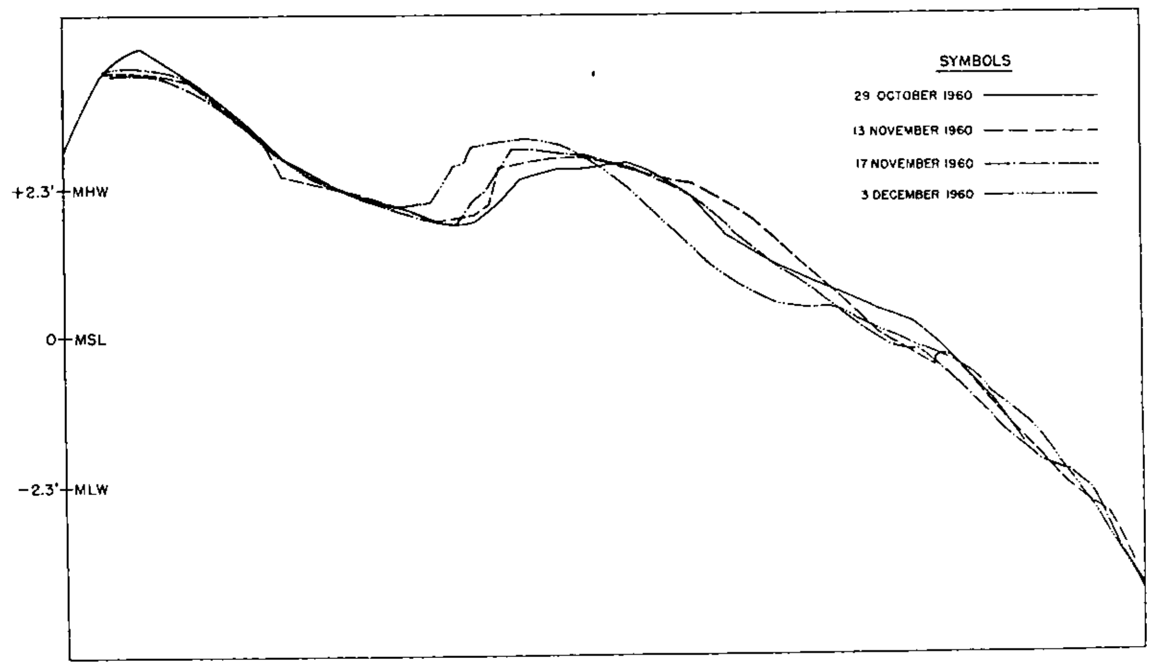


Figure5 Profiles from station S3, 1960, group 2

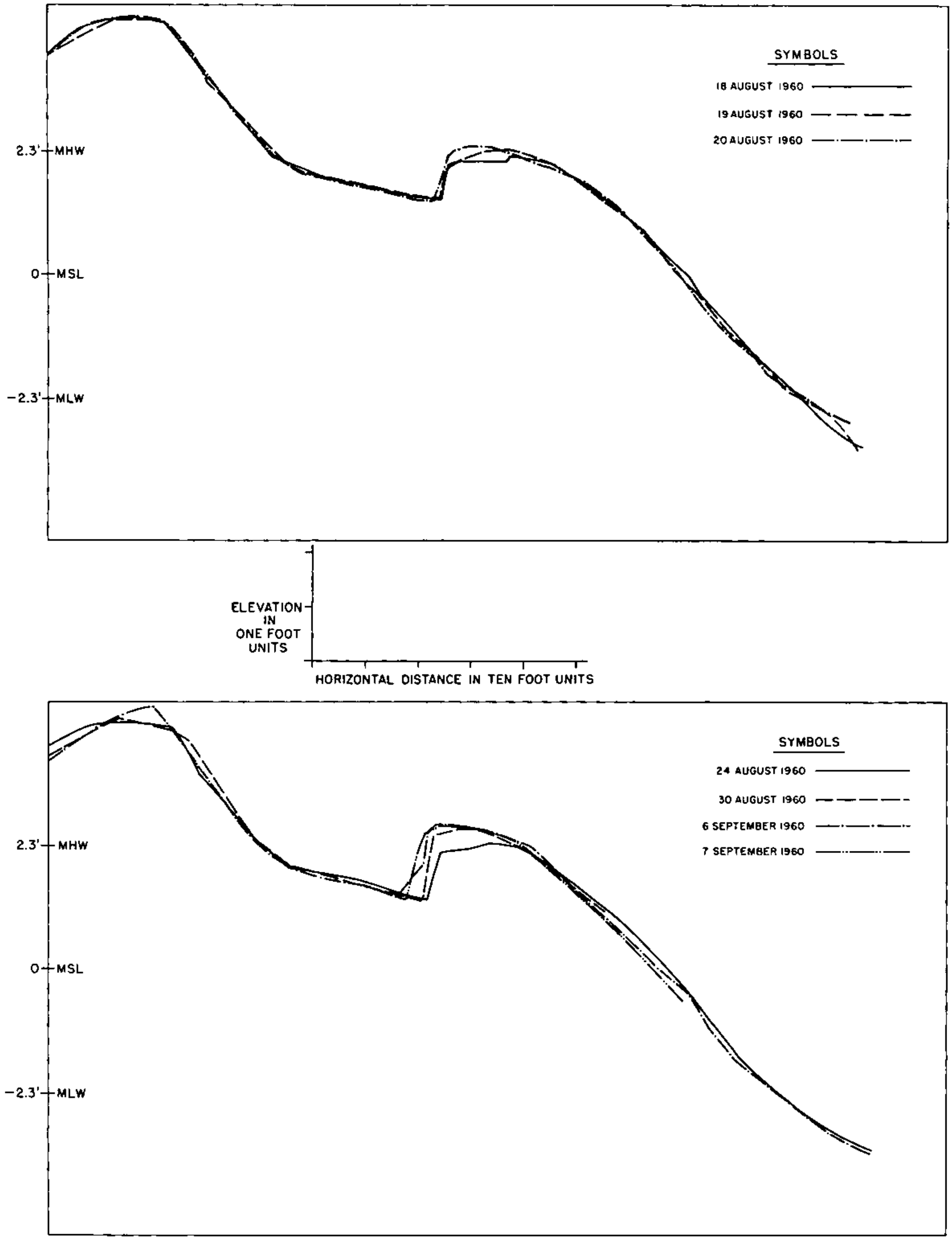


Figure 6 Profiles from station S4, 1960, group I

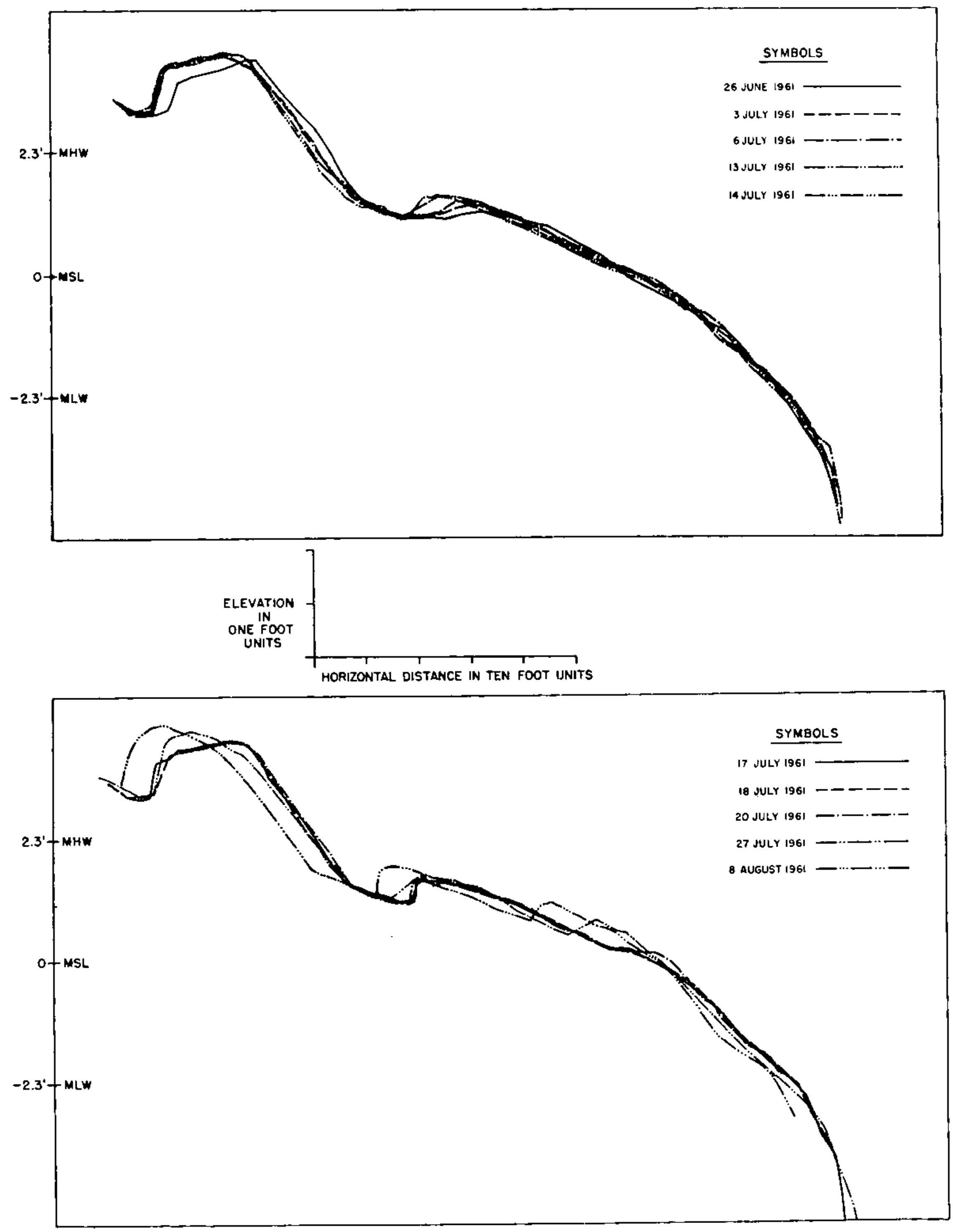


Figure 7 Profiles from station S3A, 1961, group I

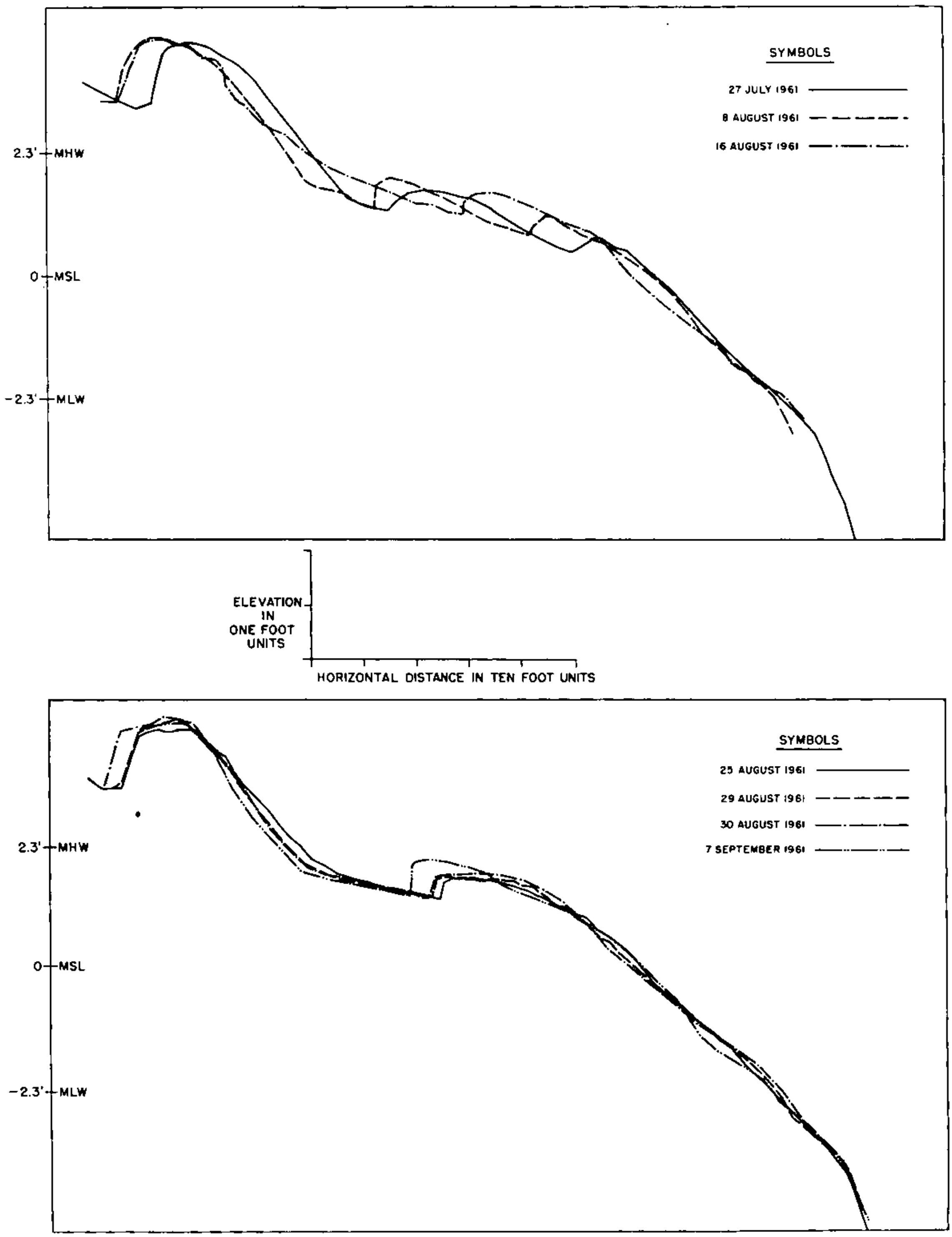


Figure 9 Profiles from station S3A, 1961, group 2

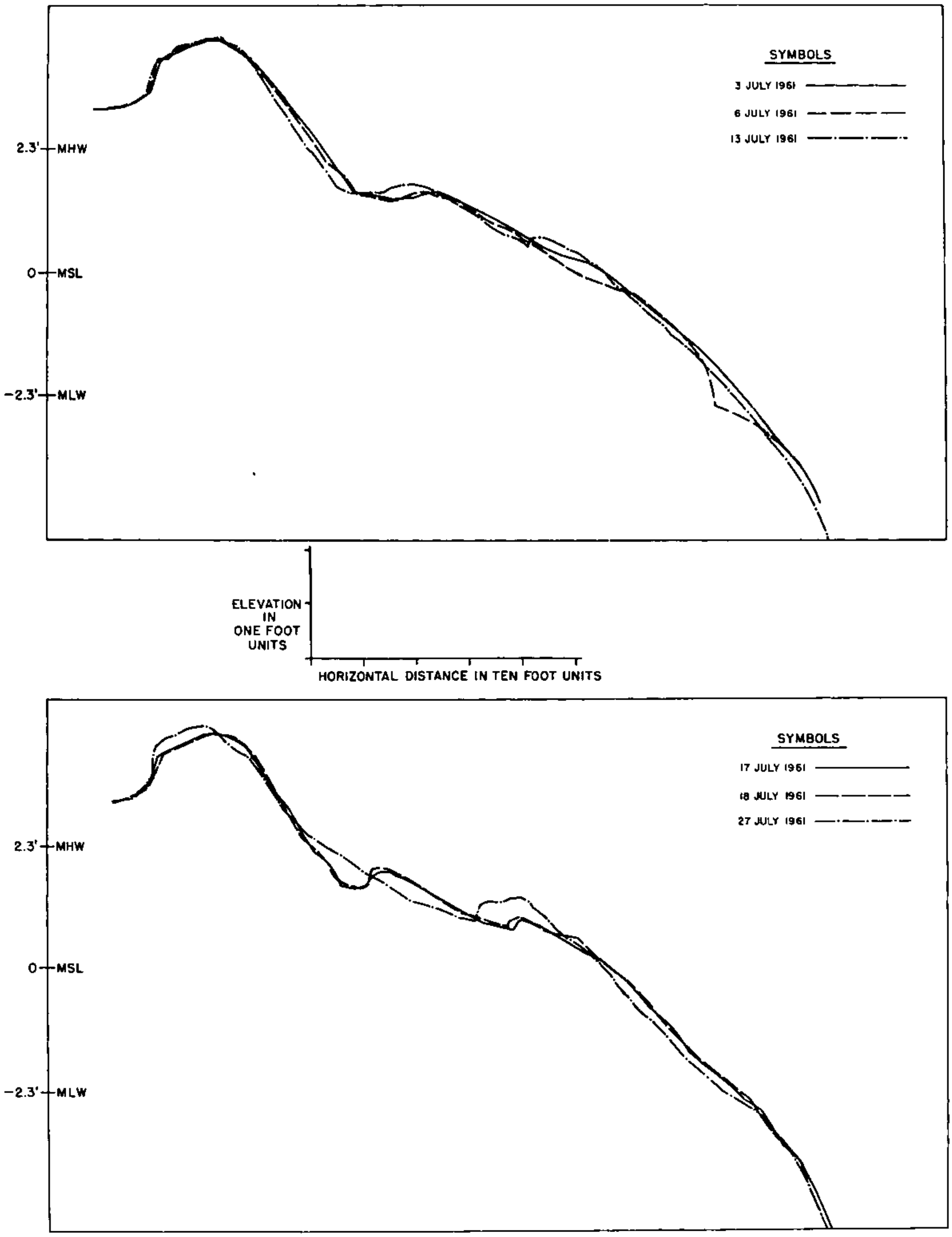


Figure 11 Profiles from station S4A, 1961, group I

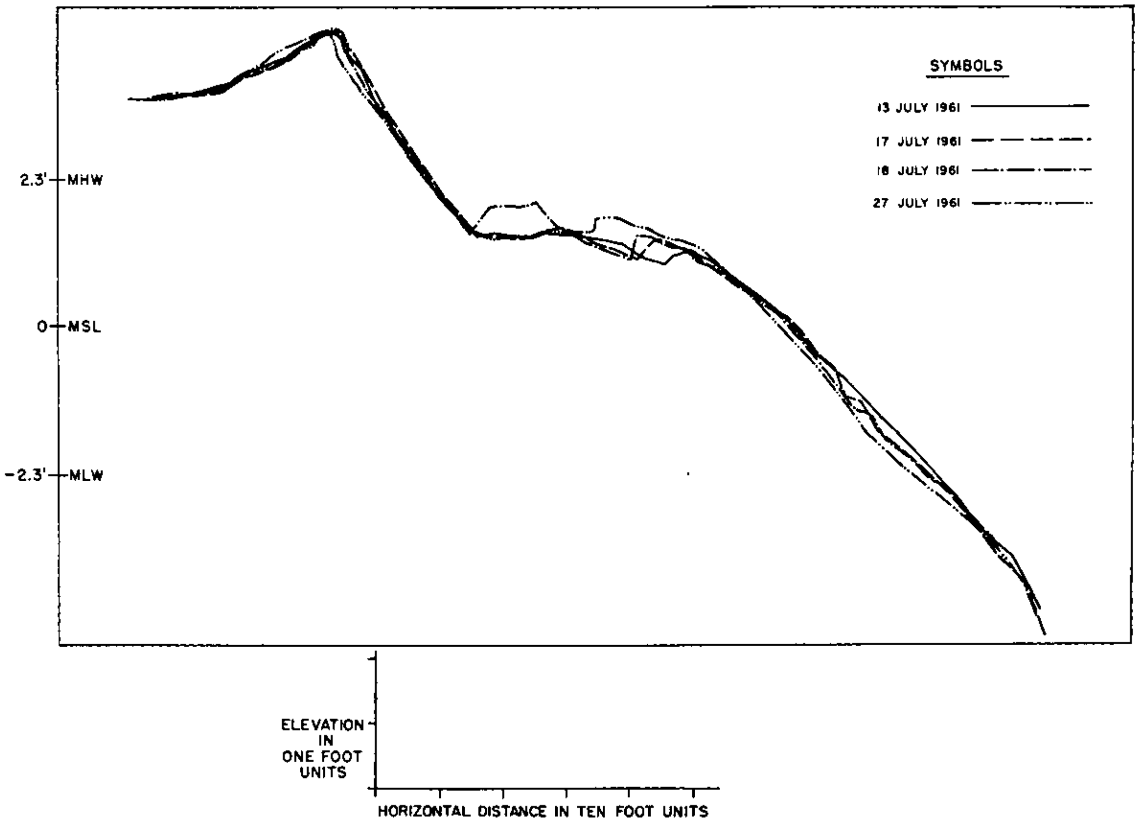


Figure 12 Profiles from station S5, 1961, group I

profiles is traced in S4A group 1 (Figure 11) and group 2 profiles of 1961; S5 group 1 profiles (Figure 12 show growth of lower-foreshore spit-bar II, while group 2 profiles trace development of a single lower-foreshore spit-bar that arose from spit-bar II after spit-bar I merged with the upper-foreshore spit-bar; S6 group 1 profiles of 1961 trace the development of lower-foreshore spit-bar I, while group 2 profiles (Figure 13) show development of lower-foreshore spit-bar II.

Before proceeding to a discussion of statistical tests applied to the grouped data, certain descriptive aspects of profile topography can be pointed out. Figures 4 and 6 show a broad, nearly flat base-to-trough region between upper- and lower-foreshore spit-bars. Its seaward slope is only one degree. Straightness and degree of slope are little affected by spit-bar development. Width of the base-to-trough region is decreased by landward migration of the lower-foreshore spit-bar and increased by landward migration of the upper-foreshore spit-bar.

In S3 group 1 profiles, the profile for 18 August 1960, (Figure 4) shows the crest of lower-foreshore spit-bar II lying 16.5 feet seaward of spit-bar crest I. By the following day all trace of spit-bar II was lost as a result of wave action that tended to smooth the profile. A different pattern of lower-foreshore spit-bar II behavior is shown by S3A group I profiles (Figure 9). Two lower-foreshore spit-bars were present on 27 July and 8 August 1961, but by 16 August the lower-foreshore spit-bar I had merged with the seaward face of the upper-foreshore spit-bar. Thereafter, a single lower-foreshore spit-bar developed on the nucleus of sediment representing the former lower-foreshore spit-bar II. As shown in corresponding profiles surveyed from station S4A

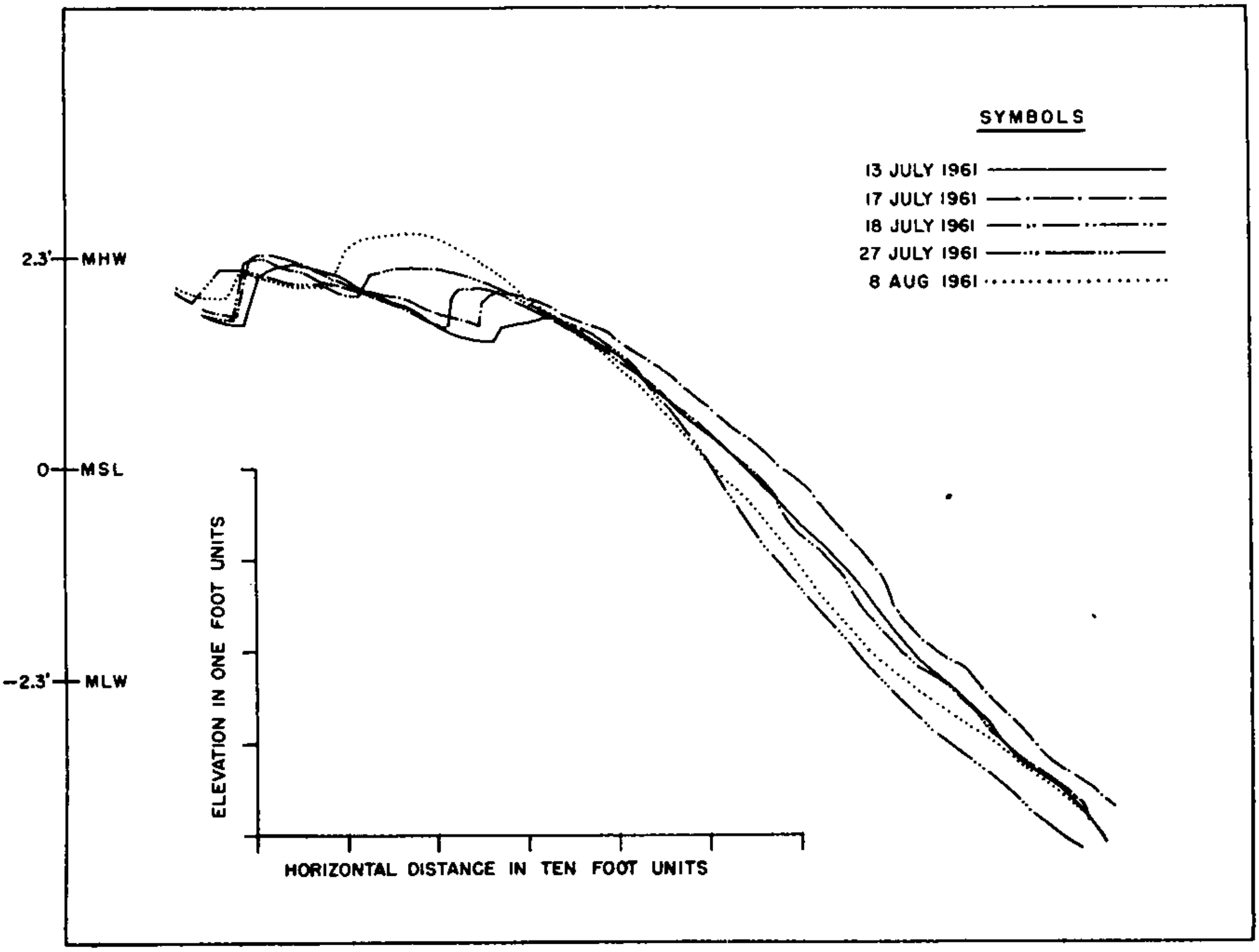


Figure 13 Profiles from station S6, 1961, group 2

(Figure 11), merging of lower-foreshore spit-bar I with the seaward face of the upper-foreshore spit-bar took place slightly earlier—on 27 July as opposed to 16 August—but the pattern of profile change is the same as outlined for station S3A.

The most common (typical) kind of upper-foreshore spit-bar transverse form is shown by S3A group 1 profiles for 1961 (Figure 7). In the profile of 3 July, slope of the seaward face of the upper-foreshore spit-bar was 7.5°, that of the landward face was 19.5°, and the gentle landward slope between spit-bar crest and edge (crest-to-edge slope) was 1°. Crest of the upper-foreshore spit-bar remained in proximity to the seaward face as landward migration took place between 26 June and 20 July. There is no noticeable change in spit-bar cross-sectional area during the migration. The upper-foreshore spit-bar shown in S5 group 1 profiles of 1961 (Figure 12) had similar transverse form to that seen in the S3A group 1 profiles for 1961, but erosion of the seaward face of this spit-bar was not accompanied by accretion on its landward face. The crest migrated landward and was reduced in elevation as cross-sectional area of the spit-bar was reduced by erosion of the seaward face.

A second, but less common, kind of upper-foreshore spit-bar transverse form is shown in S3 group 1 profiles for 1960 (Figure 4). Between 19 August and 24 August the relatively straight slopes of the seaward and shoreward faces of the spit-bar lead down from a moderately broad, slightly up-convex crest region. Slope of the seaward face was 8°; that of the shoreward face, 13°. By 24 August the seaward face had been eroded to a slightly up-concave transverse profile. Sediment eroded from the seaward face between 24 August and 31 August was deposited on the landward-facing edge-trough slope. Thus there occurred a net landward migration of the upper-foreshore spit-bar without appreciable change in transverse profile form and with less than 0.05 foot change in spit-bar crest elevation.

A third, but even less common, kind of upper-foreshore spit-bar transverse form is shown in S3A group 1 profiles for 1961 (Figure 7). Between 27 July and 8 August there occurred a profile change from the typical transverse form described above to one of a smooth up-convexity of crest and seaward face of the spit-bar. The latter form is quite similar to the transverse form usually shown by lower-foreshore spit-bars. As seen in later profiles from station S3A the more typical up-concavity in the seaward face of the spit-bar was re-established by 25 August (Figure 9).

Statistical analyses of transverse profiles

Each lower-foreshore spit-bar is potentially the nucleus of a new upper-foreshore spit-bar. It was hoped that criteria could be found for recognizing which lower-foreshore spit-bar was the precursor of the new upper-foreshore spit-bar and, of course, which lower-foreshore spit-bar was destined for removal from the profile. Validation of generalizations about spit-bar behavior would also aid in formulating a rational model for spit-bar development.

Special effort was made to pinpoint location of crest, edge, and trough of lower-foreshore spit-bars. These are three of the most obvious and easily located form elements of lower-foreshore spit-bar geometry. As a lower-foreshore spit-bar migrates landward and/or grows in size, the points marking location of crest, edge, and trough occupy a sequence of coordinate positions in the plane of the transverse profiles with respect to a given survey station.

Examples of the coordinate data of lower-foreshore spit-bar crest, edge, and trough positions used in statistical analysis are given in the lower portion of Figures 8, 10, and 14. Preliminary plots of elevation on distance for each of the form elements in a profile group seemed to show arithmetically linear trends throughout. Therefore, a simple linear relationship of elevation on distance for each form element in each profile group has been assumed to be the appropriate regression model. One of the available statistical procedures for measuring the degree of relationship between two variables is the simple linear correlation, also referred to as simple correlation, product moment correlation, and total correlation. Value of simple correlation and the computational procedures for determining simple correlation coefficient, r (dimensionless), are given by Croxton and Crowden (1941, p. 660-690), Dixon and Massey (1957, p. 198-204), and Steel and Torrie (1960, p. 183-193). Correlation coefficient can take all values between +1 and −1, where unity is spoken of as perfect correlation or completely linear interdependence between two variables. A value of zero for r indicates complete lack of linear interdependence between two variables.

Desk calculater time required for computation of statistical measures described below was prohibitively great. Thus, all operations were programmed for IBM 7090 computer. Analysis of relationship between elevation and horizontal distance for trend of all form elements in transverse profile groups yielded correlation coefficients ranging between +0.3294 and −0.9952 for trough positions, −0.0495 and −0.9988 for edge positions, and +0.4697 and −0.9952 for crest positions. These data are shown in Table 1. It will be noted that most values of r lie between −0.8 and −0.99, which indicates strong correlation between variables that was seen initially by visual examination of graphed relationships.

Again assuming linear relationship between two variables the straight line (regresion line) that best represents trend of the relationship can be found. The chosen line is usually that which makes the sum of squared vertical deviations of observations from that line smaller than corresponding sum of squared deviations from any other line. Using this least squares criterion, regression of elevation on distance for each set of data was calculated according to procedures given by Dixon and Massey (1957, p. 191-193) and Steel and Torrie (1960, p. 161-169). It was assumed that elevation was in the role of dependent variable; horizontal distance the independent variable. Equation of the best fitting straight line for each set of form element data is given in Table 1. Appropriate regression lines are plotted in figures 8, 10, and 14.

Standard error of estimate, $S_{y.x}$, provides a measure of the scatter of observations about a fitted regression line. $S_{y.x}$ has same units of measurement as the dependent variable, Y (elevation in feet). Small values of $S_{y.x}$ indicate small scatter of observations about the regression line. Discussion of $S_{y.x}$ and computational procedures are given by Dixon and Massey (1957, p. 191-192), Meiswanger (1956, p. 624-632), and Steel and Torrie (1960, p. 169-170). Tabulated values of $S_{y.x}$ indicate that scatter is usually least in regression of elevation on distance for lower-foreshore spit-bar trough positions in each group of transverse profiles.

As is indicated by the regression cœfficient in each regression equation, trend lines for positions of lower-foreshore spit-bar form elements have low slopes. These cœfficients are mostly negative with average value of 0.03320. Visual appraisal of the regression lines would perhaps lead to the conclusion that each form element occupies progressively higher elevations as it moves landward

TABLE I. SUMMARY OF STATISTICAL DATA RELATING TO TRANSVERSE GEOMETRY OF LOWER-FORESHORE SPIT-BARS

Profile line and year	*Profile Group*	*Form Element*	*Correlation Coefficient*	*Regression Equation*	*Standard Error of Estimate*	*Independence of Regression*	*Homogeneity of Regression Coefficients*
S3, 1960	1	Crest Positions	r = .4697	y = .03460 x −1.003	Sy.x = .1662	t = 1.190 N.S.	
		Edge Positions	r = −.0495	y = −.00527 x +2.383	Sy.x = .1619	t = − .1108 N.S.	F = 1.006 N.S.
		Trough Positions	r = −.8211	y = −.02476 x +3.223	Sy.x = .0270	t = − 3.217 *	
S3, 1960	2	Crest Positions	r = −.5943	y = −.02556 x +4.658	Sy.x = .2091	t = − 1.810 N.S.	
		Edge Positions	r = −.8231	y = −.04620 x +5.852	Sy.x = .1504	t = − 3.550 *	F = 1.149 N.S.
		Trough Positions	r = −.9764	y = −.02238 x +3.122	Sy.x = .0377	t = −10.10 * **	
S4, 1960	1	Crest Positions	r = −.9198	y = −.04207 x +5.856	Sy.x = .0841	t = − 5.243 * **	
		Edge Positions	r = −.8945	y = −.09200 x +9.027	Sy.x = .1033	t = − 4.475 * **	F = 13.06 * **
		Trough Positions	r = .3294	y = .00426 x +1.061	Sy.x = .0395	t = .7801 N.S.	
S4, 1960	2	Crest Positions	r = −.8815	y = −.04052 x +5.434	Sy.x = .1681	t = − 4.938 * **	
		Edge Positions	r = −.8073	y = −.02486 x +4.144	Sy.x = .1627	t = − 3.351 *	F = 1.887 N.S.
		Trough Positions	r = −.9952	y = −.03843 x +3.871	Sy.x = .0336	t = −27.07 * **	
S3A, 1961	1	Crest Positions	r = −.9371	y = −.02979 x +3.686	Sy.x = .0583	t = − 7.594 * **	
		Edge Positions	r = −.9872	y = −.02845 x +3.441	Sy.x = .0237	t = −17.47 * **	F = 7.158 * **
		Trough Positions	r = −.7646	y = −.01218 x +1.946	Sy.x = .0386	t = − 3.356 * **	
S3A, 1961	2	Crest Positions	r = −.9891	y = −.03954 x +4.837	Sy.x = .0640	t = −15.02 * **	
		Edge Positions	r = −.9902	y = −.03500 x +4.223	Sy.x = .0615	t = −15.86 * **	F = 3.845 *
		Trough Positions	r = −.9944	y = −.03078 x +3.526	Sy.x = .0372	t = −21.06 * **	
S4A, 1961	1	Crest Positions	r = −.9811	y = −.03287 x +3.844	Sy.x = .0310	t = − 8.776 * **	
		Edge Positions	r = −.9544	y = −.02993 x +3.596	Sy.x = .0461	t = − 5.539 *	F = 1.613 N.S.
		Trough Positions	r = −.9424	y = −.01944 x +2.609	Sy.x = .0218	t = − 4.882 *	
S4A, 1961	2	Crest Positions	r = −.9798	y = −.05042 x +5.478	Sy.x = .1582	t = −13.86 * **	
		Edge Positions	r = −.9976	y = −.04334 x +4.647	Sy.x = .0477	t = −38.18 * **	F = 7.248 * **
		Trough Positions	r = −.9835	y = −.03653 x +3.824	Sy.x = .1156	t = −15.38 * **	
S5, 1961	1	Crest Positions	r = −.9952	y = −.03918 x +5.044	Sy.x = .0232	t = −14.32 * **	
		Edge Positions	r = −.9988	y = −.04476 x +5.401	Sy.x = .0124	t = −28.41 * **	F = 5.073 N.S.
		Trough Positions	r = −.9750	y = −.02981 x +3.769	Sy.x = .0348	t = − 6.211 *	
S5, 1961	2	Crest Positions	r = −.8631	y = −.06059 x +7.188	Sy.x = .2409	t = − 4.187 * **	
		Edge Positions	r = −.9076	y = −.06261 x +6.736	Sy.x = .1778	t = − 5.294 * **	F = .1579 N.S.
		Trough Positions	r = −.9451	y = −.05245 x +5.008	Sy.x = .0931	t = − 7.084 * **	
S6, 1961	1	Crest Positions	r = −.7876	y = −.01604 x +3.084	Sy.x = .0775	t = − 2.214 N.S.	
		Edge Positions	r = −.8171	y = −.02914 x +3.632	Sy.x = .0838	t = − 2.455 N.S.	F = .9993 N.S.
		Trough Positions	r = −.9383	y = −.03198 x +3.214	Sy.x = .0374	t = − 4.699 *	
S6, 1961	2.	Crest Positions	r = −.9246	y = −.05038 x +5.821	Sy.x = .1353	t = − 4.202 *	
		Edge Positions	r = −.9572	y = −.04107 x +4.826	Sy.x = .0984	t = − 5.732 *	F = 2.733 N.S.
		Trough Positions	r = −.9629	y = −.02577 x +3.449	Sy.x = .0688	t = − 6.182 * **	

N.S. Indicates value of t or F is not significant at 5% level.
* Indicates value of t or F is significant at 5% level.
** Indicates value of t or F is significant at 1% level.

with spit-bar migration. On the other hand, the relatively few pairs of observational data, the considerable degree of scatter about regression lines, and low values of regression cœfficients make any statement about a general increase in elevation with landward migration open to question. It might be suggested that for all practical purposes regression lines were horizontal. Hence, there would be no real change in elevation with landward migration of a form element. A method for testing horizontality of regression lines, using the statistic t, is given by Dixon and Massey (1957, p. 196). Hypothesis to be tested, the null hypothesis, is that the variable Y (elevation) is independent of the variable X (horizontal distance). That is to say, the population regression coefficient, β, is equal to zero. A level of significance, α, is chosen. In geological problems α is usually taken as 0.05 (5 per cent) which means that the null hypothesis, if true, will be rejected in error for only 1 out of each 20 cases tested (Dixon and Massey, 1957, p. 91). Although α of 5 per cent was chosen as a reasonable level of significance, α of 1 per cent is frequently used as a more conservative criterion for accepting or rejecting the null hypothesis. Result of using α of 1 per cent is also given under the heading *independence of regression* in Table I.

Eleven of the twelve regression slopes for trough positions proved to be significant (population regression slope is not zero; the null hypothesis is rejected) at the 5 per cent level. Regression slopes for ten of the twelve edge lines and nine of the twelve crest lines proved significant at the 5 per cent level. Therefore, the writer accepts as real the characteristic increase in elevation during landward migration of form elements, as seen in the majority of cases tested.

Significance tests of individual regression coefficients do not shed light on an important question: are the regression lines for form elements in each group of profiles essentially parallel to each other, or do they diverge? Parallelism of regression lines would indicate constancy of topographic relief of a spit-bar and a geometric similarity in its development throughout landward migration. That is to say, a translation of the form in time without change in shape. Landward divergence of regression lines would indicate a relative change in relief as the spit-bar migrates landward and would include the possibility that a lower-foreshore spit-bar might continue to develop directly into a new upper-foreshore spit-bar.

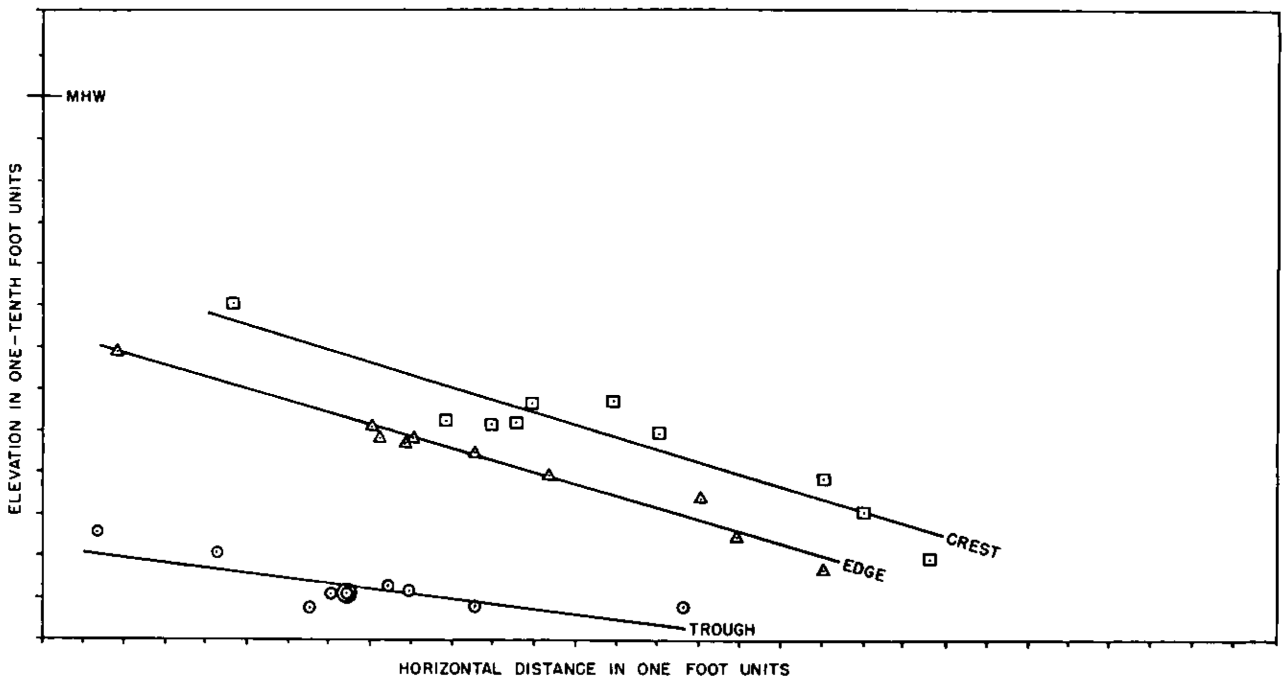


Figure 8 Regressions of elevation on distance for S3A profiles, 1961, group I

Parallelism of regression lines, referred to as homogeneity of regression, can be tested using the statistic F (Steel and Torrie, 1960, p. 173). A null hypothesis is stated that the population slopes of two or more regression lines are the same. Homogeneity of regression cœfficients is tested by analysis of covariance, for which the design of computational procedure is given by Steel and Torrie (1960, p. 319-320). The result of an F-test for homogene-

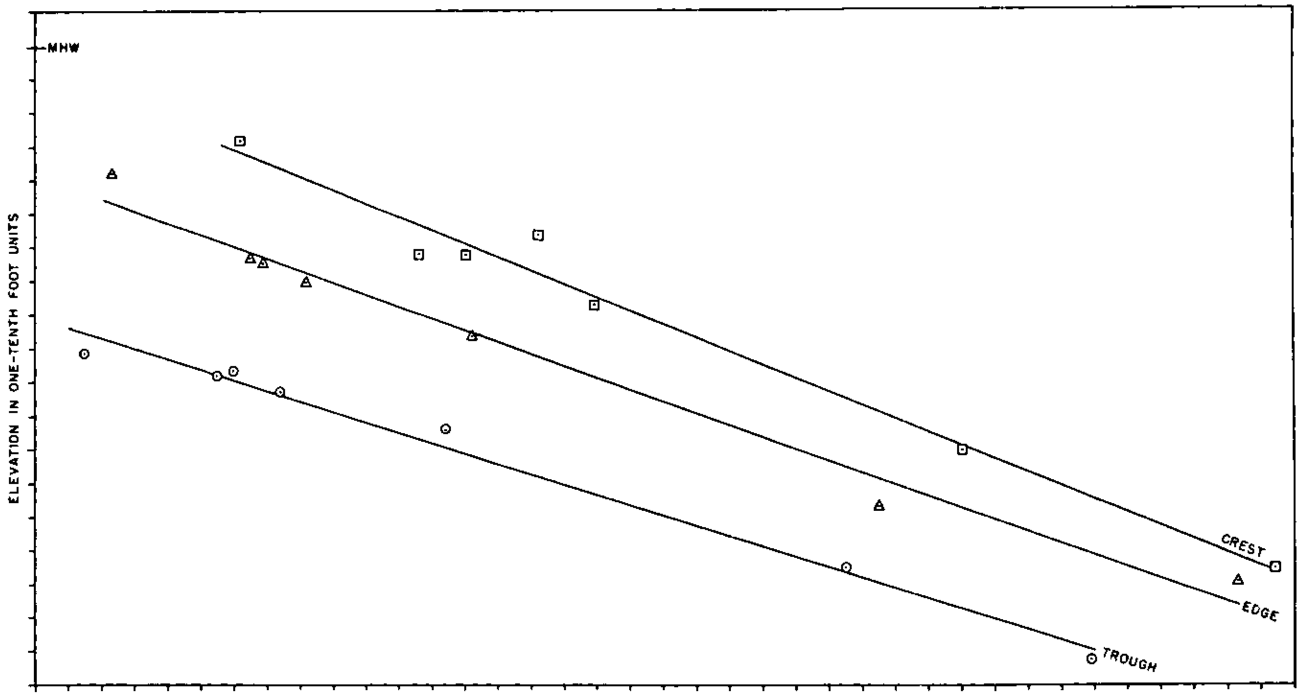


Figure 10 Regressions of elevation on distance for S3A profiles, 1961, group 2

ity of regression cœfficients in each group of transverse profiles is given in Table 1. A 5 per cent level of significance was used, but a statistical decision based on a 1 per cent level of significance is also indicated. Regression slopes are shown to be homogeneous (the null hypothesis is not rejected) within eight of the twelve profile groups tested. In addition, it is known from observation that no new upper-foreshore spit-bar grew directly from any of the four lower-foreshore spit-bars for which non-homogeneous regression slopes were found. Therefore, the well developed lower-foreshore spit-bars at Horseshoe Cove fail to comply with a diverging-slope criterion for development into upper-foreshore spit-bars. This aspect of spit-bar development is discussed further below.

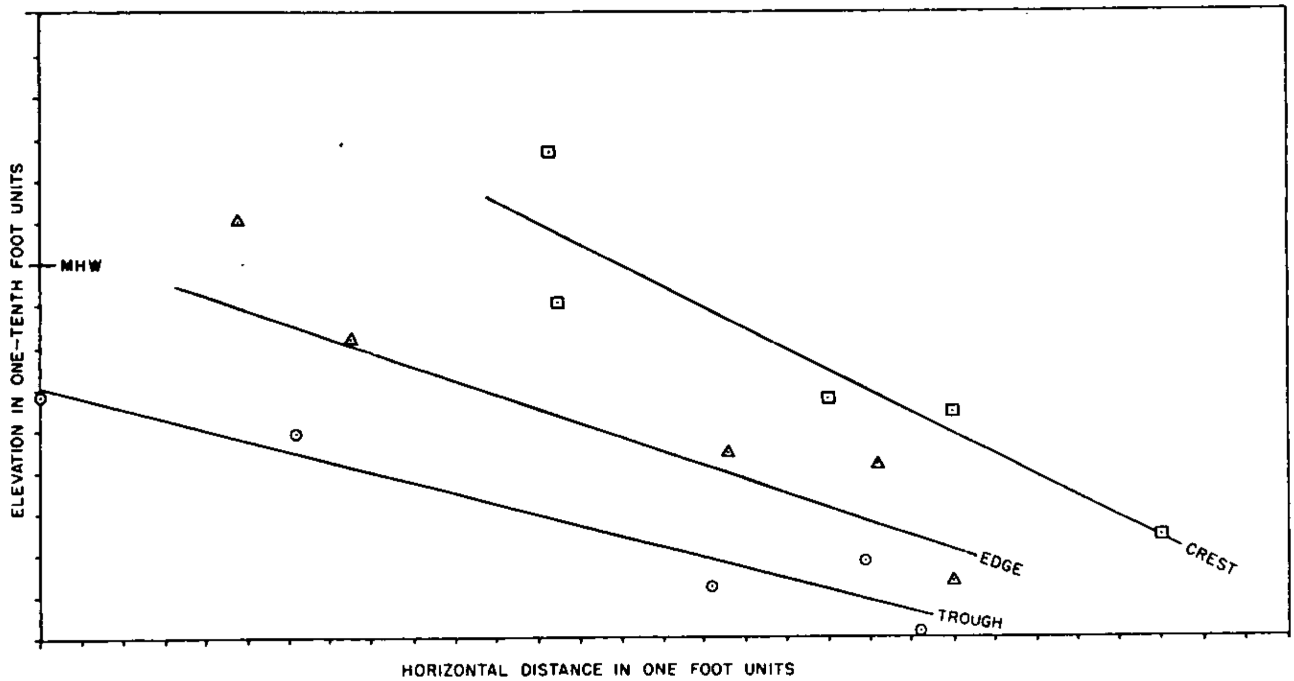


Figure 14 Regressions of elevation on distance for S6 profiles, 1961, group 2

Procedure for computing a confidence interval for the population regression cœfficient, β, is described by Steel and Torrie (1960, p. 169-171). In essence, even though there is a statistical basis for not rejecting geometric similarity in development of a majority of lower-foreshore spit-bars, the scatter of the data for each regression line is so large that an actual increase in spit-bar relief remains a physical possibility. Therefore, a greater emphasis is given to the complete record of transverse profiles rather than the results of statistical analyses. Over the period of record no individual lower-foreshore spit-bar was observed to be transformed into a new upper-foreshore spit-bar.

Composite transverse profiles

The greatest amount of transverse profile data bearing on life history of lagoonal-, upper-, and lower-foreshore spit-bars at Horseshoe Cove is represented by transverse profiles from stations S3, S3A, and S5. Selected transverse profiles from stations S3 and S3A are shown in Figure 15. The profile of 30 June 1960 shows two lower-foreshore spit-bars and an upper-foreshore spit-bar having the same transverse form as lower-foreshore spit-bars, and a lagoonal spit-bar of low relief lying landward of station S3. During the summer of 1960 the upper-foreshore spit-bar remained fairly stationary, while increasing in elevation and relief. Trend of increase in elevation and relief was temporarily reversed by waves from Hurricane Donna (12 September 1960). Recovery of trend is shown by profile of 4 October. By 13 November the upper-foreshore spit-bar had been driven landward, almost completely covering the lagoonal spit-bar. At the same time the upper-foreshore spit-bar was reduced in elevation and became almost symmetrically up-convex. Renewed seaward progradation must have occurred later in 1960 by accretion on the seaward face of the spit-bar. This accretion, also seen on profiles from station S5 (Figure 16), is inferred from height and shape of the trough landward of the 1961 upper-foreshore spit-bar, shown by profile of 3 June 1961. The

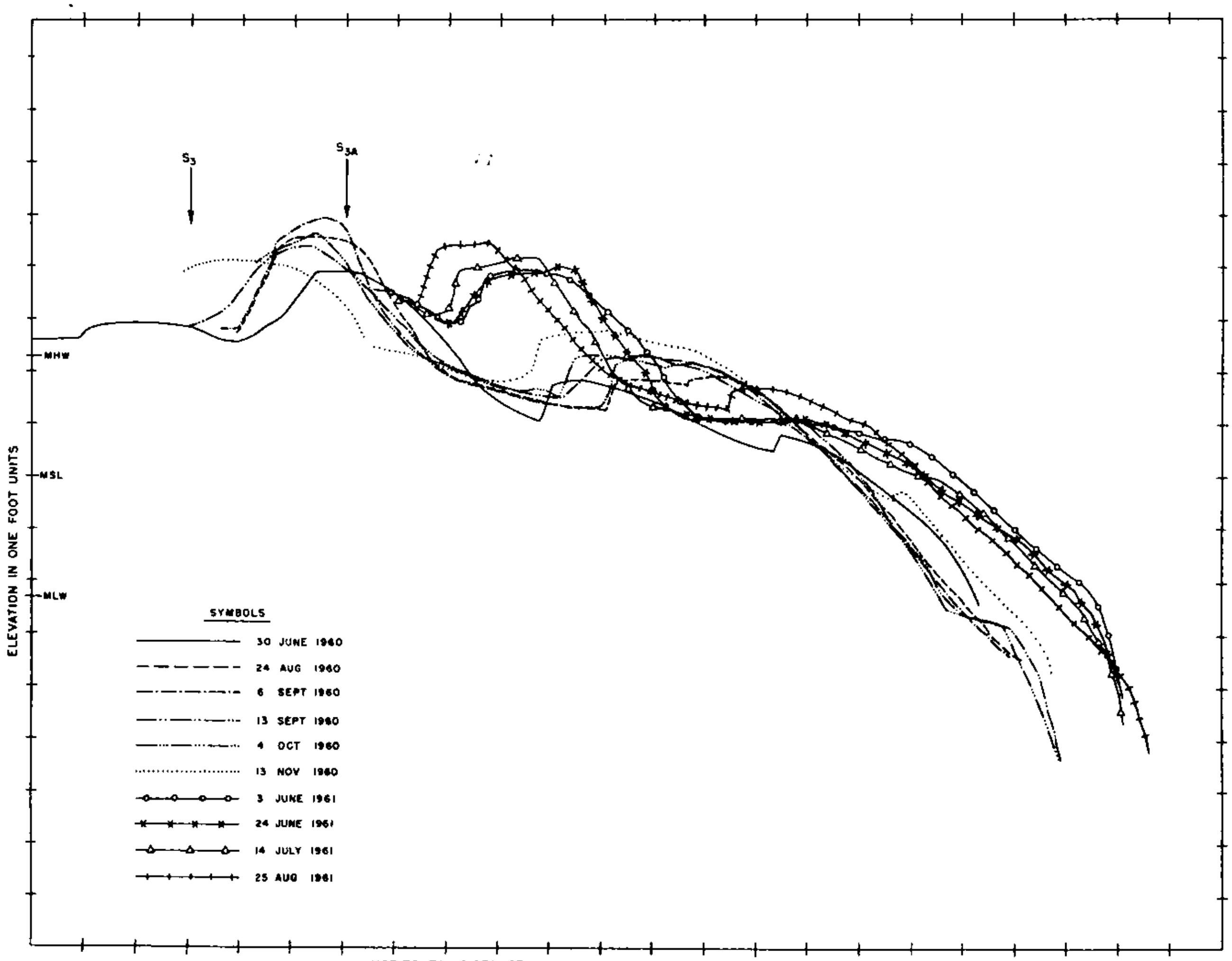


Figure 15 Representive profiles from stations S3 and S3A, 30 June 1960–25 August 1961

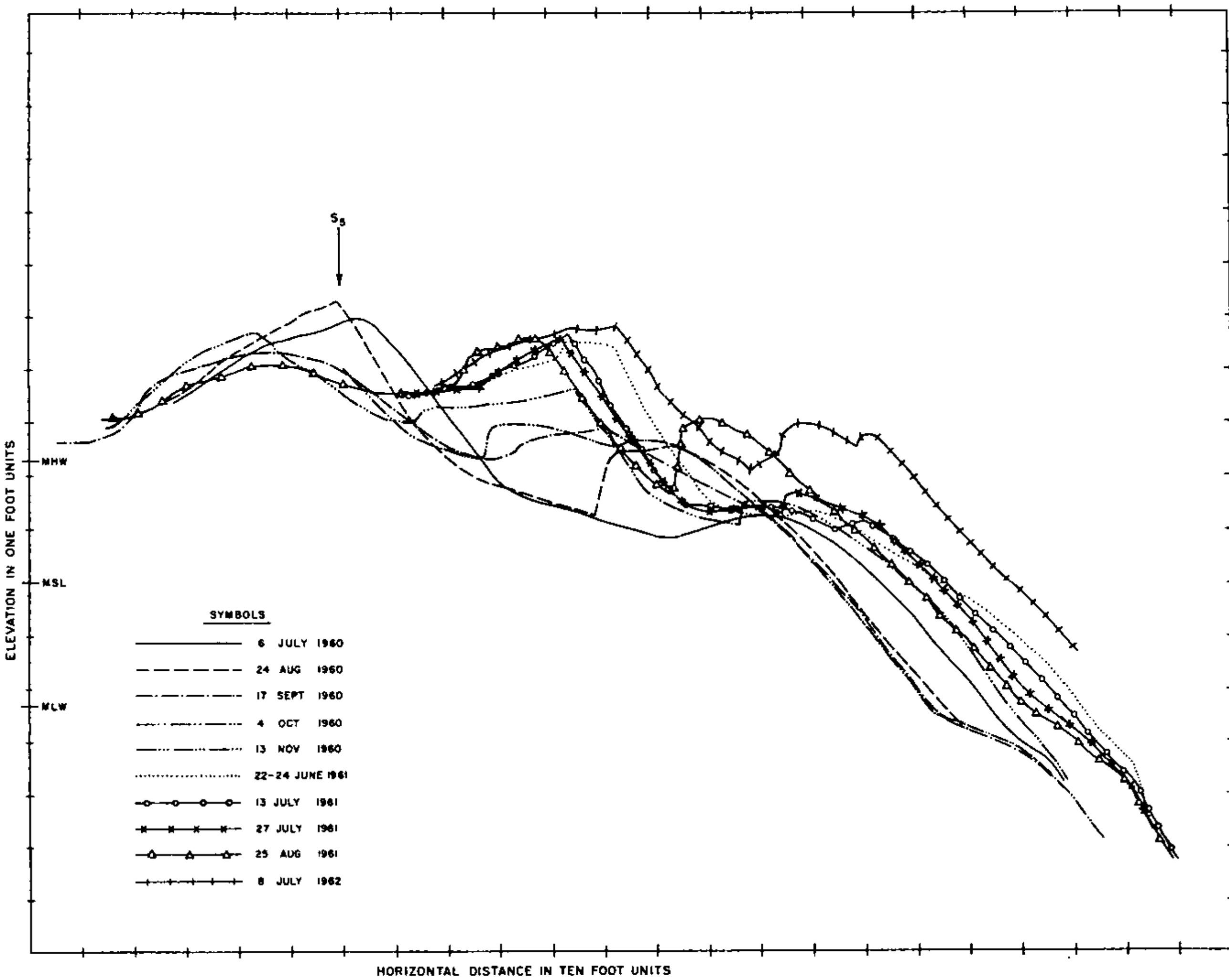


Figure 16 Representive profiles from station S5, 6 July 1960–8 July 1962

initial position of the 1961 upper-foreshore spit-bar was 15 feet landward of the 13 November 1960 position of the lower-foreshore spit-bar crest. Transverse form of the new upper-foreshore spit-bar was like that of a typical lower-foreshore spit-bar. An up-concave seaward face developed on the upper-foreshore spit-bar as its crest was built to mean-high-tide swash elevation.

Formation of a low-tide terrace and seaward progradation of the lower foreshore and offshore portions of the beach are an integral part of spit-bar growth. Annual seaward progradation of lower-foreshore and offshore portions of the beach between stations S3 and S6 amounts to about 15 feet. As shown by S3 group 2 profiles of 1960 (Figure 5) and composite profiles from station S3 shown in Figure 15, maximum development of the low-tide terrace took place between 13 September and 4 October 1960. By 13 November, accretion on the lower foreshore had reversed the trend of low-tide terrace formation. Continued accretion resulted in an 18-foot seaward translation of the mean-low-water contour between 13 November 1960, and 3 June 1961. Incipient formation of a low-tide terrace in 1961 is best shown by S3A group 2 profiles (Figure 9).

A composite of selected transverse profiles from survey station S5 for the period 6 July 1960, to 8 July 1962, is shown in Figure 16. The prevailing trend of increase in elevation and relief of upper-foreshore spit-bar was temporarily reversed by Hurricane Donna, but recovery of trend quickly followed return to usual tide range and wave action. A precursor of the 1961 upper-foreshore spit-bar developed by 13 November 1960. This form was built of sediment that earlier (4 October 1960) had comprised the lower-foreshore spit-bar. By 22 June 1961, the new upper-foreshore spit-bar had increased in elevation and relief as its seaward face underwent a 12-foot progradation. By 25 August 1961, retrogradation exactly canceled the previous progradation. Accretion and progradation took

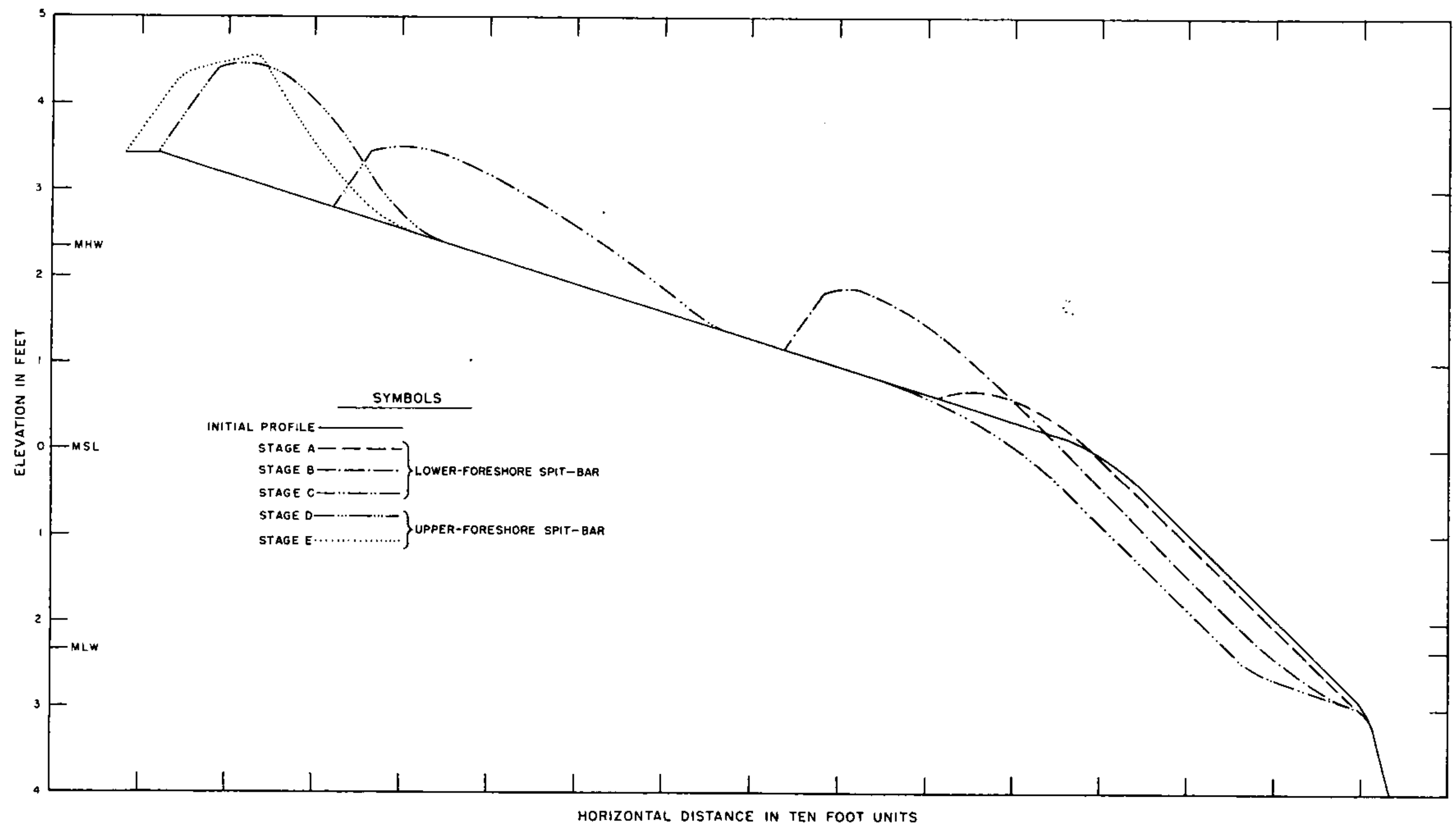


Figure 17 Idealized stages in development of typical Horseshoe Cove spit-bar profile

place on the entire profile between 25 August 1961, and 8 July 1962, but no new upper-foreshore spit-bar was created.

There was a strong tendency for creation of a low-tide terrace in 1960 as shown by S5 profiles of 6 July through 4 October 1960, (Figure 16). This tendency was also seen in the S3 profiles for 1960. By 22 June 1961, the mean-low-water contour stood 12 feet seaward of its former position. A weak tendency for creation of a low-tide terrace along the S5 profile in 1961 is shown in Figure 16. Accretion between 25 August 1961, and 8 July 1962, caused about the same seaward progradation (12 feet) as had been recorded in the winter of 1960-61. Composite profiles S4, S4A, and S6, not presented in this report, showed the same sequence of development as is outlined above. Growth of the low-tide terrace during the summer is considered evidence of at least an important landward component of wave energy. Inception of a new cycle of spit-bar building is signaled by seaward progradation of the Horseshoe Cove shoreline from supply of new sediment brought by shore drifting from Arrowsmith Beach.

Idealized spit-bar development

Generalized development of spit-bars at Horseshoe Cove is envisioned in Figure 17. This model is based on transverse profile relationships described above (Figures 15 and 16) and treated in statistical analysis of transverse profile groups. From a transverse profile initially devoid of spit-bars, the lower-foreshore spit-bar develops by idealized stages that eventually lead to creation of an upper-foreshore spit-bar.

The initial profile in Figure 17 shows a gently inclined, rectilinear foreshore slope between +3.5-foot elevation and mean sea level. The slope of this surface, about 2°, is roughly equivalent to the mean slope as determined by regressions of trough positions. A more steeply sloping portion of the foreshore profile begins at mean sea level and extends to –3-foot level on an average slope of 5°. A very much steeper offshore-zone slope begins at –3-foot level. The surveyed Horseshoe Cove transverse profiles included only the upper portion of this slope, from which an average slope of 22°, used in Figure 17, was obtained.

Stage A profile (Figure 17) shows that erosion of the lower foreshore has initiated formation of a low-tide terrace. Eroded sediment was transported landward to become the lower-foreshore spit-bar, which shows as a smoothly up-convex, edgeless form in stage A. Continued erosion of the lower foreshore results in expansion of the low-tide terrace. Eroded sediment is added to the spit-bar eventually producing stage B, in which the lower-foreshore spit-bar attains its characteristic up-convex transverse form with distinct crest, edge, and trough.

Note that in stage A of its development the lower-foreshore spit-bar lacks a steep, depositional slope between edge and trough. Bagnold (1941, p. 201-203) reports that barchan dune formation in a unidirectional wind-stress field begins with a slightly asymmetrical embankment of sediment whose transverse form is quite similar to spit-bar form shown in stage A (Figure 17). As the desert dune grows in elevation and relief, sediment from its windward face is driven over the dune crest to be deposited on a leeward repose slope, called the *slip face*. Transverse form of the sand dune is then similar to, perhaps basically identical with, spit-bar form shown in stage B. Formation of the edge-trough slope of the spit-bar by deposition of sediment carried over its crest in swash of waves is crudely analagous to formation of dune slip-face by saltation of sand. Maintenance of characteristic transverse form and cross-sectional area of both barchan dune and spit-bar, as these forms migrate in response to directional influence of wind and waves, can be accomplished only if there is constant supply of reworked sediment to the slip face and edge-trough slope respectively.

Erosion and accretion continue as the spit-bar is driven landward to a position shown by stage C in Figure 17. Constant topographic relief of the spit-bar, inferred from homogeneity of regression slopes of form elements in the majority of profile groups, results in geometric similarity of spit-bar development between stages B and C. When the first lower-foreshore spit-bar reaches a position shown by Stage C it remains relatively immobile as a succession of newly-formed lower-foreshore spit-bars migrates landward. Each of the new spit-bars maintains parallelism of regression slopes of form elements as it migrates landward and eventually adds its sediment to the growing embankment. When crest elevation and relief of this transitional spit-bar have increased such that waves can barely carry sediment over spit-bar edge, stage D has been reached. Thus, stage D represents initial form and position of the newly-created upper-foreshore spit-bar. Subsequent to this time, the seaward face of the spit-bar is eroded to produce the typical upper-foreshore spit-bar transverse form of stage E (Figure 17). From transverse profile evidence, stages D and E could take place within about 20 feet (horizontal distance) of the lower-foreshore spit-bar position at stage C.

A final analogy relating to spit-bar transverse form can be drawn from the literature. In non-tidal wave tank experiments, Keulegan (1948) was able to create off-shore-zone bars (submarine bars)

RECORD OF WIND VELOCITY AND DIRECTION
U.S.W.B. OFFICE THE BATTERY, NEW YORK
1911 TO 1957

Month	Mean Hourly Velocity	Prevailing Direction
JAN	16.4 MPH	NW
FEB	16.7	NW
MAR	17.1	NW
APR	15.2	NW
MAY	13.5	NW
JUN	12.8	S
JUL	12.1	S
AUG	11.7	S
SEP	12.4	N
OCT	13.9	NW
NOV	15.7	NW
DEC	16.2	NW
Yearly Mean	14.5	NW

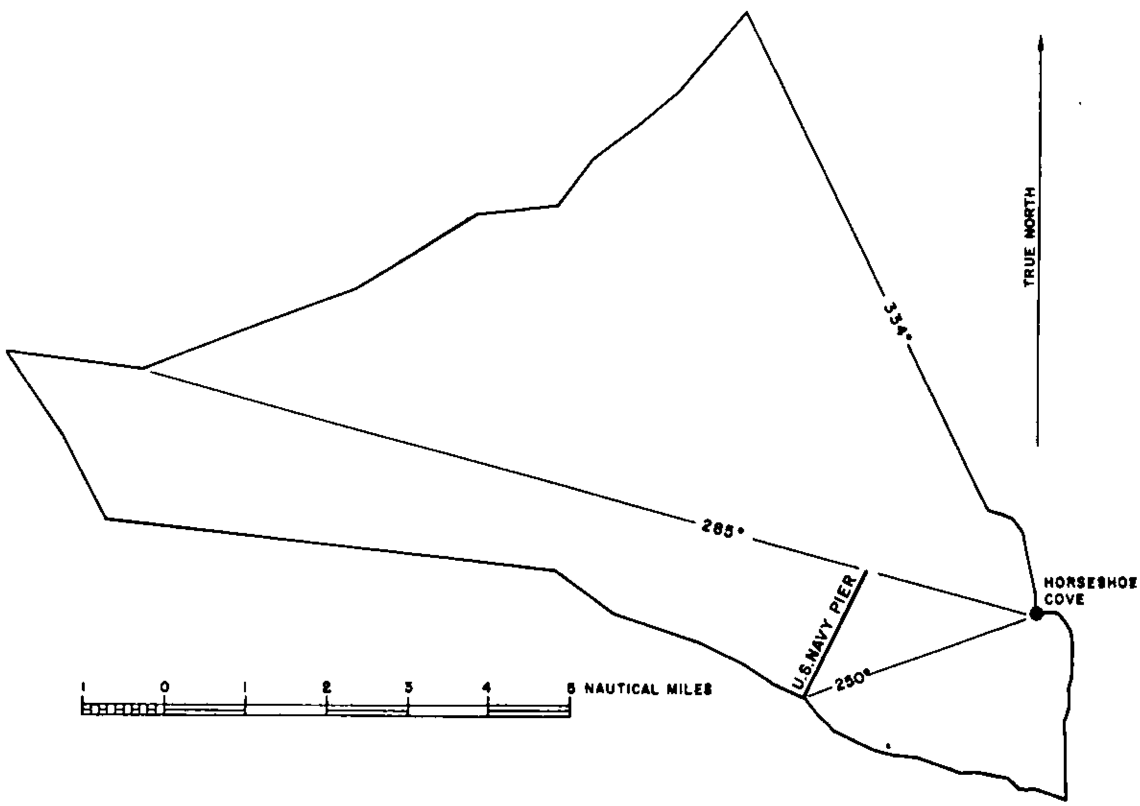


Figure 18 Fetch diagram and average wind conditions for Sandy Hook Bay

that compared favorably in form with natural bars formed in non-tidal waters. If difference in transverse profile landward and seaward of the wave-tank bar can be ignored for purposes of this discussion, it is noted that the wave-tank bar goes through a sequence of form changes below still-water level, almost identical with the sequence shown in Figure 17 stages A through D (Keulegan, 1948, p. 12). The laboratory study cited is one of the few in which other than final (steady-state) profiles have been reported from wave-tank experiments.

FETCH AND WIND REGIMES FOR SANDY HOOK BAY

Figure 18 shows a fetch diagram for Horseshoe Cove in conjunction with a table giving average wind velocity and direction at The Battery, New York, measured over a 57-year period. Six nautical miles separate Horseshoe Cove from The Battery. Battery data were chosen because the relatively great elevation and open exposure of wind recording instruments there (U. S. Weather Bureau) seemed to offer the best possibility of obtaining a wind record representative of conditions over the bay to the west of Sandy Hook.

Wind directions listed in Figure 18 correspond with wave directions already described as responsible for spit-bar growth and shoreline progradation at Horseshoe Cove accompanying retrogradation of the Arrowsmith Beach shoreline. Local wind-generated waves from the northwest striking the Arrowsmith Beach shoreline obliquely, dominate the regime from October to May. Waves from the south strike the Horseshoe Cove shoreline most directly during summer months. Wave height and period are a direct function of fetch distance, wind velocity, and duration of wind blowing from a given direction. Assuming that duration of wind may be taken as constant for any wind direction, the fetch diagram shows that average waves from the northwest will be much larger and of longer period than waves generated by winds from any other direction. Deep-water conditions (relative to wave length) are most nearly approached in the sector between 285° and 334° true azimuth. Such a condition is favorable to formation of large waves by minimizing bottom friction. U. S. Navy pier located in sector between 250° and 285° probably interferes with wave generation in that sector, which is one of rapidly shortening fetch.

SUMMARY OF HORSESHOE COVE INVESTIGATION

Because only local wind-generated waves are effective in causing beach changes along the Sandy Hook Bay shoreline, the Horseshoe Cove-Arrowsmith Beach area offers an uniquely simplified environment for study of foreshore spit-bar growth. The foreshore zone is bounded below by the mean-low-water contour and above by the upper limit of swash at mean high water. Foreshore spit-bars, growing within this zone, are ridges of non-cohesive, clastic debris attached to a headland feeder beach at the proximal terminus and extending to a distal terminus in deep water. As a result of wave refraction, planimetric curvature of spit-bars is convex toward open water. Lower-foreshore spit-bars present a smoothly up-convex transverse profile outline of the seaward face that changes abruptly in the landward direction into a steeply descending depositional slope between spit-bar edge and trough. This depositional edge-trough slope is built of sediment eroded from seaward face of the spit-bar by swash of waves. When sediment-laden swash passes landward of spit-bar crest, it encounters standing water in the trough; swash velocity is checked and sediment load is deposited on the edge-trough slope. The lower-foreshore spit-bar migrates landward by this accretional mechanism. This maximum of record during a single tidal period was a two-foot landward migration of the edge-trough slope. A fluorescent tracer-particle experiment conducted at the same time established that sediment added to edge-trough slope was derived from entire seaward face of the spit-bar. Measured decrease in mean diameter of lower-foreshore spit-bar crest sediment toward the distal terminus is consistent with wave processes causing spit-bar development.

Lower-foreshore spit-bars are formed and driven landward primarily by small summer-waves of low steepness until a ridge, representing sediment accumulated from many lower-foreshore spit-bars, is built to an elevation where swash at mean-high-tide can no longer transport sediment over the crest. The resulting ridge is called an upper-foreshore spit-bar. Seaward face of this spit-bar is sharply sigmoidal in transverse profile. Further landward migration is slight except under conditions of extreme tide and high waves. When the new upper-foreshore spit-bar is formed the previous upper-foreshore spit-bar, no longer subject to wave attack, eventually becomes a broadly convex ridge of low relief. Flattening and lowering and primarily caused by deflational removal of the spit-bar sediment. Because this relict spit-bar is located in the tidal lagoon landward of younger, actively growing spit-bars it is called a lagoonal spit-bar.

Examination of all transverse profiles taken at Horseshoe Cove survey stations reveals that upper-foreshore spit-bars have greater stability in position and cross-sectional area than lower-foreshore spit-bars. In a sense, the upper-foreshore spit-bar represents an equilibrium form intermediate in evolutionary stage between the lower-foreshore spit-bar, growing by sporadic accretion of sediment eroded from Arrowsmith Beach, and that of the lagoonal spit-bar, being reduced primarily by deflational processes.

Regression analyses of geometric elements of lower-foreshore spit-bar transverse profiles showed that no single lower-foreshore spit-bar can accurately be described as the progenitor of the next upper-foreshore spit-bar to be formed. An upper-foreshore spit-bar grows from a nucleus of sediment contributed by many lower-foreshore spit-bars, each of which maintains geometric similarity in transverse shape while migrating landward on the beach.

The annual cycle of spit-bar formation is seen as caused by an annual cycle of prevailing wind blowing over Sandy Hook Bay. Average fall, winter, and spring waves acting from the northwest cause shore-drift transport of sediment from the Arrowsmith feeder beach southward to Horseshoe Cove, where deposition on foreshore and offshore slopes results in seaward progradation of the spit-bar shoreline. In summer, small, low waves from the south rework this sediment into a series of lower-foreshore spit-bars which are pushed landward on the foreshore until they coalesce and emerge as a new upper-foreshore spit-bar.

IMPLICATIONS OF THE STUDY

Similar geomorphic features to those studied at Horseshoe Cove have been reported by other investigators. Zenkovich (1967, p. 271) describes a similar profile to that found at Horseshoe Cove as being produced wherever there is an open expanse of lagoon, a river mouth, or a flat landward of the beach. Dolan and Ferm (1966) in a study along the Outer Banks of North Carolina found that foreshore bars formed under low energy wave conditions (highest ⅓ of waves in deep water having height between 1 and 2 feet) but were absent from the transverse profile during higher energy wave conditions. Therefore, it is to be expected that foreshore bars, as the general case of the foreshore spit-bars described in this report, are a more common feature of the beach profile than has been suggested in the literature.

Foreshore bars with the same transverse profile as those at Horseshoe Cove have been observed by the writer at Virginia Beach, Virginia; Lloyd Neck, Long Island; and during many occasions at Kingmill Beach, Sandy Hook. They are considered by the writer to originate in the landward movement of submarine (offshore-zone) sand bars during summer-wave, or equivalent long-period wave conditions, on exposed beaches. If not destroyed by other types of waves, the foreshore bar will migrate upward on the beach until its crest is at the limit of high-water swash. At such time its transverse shape changes because of the loss of the steep, landward depositional slope as the foreshore bar becomes one of a series of summer berms on a prograding shoreline. During summer-wave conditions, many foreshore bars add their mass to the enlarging berm. Foreshore erosion that may temporarily interrupt summer-berm building may cause partial or complete removal of the up-convex shape of the developing berm. The resulting berm may then show the sharp crest and steeper seaward slope than landward slope considered typical of beach berms. A similar developmental sequence to the one described is suggested in the series of profiles surveyed by Strahler (1966, p. 254) at Spermaceti station, Sandy Hook.

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REFERENCES CITED

BAGNOLD, R. A., 1940, Beach formation by waves; some model experiments in a wave tank. Institution Civil Engineers Jour., v. 15.

———, 1941, The physics of blown sand and desert dunes. Methuen and Co., London.

CORNISH, V., 1898, On sea-beaches and sandbanks. Geogr. Jour., v. 11.

CROXTON, F. E., and CROWDEN, D. J., 1941, Applied general statistics. Prentice-Hall, Inc., New York.

DAVIES, J. L., 1957, The importance of cut and fill in the development of sand beach ridges. Australian Jour. Sci., v. 20.

DAVIES, J. H., 1957, Dune formation and stabilization by vegetation and plantings. U. S. Army, Beach Erosion Board Tech. Mem. No. 101.

DIXON, W. J., and MASSEY, F. J. JR., 1957, Introduction to statistical analysis. 2nd ed., McGraw-Hill Book Co., Inc., New York.

DOLAN, R., and FERM, J., 1966, Swash processes and beach characteristics. Professional Geographer, v. 18.

EAGLESON, P. S., GLENNE, B., and DRACUP, J. A., 1961 Equilibrium characteristics of sand beaches in the offshore zone. U. S. Army, Beach Erosion Board Tech. Mem. No. 126.

EVANS, O. F., 1942, The origin of spits, bars, and related structures. Jour. Geology, v. 50.

GIERLOFF-EMDEN, H. G., 1961 Nehrungen and lagunen: Petermanns Geographischen Mitteilungen, 2. Quart.

GRESSWELL, R. K., 1957, The physical geography of beaches and coastlines. Hulton Educational Publications Ltd., London. 128 p.

GROVE, A. T., 1960, Beach profiles at Scolt Head Island, p. 67-69 in Steers, J. A., Editor. Scolt Head Island. W. Heffer and Sons Ltd., Cambridge.

GUILCHER, A., 1958, Coastal and submarine morphology. Methuen and Co., London.

GULLIVER, F. P., 1899, Shoreline topography. Am. Acad. Arts and Sci., Proc. v. 34.

HOYT, J. H., 1962, High-angle beach stratification, Sapelo Island, Georgia. J. Sed. Pet., v. 32.

INMAN, D. L. and Bowen, A. J., 1963, Flume experiments on sand transport by waves and currents. 8th Conf. Coastal Engineering, Proc.

JOHNSON, D. W., 1919, Shore processes and shoreline development. John Wiley and Sons, Inc., New York.

KEULEGAN, G. H., 1948, An experimental study of submarine sand bars. U. S. Army, Beach Erosion Board Tech. Rep. No. 3.

KIDSON, C., 1963, The growth of sand and shingle spits across estuaries: Zeitschrift Geomorph., N. F. Bd 7.

KING, C. A. M., 1959, Beaches and coasts. Edward Arnold Ltd., London.

KRUMBEIN, W. D., 1954, Statistical significance of beach sampling methods. U. S. Army, Beach Erosion Board Tech. Mem. No. 50.

KUENEN, PH. H., 1950, Marine geology. John Wiley and Sons, Inc., New York.

McKEE, E. D., and STERRETT, T. S., 1961, Laboratory experiments on form and structure of longshore bars and beaches, p. 13-28 in Geometry of Sandstone Bodies. Am. Assoc. Petroleum Geologists, Tulsa.

McKENZIE, P., 1958, The development of beach sand ridges. Australian Jour. Sci., v. 20.

NEISWANGER, W. A., 1956, Elementary statistical methods. The Macmillan Co., New York.

OLIVER, F. W., 1926, Blakeney Point in 1924-1926. Great Britain, The National Trust, Joint Management and Scientific Rep.

RECTOR, R. L., 1954, Laboratory study of equilibrium profiles of beaches. U. S. Army, Beach Erosion Board Tech. Mem. No. 41.

SHEPARD. F. P., 1937, Revised classification of marine shorelines. Jour. Geology, v. 65.

———, 1952, Revised nomenclature for depositional coastal features. Am. Assoc. Petroleum Geologists, Bull., v. 36.

STEEL, R. D. G., and TORRIE, J. H., 1960, Principles and procedures of statistics. McGraw-Hill Book Co., Inc., New York.

STEERS, J. A., 1946, The coastline of England and Wales. Cambridge Univ. Press, Cambridge.
———, (Editor), 1960, Scolt Head Island. 2nd ed., W. Heffer and Sons, Ltd., Cambridge.

STRAHLER, A. N., 1966, Tidal cycle of changes in an equilibrium beach, Sandy Hook, New Jersey. Jour. Geology, v. 74.

U. S. COAST AND GEODETIC SURVEY, 1962, Tide tables—east coast North and South America. U. S. Govt. Print. Office, Washington.

WATTS, G. M., and DEARDUFF, R. F., 1954, Laboratory study of effect of tidal action on wave formed beach profiles. U. S. Army, Beach Erosion Board Tech. Mem. No. 52.

WILLIAMS, W. W., 1960, Coastal change. Routledge and Kegan Paul Ltd., London.

YASSO, W. E., 1962, Flourescent coatings on coarse sediments; an integrated system. Office Nav. Res. Project NR 388-057. Tech. Rep. No. 1.

YASSO, W. E., 1964, Geometry and development of spit-bar shorelines at Horseshoe Cove, Sandy Hook, New Jersey. Office Nav. Res. Project NR 388-057. Tech. Rep. No. 5.

ZENKOVICH, V. P., 1967, Processes of coastal development. Oliver and Boyd, Edinburgh.

CORRELATION BETWEEN PRECIPITATION, FLOOD, AND WINDBREAK PHENOMENA OF THE MOUNTAINS—A CASE STUDY FROM CENTRAL EUROPE (1946-1956)

by

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The wind-diminishing effect of fences is a commonly known occurrence both in connection with snow fences along roads in winter and in windbreak forest belts in some dry areas of the Prairies, as well as in the South Russian Steppe of Europe and Asia—where they are used to preserve or to increase the moisture-content of the farm lands. If this effect can be seen on a small scale, and with relatively low fences, it should also be possible on a bigger scale. For example, mountains of greater relative height would affect the adjacent area as to moisture content of the air and of the soil. The mountains would influence the yearly precipitation and the strength of steady winds not only on a huge scale such as along the Himalayas of Asia, the Alps of Europe, and the Rocky Mountains of our continent, but also to a lesser extent in any area where relative height of the mountains combined with steady wind-direction create favorable conditions for the windbreaking phenomena to alter the climate.

In accordance with the previous reasoning, High Tatra of the Carpathian Mountains in Czechoslovakia, an orographic barrier, was chosen as a wind shield for analysis of these aspects because of the generally constant wind-direction from West-Northwest to East-Southeast. Also, the shape of the massif—with its 20-22 mile length, 1-2 mile width, and almost 1½ mile relative height above the adjacent area—is very good for purposes of study. Furthermore, the windward side of the massif (the German-Polish plains) allow full development of these prevailing winds; and the lee side has much lower mountains, and only the Dukla Pass in the Carpathian mountains, with its prevailing North-South wind along the valleys of Topla, Ondava, and Laborec, can disturb or at least influence these phenomena.

For proper analysis and for establishing the correlation between windbreak effect, precipitation, and flood, a further description of the area is essential.

Forests in observed watershed areas adjacent to High Tatra are, in km² or % of area as follows:[1]

	Name of Watershed	*Watershed area in km²*	*Forest*		*Conifers %*	*Hardwood %*	*Mixed Wood %*
			in km²	*in %*			
Windward Side of High Tatra							
	Orava	1,991.6	623.6	31.3	90	10	. .
Windshade Side of High Tatra							
	Vistula (Dunajec-Poprad)	1,953.0	679.8	34.7	86	7	7
	Hernad	4,432.6	1,889.0	42.6	52	26	22
	Bodrog (Topla, Ondava, Laborec)	7,216.5	2,212.7	30.6	7	90	3
	Total in Windshade	13,602.1	4,781.5	35.0	36	53	11

[1] Data of the Central Institution for Geodesy and Cartography in Prague, 1965.

The relation between yearly precipitation in mm and the height above sea level, as gathered from available records for Czechoslovakia, can be put in the following equations:[2]

Below 600 m height above sea level:

$$\text{Precipitation in mm} = 0.54 \cdot H + 546$$

Above 600 m height above sea level:

$$\text{Precipitation in mm} = 0.63 \cdot H + 492$$

In both equations, H = altitude above sea level in meters of the investigated geographical place.

In the equations expressed correlation between the height above sea level and the yearly precipitation is generally valid and in each case, when deviation occurs, this can be explained by the windbreak effect of the mountains, as will be discussed further.

There were several researches conducted in Czechoslovakia concerning the windbreak effect of the air in the adjacent areas. Reports on their findings were published by P. D. Isin in 1951 and by St. Fekete in 1954.

The wind velocity was diminished because of the wind shield, according to observations conducted in Zikarovce (District of Galanta, Slovakia) on August 28, 1953 and October 27, 1953 and in Vlcany (District of Galanta, Slovakia) by the Water Resources Research Institute of Bratislava:[3]

15% in a distance 3 times the height of the windshield.

40% in a distance 10 times the height of the windshield.

80% in a distance 25 times the height of the windshield.

The above data were obtained at windbreak fences of 5.0 m height, measured at an elevation of 1.50 m above the ground and by wind velocities 4.50 to 5.10 m/sec.

A report was issued by P. D. Isin on the same subject, but for different types of windshields, as shown on the following graph which has data similar to Fekete's, only more elaborated.[4]

In the same report, the measured evaporation at windbreak forest belts of 10 to 14 m height and 14 to 16 m width were as follows:

Distance in m from the windbreak forest belt:	10	50	100	200	300	400
Evaporation effect in %, compared to similar areas without windbreak:	77	82	86	94	95	100

The report of Fekete about the decrease of evaporation effect is based on observation on October 14 to October 15, 1953, conducted by the Water Resources Research Institute, Bratislava, in Vlcany (District of Galanta, Slovakia) with protective fences of 5.0 m high, measured at an elevation of 1.50 m above the ground and at 4.20 m/sec wind velocity. The results are as follows:

Windward Side of the Forest Belt:							
Distance in m from the forest belt:	207	157	107	67	37	17	7
Evaporation effect in % compared to similar area without forest belt:	91.1	91.1	87.5	79.7	81.2	77.1	75.0
Windshade Side of the Forest Belt							
Distance in m from the forest belt:	7	17	37	67	107	157	207
Evaporation effect in % compared to similar area without forest belt:	37.5	35.4	52.1	77.1	91.1	91.9	93.7

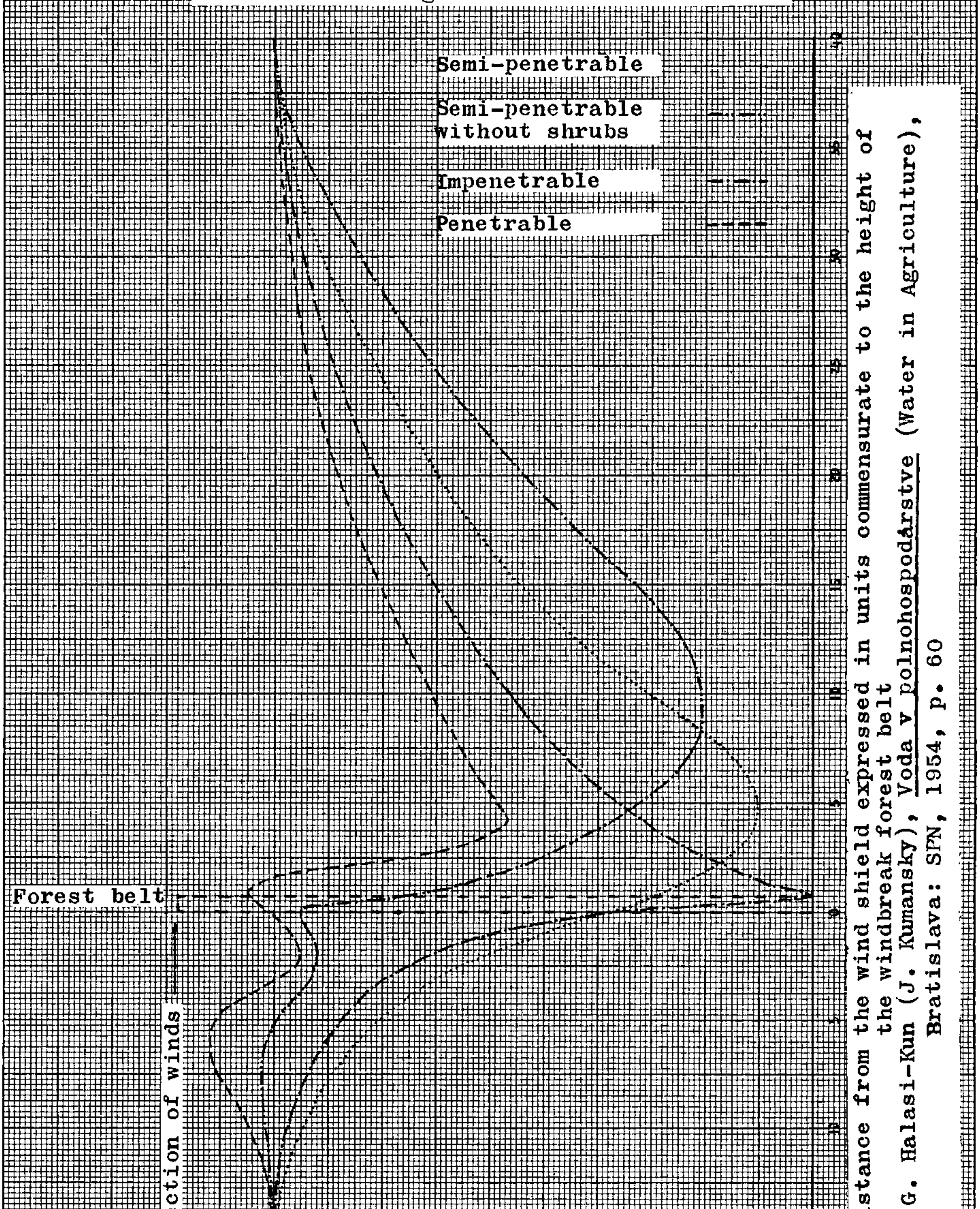
Analogical to the previous data, High Tatra as an orographic barrier, with its steady West-East winds, acts like a wind shield and therefore it should affect the precipitation in an area from a distance of 15 times the relative height of the massif on the windward side to a distance of 40 times the height of the natural wind shield on the windshade side. Practically, this applies to the whole watershed area of Orava on the West side and to the watershed areas of Vistula (Dunajec and Poprad), Hernad and Bodrog (Topla, Ondava, and Laborec) on the East side of the observed windbreaker complex. On this principle and based on the previously described two precipitation formulas for Czechoslovakia Fig. 3 to Fig. 6 show the cross sections of these areas with the computed and observed yearly precipitation. The location of these cross sections, numbered 1 to 4, is shown in Fig. 2. The deviations in % between the expected (computed) and observed yearly precipitation data and their distance from the windbreaking massif is graphically expressed in Fig. 7, and in general shows a wavy shape and has a similarity to the graph of the wind velocity-diminishing effect of the forest belts or of the decrease in moisture-content at windbreak fences as shown in Fig. 1.

[2] Smetana, Jan, Hydrologie a uprav toku (Hydrology and Flood Control), Prague: CVUT, 1952, pp. 18-27.

[3] *Vodohospodarsky Casopis* (Water Resources Periodical), Bratislava: Fekete, St., "Prve poznatky z vyzkumu vetrolamov na Slovensku" (First Reports about the Research on Windbreak Forest Belts in Slovakia) *II* (1954) pp. 3-20.

[4] Isin, P. D., *O problemech premeny prirody v CSR* (Problems of Changing the Climate of CSR), Prague, 1951.
Halasi-Kun, George, *Voda v polnohospodarstve* (Water in Agriculture), Bratislava: SPN, 1954, pp. 41-60.

Fig.1. Wind Shield Effect Diagram of Windbreak Forest Belts with 10-14 m Height and 14-16 m Width : *



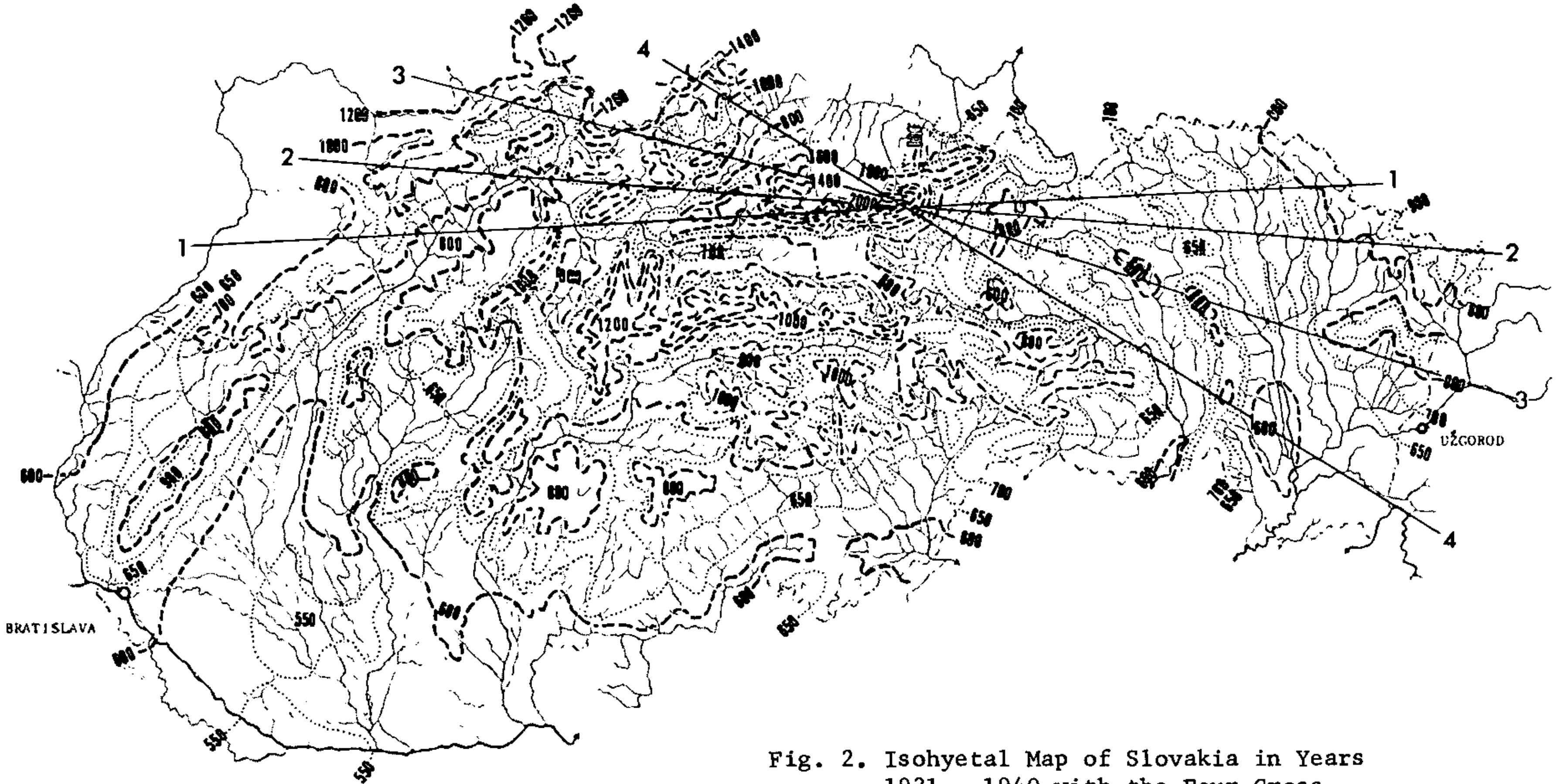


Fig. 2. Isohyetal Map of Slovakia in Years 1931 - 1940 with the Four Cross Sections 1 - 4 through High Tatra where the Windbreak Phenomena's Influence on Rainfall Was Studied

1 : 1.250.000

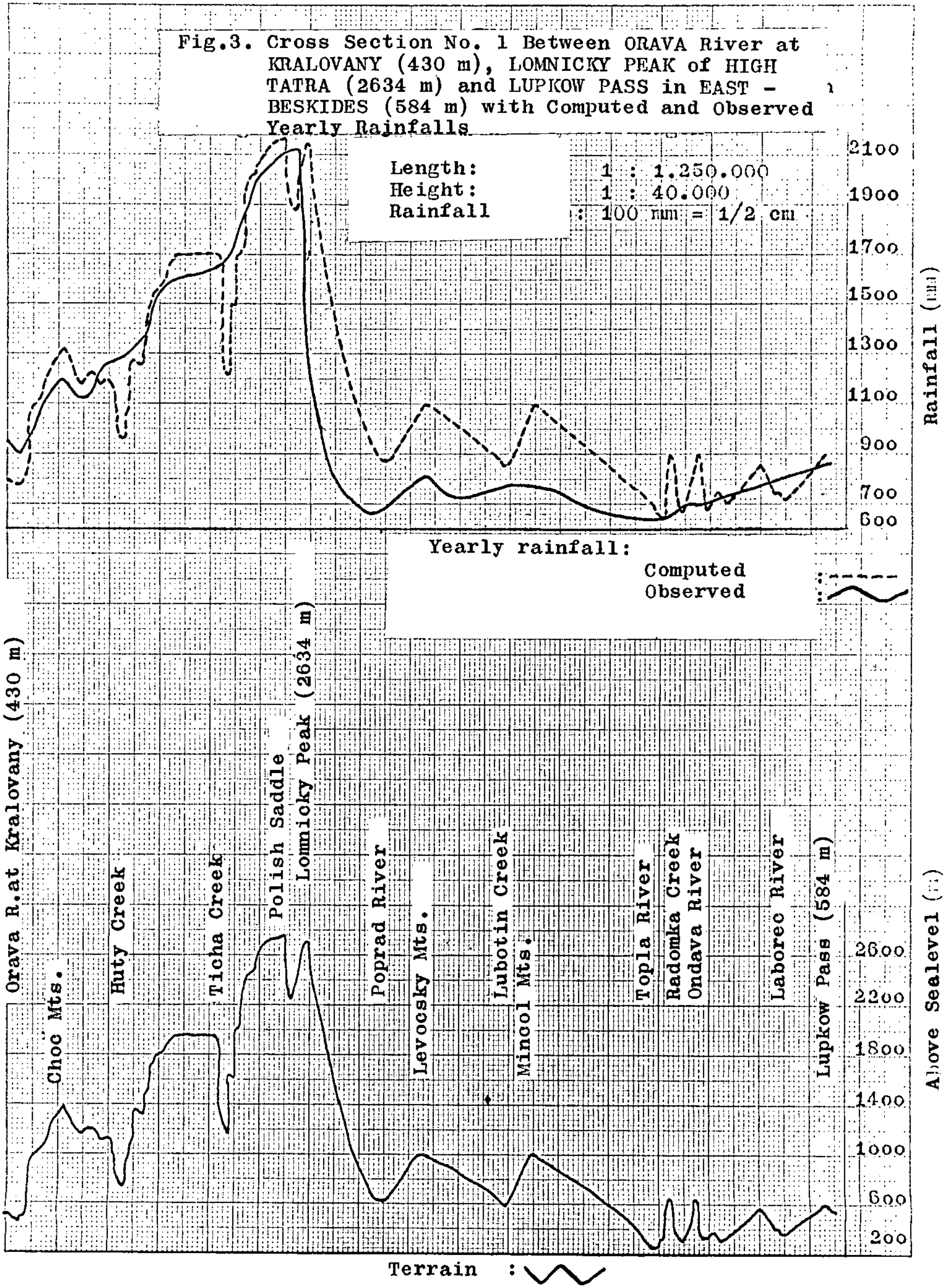
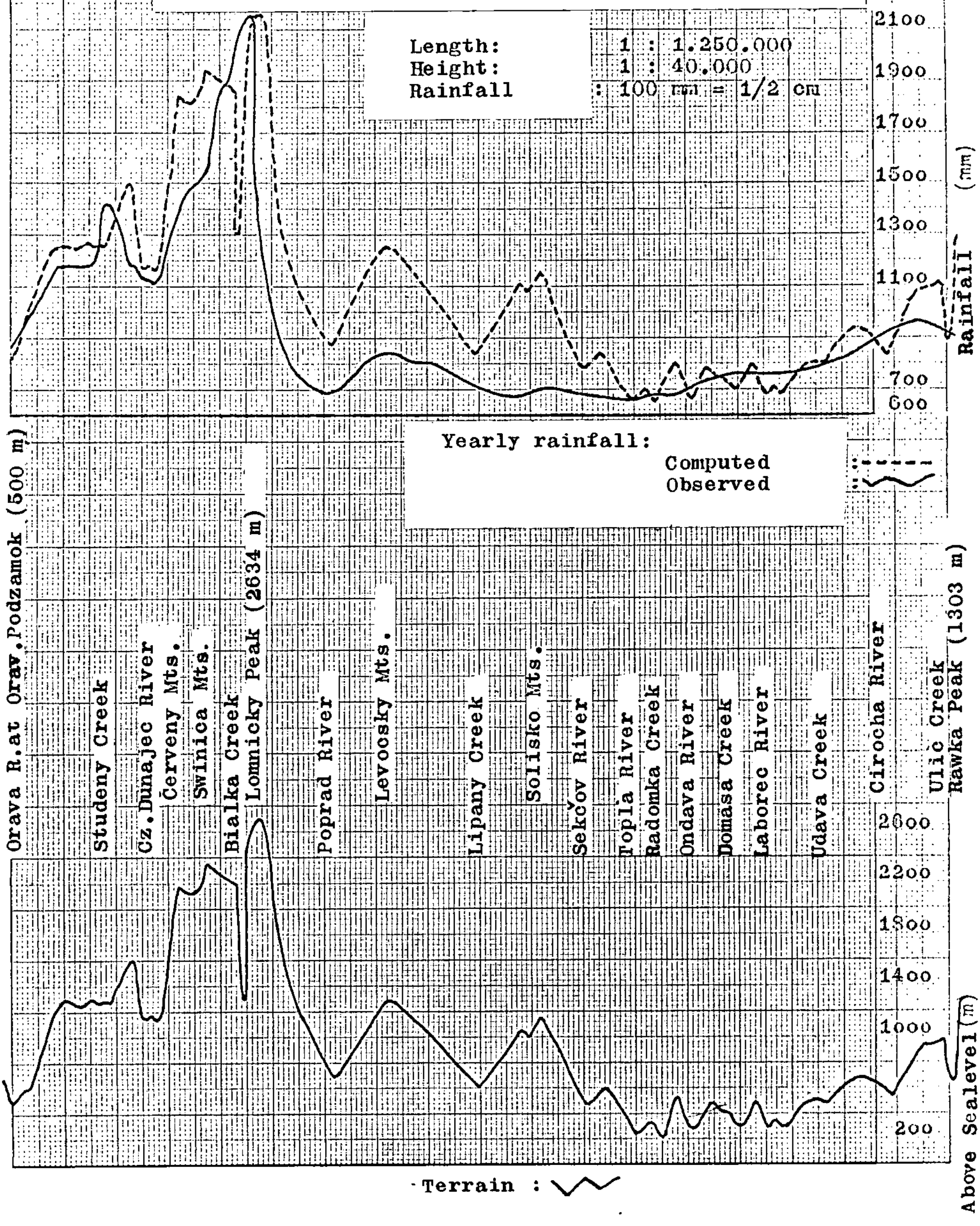


Fig.3. Cross Section No. 1 Between ORAVA River at KRALOVANY (430 m), LOMNICKY PEAK of HIGH TATRA (2634 m) and LUPKOW PASS in EAST - BESKIDES (584 m) with Computed and Observed Yearly Rainfalls

Fig.4. Cross Section No. 2 Between ORAVA River at ORAV.PODZAMOK (500 m), LOMNICKY PEAK of HIGH TATRA (2634 m) and RAWKA PEAK (1303 m) with Computed and Observed Yearly Rainfalls



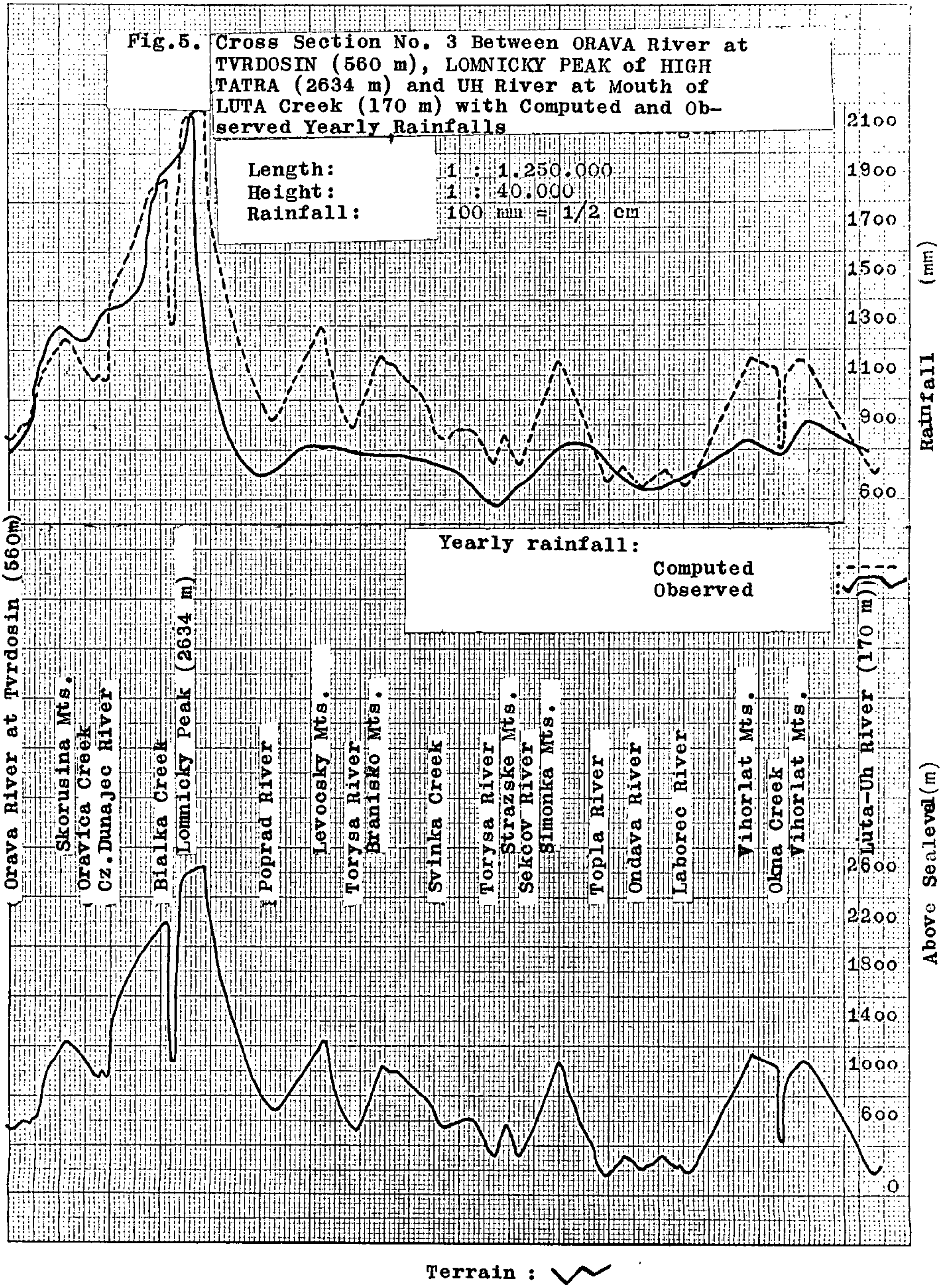


Fig.5. Cross Section No. 3 Between ORAVA River at TVRDOSIN (560 m), LOMNICKY PEAK of HIGH TATRA (2634 m) and UH River at Mouth of LUTA Creek (170 m) with Computed and Observed Yearly Rainfalls

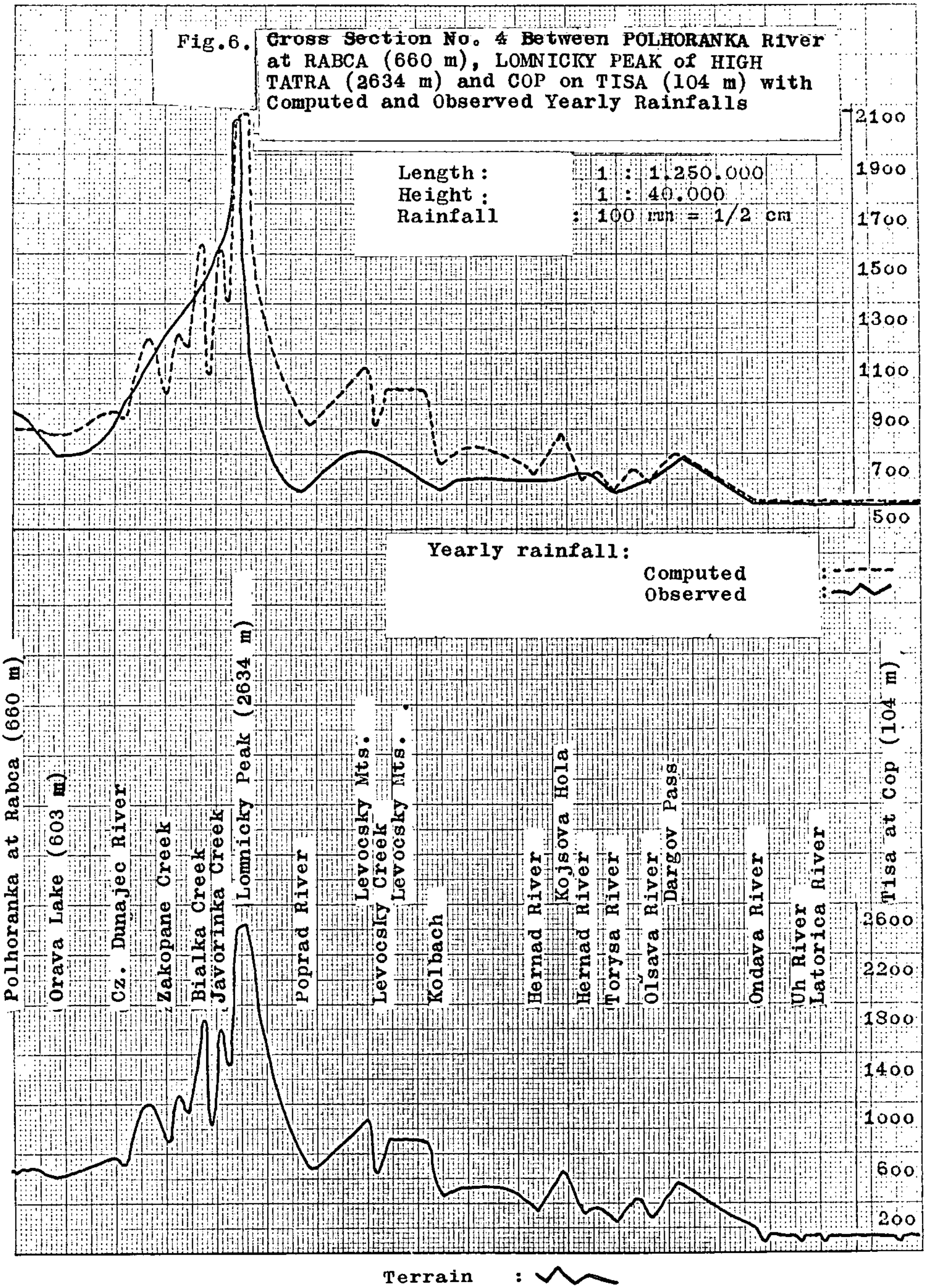
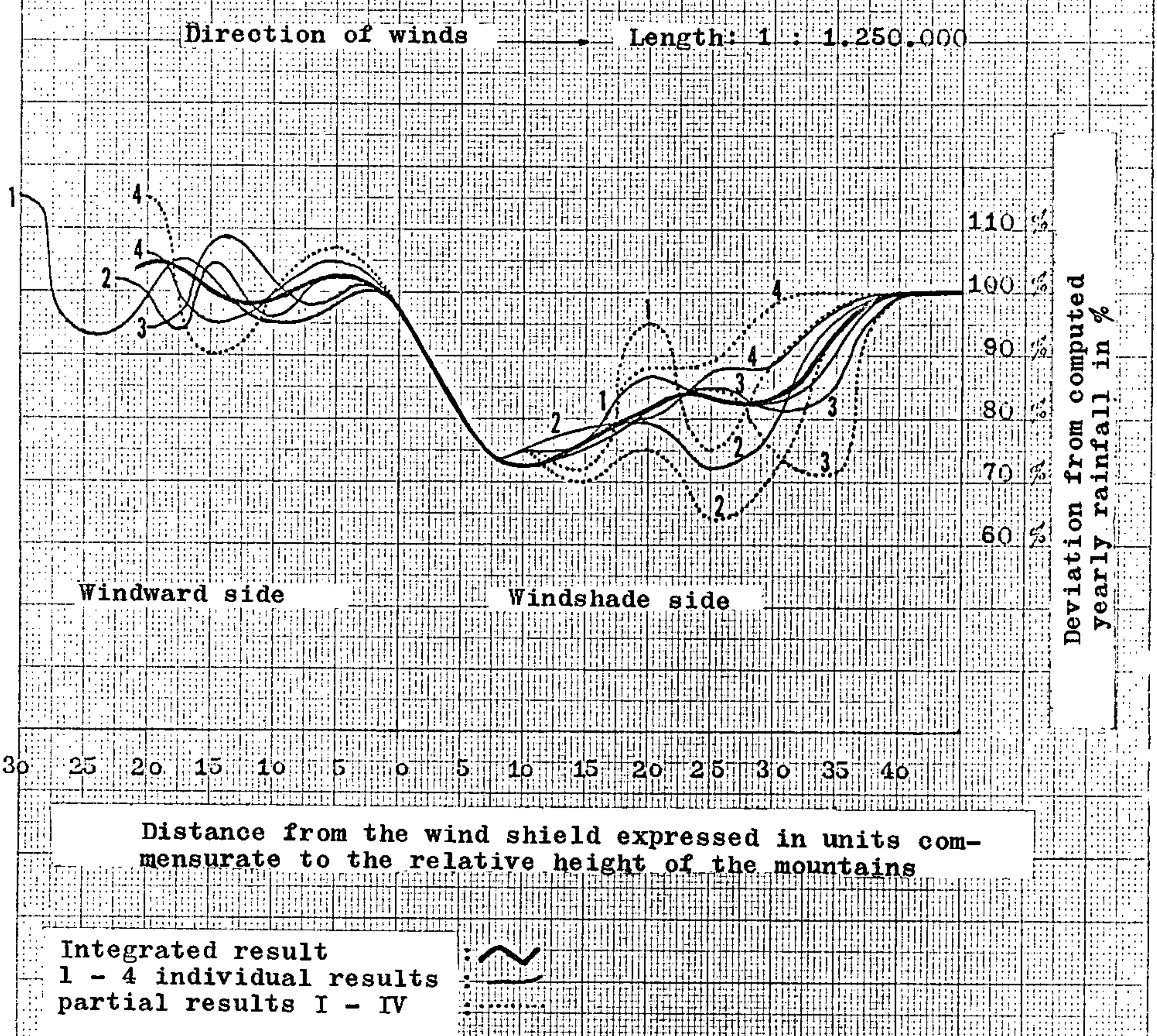


Fig.6. Cross Section No. 4 Between POLHORANKA River at RABCA (660 m), LOMNICKY PEAK of HIGH TATRA (2634 m) and COP on TISA (104 m) with Computed and Observed Yearly Rainfalls
Length : 1 : 1.250.000
Height : 1 : 40.000
Rainfall : 100 mm = 1/2 cm
2100
1900
1700
1500
1300
1100
900
700
500
Rainfall (mm)
Yearly rainfall:
Computed
Observed
Polhoranka at Rabca (660 m)
Orava Lake (603 m)
Cz. Dunajec River
Zakopane Creek
Bialka Creek
Javorinka Creek
Lomnicky Peak (2634 m)
Poprad River
Levocsky Mts.
Levocsky Creek
Levocsky Mts.
Kolbach
Hernad River
Kojsova Hola
Hernad River
Torysa River
Olsava River
Dargov Pass
Ondava River
Uh River
Latorica River
Tisa at Cop (104 m)
2600
2200
1800
1400
1000
600
200
Above Sealevel (m)
Terrain :

Fig.7. HIGH TATRA Mountains as Wind Shield Against West Winds and Their General Effect on Yearly Precipitation (Based on Figures 3 to 6)



The effect on precipitation is greatest on the lee side of the massif at a distance 6-15 times the height of High Tatra (12-16 km: the Spis/Zipser/Basin along the valley of Poprad). The observed yearly precipitation in this area is 200-220 mm less, which means 20-22% lower than computed. This diminishing effect on precipitation can be found up to a distance of 30-40 times the height of the mountains (90-100 km). Between the valley of Poprad and the East border of Czechoslovakia, especially on the windshade side of Levoca, Mincol, Branisko and East Beskides mountains and in the valley of Topla and Ondava, the effect gradually diminishes from West to East with a value 15-5%. In the river-head of Hernad, down the valley as far as Levoca and the valley of Torysa at Sabinov and Presov, the effect is greater and has a 15-10% decrease.

In comparing Fig. 6 to Fig. 3-Fig. 5, it is obvious that the first graph shows a far smoother wavy diminishing effect on precipitation than do the three others. The reason of this is that the cross-section is taken in general along the valley of Hernad, where the diminishing influence—because of the mountain ranges' West-East tendency on both sides of Hernad—developed without any disturbance. On the other hand, Fig. 3 to Fig. 5 demonstrate, in the valley of Torysa near Sabinov and Presov, a locally greater percentual decrease in precipitation caused by the double windbreak influence (of High Tatra and Levoca-Mincol-Branisko mountains). In addition to High Tatra's effect, the massif Levoca-Branisko is an added factor. Farther East, in the valley of Topla, Ondava, and Laborec, the decreasing phenomenon is essentially disturbed because along these valleys through the Dukla-Pass of the Carpathians there is a constant North-South wind, which substantially alters the windshield effect of High Tatra.

On the windward side, in the valley of Orava, a different influence on precipitation can be observed. Naturally, on this side of the massif, the modification of the yearly rain occurs in a much smaller area—starting at a distance of 15-20 times the height of High Tatra. This is similar to the reports of Isin and Fekete concerning the increased wind velocity and moisture-content of the adjacent affected air on the windward side of the shield. Here, too, the precipitation phenomenon shows a wavy tendency which at a distance of 3-5 times the height of the mountains (6-10 km) reveals a higher yearly precipitation rate: 50-100 mm more than computed, which corresponds to a 5% increase (see Fig. 3 to Fig. 5).

Besides the increase and decrease in precipitation caused by the windbreak influence of the High Tatra, there is a windfall effect as well. Forest Service experts of Central Europe described the damages caused by windfalls many times and explained their origin by the fact that these forests were exclusively conifers. As a remedy, it was recommended that instead of conifers, mixed woods be planted in these areas. However, it was overlooked that in all cases the windfalls occur without exception on the windward side of the mountains and within a distance range of 3-5 times the height of the massifs, as has happened for the past half century in these Slovak areas: Ticha valley of High Tatra including valley of Koprova; Hodrusa valley and the river-head of the river Hron in Lower Tatra, between the two World Wars (in 1925); and Studeny valley of West Tatra (June 8, 1948). In general, the winds increased in a wavy pattern starting from a distance 15 times the height of the windbreak massif. The increase was greatest at the distance previously mentioned and reached a value of 110% compared to the adjacent windward area. This increase is the primary cause of the windfalls, which virtually cease at a distance of 1-2 times the mountains' height before the winds reach the mountain ridges.

Finally, everyone would assume that these phenomena should have influence on floods, too. It is interesting to note that this is not so. The maximum flood, for obvious reasons, is not necessarily bound by the wind shield effect because—regardless of the prevailing direction of winds—the precipitation which causes the greatest floods can occur with a wind which does not follow the pattern of the prevailing ones. The floods of June 28 and 29, 1958, in the area of High Tatra, are recorded as the worst in the past hundred years. They developed on the East and South slopes of the massif under steady West-East winds and at a maximum precipitation of 6.5 mm per 15 minutes, or 24.8 mm per hour respectively with a rate of 191.7 mm per day with over 200 mm in two days. On the windward side of the mountains, the maximum flood occurred on June 29, 1958, after the winds in the final 24 hours of the storm changed from their normal direction and blew from East to West. This increased the precipitation in the watershed of Orava and all over in the area of High Tatra causing peak rates of run-off.

The results of further researches in the same area proved that in general the floods, especially the greatest ones, are governed rather by the geological subsurface of the watersheds than by the precipitation, the surface and the shape of the area, or by the windshield phenomena.

BIBLIOGRAPHY

CABLIK, JAN, *Ochrana zemedelstvi pred klimatickym suchem* (Protection of Agriculture against Climatic Drought), Prague: Brazda, 1951.

DUB, OTO, *Hydrologia, Hydrografia, Hydrometria,* Bratislava: SVTL, 1963.

ISIN, P. D., *O problemech premeny prirody v CSR* (Problems of Changing the Climate of CSR), Prague, 1951.

Hydrologicka Rocenka CSR—Cast I, Povrchove vody (Hydrological Yearbook of CSR—Part I, Surface Water), Bratislava, 1946-1960.

HYDROMETEOROLOGICKY USTAV (Hydrometeorological Institute), *Hydrologicka Rocenka CSSR—Cast I, Povrchove vody* (Hydrological Yearbook of CSSR—Part I, Surface Water), Prague, 1960-1966.

KUMANSKY (Halasi-Kun), JURAJ, *Voda v polnohospodarstve* (Water in Agriculture), Bratislava: SPN, 1954.

Sbornik Prac o Tatranskom Narodnom Parku (Work Report on National Park Tatra): Pacl, J., "Katastrofalna povoden v oblasti Tatier v juni 1958" (Catastrophic Flood in Region of Tatra in June 1958), *III* (1959), pp. 17-56.

SMETANA, JAN, *Hydrologie a uprav toku* (Hydrology and Flood Control), Prague: CVUT, 1952.

Vodohospodarsky Casopis (Water Resources Periodical): Fekete, St., "Prve poznatky z vyzkumu vetrolamov na Slovensku" (First Reports about the Research on Windbreak Forest Belts in Slovakia), *II* (1954), pp. 3-20.

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