PROCEEDINGS OF UNIVERSITY SEMINAR ON POLLUTION AND WATER RESOURCES COLUMBIA UNIVERSITY

Volume II 1968–1969

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Edited by
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CONTENTS

Further lectures were given by the Seminar at: the Newark College of Engineering, Department of Civil and Environmental Engineering; the Columbia University Department of Biological Sciences.

Recently an additional Seminar was established. Special Problems in Ocean Engineering, a joint effort of the University Seminars on Pollution and Water Resources, and the Ocean Engineering Division of the Columbia University School of Engineering and Applied Sciences, Henry Krumb School of Mines. This Seminar is devoted to the environmental problems of the coastal and offshore areas (hydrology, estuarine biology, thermal pollution and submarine ecology). Participant students (junior scholars) are registered also in the Division of Ocean Engineering.

George J. Halasi-Kun
Chairman of the University Seminar
on Pollution and Water Resources
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INTRODUCTION

The Steering Committee of the Seminar gratefully acknowledges the generous participation in financing, coediting and printing of this book by the U.S. Geological Survey and the generous contributions of time and effort on the part of the distinguished members of the Seminar in preparing the published lectures and articles. Only their unselfish dedication made possible the appearance of the second and third volume of the Seminar Proceedings.

To give an account about the main phases in the development of the Seminar in 1968-1969 and 1969-1970, these important events should be noted.

The membership was enlarged to seventy-seven members and six guest members. Furthermore, in the spirit of the constitution and in accordance with the charter of the University Seminars, eight graduate students, each of them with an advanced degree, were admitted as student members (junior scholars) in the spring and fall of 1969.

Besides conducting the regular lecture series, the last meeting of the academic year 1968-1969 was experimentally organized as a field meeting in Trenton, New Jersey, with the State Geologist of New Jersey as host, to present general information about the problems of pollution and water resources in that area. Its great success inspired the members of the Seminar to adopt a policy of organizing each year a field meeting at a place where a Seminar member would be host, such as for instance the area of Boston or Washington, D.C.

With the help of the Environmental Protection Department of the State of New Jersey the first book of the Seminar "Proceedings of University Seminar on Pollution and Water Resources: 1967-1968" was published. This book dealt with activities of the Seminar and its members and consisted of lectures and articles.

In cooperation with the Technical University, Brunswick, research activities on hydrology of smaller watersheds have been organized abroad, starting with the academic year 1969 - 1970. The results are intended to serve as a basis for further research in the areas of New Jersey, New York, and Connecticut.

In the academic year 1969-1970 with the World Bank (International Bank for Reconstruction and Development) as host, the "Annual Meeting in Washington D.C." was fostered. The lecture together with the meeting gave a general inside view of the water resources policy and procedures of this world organization. The vivid response of the Seminar members, expressed during the meeting in Washington, D.C., initiated in March 1970 a three-day conference on international and interstate regulation of water pollution.

The conference was organized together with the Columbia University School of Law and its "Proceedings" were published separately. Fourteen lectures were delivered by the Seminar members and the intensive participation (over 250) especially of experts, diplomats and visitors from abroad secured the international character and success of the meeting.

THE INTEGRATION OF DESALINATION INTO WATER AND POWER PROBLEMS--SOME UNEXPLORED ECONOMIC PROBLEMS

by

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1 The views and opinions expressed in this paper are those of the lecturer and do not necessarily reflect the views of the United Nations.

This lecture will deal very largely with the question of dual-purpose desalination plants which produce both power and desalinated water. The critical review of the potentialities of this type of dual-purpose plant should not create the impression that I am against dual or multi-purpose plants. Multi-purpose river projects are standard today, and industrial plants producing both steam and power are very often economic under specific industrial conditions. In this lecture, however, we are dealing with dual-purpose desalination plants in which the output is destined for the market.

In the lecture today I would like to discuss first some of the problems of a dual-purpose plant; namely, a plant which produces both power and desalinated water and, subsequently, the problems of system cost and growth in a water supply system based exclusively on desalination.

Dual-purpose plants appear to possess an unusual attraction for many engineers and many water planners. I believe that existing experience, as well as potential problems, will indicate that such dual-purpose plants will be economically successful only in exceptional cases. The United Nations' survey on desalination plants in 1964 found that very few dual-purpose plants were operating successfully in economic terms, and only a small number operated at high load factors. The worldwide survey of operational experience of desalination plants in 1965, to be released soon, shows the same results. There is, therefore, little in the experience known to us which would justify the assumption that dual-purpose plants, apart from engineering efficiency calculations, are superior to single-purpose installations. However, the experience we have relates exclusively to small dual-purpose plants and it might be argued that the situation would be different for large dual-purpose plants. As the following analysis will attempt to show, large dual-purpose plants will have big problems and the experience of existing small dual-purpose plants remains relevant. In the following discussion, I will restrict myself to the application of dual-purpose plants for permanent water and power needs; in other words, we will disregard the water and power needs for a mine which will last about twenty years, and other types of temporary water and power needs.

If we consider a dual-purpose plant for an area with permanent water and power needs, then it is essential to plan the dual-purpose plant in such a way that it can operate continuously and produce at its optimum output of both water and power; in other words, the dual-purpose power-desalination installation should be operated on base load. If, for a considerable period of the year, it should operate to produce water only or power only, it is assumed that, under these conditions, operation would be uneconomic.

We, therefore, stipulate that a dual-purpose plant should be operated on base load for water and power over its productive life. In most calculations for these installations, productive life is now assumed to be around 30 years, although this appears to me to be unduly long. What I would like now to discuss in detail are the consequences of keeping a dual-purpose plant on base load for a considerable number of years, say 20-30 years, and the problem of the proper base load penalties which may apply to both water and power outputs. The power base load penalty is a problem which is already known but often misunderstood and the water base load penalty is a problem which, I believe, has not yet been recognized.

Let me first deal with the power base load penalty. In an electricity system, it is customary to put on base load the power unit which produces electricity at the lowest cost per kilowatt hour. Therefore, large modern units incorporate, through economics of scale, lower capital investment per kilowatt of capacity and lower fuel cost because of increased efficiency in the use of fuel per kilowatt hour of output. Consequently, keeping a power unit on base load beyond the period when it would otherwise have been displaced by new units at lower unit capital cost (through bigger size) and at higher fuel efficiencies, will clearly raise the electricity system costs beyond the achievable minimum. In other words, it may prevent the normal reduction in generating cost as a result of modern engineering developments or system growth. This cost of keeping a power unit on base load beyond the normal period of 8-12 years, which is clearly required in the case of dual-purpose plants, is called the "power base load penalty" and this cost, based on certain assumptions, could be estimated.

The power base load penalty as defined appears now to be clear but, apparently, it is not clear to many of the engineers involved in dual-purpose plant design and, as an example, I am quoting a paragraph from a lecture by Mr. Adar, held at the Second European Symposium on Fresh Water from the Sea in May 1967, which has appeared in the Journal "Desalination", No.1, 1967. Mr. Adar states:

> "The cost of off-peak electricity output from any dual-purpose plant is that of the fuel only at a thermal efficiency of about 95%; therefore this cost for a nuclear dual-purpose plant is about one third of the marginal electricity cost from a nuclear power-only plant and one sixth from a fossil-fueled power-only plant. This fact ensures that such plants need never suffer a base load penalty and also means that economic obsolescence is very unlikely within the normal plant lifetime."

Normally, this base penalty is composed of two elements, namely (a) the lower capital requirements per kilowatt of newer and bigger power units and (b) the higher efficiency or lower fuel requirements per kilowatt hour output in newer power units. The second point (b) is, however, not always applicable. In a country like Kuwait, where natural gas is made available free of cost for desalination, or in geothermal power units where there are no fuel costs, item (b) would not be applicable. The power base load penalty will vary according to the power generating system employed. In the case of nuclear power, the economics of scale will be more important than the increased efficiency in the use of fuel; whereas, in the case of conventional fuel, the economics of scale is somewhat less important and the efficiency in fuel use somewhat more important. The size of the power base load penalty will also depend on the growth rate of the system. If the electricity demand of a system should grow at a rapid rate, say 7-14% per year, then the power base load penalty would be much higher than in a system where the annual power demand growth was only 3-6%. It should be noted that most electricity systems, including those in developing countries, still exhibit a comparatively high annual growth rate.

It could be argued that, in order to overcome the power base load penalty, such dual-purpose plants should be calculated as operating, say, only 10 years on base load and being shifted after 10 years to a reduced output. The problem then arises of how to find a reduced production pattern which fits both the power and water demand since everywhere power demand is growing on average much more rapidly than water demand. No one has been able to propose a practical solution for a synchronized reduction in output wherein dual-purpose plants would only operate a limited number of hours but, during this limited time, would operate as a dual-purpose plant. It should be noted, furthermore, that, if a dual-purpose installation was intended to be operated on base load for a limited period only, then this must necessarily affect the over-all cost calculations.

Clearly there must be a similar problem, namely a water base load penalty, applicable to dual-purpose plants. Let me first make it clear that, for a dual-purpose installation to operate on base load, it also requires, on the water side, a water supply or grid system because the pronounced differences in water demand between day and night and winter and summer would make it impossible for an individual plant to operate on base load, unless there existed a tremendous storage capacity. It is intellectually difficult to assume that the technology of desalination would be so stationary for, say, 30 years that newer desalination units, at lower unit capital cost, could not become available over such a long period of time. If, however, it is assumed that cheaper systems will come into operation in future then clearly those systems operating at lower capital cost per unit of water, better corrosion control and higher output per pound of metal could be put on base load in the water supply system, whereas the older units should be put on lower loads so as to obtain the lowest possible water system cost.

It is today not sure to what extent the economy of scale will be applicable to desalination units once sizes of, say, 10 million gallons per day, are in operation (nevertheless, to a certain degree, economy of scale will be applicable, at least insofar as savings in management, maintenance and repair and purchasing are involved). Similarly, it is reasonable to assume that newer desalination systems will achieve better efficiency in the energy requirements per unit of water output. It is for these reasons that a water base load penalty exists too, although there is less awareness of this problem than of the power penalty.

One may perhaps argue that the described problems of power and water base load penalties might be reduced by assuming for cost calculation a shorter period of life for the installations, meaning that the amortization period could be reduced to, say, 10 years. In such cases, the capital investments would be amortized and the cost of maintaining a dual-purpose plant on base load would only involve fuel cost and maintenance. Such a shortened period of amortization is, however, hardly practical because, for the first ten years, it would make water and power so expensive that there would be very little economic justification for the dual-purpose plants. One might also want to consider pre-building as a way of overcoming the power and water base load penalty. In the case of water, pre-building might allow time so that, in such a way, the groundwater reserve is overdrawn during the years of construction of the dual-purpose plant and the installation is kept fully operative by using the water output during the first few years, partly in order to recharge the overdrawn groundwater reserve. Unfortunatley, however, electricity cannot be stored economically and so the solution is not applicable to a dual-purpose plant. Such a solution might also raise certain other problems which will not be discussed here in detail. The conclusion of these discussions leads, therefore, to the warning that a dual-purpose plant, in its economics, should include also the negative cost of the water and power base load penalty.

In addition to the clearly definable economic problems arising from the shifting of a dual-purpose plant from base load to lower loads, there are also certain technical problems which require detailed study and which may involve serious unexplored technical problems. By reducing a dual-purpose plant from base load to lower load, lower production could be achieved in two different ways or in a combination of the two different ways. One is to operate the dual-purpose plant on partial load and/or shut it down for certain given periods, say, every night or so.

Let us first turn to the problems which may arise in running the dual-purpose installation at partial load. Apart from the obvious economic consequences of the cost involved in running production facility below its design capacity, there is first the question of the effect of partial operation on desalination plants. Our experience indicates that this is bad for the equipment and that technical problems such as enhanced corrosion tend to arise. The difficulties associated with the partial operation of the desalination part of a dual-purpose plant could perhaps be overcome, at least to some extent, by avoiding one large desalination unit and substituting several smaller distillers so that, when partial load operations were required, one or two units would be shut down and, say, one or two units would operate at full load. Such a solution is feasible but would, however, have the following drawbacks: (a) it would reduce the possible economics of scale on the desalination side; (b) the cost of maintaining the shut down desalination plant under water pressure or vacuum to avoid corrosion build-up would be raised. On the power side, enough experience is available to substantiate that no technical difficulties would arise from the operation of a power unit at reduced load. There is less experience on the effect of partial loading in the case of nuclear power plants but, broadly speaking, one may assume that, apart from considerably increased unit cost, no major problems could be expected.

In the second case, we have assumed that, instead of partial loading, the dual-purpose plant can be shut down for given periods, such as, say, every night and the following problems will be encountered; as regards desalination plants, the cost of maintenance will rise rapidly, as pointed out earlier, because of increased corrosion. From a practical point of view it is doubtful whether such an operating regime would be acceptable since the warming up of an evaporating plant can extend over several hours and involve a nonproductive heat consumption which could be prohibitive on a daily basis. As regards the power plant, the frequent shut down and the starting again of the boiler, apart from increased cost, will also tend to decrease the life of the boiler, although this is a consideration which applies to any generating plant operating on a two or one shift basis. Fuel efficiency will decrease and it could probably be assumed that plant life will be reduced too. In some forms of nuclear reactors, the frequent shut down of the reactor may lead to a build up of poison in the fuel elements and perhaps to other problems which need more study.

In view of these numerous and largely unexplored problems and the possible effects of under-loading or reduced hours of operation on a dual-purpose plant, caution is necessary in assuming that, in order to avoid the base load penalty for water and power, these plants could be operated technically and economically on lesser loads after a number of years of use on base load. If calculations on a given project indicate that a dual-purpose plant can be left on base load only for a given number of years because of base load penalty, then the design of the dual-purpsoe plant should also be tested for its effectiveness under partial loading or reduced hours of operation.

Let me now turn to other problems involved in a dual-purpose plant. And here we enter a field which is totally unexplored as well as speculative but, nevertheless, I believe that the problems involved need to be studied, defined and perhaps, later on, to be costed, if possible, in economic terms. I suggest that in addition to the base load penalties for power and water, there are other categories of potential future unrecognized costs in dual-purpose plants which I would like to define as "other power cost penalties" and "other water cost penalties". The definition of "other power cost penalties" and "other water cost penalties" is power or water costs arising because the power and water production may not be needed by consumers.

Let me now explain what the other power cost penalties may mean in practice, both for power and for water. In the case of individual dual-purpose plants, not connected to a power grid, the power output of dual-purpose plants may not be needed if the grid system reaches the area of a dual-purpose plant. This has just happened in the case of a dual-purpose plant operating in Elat, Israel, where the extension of the grid system is now reaching Elat. Grid power in Elat will, therefore, become available far below the power cost of the small steam power plant which forms part of the dual-purpose plant. There are many other similar problems which may arise in the future and even the power grid system we know today may undergo considerable changes. You all know that considerable work is being done on the development of a fuel cell to operate on natural gas in the basement of private houses so that each house can produce its own electricity in addition to using natural gas directly. In case this development should, in the next ten years, prove successful, it will have a considerable impact on the grid systems as we know them today because it might eliminate from the income of the big power companies the consumers who pay the highest rate for electricity. In some form this devlopment has already started with the so-called "total energy system" under which, for a big housing complex, a system is introduced which provides all energy needs of the housing complex from one fuel source. This provides for electricity generation, for air-conditioning, for heating, etc. We cannot foresee the future, over 30 years, of the electricity systems or of the energy systems but we do know that new technologies may come into play and they may lead to conditions under which a power station may not be able to sell all its output to a grid system. In this connection, it is relevant to recall that, in modern power systems, generating costs form less than one-third of the total systems cost and that transmission, distribution and commercial activities form the bulk of the total systems cost. Under those circumstances, the economic attractiveness of a new technology which would cut out all or a substantial part of transmission, distribution and commercial costs, should not be disregarded.

Equally complex is the outlook for water. The output of a desalination plant might become redundant at any time in the future, either because of the discovery of conventional water at lower cost, and this would include water becoming available through weather engineering or because of decline in the cost of the re-use of water. There are some additional specific problems in water, such as a decline or shift in industries using large amounts of water or the large-scale introduction of air or other systems of cooling, replacing the large-scale use of water for cooling and other similar purposes. Rainmaking has recently been introduced on a systematic basis in Israel and it has already led to a decision to postpone the time when large-scale desalination will be required.

The problems discussed above and the examples given for the "other power or water cost penalties" are penalties, which, it might be argued, are part of the risk of the introduction of new technologies and, therefore, would be applicable also to individiual power or desalination plants and so are not costs properly to be considered specific to dual-purpose plants. Such an argument, however, is only partly correct because the burden of the problems is that two combined systems, mutally dependent, carry double the normal risk of obsolescence.

In effect, however, it might be more than a double risk and we now turn to the question which is rarely asked so far and is quite important; namely, who will pay for the base load or other penalties? We believe that the problem is both complicated and very important. Let us assume that, in an area where dual-purpose plants have been constructed, rainmaking or some other technology has provided large-scale new sources of potable water at very low cost and that the decision is taken to close down the desalination part of a dual-purpose plant. Who will pay for the additional cost involved in running a dual-purpose plant as a single-purpose power plant? Also, there will be capital losses involved in the closing down of the desalination installation. Let us remember that we have two additional cost factors to consider here and in some cases even three; namely: (1) the increase in operating costs created for the power sector by shutting down the desalination plant; (2) the capital losses of the desalination plant; and (3) if applicable, the cost of keeping for a few years the dual-purpose plant less desalination on base load, when units producing power at lower cost could be put on base load. The allocation of base load penalties as between water and power and of other penalties is quite a complex matter and, among the small dual-purpose desalination plants now in operation, this is a matter which has created consistent argument between the water and power authorities involved.

The allocation of the "other penalties" will become even more complex as the size of the cost may be even bigger. Wherever a dual-purpose plant is Government-owned or owned by other public bodies, it is obvious that the base load or other penalties will have to be paid for by the taxpayer. However, if the dual-purpose plant is privately-owned or partly privately-owned, the problems may not easily be solved.

Now let me turn to a second interesting economic problem; namely, the growth problem of a desalination system. Let us assume, for simplification, that we are dealing with a water supply problem in an area where all or most of the water supply has to be provided by desalination and where there is a permanent population and a growing water demand, such as, for instance, in the case of Kuwait. In such an area the first desalination plant will be followed by a second and, over the years, further desalination plants will be required so that gradually a desalination water supply system will come into being.

The question now arises as to what size does one build the first plant and each of the subsequent plants so as to obtain the best system economics.

The United Nations has recently studied that problem, in conjunction with storage requirements, so as to obtain both a reliable and low-cost system of desalinated water supply. To our great surprise, detailed studies reveal that it will be difficult under such systems to determine properly the size of the first plant without a careful study of the pattern of systems growth over a 20-30 year period and without estimating the annual water demand growth over that period. We also found that, unless the annual increase in water demand surpasses 8% per annum, and that is a very high annual water demand growth rate, the size of the second plant, when needed, will be smaller than the first plant. Assuming other factors as being equal, we would obtain, over a 20 or 30 year period, increased systems cost per thousand gallons of desalinated water instead of lower costs.

This surprising result is due to the fact that, once a population has reached a comparatively high water level, the per capita consumption of water does not rapidly increase and the same applies to industrial use. There is, therefore, a basic difference in the pattern of growth between the power demand and the water demand and this requires, for the future, a much more detailed and long-term study of water demand growth patterns. Once, however, the problem of the danger of the growth in system costs of desalination was recognized, the United Nations team has tried to find ways in which an increase in system costs could be avoided. Much will depend on local conditions and it was found that, where desalination can be combined with groundwater supplies, a joint desalination groundwater scheme could avoid increased systems cost. In certain cases, a combined development with surface water could also avoid the danger of increased systems cost.

There are many other factors which need careful study in a desalination supply system and the time has come for water economists and water engineers to look at these problems and to study them carefully. I believe also that it is insufficient today to look at a single desalination plant, calculate its cost and determine its size by the present or immediately foreseeable future demand as if a desalination plant could be judged over its long life on a single plant basis only. This type of single plant economy is applicable only to temporary requirements but not for areas with permanent populations and permanent demands.

There will be cases when studies will be carried out where the cost evaluation for a single plant may have to be modified considerably by the systems analysis over a considerable period ahead. I also believe that further studies in this direction will indicate that an integrated development between desalination and conventional water resources will, in a systems study, probably show better economic results than the development of desalination-only systems.

In some countries, the belief is held that, once the first desalination plant is in operation, all future water demand has to come from desalination and, in one country which I know quite well, an announcement was recently made that all power plants in the future will be dual-purpose plants. I believe that the systems problems of dual-purpose plants and of desalination are too little explored, too little recognized and are not incorporated into the costing of desalination today. The development of proper economic tools for this type of economic evaluation is now needed, and I hope that my lecture will contribute to the awareness of those problems and that perhaps somewhere universities or other bodies or firms will delve into these important problems.

WATER RESOURCES DEVELOPMENT OF SOUTHEAST LOWER-SAXONY

F. R. GERMANY

by

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In this article, I shall discuss the problems of water resources and water development in West Germany. For all this country's being located in the humid climate of Central Europe, a region with what would seem to have sufficient precipitation, the surprising truth that is a supply of water of suitable quality is becoming scarce. This is a situation that requires legislative, administrative, and technical measures if the region is to develop in a way that will be advantageous to the whole national economy. Because in the future, water will be a critical factor in human welfare and industrial growth.

1. Legislation and Planning (the Water Act)

Before going into technical detail, I shall make some general remarks concerning the problem in Germany.

First, let me refer briefly to German water legislation. In view of the increasing difficulties in German water development, several authorities demanded that the government give immediate consideration to a new Water Act. In accordance with this request, the Federal Government put forth a bill for a new Water Act, which was passed by the parliament and went into effect in 1957, replacing a sixty-year-old law which had become obsolete.

The new law was completely different from the old. Whereas formerly water use was granted to everyone, with the legislative body reserving to itself the right of restriction, the new law undertakes to regulate the overall use of water on a coordinated basis. The long-term needs of the area are taken into account.

The law has six parts, which constitute a comprehensive whole.

One part deals with a water development plan which would be of interest. This plan involves an entire river basin -- an area distinguished by an interdependent economy. It provides for future possibilities concerning water resource management, flood control, and the prevention of water pollution in watercourses.

Such a water development plan is not merely a technical plan which, in its working out, only makes provisions of building or improving hydraulic structures. It is rather intended to outline a basis for the large scale utilization of water resources and to achieve a water balance which will meet future requirements. It has the character of a recommendation to the authorities.

1.2 Development Areas in West Germany

As the increased need for water is part of the general economic development, the law requires coordinating any water development scheme with the general development plans of the region in such a way as would be valid for a period of thirty years.

Regions covered by water development plans often cross political or administrational boundaries, and may include complete river basins and even be related to areas in another country in compensating water supply between sectors of surplus and deficit. Therefore, it is necessary to design water development plans in accordance with standardized principles.

The total area of the Federal Republic of Germany is 96,000 square miles with a population of 60 million, which means an average population density of some 620 inhabitants per square mile. And the many water development schemes for individual river basins are being set up according to modern guidelines.

The following figures will add to a better understanding: the population of West Germany is about 3/10 of that of the United States; the area of West Germany, however, is only 3/100 that of the U.S. This means that the area of West Germany is equal to the combined area of New York State and the state of Pennsylvania.

The areas covered by water development plans vary in size from 200 square miles for municipal areas to less than 4 square miles for rural areas. Thus, Lower-Saxony, or as we call it, Land Lower-Saxony, with an area of 18,000 square miles and with 6.5 million inhabitants, is subdivided into 17 development sections.

A close connection between water resources and development planning was initiated originally in the individual agglomeration areas such as Hamburg, Frankfort, Stuttgart, Munich, and the Ruhr district. Such density planning is now also being carried out in less-populated areas, in order to provide for the course of increased industrialization.

Southeast Lower-Saxony is this type of area.*

Southeast Lower-Saxony has an area of 1800 square miles, and a population of 1.18 million living in 550 communities. This relatively small area is characteristic of many in Germany. And it is used as a kind of unbroken thread in what follows and to serve as an example in explaining the several methods of water resource development and their simultaneous relation to general development planning.

In this context, another comparison with the United States can be made. Southeast Lower-Saxony is a little smaller than the State of Delaware: the population is nearly twice that of Delaware. (See Figure 1)

Southeast Lower-Saxony consists chiefly of the region around Braunschweig (which has the Old Saxon name of Brunswick), and is situated approximately 120 miles south of Hamburg and the North Sea. The southern boundary is formed by the Harz Mountains, and in the north it is bordered by the Luneberg Heath. The demarcation line between East and West Germany forms a strict border in the East.

Because of the Iron Curtain (and you can see Check Point Helmstedt on the illustration), the economic relations of the Brunswick district -- which formerly radiated in all directions -- were unnaturally cut off from the East. The region has changed from being in a central position in Europe to being a peripheral area in free Western Europe.

*In the illustrations, the abbreviation SEL stands for Southeast Lower-Saxony.

The economic affairs of any region are always in a state of change, not only because of political events but also as a result of new techniques and new resources such as are now coming about.

2. Southeast Lower-Saxony

2.1 Survey of the Report on National Planning

In 1961, representatives of the communal central organizations and of the commercial boards in Southeast Lower-Saxony agreed that it was necessary to do research in order to work out guidelines for regional development. They realized that, in addition to water resources, other factors in development planning, such as population, transportation, and the general economy, had to be taken into consideration.

The man put in responsible charge of this research was Professor Friedrich Zimmermann, director of the Leichtweiss Institute for Hydraulics and Foundation Engineering. He had a team of researchers which included thirteen professors at the Technical University of Brunswick and other experts.

The results of their work were collected in thirty-five volumes, and the project took five years to complete. The total cost amounted to about 20¢* per capita for the population of Southeast Lower-Saxony.

Significant points developed were the following:

1. The composition of the population by profession or job and its expected development (of utmost importance in any such investigation).

2. The economic development of the area for which plans were to be made and its anticipated growth.

3. The distributory network, which is to say, the design of accepted development principles which serve as the basis for all future development measures.

4. The limit of built-up areas.

5. Transportation and its expected development.

6. Technical supply and refuse disposal, including the fields of energy supply, water supply, sewage and refuse disposal, and flood control.

7. Problems concerning national parks, landscaping, and recreational areas.

In the following, measures concerning water resources are treated in some detail, and other objectives are discussed only insofar as they relate to these measures.

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*The costs originally caluclated in DM are expressed in United States dollars (4 DM = $1)

2.2 Economy and Population

Some information on the economy and population of an area -- and their interdependence -- is essential to an understanding of the condition of water resources. (See Figure 2)

Every area with a varied structure reflects, in its pattern of zones of agglomeration, the results of artery transportation and the location of its industries.

Without going into detail, it can be pointed out that in Southeast Lower-Saxony no less than eight types of communities were observable, ranging from the single farm to the town or regional center.

The spectrum of industrial activity is divided to the same degree, and includes single farms and industrial plants. The over-concentration of industrial plants in one area and workers' living in another area causes many problems which otherwise could be avoided, e.g., daily commuting and traffic jams. And there are others, which I shall not mention.

The present structure of population is subject to steady change. In Germany and in other industrialized countries, there is a decrease in the number of people engaged in agriculture. But the percentage engaged in industry is slightly increased. And Lower-Saxony shows a relatively rapid increase in the number of people engaged in commerce and trade. (See Figure 3)

The regional distribution of the population and economic activity is the main factor that must be considered in the needs and demands of water supply.

In Germany, agriculture is still an essential part of the economy. In Southeast Lower-Saxony, about 60% of the total area is devoted to agriculture; this is also the average for the Federal Republic of Germany, and an additional 25% is reserved as forest in Lower-Saxony.

Industrial development expanded, between 1957 and 1961, at an annual rate of 8.4% in Lower-Saxony, and this exceeded the 6.7% rate for the Federal Republic of Germany.

What has been called a "distribution of branches" may be considered a measure of a stable economy. Generally speaking, this is a plan for achieving a roughly equal distribution of industrial centers in Germany, and also to avoid concentration of specialized industries.

The distribution of industrial branches is very complex in this example. It includes mining and basic-material production; it includes automated production plants (see Figure 4 showing the Volkswagen works in Wolfsburg) and the photographic plant in Brunswick. In addition, there is a large number of canning plants and sugar refineries which use large quantities of water in the processing of agricultural products.

Concerning the population of Germany, the following may be postulated.

West Germany now has 60 million inhabitants, and the projectd population in the year 2000 is 75 million.

In Southeast Lower-Saxony, there are now 1.18 million people -- or 2% of the Federal Republic of Germany -- and, considering the age-range, it is expected that there will be 1.48 million in 2000. This would be an increase of 23%.

In order to get a reasonable distribution of the total population, any unnecessary concentration should be avoided.

In areas within an evenly distributed industry, as in the case of Southeast Lower-Saxony, the concentration of population shall be limited in some communities; in others, however, an increase is desirable. The structure of the many small communities, on the other hand, is to be improved by the development of core centers with more than 5,000 inhabitants.

2.3 Hydrologic Facts

The climate in Germany varies from region to region and has to be analyzed for each river basin. Since the country is in the center (not the periphery) of the continent of Europe, it has no variations in climate that would compare with those in the western United States.

For example, the annual precipitation in Southeast Lower-Saxony has a uniform distribution over the year. In the mountains, it ranges up to 55 in. (1400 m.m.), and in the lowlands it may be as little as 20 in. (500 m.m.).

These hydrological relations are reflected in the runoffs of the rivers. In Saxony, the density of rivers is, for each square mile of land, 0.77 miles of river. The main river is the Aller, which, as the last big right contributary, flows into the Weser at a point about 60 miles upstream from its mouth, the North Sea. The quantity of ground water available depends mainly on hydrological conditions. These vary considerably for each of the many river basins in Germany. But according to these conditions and the analysis thereof, Southeast Lower-Saxony consists of the following:

1. The Harz Mountains, with a thick layer of podzol soil and high precipitation. Chiefly a source and site of lumber industry.

2. The northern foothills of the Harz Mountains are preferred for agricultural use. This area has fertile loam and clay soils that are easily washed away from the surface.

3. The Lowland, which extends north of Brunswick into the heath. It has sandy soil and a low rate of precipitation.

The discharge of the rivers, therefore, differs and reaches values of 1.230 million gallons per day per square mile (25 l/sec. km^2) in the South and 0.148 million gallons per day per square mile (4 l/sec. km^2) in the North. The volume of the annual discharge of the Aller River at the mouth of the Oker River is 0.47 acre feet per year (580 million m^3/year).

The purpose of all work connected with flood control of the rivers is the control and management of surface runoff. Recently, these aims have been achieved by "flood water retention basins."

At the end of the Middle Ages, storage ponds were constructed in the Harz Mountains to prevent floods and make possible the utilization of mountain rivers and to create water power for gold and silver mines. Later, they were used to improve runoff conditions and for surface irrigation. By the beginning of the 18th century, larger dams had been built in the Harz Mountains, and the last dams were built in recent years. Today, however, a floodwater retention basin system located in the lowlands is preferred.

These water resource problems were increased in recent years by problems of public water supply and improving conditions for the period of low flow in the rivers as related to the need for irrigation water and improving the quality of the water (pure water).

For these purposes, a dam such as that in the illustration is used. The construction of the wall consists, in the lower part, of an arch dam and in the upper part, of a gravity dam. The whole dam has a height of 240 feet and a width of 26 feet.

3. Water Ways

In order to describe hydrologic relations, it is necessary to give an account of artificial waterways. In West Germany, the network of waterways has a total length of 1800 miles and consists of rivers, channeled rivers, and canals, on which about 30% or 200 million tons of the total traffic volume, is transported. At the present time, a number of canals are being improved or constructed. One of the projects under construction is the Rhine-Main-Danube connection -- 2200 miles in length -- which provides a waterway and enables larger vessels to move from the North Sea to the Black Sea.

The construction of waterways alters the landscape considerably, but the conditions for location of industries are improved. In several cases, it was shown in Germany that because of the construction of waterways, the number of persons employed in industry increased threefold in the so-called "wet" areas as compared to the so-called "dry" areas untouched by the canal. The turnover of industry became fourfold what it had been in "wet" areas and this fact has natural consequences for water management.

In our example, the Middle-Land Canal crosses the northern part of Germany. The Middle-Land Canal is an artificial waterway system 320 miles in length. (See Figure 5) It passes north of the central German mountain chain and connects rivers such as the Rhine, Weser, Elbe, and Oder Rivers. The big German rivers flow from south to north. A natural crossing link from west to east is missing, and was created by the Middle-Land Canal, which was completed in 1938. The canal system has been modernized and improved to meet the steadily increasing requirements of traffic. At present, the remaining sections of the canal are being improved to facilitate traffic with the so-called "Europe vessel," which has a draw of 1350 tons. The total cost for improving the whole canal system will be $370 million.

In the Southeast Lower-Saxony area, the Middle-Land Canal passes through a summit pond, 40 miles in length, which is artificially fed by repumping the water from the lower areas, where a surplus of water is available.

Investigations were made to find out whether canal losses of 81,000 acre feet (100 million m^3) could be compensated by drawing water from the natural water courses in Southeast Lower-Saxony. This, however, constitutes a severe interference with the water resources, especially in times of droughts.

At present, a connection between the Middle-Land Canal and the North Sea -- the North-South Canal, 70 miles in length -- is under construction. The summit pond of the canal system is thus increased by an additional 35 miles. The construction costs of the new canal are $160 million.

In order to save water, the Leichtweiss Institute proposed erecting two ship elevators at the end of the reaches; the type of ship elevator shown in the illustration will be built. (See Figure 6)

4. Water Power

There is a demand for electric power in West Germany of about 180 billion kwh. The main part of the demand, the so-called base load, is more than 80% of the whole demand and is generated by power stations using coal, peat, natural gas, or nuclear power.

Water power is used to cover the base load only in the Alps, where rivers with high discharges make possible an economic operation of river power stations. An effective use of water power is gained by the use of storage power stations or pump storage stations to meet the demand in peak hours.

Generally, in Germany, the distance between a storage power station and the center of power consumption is considerable. The capacity of such stations is too small to meet the total requirements, and in addition to that transportation through overhead lines causes high energy losses.

Nowadays, pumped storage generating stations are installed within a short distance of the centers of electric power consumption. They are no longer connected with major water courses. The plants are only bound to the minimum hydraulic head, which does not exceed 500 feet (the elevation difference between the two basins).

In Germany, pump storage stations have been in operation for several decades. Their construction has been steadily improved; especially, the time required to reach full load operation has been shortened to 1.5 minutes. This time is very essential in avoiding blackouts. Because of this technique, major blackouts are almost unknown in Germany. Today there are 33 larger power plants in Germany with a total of 2,255 MW. A further plant, generating 880 MW, has reached an advanced stage of planning.

In Southeast Lower-Saxony, there is also a pumped storage generating station with a capacity of 220 MW which is available for the compound supply network within 60 seconds of starting time. (See Figure 7) There is no connection either from the upper basin or from the lower basin to a natural watercourse. Losses caused by evaporation and infiltration from the basins having a surface of 14 acres amount to 3 cfs (100 L/sec) and are replaced by taking water from the bypassing river. The long-shaped form of the lower basin situated in the flood-plains was one of those tested by an hydraulic model at the Leichtweiss Institute.

5. Water Supply

Analysis and prognosis of the economic situation in a development area permit the assessing of tendencies which will determine future requirements of water resources in the field of water supply, flood control, sewage disposal, and recreation.

The water demand is connected with the degree of industrialization, and may differ in a development area.

In larger and relatively equally populated agglomeration areas, the river with the higher discharge is used for the supply of fresh water stored by dams or infiltration of surface water into the groundwater. The river with the lower discharge received the treated sewage water.

Examples can be given in Germany in the Ruhr River basin and the neighboring Emscher River basin, where there are 4,500 million gallons (17 million m^3) sewage water and 185 million gallons (700,000 m^3) dry sewage sludge each year. Central Wurttemberg and the Stuttgart area are supplied with 50 million gallons each day by a pipeline 100 miles in length coming from Lake Boden. The sewage water is carried by a culvert system into the Neckar River.

Conditions are similar in Southeast Lower-Saxony, where rivers rich in water from the Harz Mountains are used to provide water supply; and the neighboring smaller rivers, passing through the northern extensions, collect the treated sewage water.

From the standpoint of the water supply in the area to be developed, there are two different regions, the South and the North. Some figures may illustrate these facts: in the southern part, 74% of the inhabitants used a central water supply system in 1948; and by 1963 this group was increased to 96.6%. In the northern part, in 1963, only 64% of the inhabitants had access to a central water supply. The average for West Germany amounted to 88% at that time. (See Figure 8)

In West Germany, the total demand for water from the public supply system reaches some 3,700 billion gallons (14 million m^3) for private and industrial use, the latter consuming 2,800 billion gallons (1,700 million m^3) of the total. The annual increase rate was 2% in the last few years. The average consumption by households and excluding industries has at present a value of 30 gallon/inh.day (110 l/ET).

The annual investments in the public water supply approach a total of about $250 million, which averages out to one cent per 22 gallons of water.

In the example of Southeast Lower-Saxony, the present average demand for water for domestic and industrial use is 26 billion gallons a year (105 million m^3/year), which equals a total specific water demand of 66 gal/inh. day (250 l/ET). (See Figure 9)

Underlying the assumptions mentioned so far, and the expected evolution, the total demand for water will reach 80 gal/inh. day (300 l/ET) in the year 2000.

The total demand for water will be 45 billion gallons a year (170 million m^3/year) by the year 2000. That represents an increase of 70%, mainly because of a greater demand for water for domestic use; while the water used for industrial purposes will increase only slightly because of improved systems of water circulation in industry.

In addition to water demand, the water resources available for economic utilization can also be roughly assessed.

In West Germany, the annual precipitation has a volume of 160 million acre feet (200,000 million m^3), from which 70 million acre feet (90,000 million m^3) are added to the surface runoff and 90 million acre feet (110,000 million m^3) are lost through evaporation and transpiration. The total water supply, including the provision of cooling water, requires 11 million acre feet (1,400 million m^3), of which 2.40 million acre feet (3,000 million m^3) are lost due to evaporation. The main problem is to produce water of good quality at specific places in a sufficient quantity. This can be solved only after detailed investigation.

In the example, the demand for future irrigation water is covered by drawing upon rivers or from the North-South Canal, which can supply irrigation water taken from the Elbe River. Besides these resources and the extension of existing plants, there are no other sources of surface water. Groundwater, as a resource, however, is available to the extent of 8.5 billion gallons (32 million m^3). (See Figure 10)

The capacity could be increased manyfold if a retention basin of 80,000 acre feet (100 million m^3) were constructed as a infiltration basin. This project, however, requires a certain quality of water which cannot be guaranteed at present.

Accessible surface water stores with a total volume of 10 billion gallons (38.7 million m^3), available only in the Harz Mountains, can be used by constructing new dams.

The withdrawal of fresh water from the existing dams should be deferred as long as possible, at least as long as other measures for upgrading the quality of river water remains effective. This is necessary to maintain normal biological conditions in the river and the possibility of obtaining drinking water by infiltration from the rivers into the ground water area.

The quantity of water demanded in the year 2000 will amount to 19 billion gallons a year (72 million m^3/year) as compared with the economically available water supply of 20 billion gallons a year (76 million m^3/year). In the year 2000, the demand will reach the present available supply. Meeting water demand in the future will be possible only through using groundwater and having reservoirs in which to store surface water. (See Figure 11)

The total water supply in future has to be based on having an adequate compound network which can supply all larger coherent communities with water of good quality and sufficient quantity. (See Figure 12)

In the year 2000, the water supply cannot be safe without a compound network based on a long-distance pipeline. The measures necessary to meet these demands require investments of $232 million. (See Figure 13)

A long-distance circuitline used in conjuction with large waterworks has the following advantages:

1. Sufficient water supply of satisfactory quality can be guaranteed even during droughts.

2. Large scale landscaping measures are made possible by combining several components of water management. If, in this case, the quantities supplied by the individual large waterworks are seasonally regulated, it might be possible to hold the groundwater table for the waterworks on a favorable level during the crop-growing period. Furthermore, the river valley downstream of a dam gets a better protection against flood by increasing the output of drinking water from the dam. Thus in summertime it is better to take the drinking water from dams. In wintertime, it is preferable to use groundwater for this purpose.

6. Sewage Treatment

The purification of groundwater and surface water will be an essential issue in the future. The state of development of sewage treatment is even worse than that of water supply. Although during the last 15 years in West Germany a sum of $3 billion was spent on sewage disposal, in the year 1967 alone $450 million, only 50% of the population are joined to a central sewage treatment plant. However, it must be mentioned that in Germany only 60% of the population live in towns with more than 10,000 inhabitants. (see Figure 14)

These towns are provided with sewage treatment plants of good standards, as is shown in the example. The smaller communities have no satisfactory sewage treatment plants and they deal with the problem on an individual household basis. The costs of measures of sewage disposal and treatment in the example amount to $125 million.

The insufficient number of larger sewage treatment plants results in an unsatisfactory water quality in the rivers. In Germany, methods for classification of water quality are the so-called saprobic system (using bacterial criteria) or the improved system of purification degrees (representing a synthesis of biological and physical examinations.) According to the latter system, the following four degrees are distinguished (Figure 15):

class 1	nearly pure
class 2	slightly polluted
class 3	moderately polluted
class 4	heavily polluted

The techniques of sewage treatment have been developed to such an extent that excessive pollution of a water course, exceeding its natural process of repurification, can be avoided.

An economically unsolved problem, however, is how to avoid a process which is caused by fertilizing sales, originating from washed out fertilizers, or concentrated sewage feed which can result in the formation of a rich water flora having negative consequences.

Although the use of internal water circulating systems and the extension of sewage treatment plans improved the conditions in the water courses in the past decade, they are still worse than sixty years ago.

The aim for the future is to restore conditions allowing for watersports and fishing.

In addition to the extension of sewage treatment plants, measures are urgently needed to improve the biological repurification through technical means, e.g., artificial aeration or the construction of sedimentation basins. Water quality of a river so treated is not so much endangered by sudden failures in the operation of sewage treatment plants. Furthermore, the requirements of improving water quality are related to the aims of flood protection.

7. Flood Protection

It may seem a bit unusual to close a discussion of the measures of water management with a few remarks about flood control. However, it must be pointed out that a change in concept has occurred.

In contrast to the traditional concept of flood control -- that it is a line fashioned to protect against flooding by constructing dams or dikes -- an areal fashioned flood control, combined with an improvement in water quality, is preferred today.

This is accomplished by use of flood-water retaining basins with the function of retarding the uncontrollable peak flood and diminishing it to an acceptable discharge. The bottom of these basins is agriculturally or silviculturally used to the extent that it does not serve as a permanent pool to improve the water quality. The large dike constructions that have been used in the past are no longer practical, and have been abandoned for reasons of water management and of reduction of maintenance costs.

The basins have different functions. If the downstream reach has a limited capacity, as might be caused by new industrial or living areas along the waterfront of a river, basins have to retain the highest flood. In another case, the basins may have compensating functions in order to avoid a more frequent flooding that could be caused by an increase of the above-mentioned areas. Finally, they would improve protection against floods during the crop-growing season.

The basins are constructed according to their functions and vary in a wide range. In Germany, every form can be found -- ranging from a simple weir to a traditional dam. In every case it is desirable to prevent an areal extension of floods in the upper region. Thus, only a small storage space is required in an optimal operation. (See Figure 16)

For our example in Southeast Lower-Saxony, a system of retarding basins in the Aller River watershed is planned and is already under construction. There will be 13 large retention basins with a capacity of from 400 to 90,000 acre feet (0.5 to 110 million m^3), providing a storage of 100,000 acre feet (123 million m^3). Thus some 9,100 acres (22,500 hektar) of agriculturally usable land will be protected against floods. The costs of the retention basins amount to some $60 million. (See Figure 17)

There will be an interconnected operation for the basins. Some of the basins are automatically operated by employing new kinds of outlet structures. Other basins require operation personnel. At present, our Institute is studying the provisions necessary for a central operation of all basins. The operation is based on hydrological data, especially since the daily precipitation allows only a short-range forecast. An optimal operation will be reached by the use of computers.

Another example which demonstrates how closely retention basins are arranged in metropolitan areas exists in the city of Hamburg, where in an area of 300 square miles (780 km^3) about 80 basins have been constructed or are planned with very interesting concepts of design and operation.

8. Distributory Network

To conclude, all aspects may be combined in a distributory network. A region is organized by zones of congestion, zones reserved for agricultural use, zones of transition, areas of underdeveloped industries, areas with unchanged structures, and recreation areas. (See Figure 18)

The adequate providing of a region with national parks and recreational areas both close to larger towns and in remote and thinly populated areas is essential to improve the living conditions. According to this concept, the region must be developed in a way that will support the existence of the population in the area concerned.

9. Conclusion

Finally, let me give a summary of all measures of water management in a region and make some comparisons on how far man can influence the hydrologic cycle.

The comparison of the hydrologic cycle of today and the future shows that in the example of Southeast Lower-Saxony more than 70% of precipitation forming the basis of natural water supply is lost by evapotranspiration and thus removed from human interference. At present, the water supply for domestic and industrial use is only 0.75 in. (19 mm.) or 3% of the annual precipitation of 26.6 in. (676 mm.). After thirty years it will be built up to 6%, which appears to be only a small increase. (see Figure 19)

Taking into account the fact that surface runoff has a share of 20%, and that only a small percentage of this amount can be used because of quantity and quality of water, it will be clear what has to be done in the field of water resources development.

The kind of measures that can be taken has been shown for the relatively small area of Southeast Lower-Saxony. The problems to be solved are of importance, also, for other comparable areas in Germany where the structure is steadily changing because of the increasing degree of industrialization.

Definite measures to secure natural water resources and, so, the existence of an increasing population of Germans in the coming decades, are needed.

With the help of comprehensive water resource planning, man endeavors to become independent of his natural environments. The whole economy must be able to function apart from the natural cycle, e.g., the cycle of a hydrologic year. The economy becomes more and more sensitive to fluctuations in the supply of water because water is a vital element.

It should be the task of every specialist in the field of water resources to draw attention to this bottleneck problem. He should try to avoid such crises through the economic employment of all modern techniques.

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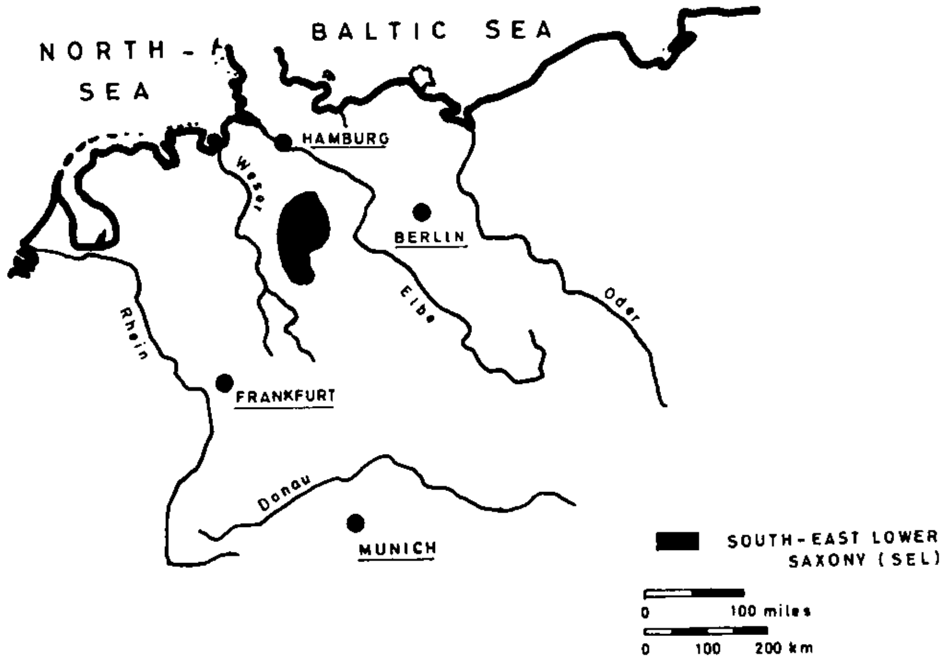

NORTH-
SEA
BALTIC SEA
HAMBURG
BERLIN
FRANKFURT
MUNICH
Weser
Rhein
Elbe
Oder
Donau
SOUTH-EAST LOWER
SAXONY (SEL)
0
100 miles
0
100
200 km

Figure 1

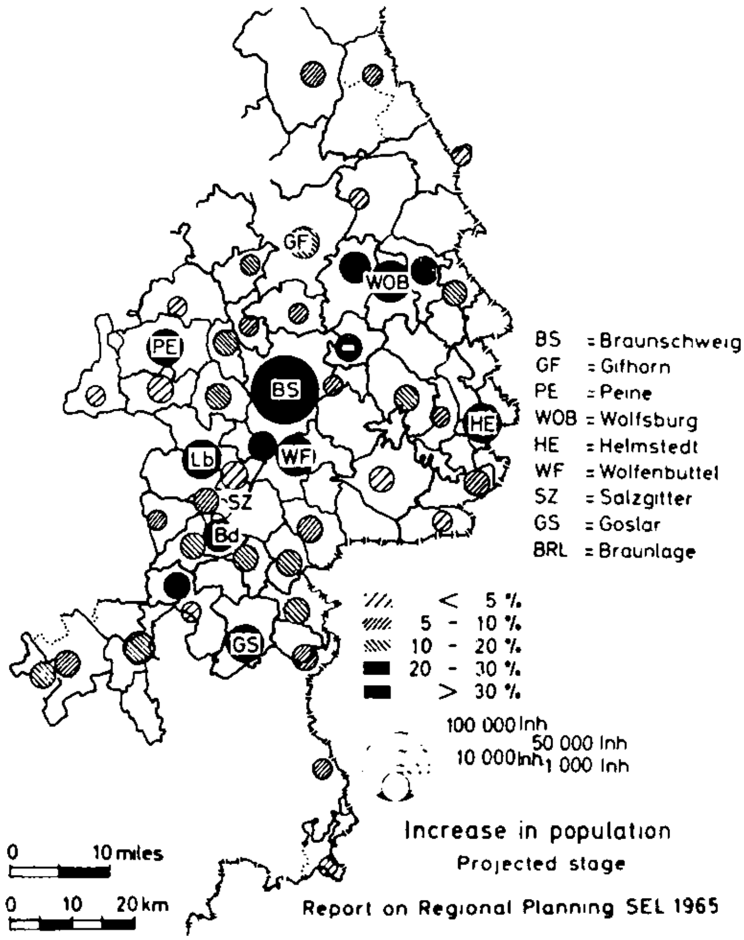
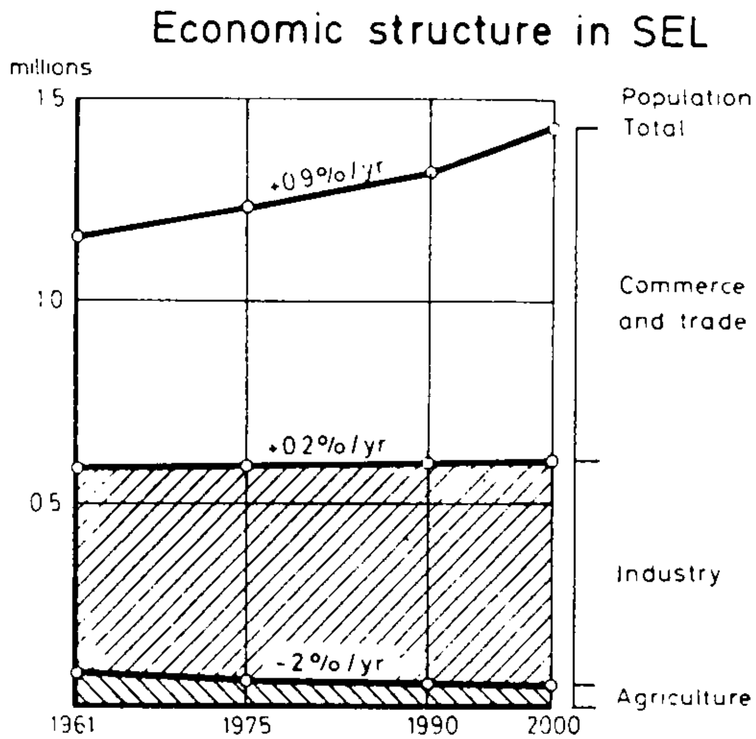

BS = Braunschweig
GF = Gifhorn
PE = Peine
WOB = Wolfsburg
HE = Helmstedt
WF = Wolfenbuttel
SZ = Salzgitter
GS = Goslar
BRL = Braunlage
< 5 %
5 - 10 %
10 - 20 %
20 - 30 %
> 30 %
100 000 Inh
50 000 Inh
10 000 Inh
1 000 Inh
Increase in population
Projected stage
Report on Regional Planning SEL 1965
0 10 miles
0 10 20 km
Lb
Bd

Figure 2



Report on Regional Planning SEL 1965

Figure 3

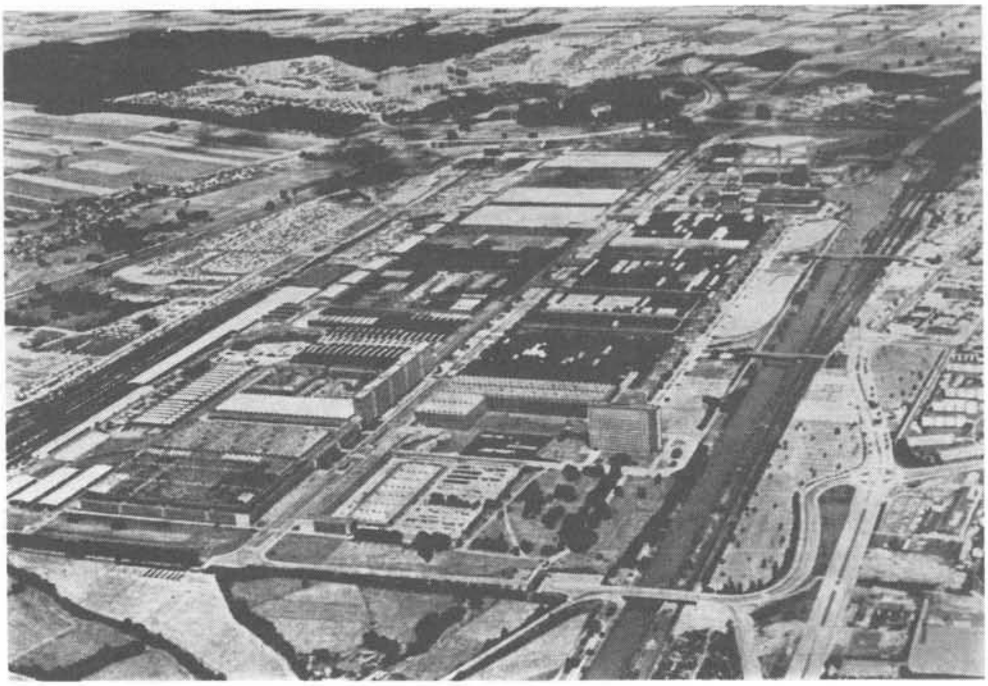
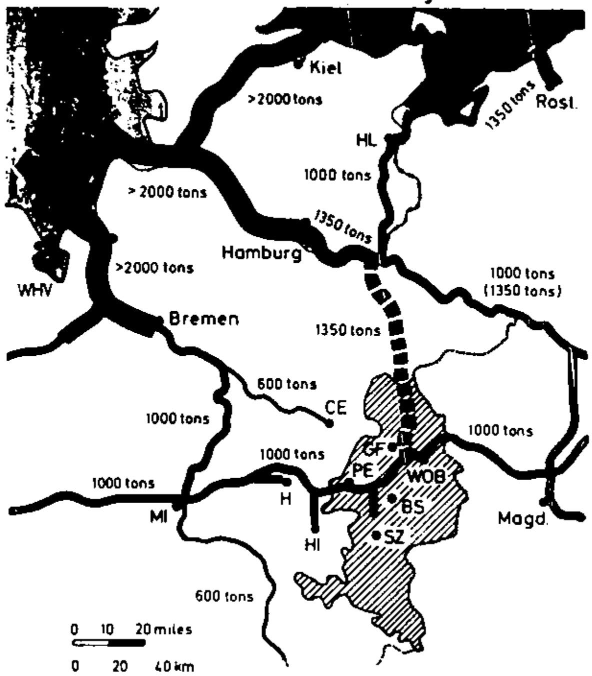


Figure 4

SEL in the network of waterways
in Northern Germany



Report on Regional Planning SEL 1965

Figure 5

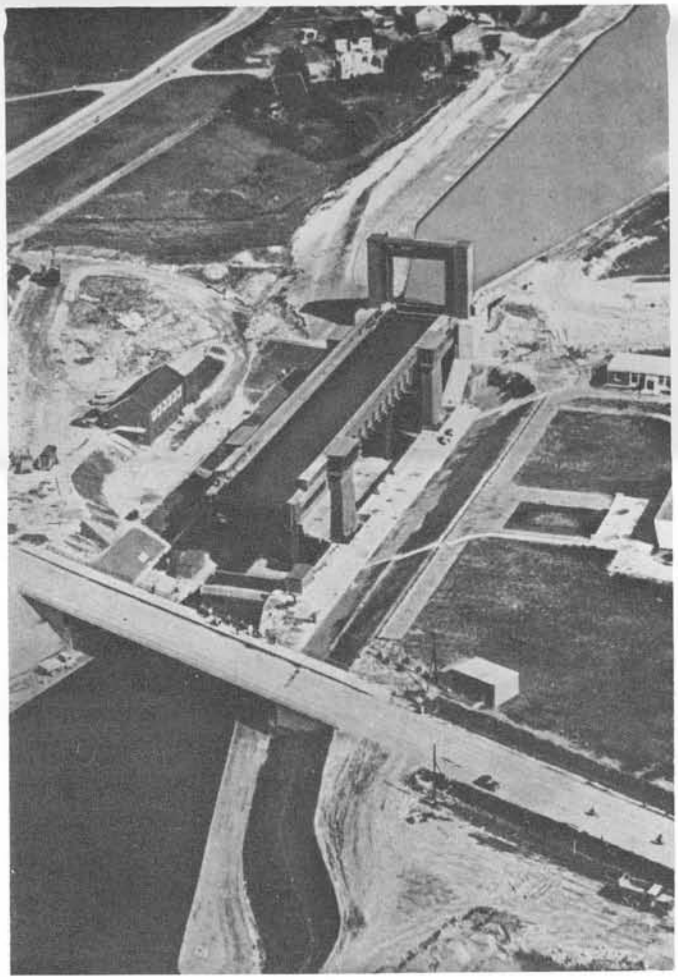


Figure 6

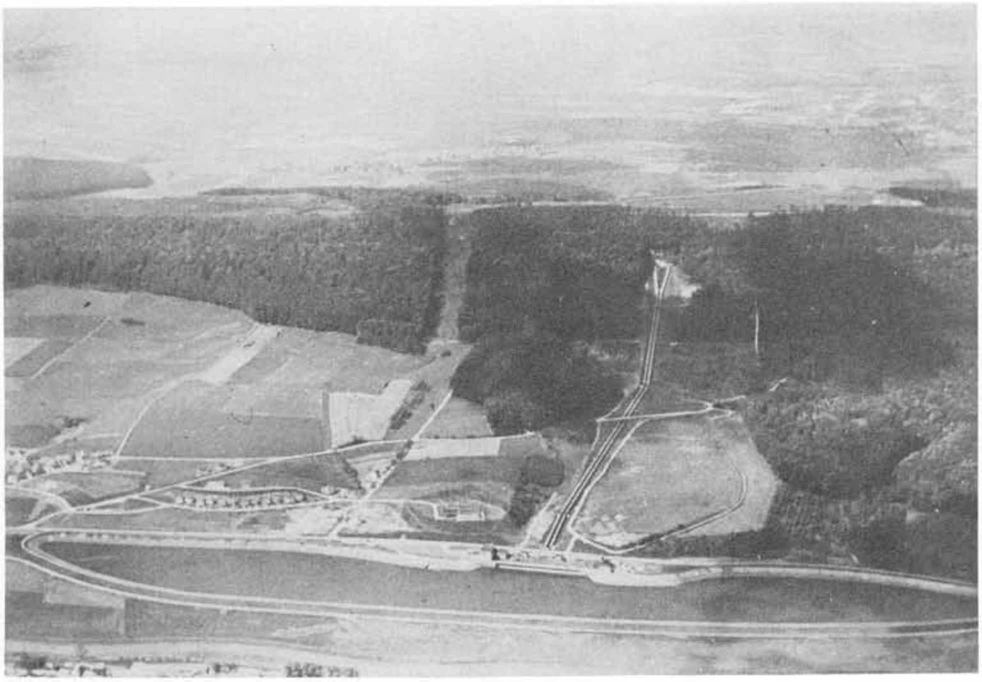
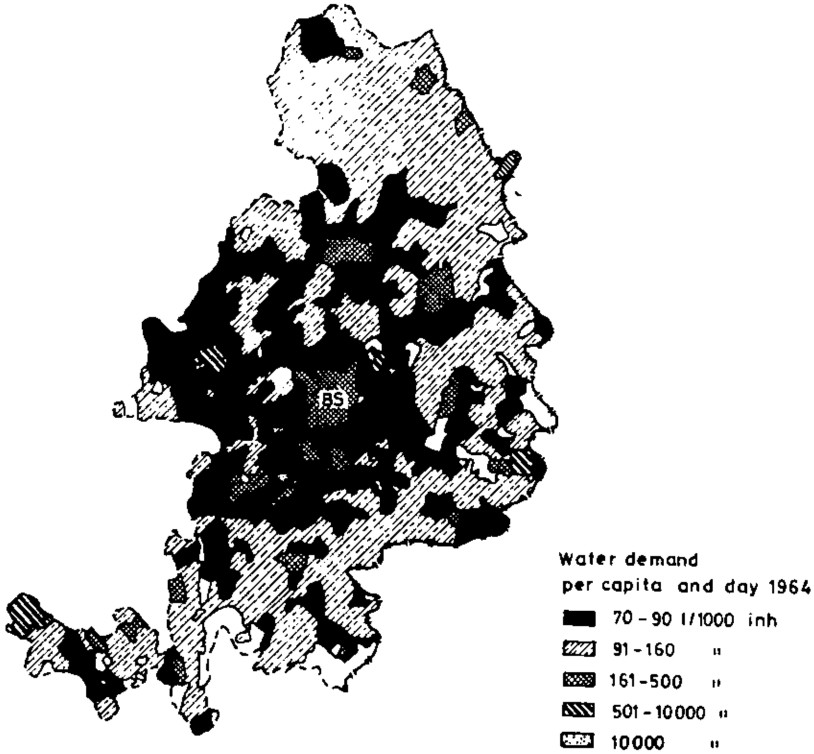


Figure 7



Regional Planning SEL-Water development

Figure 8

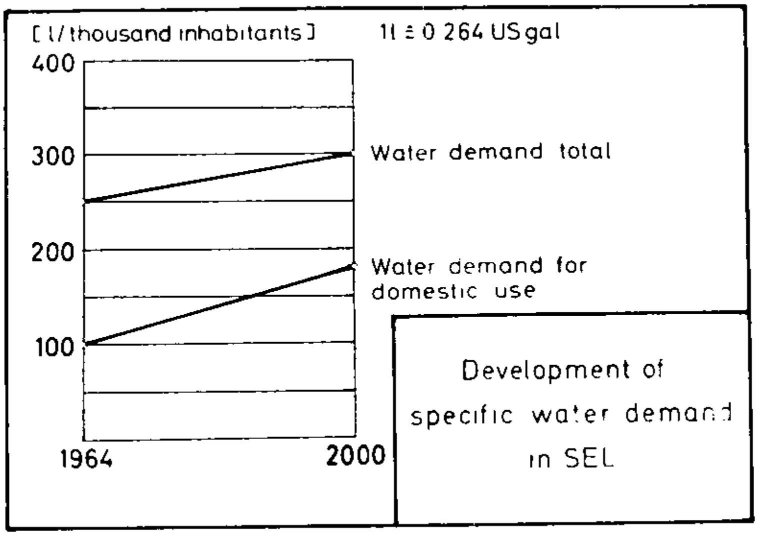
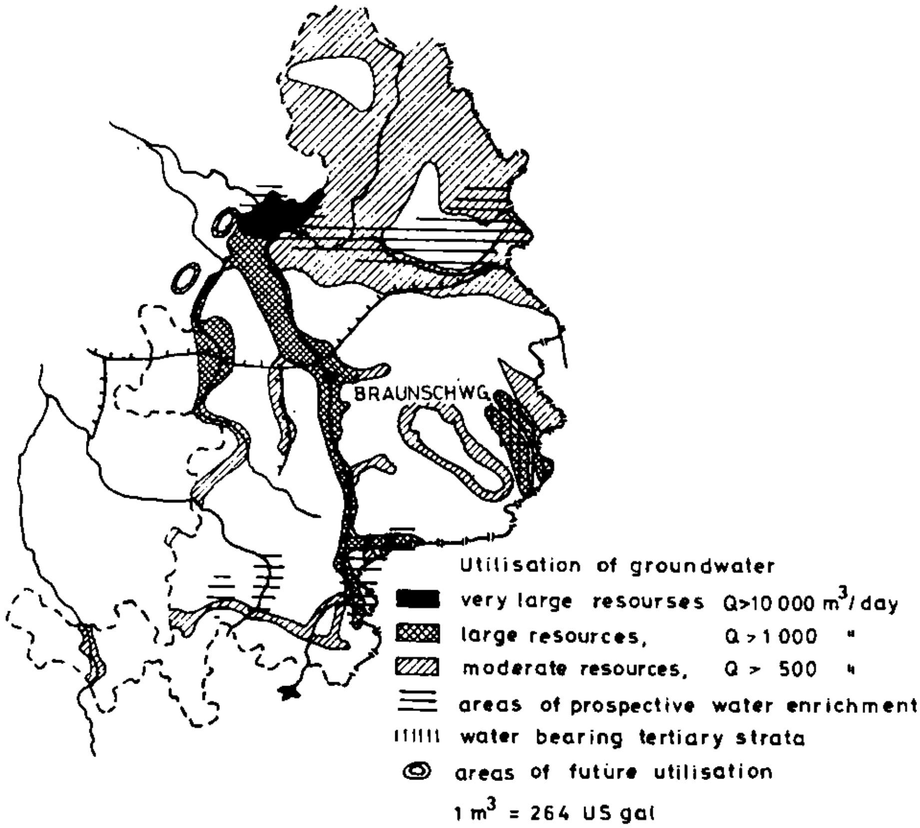

[l/thousand inhabitants]
1l ≙ 0 264 US gal
400
300
200
100
1964
2000
Water demand total
Water demand for domestic use
Development of specific water demand in SEL

Figure 9



Regional planning SEL - Water management

Figure 10

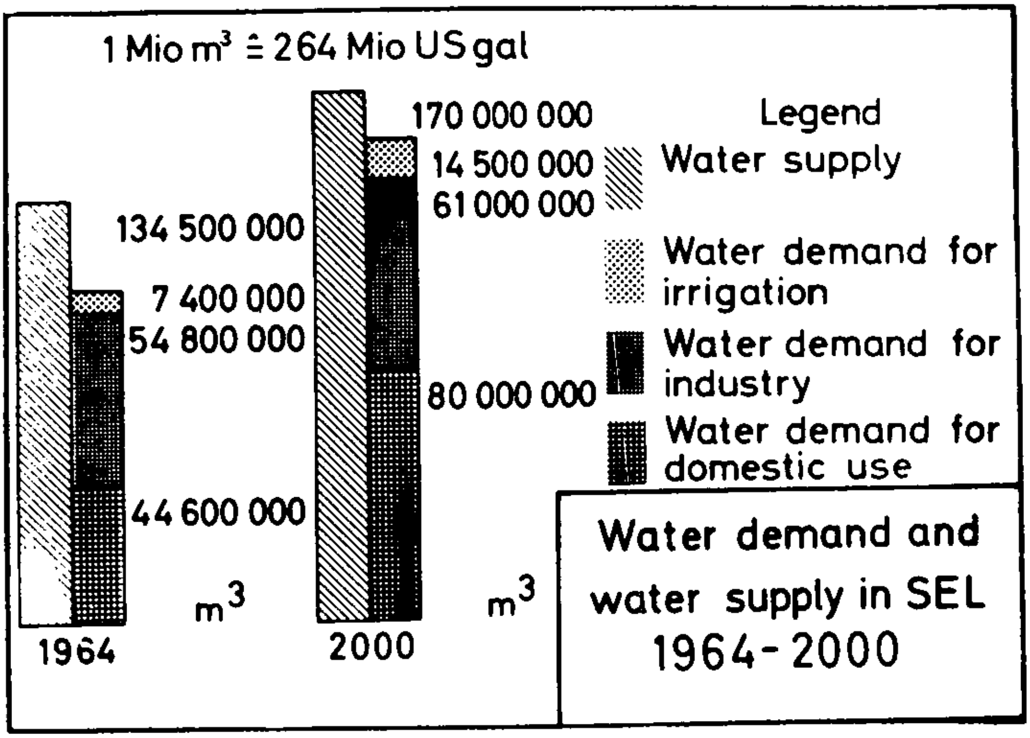

1 Mio m³ ≙ 264 Mio US gal
134 500 000
7 400 000
54 800 000
44 600 000
m³
1964
170 000 000
14 500 000
61 000 000
80 000 000
2000
m³
Legend
Water supply
Water demand for irrigation
Water demand for industry
Water demand for domestic use
Water demand and water supply in SEL 1964-2000

Figure 11

Balance of Water Supply in SEL

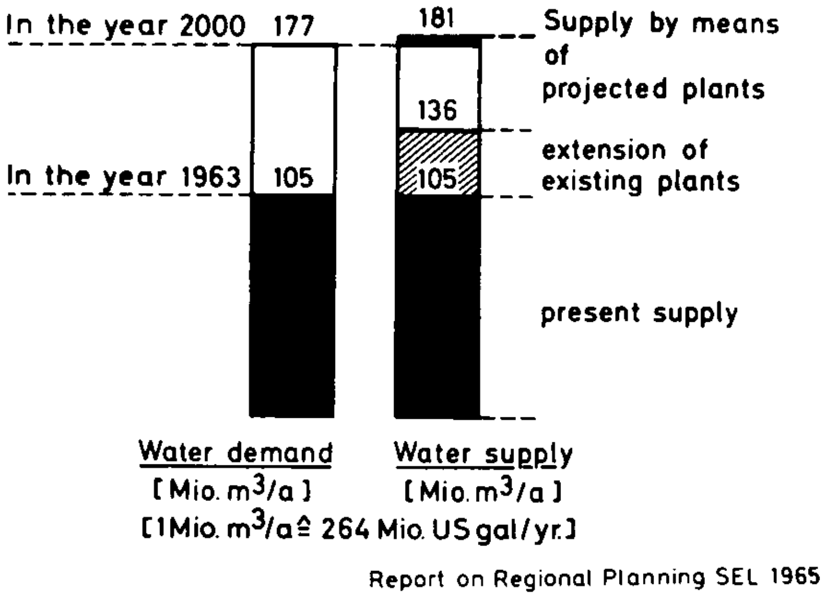
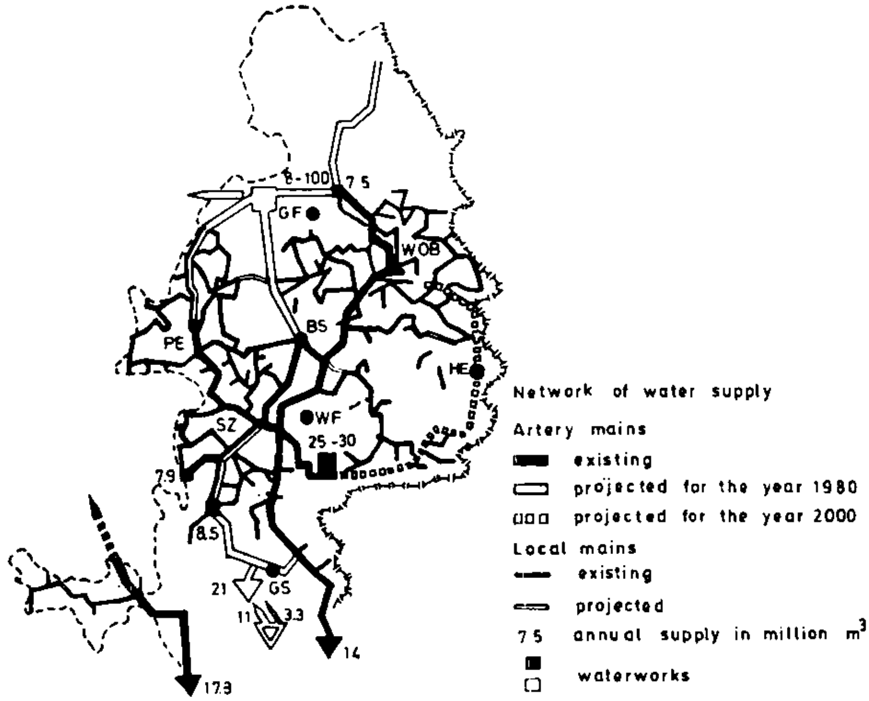


Figure 12



Regional planning SEL-Water management

Figure 13

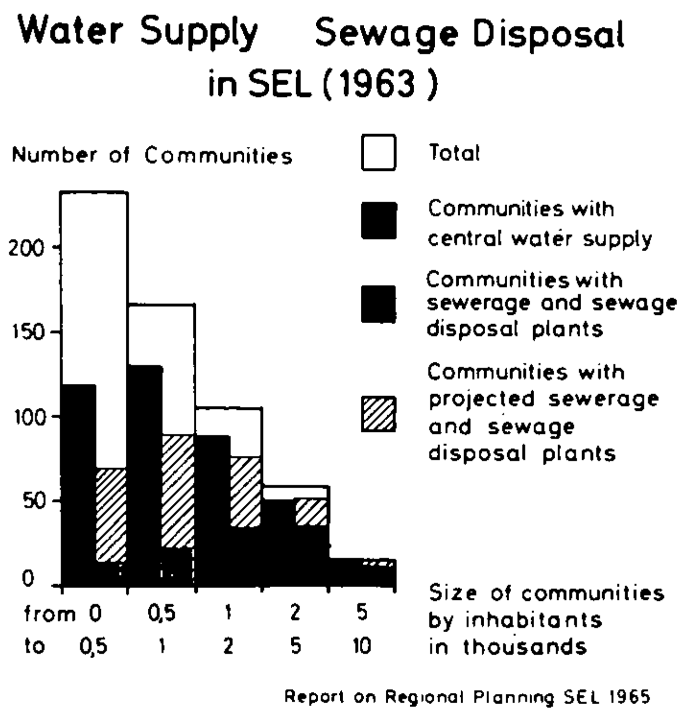
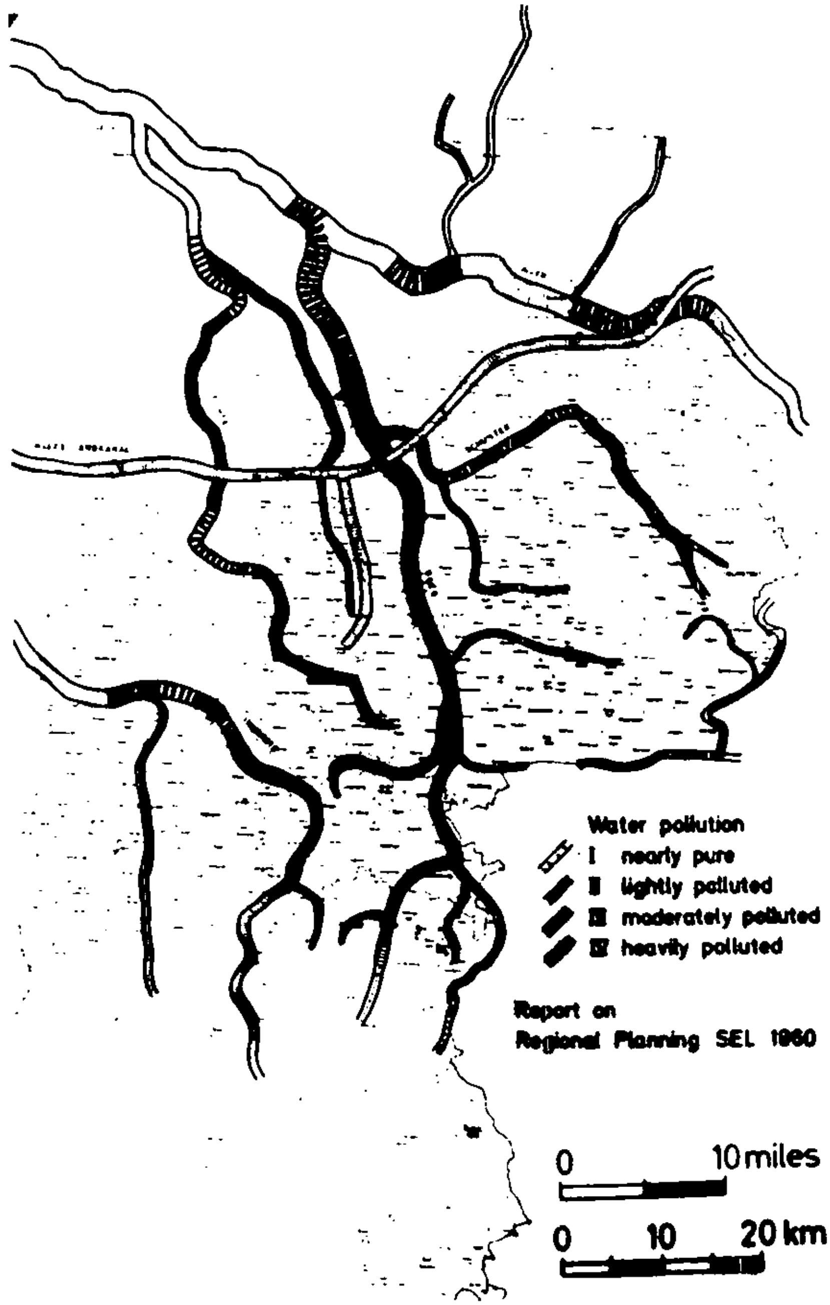

Water Supply Sewage Disposal
in SEL (1963)
Number of Communities
Total
Communities with central water supply
Communities with sewerage and sewage disposal plants
Communities with projected sewerage and sewage disposal plants
200
150
100
50
0
from 0 0,5 1 2 5
to 0,5 1 2 5 10
Size of communities by inhabitants in thousands
Report on Regional Planning SEL 1965

Figure 14



Water pollution
I nearly pure
II lightly polluted
III moderately polluted
IV heavily polluted
Report on
Regional Planning SEL 1960
0 10 miles
0 10 20 km

Figure 15

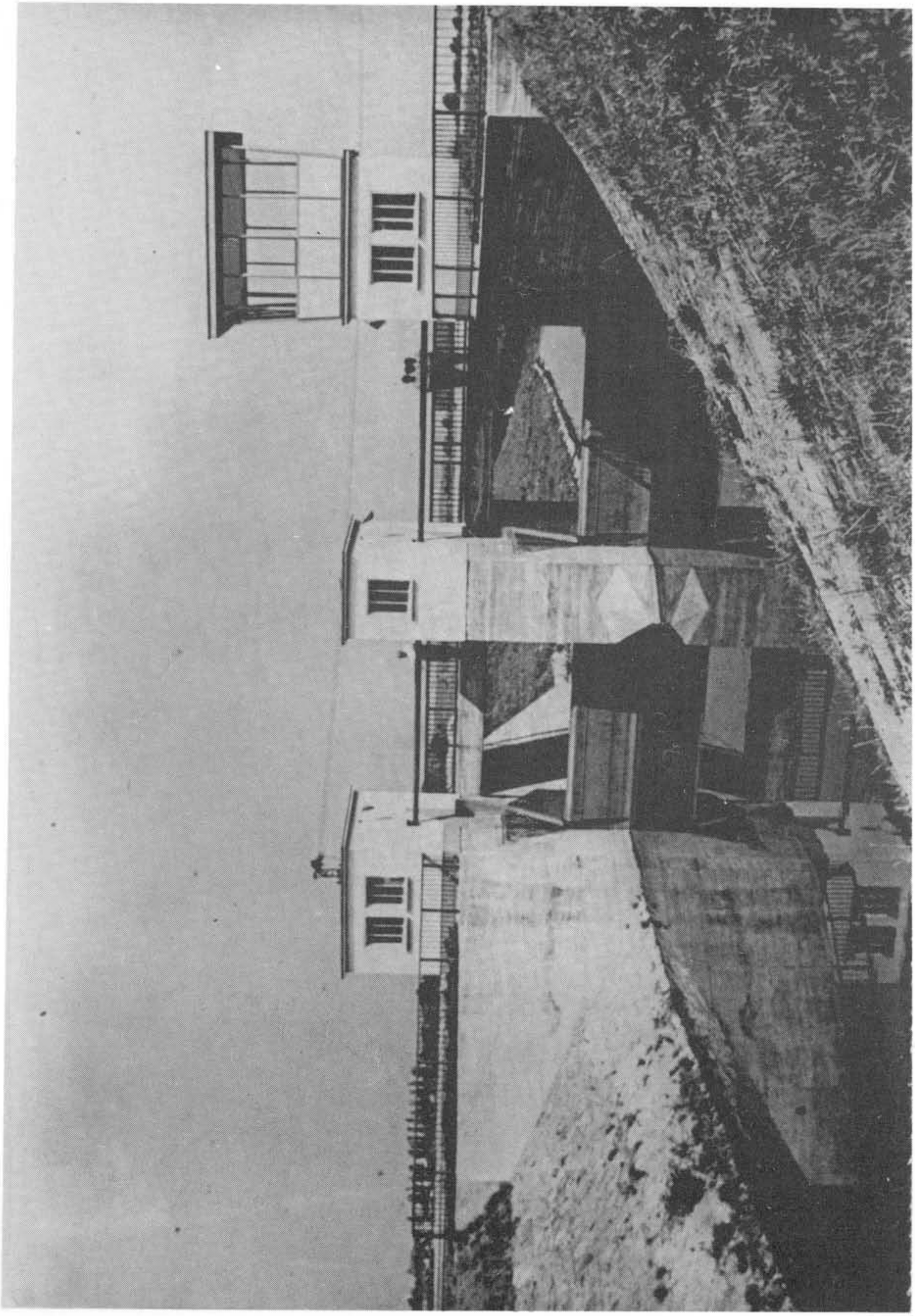
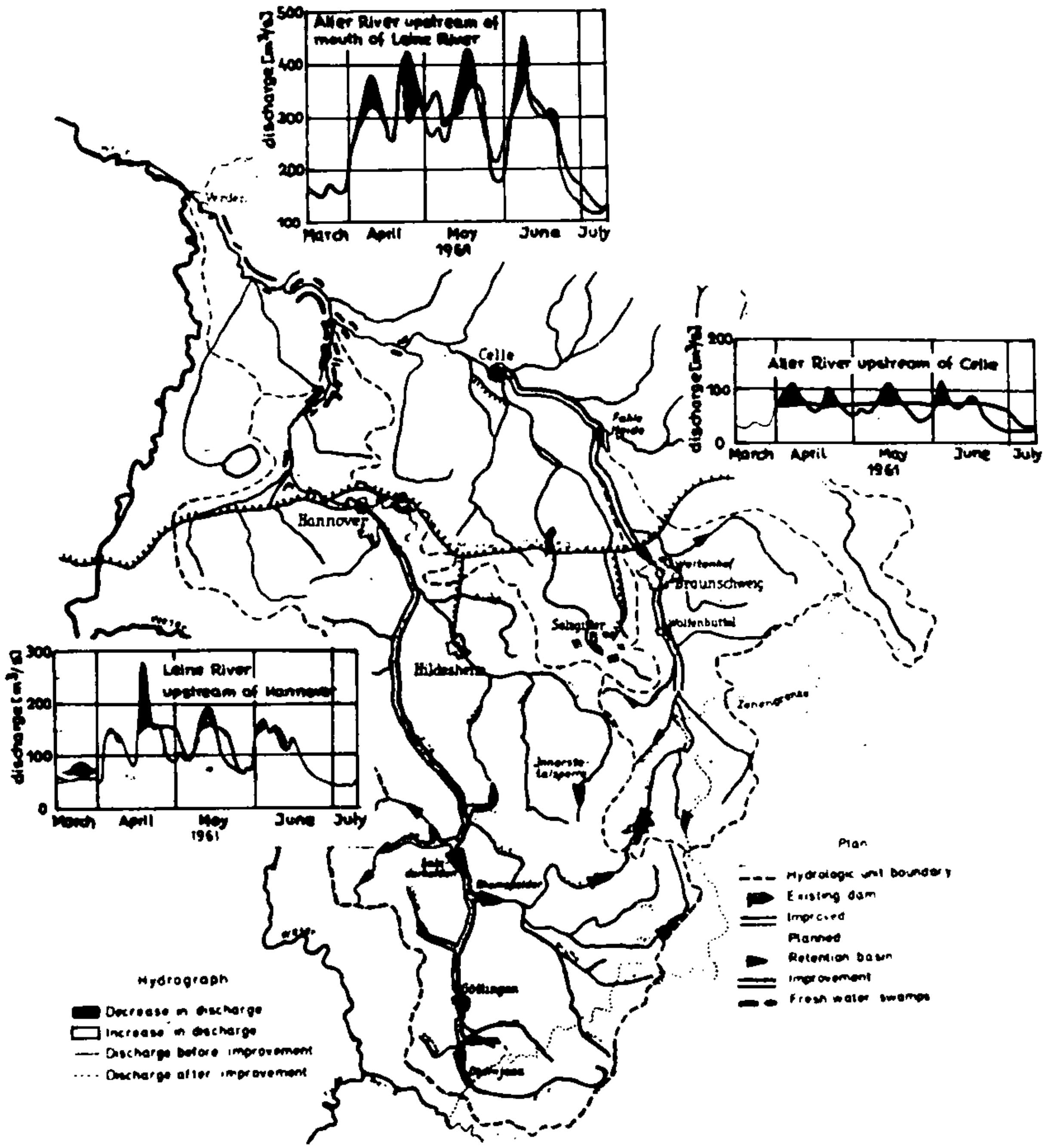


Figure 16



Flood control
in the Aller River Basin
Effect of Retention Basins on Hydrograph

Figure 17

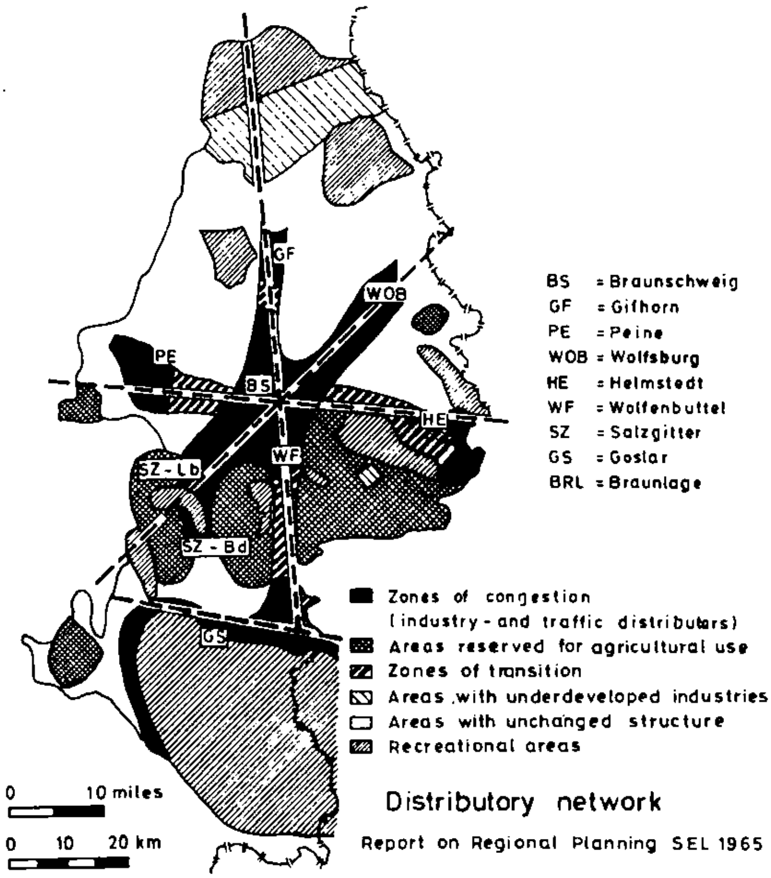

GF
WOB
PE
BS
HE
SZ - Lb
WF
SZ - Bd
GS
BS = Braunschweig
GF = Gifhorn
PE = Peine
WOB = Wolfsburg
HE = Helmstedt
WF = Wolfenbuttel
SZ = Salzgitter
GS = Goslar
BRL = Braunlage
Zones of congestion (industry - and traffic distributors)
Areas reserved for agricultural use
Zones of transition
Areas with underdeveloped industries
Areas with unchanged structure
Recreational areas
0 10 miles
0 10 20 km
Distributory network
Report on Regional Planning SEL 1965

Figure 18

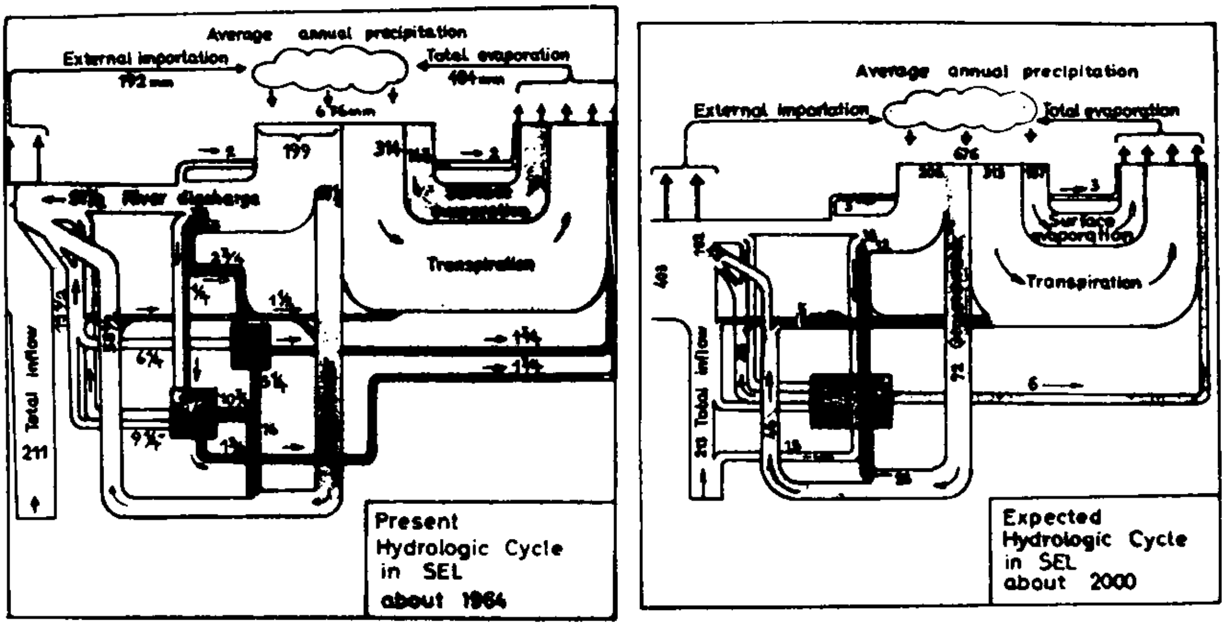

Average annual precipitation
External importation
192 mm
Total evaporation
484 mm
696 mm
199
River discharge
Transpiration
Total inflow
211
Present
Hydrologic Cycle
in SEL
about 1964
Average annual precipitation
External importation
Total evaporation
Surface evaporation
Transpiration
Total inflow
Expected
Hydrologic Cycle
in SEL
about 2000

Figure 19

WATER CONDENSATION FROM THE AIR

by

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The following scheme was devised, by R. D. Gerard and myself, to try to utilize the environment and make full use of all of its ensuing products.

The basic system utilizes the cold water found at depth in the ocean. When the cold water is extracted and passed through a suitable condensing system and moisture-laden air from the sea is passed over it, the moisture from the air is condensed on the coils. The water, when collected, forms a reservoir of useful fresh water. The cooled, dried air, being heavier than the surrounding air, will flow downhill, hugging the ground. With a suitably large plant and the proper arrangements, the cooled air can moderate the temperature of a whole community. The water brought from the depth has been warmed in the process; and, if led to a suitable enclosure, will be of importance to fish culture, for it is from the depth where maximum nutriments are present. The upwelling currents of the ocean occurring naturally, for instance off of the Peruvian Coast, are noted for their fabulous productivity.

What natural circumstances are most favorable for this scheme? Basic conditions can be satisfied in warm coastal areas that are humid because the breezes bring moisture from the sea. Another requirement is the availability of deep ocean water (about 600 meters) close to shore. This can be found near steep coasts, or where submarine canyons penetrate to a considerable distance into the continental shelves. Most of the coasts favorable to this system are in the region between 30° south latitude and 30° north latitude. Many of the areas in this region have a dearth of fresh water. (Figure 1 shows some of the world areas where offshore conditions are favorable.)

The trade winds, flowing predominantly in these regions, are noted for their regularity and reliability.

One site, especially favorable to this system, is St. Croix, one of the Virgin Islands. (See Figure 2) Approximately two miles from shore, water depths of 1000 meters and more can be found. At 600 meters sea water temperatures of 5°C are encountered. If a pipeline from this depth were connected to a tank near the shore, water very close to this temperature could be pumped from the tank into a condensing system, located on the shore at the top of the steep-sloped shore-line. The pump would only have to raise the water from a few feet below sea level (because of the friction in the pipeline) to the top of the condenser. The proposed system is shown schematically in Figure 3.

The condenser may require some engineering, as it will be different from most current condensing applications. A consideration of the design will be the choice of either running the sea water through the condenser only once, or recirculating or cascading it.

If the latter is desirable, more power will be required for pumping, but the condenser can be made much smaller. The condenser must be designed to allow the sea air to pass through it readily, contacting the cooled surfaces for rapid heat exchange, and must be of such materials (or coated with materials) as would facilitate the run-off of the condensate.

When the sea water is returned from the condenser to the sea, some of the power used to lift it to the condenser can be recovered by a suitable turbine system. We estimate this amount to be about one third.

If the condenser is placed atop a hill along the shore, the natural flow of air from the onshore breezes, as well as the fact that the cooled air is heavier than the surrounding air, will cause the air to flow inshore, staying close to the ground. The cooled region will create a climate more favorable for human habitation.

The effluent from the condenser, instead of being obnoxious as most effluents are, is, in contrast, highly desirable because the seawater contains the highest concentration of nutrients within the seawater column. Any sea area in which this is deposited will burst into violent life activity, since surface waters have generally been depleted of nutrients by the life processes.

The effluent can also be put into tanks for the culture of desirable organisms; or into bays with controlled closures where the local life forms can be expected to flourish. It is believed that this portion of the idea may well be its most important product. Tests of this part of the scheme are already underway.

In trade wind regions, the average humidity of the air sweeping onshore is about 80% and averages 25°C in temperature. With normal trade wind velocities, averaging about 6 cm/sec, more than 200 million gallons of fresh water, contained in the air as vapor, sweep across each kilometer of shore-line in the first hundred meters above the shore.

If it is assumed that the air, originally at 25°C, will be cooled to 12.5°C in our condenser, then about six grams of water could be recovered from each cubic meter of air, and about 540 grams of seawater would be raised for 5°C to 12.5°C to achieve this; i.e., ninety units of sea water would be needed to make one unit of fresh water.

While many factors cannot be precisely calculated, we have made some estimates of a system. For a plant providing one million gallons of fresh water per day, a pipeline would be required, around two meters in diameter, approximately one mile long, and requiring about 2000 horsepower to pump the sea water 60 m above sea level. For this purpose, about 90 million gallons of sea water, high in nutrients available for marine life cultures, would be handled.

Until many of the engineering criteria can be settled, it seems likely that only crude estimates of the requirement, such as those given above, can be made and consequently an evaluation of the economic factors of capital and running costs also.

TABLE 1.

CHANGES IN TEMPERATURE AND NUTRIENT CONCENTRATION OF SEAWATER IN FUNCTION OF DEPTH

Depth (m)	Temp. (°C)	PO_4 (μgA/lit.)	Nitrate (μgA/lit.)
1	26.97	0.02	0.08
99	24.67	0.03	0.40
197	19.54	0.12	2.53
395	14.85	0.76	11.98
592	9.62	1.65	23.01
987	5.56	1.71	17.31

ATLANTIS II, Cruise No. 14, hydrographic station No. 512. 9 December 1964, in the Virgin Island Basin, 17°57'N, 65°00'W, to depth 4453 m.
Abbreviation: μgA = microgram-atoms.

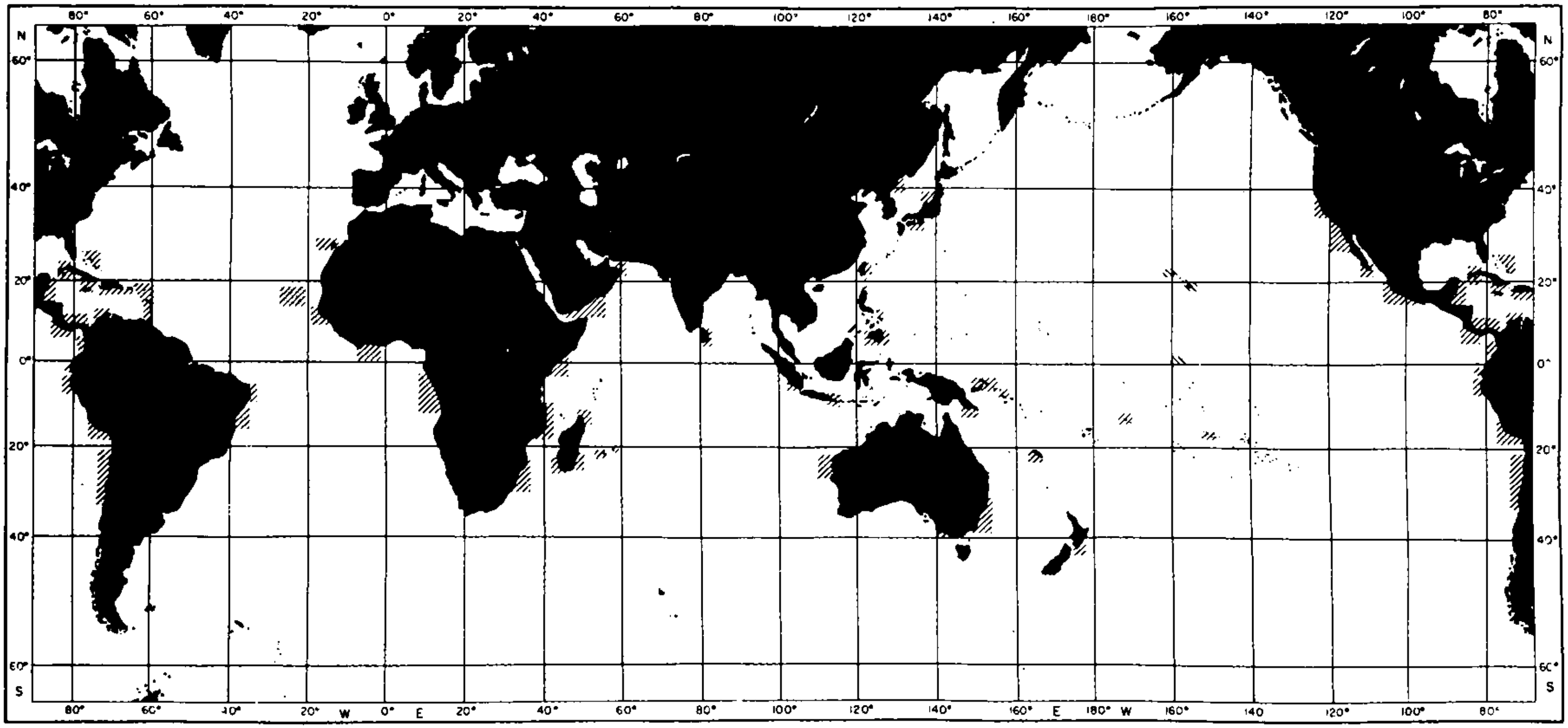

Figure 1 -- Principal tropical and mid-latitude areas where seawater of 500-meters depth may be found within 30 kilometers of the shore.

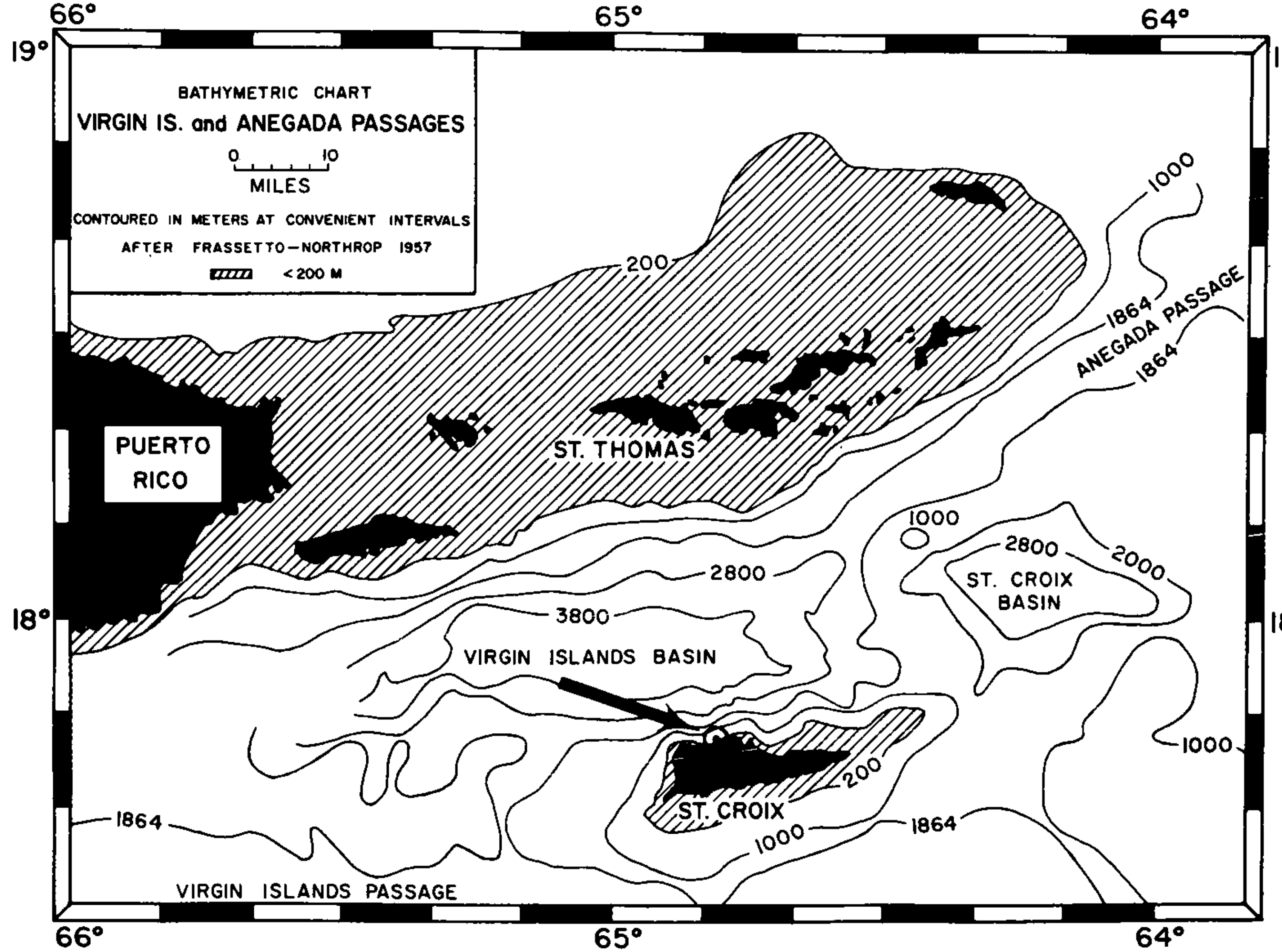

Figure 2 -- Bathymetric chart, Virgin Islands and Anegada Passages.

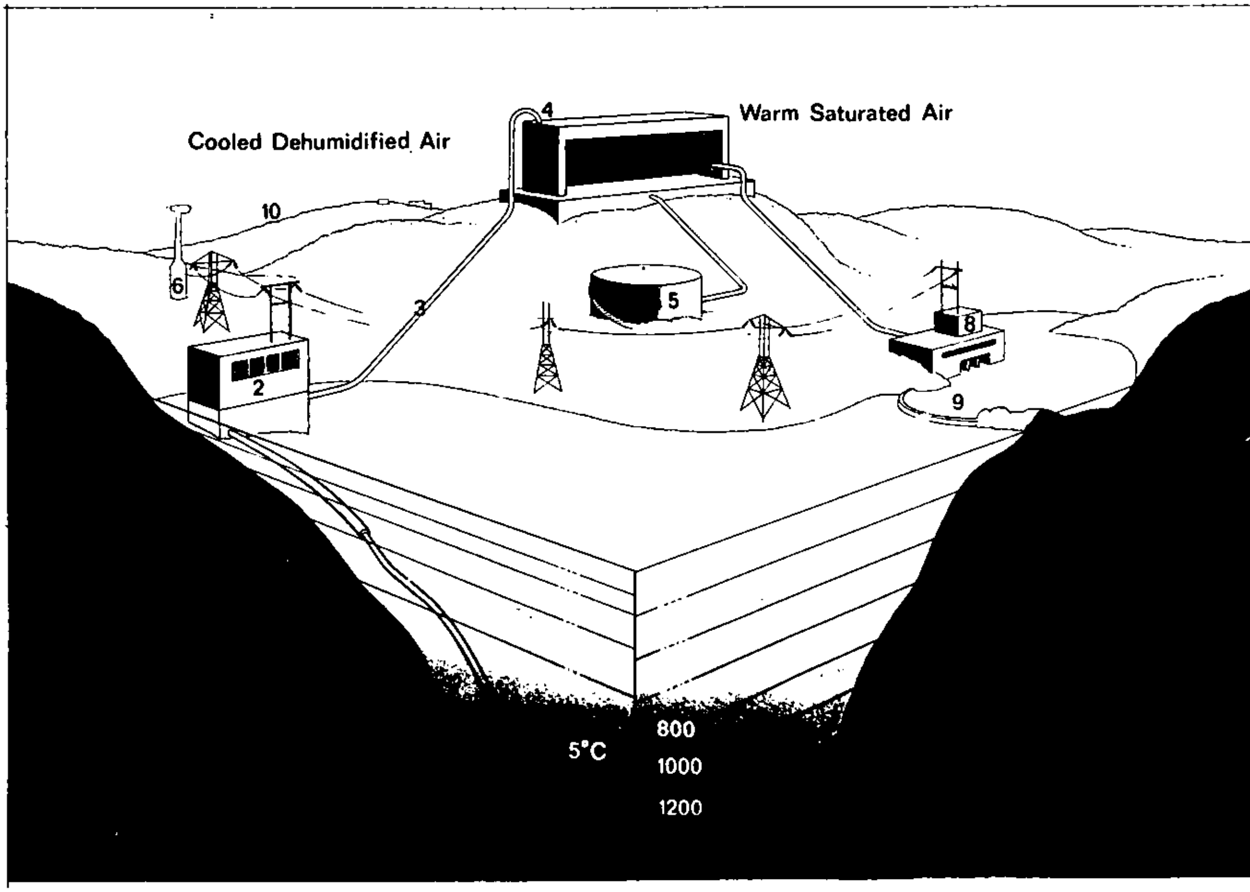

Figure 3 -- Schematic of proposed water recovery plant.

(1) Large-diameter pipe to deep water;
(2) Pump;
(3) Connecting pipe;
(4) Condenser;
(5) Freshwater reservoir;
(6) Windmill electric generator;
(7) Baffles to direct wind;
(8) Small turbine to recover water power;
(9) Lagoon receiving nutrient-rich water for aquiculture;
(10) Community enjoying cooled dehumidified air.

COMPUTATION OF PEAK DISCHARGE FROM SMALLER WATERSHEDS IN EAST CZECHOSLOVAKIA

by

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A research was conducted in the years 1963 - 1966 based on data previously collected from the eastern part of Czechoslavakia (Slovakia), where precipitation values, topographic characteristics, and rock formations of the different watersheds with areas 300 km^2 or less are of sufficient variety as to make possible an evaluation of peak rates of runoff from smaller watersheds. The aim of the research was to suggest such equations for maximum runoff as could be used generally not only for the chosen area, but anywhere the climatic and physiographic characteristics are the same.

Analogical aims were fostered by the U.S. Department of Commerce Bureau of Public Roads in establishing runoff formulas for small watersheds.[1] In accordance with these formulas, the entire United States is divided into only four runoff zones corresponding to the four main geological regions of North America. Similar investigations were pursued by Russian hydrologists as Ogievskij or Sokolovskij in working out equations to compute peak floods for drainage areas in accordance with the latitude and regions of the USSR but generally not taking in account the geological conditions.[2] [3] In both cases the formulas are rather more arbitrary than adapted to the physiographic regions and embrace too large territories without respecting the local characteristics. Classifying the watersheds on such a big scale, the peak rate runoff of small areas can not be expressed properly by these formulas because they become too generalized and are not governed by the local hydrological conditions which correspond to their size.

Slovakia serves well for this research, not only because of its relatively small area (49,008 km^2) and large variety of the factors which can influence the peak rate of runoff but also because of its unusually well preserved hydrological records about floods from smaller watersheds and covering a longer period of time.

The relief of Slovakia was created mainly by geomorphologic forces before the Tertiary Events. In Diluvium and Alluvium, they were only erosion activities. The ridges of the high mountains were furrowed by the Glacial Events and the surface erosion created typical Alpine formations. In areas where the glaciers were not effective anymore there came into being the mountain regions with transition toward the hilly country in the South. As the end result of all these geomorphological activities, is the region called Slovakia with its slopes from North to South and its surface waters following the original valleys or the valleys created by erosion. All the water of these valleys, except that of Poprad, flows toward the Hungarian Plains.

The runoff conditions are governed as much by the elevation (above sea level) as by the slope and permeability of the geological subsurface. Most characteristic are the Flysch zones of the Carpathians, where, at steep slope and at almost watertight subsurface, the runoffs have the highest values in

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1 Potter, William D., Peak Rates of Runoff from Small Watersheds, Washington, D.C.: U.S. Dept. of Commerce, 1961.

2 Ogievskij, A.V., Gidrologij Suši (Hydrology of the Deserts), Moscow: ONTI NKTP, 1952.

3 Sokolovskij, D.L., Rečnoj Stok (River Flow), Leningrad, 1952.

all Slovakia. On the other hand, the lowest runoff values occur in the South Slovak Plains of diluvial and alluvial origin, where thick layers of gravel with very slight slopes and greatest permeability are characteristic.

The precipitation ratio - one of the most important runoff factors - is influenced by the geographical configurations, especially the mountains on the North-West border of Slovakia. These influence the climate chiefly by determining the direction of the winds; reducing the strength of the winds; modifying the distribution of rainfalls and temperature. Generally, the average climate of Slovakia is neither Continental nor Oceanic, because the Carpathian Range isolates it. In the South, the area is wide open toward the Hungarian Plains, and therefore the climate is moderate. The yearly precipitation ratio in Slovakia amounts to 600 - 1400 mm, which is higher than elsewhere in the Carpathian Basin. The prevailing winds blow from the West in West-Slovakia and from the North in East-Slovakia. As a result of uneven distribution of the yearly rainfall, North-Slovakia has more drainage problems than South-Slovakia, where - because of lower precipitation ratio - not only drainage but also irrigation problems are common.

Account of Formerly Used Equations for Peak Rates of Runoff from Watersheds Expected Once in 100 Years.

To eliminate the possibility of errors, originating from too large scale or from generalizations, the research was concentrated, at first, on a survey of formulas which were in use in the chosen area and on a critical appraisal of these formulas.

Iszkowski's Equation[4]

Iszkowski's equation was developed for areas of 100 km^2 in South-Poland before the First World War, and can be used for areas of 50 - 300 km^2. The results of computation can give as much as $\pm$ 35-50 % errors. The general equation is the following:

$Q_{max} = m \cdot C \cdot P \cdot A \cdot 0.001$, where

Q_{max} = Peak Rate of Runoff in m^3/sec
m = Coefficient (see Chart #1)
C = Coefficient (see Chart #2)
P = Yearly Precipitation in mm
A = Area of Watershed in km^2

4 Novotný, J., Hydrologie, Prague: ČMT, 1925, pp. 22-27.

Chart #1:

Values of Coefficient "m"

Area (Km^2)	"m"	Area (Km^2)	"m"
1	10.00	80	7.50
10	9.50	90	7.43
20	9.00	100	7.40
30	8.50	150	7.10
40	8.23	200	6.87
50	7.95	250	6.70
60	7.75	300	6.55
70	7.60	400	6.22

Chart #2:

Value of Coefficient "C"

Conditions of Watershed:	Categories:			
	I	II	III	IV
Swamps	0.017	0.030	-	-
Plains	0.025	0.040	-	-
Plains with Hills	0.030	0.055	-	-
Hilly not too steep	0.035	0.070	0.125	-
Steep hills in mountains or extremely steep hilly country	0.040	0.082	0.155	0.400
Mountains, margin of high mountains, average steep, wooded area	0.045	0.100	0.190	0.450
Mountainous (Little and White Carpathians)	0.050	0.120	0.225	0.500
Bohemian Ore Mts, Sudetes, Beskides, Magures, Low Tatra	0.055	0.140	0.290	0.550
High mountains in accordance with steepness (High Tatra)	0.060-0.080	0.160-0.210	0.360-0.600	0.600-0.800

As explanation for Coefficient "C" is given by Iszkowski as follows:

a) For soil very permeable with average vegetative cover at all elevation, Category I should be used. Soils abundant in groundwater belong to Category II.

b) Standard soils with a recognizable combination of species and average vegetative cover in hilly country and in the mountains; watersheds with little permeability and average vegetative cover in the Plains or slightly wavy surface up to 150 km^2 should be in Category II. If the surface is wavier and the area greater than 150 km^2, it belongs in Category III. In all other cases with an area greater than 150 km^2, a combination of the Categories II and III be used.

c) Impermeable soils with average vegetative cover at steeper hilly country and mountainous area belong to Category III. Area less than 50 km^2 with considerable steepness is Category IV. Area from 50 km^2 to 300 km^2 should use a combination of the Categories III and IV.

As previously mentioned, the equations of Iszkowski were established for South-Poland (Galicia), where conditions are quite different from those in Slovakia. South of Carpathians, the geological structure is far more complicated, for which reason Iszkowski's equation for runoff - with its elaborated coefficients based on observation from the relatively geologically simple Galician watersheds - can hardly be applied. The equations of Iszkowski are based on the size of the watershed area and this is expressed by two coefficients. The third coefficient is that of the yearly rainfall. The coefficient "C" is influenced by the characteristics of the configuration of the surface, elevation, vegetative cover, permeability of the ground and again by the size of the watershed area. This latest coefficient, with its 33 values, is extremely difficult to apply and leaves too many alternatives to choose from. On the other hand, in the equations not only the form of the watershed, the intensity and distribution of the precipitation but also the hydrogeological conditions are completely disregarded.

Lauterburg's Formulas[5]

Lauterburg's formulas were based on observations mainly in Switzerland. They give very reliable values for watersheds with areas 50 km^2 or less. The observed deviations were $\pm$ 6-10 %. The equations are not applicable for areas over 50 km^2. Despite applying local corrective coefficients, the deviations of computed values from those observed are smaller in the mountains than in the plains or hilly country.

The following equations are in use:

a) At precipitation in four days with 50 mm altogether:

$$Q = 0.96 \cdot A \cdot \frac{7}{6 + 0.001 \cdot A}$$

b) At precipitation of longer duration:

$$Q = Q_o + C \cdot A \cdot \frac{1 + 0.5a}{1 + a(1 + 0.1a)}$$

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5 Halasi-Kun, G. (Kumansky), Voda v polnohospodárstve (Water in Agriculture), Bratislava: SPN, 1954, pp. 31-32.

c) At precipitation of intensity up to 250 mm in 24 hours:

$$Q = Q_o + C \;.\; 2.9 \;.\; A \;.\; \left(\frac{114}{115 + 0.05A} + 0.007\right)$$

d) At cloudburst intensity 0.035 mm/sec or 126 mm/hour:

$$Q = Q_o + C \;.\; A \;.\; \frac{32}{31 + A} \;.\; 35$$

In all formulas:

Q = Peak runoff (m^3/sec)

Q_o = runoff at start of rainfall (m^3/sec)

A = Area of watershed (km^2)

C = Coefficient (see Chart #3)

a = 0.001 . A

Chart #3

Values of Coefficient "C"

Watershed conditions:	Subsurface:					
	Impermeable		Permeable		Extremely permeable	
Terrain:	steep	flat	steep	flat	steep	flat
I Forest, loose soil, stony or sandy as in desert	0.55	0.55	0.45	0.35	0.35	0.25
II Agricultural land, light vegetative cover	0.65	0.55	0.55	0.45	0.35	0.35
III Meadow and pasture	0.75	0.65	0.65	0.55	0.60	0.50
IV Barren, stony mountains, also seldom in hilly country	0.80	0.70	0.70	0.60	0.60	0.50

In equations "c" and "d" assumed heavy rainfalls were not observed in Slovakia in the last 50 years. Therefore the computation of peak runoff should be adjusted in accordance with the data of the next gage station to comply with the local conditions. The equation "d" applies 126 mm/hour rain intensity but in Slovakia, there were observed only maximum 30 mm/hour in South-Slovakia and 40 mm/hour in North-Slovakia. If data on rain for computation are not available, the result in equation "d" should be reduced by 30/126 and 40/126, respectively.

The equations take into consideration the permeability of the soil, the vegetative cover and the slope of the surface in its coefficient "C" with 24 values altogether. The form of the watershed, the elevation, the natural flood areas and the hydrogeological conditions are still disregarded. The equations have only limited local character and can be used in North-Slovakia for areas 0 - 10 km^2.

Bogdánfy's Diagram (dated 1906)[6]

Bogdánfy's diagram is based on more than 100 years' observations from the Carpathian Basin. This could be used for double-checking runoff values computed by other methods, but its accuracy is very doubtful, because of the possibility of almost ± 80 % error in value.

The same could be said about the formula used by hydrologists in Hungary and in South-Slovakia as it follows:[7]

Chart #4

Runoff Formula Used in Hungary:

Watershed Area:	Runoff Values:
1 - 2 km^2	300 - 1000 l/sec-km^2 for torrents in the mts.
5 - 10 km^2	100 - 500 l/sec-km^2
10 - 500 km^2	50 - 60 l/sec-km^2 in mountains over 300 m.
10 - 500 km^2	25 - 40 l/sec-km^2 in hilly country 50-300 m.
10 - 500 km^2	15 - 20 l/sec-km^2 in plains below 150 m.

Hlavinka's Equation[8]

Hlavinka's equation is based on observation in Bohemia and Moravia and is not used any more because of too large deviations and the complicated computations.

$$Q = A. P. C. V. 0.27778 \quad \text{, where}$$

Q = Peak runoff (m^3/sec)

A = Area of watershed (km^2)

P = Peak precipitation (mm/hour: 40-80 mm/hour)

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6 Halasi-Kun, G., Die Ermittlung von Höchstabflüssen für Einzugsgebiete Kleiner als 300 km^2 (Peak Rates of Runoff from Watersheds with Areas 300 km^2 or Less), Brunswick, Germany: Techn. University, 1968, pp. 14-17.

7 Möller, K., Epitési zsebkönyv (Construction Pocket Book), II, Budapest: Kir.Egyetemi Nyomda, 1938, p. 241.

8 Halasi-Kun, G., op. cit., pp. 17-19.

C = Coefficient depending on geographical factors with a value 0.1 - 0.8

$V = \frac{1}{n \sqrt{A}}$; n = coefficient depending on the shape of watershed and steepness of the surface.

The formula assumes a precipitation intensity of 72 to 120 mm/hour which value is 2-3 times higher than that observed. Hlavinka's formula is not used any more, even in Moravia where it originated.

Čerkašin's Formula[9]

Čerkašin's formula was introduced by the Hydrometeorological Institute in Brno for the area of Moravia and is based on Sololovskij's equations as developed for Russia. The formula was developed in the late 1950's for areas 300 km^2 or less. The equation is as follows:

$$Q = \frac{385 . \sqrt{v} . c}{a . \sqrt{L}} . A$$, where

Q = Peak flow (m^3/sec)

v = Velocity (average) of flow (m/sec)

c = 0.45 - 0.75 , coefficient depending on vegetative cover and slope of the streambed

a = 0.oo - 3.8o , coefficient depending on $\frac{L^2}{A}$ = 0.oo - 10.oo

A = Area of watershed (km^2)

L = Length of the stream (km)

The basic assumption of Čerkašin, in his computations, is that the rainfall lasts at least as long as the peak flow reaches the hydrographic station for which the maximum flood is considered. It means also that the precipitation should cover the whole drainage area for this period. In the equation, slope of the streambed, vegetative cover, length of the stream, size and form of the watershed, direction and intensity of the precipitation are the most important factors; the geological conditions are less significant. Coefficient "a" is expressed, besides its numerical value, in isograms based on observed specific runoff values collected for the past 100 years in the region of Moravia for watersheds with areas 300 km^2 or less. Practically, the equation was not accepted by Czechoslovak hydrologists because its adjustment to various watersheds was found to be too difficult and complicated.

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9 Vodni hospodárstvi (Water Management): Čerkašin, A., "Velké vody na malých povodich," (Peak Runoff of Small Watersheds) VI (1956) pp. 295-299.

Bratránek's Equation[10]

Bratránek's equation is successfully used in obtaining values for watershed areas in Czechoslovakia, where data for a duration of at least 20-30 years are available. In cases of less than 3 years data, the equation cannot be applied at all, because it is based on probability.

$$Q_n = a_n \cdot Q_{max}$$, where

Q_n = Peak flow in "n" years (m^3/sec)

Q_{max} = Peak flow once in 100 years (m^3/sec)

a_n = Coefficient (values see in chart #5)

Chart #5:

The Values of Coefficient "a_n":

Peak flow once in "n" years:	Zones: a	b	c	d
1	0.25	0.29	0.34	0.o3 - 0.o6
2	0.37	0.4o	0.46	0.o5 - 0.1o
3	0.53	0.57	0.62	0.1o - 0.16
10	0.63	0.69	0.75	0.18 - 0.23
20	0.74	0.79	0.86	0.3o - 0.37
50	0.88	0.9o	0.95	0.6o - 0.64
100	1.oo	1.oo	1.oo	1.oo

The zones "a" to "c" are for watersheds with areas 200 to 500 km^2, the zone "d" is especially for torrents and for smaller watersheds and of impermeable soil.

Zone "a" could be used in the mountains for watersheds with small natural inundation areas. Zone "b" is for average conditions especially in hilly country, and zone "c" is for watersheds in the Plains with greater natural inundation areas.

10 Cablik, J., Základy stavby rybniku a hospodárských nádrži (Basic on Construction Fish-pounds and Farmers' Pools), Prague: SZN, 1960, pp. 93-94.

Dub's Equations[11]

Dub's equations are basically identical with those of the Russian hydrologist Ogievskij

$$q = \frac{b}{\sqrt{A}}$$, where

q = Peak rate of runoff ($m^3/sec.km^2$)

A = Area of watershed (km^2)

b = Coefficient

Ogievskij in his formula assumed a slope of the streambed 0.0025 and a shape of watershed with a ratio of length to width, 2-2.5 to 1. The vegetative cover is disregarded in the equation and an average permeability of the soil is considered.

The equations for Slovakia were compiled by Dub from data available at the Hydrometeorological Institute in Bratislava, Czechoslovakia. The accuracy of these equations is ± 11-20 % in computation of peak rate of runoff from watersheds. They are developed for four regions of Slovakia: 1. Flysch-Sandstone zone; 2. Mountainous area of Central-Slovakia; 3. Hilly country, and 4. Plains of Southwest-Slovakia.

The equations are as follows in chart #6.

11 Dub, O., Hydrologia, Hydrografia, Hydrometria, Bratislava: SVTL, 1963.

Chart #6:

Regions:	I	II	III	IV
Equations:	$q = \frac{17.60}{A^{0.443}}$	$q = \frac{10.10}{A^{0.494}}$	$q = \frac{2.30}{A^{0.364}}$	$q = \frac{1.00}{A^{0.316}}$
Yearly precipitation:	940 mm	840 mm	720 mm	690 mm
Probable deviation in accordance with Dub in %	0.83	1.09	1.46	2.01
Equation is based on "n" observations	17	31	15	7
Average deviation in %	2	2	3.8	7.7
Average deviation in observed cases (chart #22) in %	7.05			
Maximum deviations	-11 %	to	+28 %	

q = Runoff (m^3/sec.km^2) ; A = Area of watershed (km^2)

Dub recommends correction of the computed data:

1) Recorded yearly precipitations are available and these differ from the assumed ones.

2) 50-60 % of the area of the watershed is supposed to be forest. The computed values are, accordingly, to be corrected by $\pm$ 5 %.

3) The anticipated form of watershed is identical with that of Ogievskij. In case of a fan-shaped watershed, the runoff is increased by 5 %. The longated form of drainage basin requires a decrease by 10 % in value of the runoff.

Dub's equations are far better adapted to the hydrological conditions of Slovakia, despite the fact that they need a whole series of corrections. On the other hand, these four formulas are still inadequate to express properly the peak rate of runoff from smaller watersheds because of the great variety in hydrogeological conditions of this area. Besides these problems the excessive number of corrections in the computations is a source of further errors as the interpolations increase the chances for individual mistakes. To facilitate more exact results in computation Dub introduced further coefficients in 1954, based on available records.

Author's Equations for Peak Rates of Runoff from Watersheds Expected Once in 100 years

For developing a more efficient equation for peak rates of runoff, it is essential to conduct further analysis of the area in question. As we will see later, it seems that according to the precipitation, the runoff and the subsurface, the whole area of Slovakia can be divided in seven characteristic regions. All further analysis and research will be conducted from this viewpoint.

Among the very first factors which were analysed was the correlation between precipitation, altitude above sea level, flood and windbreak phenomena. In accordance with the research conducted in this area - as it has been pointed out in a previous study - the windbreak phenomena, precipitation and elevation have no practical effect on peak rates of runoff from watersheds of any size at a moderate climate like Central Europe.[12] On the other hand, the yearly precipitation values are governed in the same area by the altitude above sea level of the region in question.

Intensity of Rainfall and Snoweffect

Observations for longer period in Slovakia show the following correlation between peak intensity and duration of rain fall:

$$P = 3. \quad \sqrt{6 \, . \, t} \qquad \text{, where}$$

p = Precipitation in mm

t = Duration of rain in minutes

<u>Chart #7</u>:

Intensity:	Duration of rain in minutes:							
	5	10	15	20	30	60	90	100
mm/min	5.4	3.82	3.o6	2.59	1.95	1.2o	0.87	0.72
l/sec.ha	900	637	510	431	325	200	145	120

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12 Halasi-Kun, G., <u>Pollution and Water Resources, Volume I: 1967/1968</u>: "Correlation Between Precipitation, Flood, and Windbreak Phenomena in the Mountains," New York - Trenton: Columbia University - State of New Jersey, 1969.

Snowfalls, except in extraordinary cases in the area of High Tatra, did not show any influence on peak rate runoff of smaller watersheds. The yearly snow ranges in the mountains from 200 mm to 2450 mm and its duration is 40-90 days except in the Tatra region where this period is longer.

The Monthly Distribution of the Different Climatic Factors in Slovakia

Further analysis of the hydrological conditions is given in the next few charts: a monthly analysis of distribution of the moisture content of the air, rainfalls, evaporation, periodicity of precipitation for selected hydrographic stations and a general information of runoff for rivers in the same area.

Chart #8:

Monthly Distribution of Moisture Content of the Air in % for the Years 1926 - 1950, Values Given in % (Data of Hydrometeorological Institute in Prague):

	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Yearly	IV-IX
Bratislava	84	79	71	63	65	64	64	65	67	76	84	86	72	65
Žilina	85	83	80	75	72	73	76	79	81	86	88	89	80	76
Orav. Podzámok	81	79	76	72	71	72	73	75	77	79	83	84	77	73
Zvolen	85	80	75	68	70	69	70	72	75	79	85	86	76	71
Lipt. Hrádok	82	81	77	71	70	72	72	74	76	80	84	84	77	72
Štrbské pleso	77	80	75	73	71	72	71	73	74	78	82	83	76	72
Košice	84	82	75	69	68	70	70	71	74	80	85	85	76	70
Hurbanovo	85	81	74	68	70	69	68	70	74	80	86	88	76	70
Kr.Chlmec	86	83	75	70	69	72	70	73	76	82	87	87	78	72

The average moisture content of the air in Slovakia is in the mountainous regions 80 %, in the forests 85 % and in the plains 75 %. The monthly peaks are 10 % higher than these and the lowest values slightly below average.

Chart #9:

The Monthly Peak Values of Precipitation in Slovakia for the Years 1901 - 1950, Values Given in mm (Data of Hydrological Institute in Bratislava):

	I	II	III	IV	V	VI	VII
Bratislava	107	116	146	137	179	112	153
Žilina	130	122	114	170	194	219	183
Orav. Podzámok	140	162	105	169	212	192	279
Zvolen	110	105	131	115	165	182	148
Lipt.Hrádok	110	107	151	96	173	172	232
Štrbské pleso	164	183	172	162	215	241	363
Košice	114	84	141	118	168	157	168
Hurbanovo	95	107	138	128	152	146	143
Kr.Chlmec	102	85	122	103	160	168	196

	VIII	IX	X	XI	XII	Yearly	IV-IX
Bratislava	151	175	195	209	140	981 (1944)	593 (1925)
Žilina	254	139	165	210	120	1175 (1903)	712 (1913)
Orav. Podzámok	228	194	137	197	114	1158 (1912)	817 (1939)
Zvolen	156	168	167	139	123	995 (1937)	636 (1913)
Lipt. Hrádok	173	167	142	148	96	997 (1903)	669 (1913)
Štrbské pleso	265	226	159	175	123	1230 (1937)	882 (1913)
Košice	222	154	131	148	93	969 (1915)	637 (1940)
Hurbanovo	153	147	151	123	102	863 (1937)	473 (1937)
Kr.Chlmec	197	134	137	134	113	913 (1912)	661 (1913)

Chart #10:

The Monthly Lowest Rate of Precipitation in Slovakia for the Years 1901 - 1950, Values Given in mm (Data of Hydrological Institute in Bratislava):

	I	II	III	IV	V	VI	VII	VIII
Bratislava	9	2	1	1	7	8	10	8
Žilina	11	9	2	8	11	3	9	16
Orav.Podzámok	5	3	2	10	14	30	15	26
Zvolen	7	1	7	8	10	23	14	3
Lipt.Hrádok	6	7	0	13	15	24	15	23
Štrbské pleso	12	8	6	24	33	22	10	27
Košice	3	6	1	8	13	24	28	13
Hurbanovo	5	4	2	4	3	4	11	1
Kr.Chlmec	9	7	4	5	4	21	7	9

	IX	X	XI	XII	Yearly	IV-IX
Bratislava	4	1	2	6	535 (1924)	148 (1917)
Žilina	11	1	2	8	475 (1917)	199 (1917)
Orav.Podzámok	10	3	4	5	559 (1921)	254 (1921)
Zvolen	8	2	4	16	452 (1921)	194 (1947)
Lipt.Hrádok	8	4	2	6	522 (1946)	269 (1917)
Štrbské pleso	12	8	5	9	647 (1921)	314 (1917)
Košice	2	1	2	11	412 (1942)	221 (1917)
Hurbanovo	1	3	3	7	370 (1921)	148 (1947)
Kr.Chlmec	3	3	2	2	305 (1946)	177 (1946)

Very good information is furnished about precipitation recorded in Hurbanovo, Southwest-Slovakia, where the systematic data collection goes back as far as to 1871:[13]

Chart #11a:

Characteristic Precipitation in Hurbanovo, Southwest-Slovakia, for the Years 1871 - 1950 :

	Month :											
Precipitation in mm:	X	XI	XII	I	II	III	IV	V	VI	VII	VIII	IX
Average :	53.6	48.8	50.1	35.7	31.5	39.8	49.8	64.3	58.8	57.o	51.2	48.o
Minimum :	5	2	5	1	0	2	4	3	4	11	1	1
Maximum :	166	151	176	109	120	138	128	189	146	142	152	147

	Period:						
	Year :	Season :					Half-Year :
Precipitation in mm:		X-XII	I-III	IV-VI	VII-IX	X-III	IV-IX
Average :	588	152	107	173	156	259	329
Minimum :	386	37	28	83	48	103	148
Maximum :	909	217		289	297	484	460
	Index:						
Minimum/Average :	0.65	0.24	0.24	0.5o	0.31	0.4o	0.45
Maximum/Average :	1.5	1.8	2.o	1.7	1.9	1.9	1.4
Maximum/Minimum :	2.4	7.4	8.o	3.4	6.o	4.7	3.1

13 Vodni hospodárstvi (Water Management): Bella, Št., "Zavodnovanie na Slovensku" (Irrigation in Slovakia), II (195 pp. 18-21 and 42-44.

Chart #11b:

Periodicity and Occurrence of Dry Seasons in Hurbanovo, Southwest-Slovakia, in Years 1871 - 1950 :									
Season :	Number of wet, normal and slightly dry seasons :			Number of very dry and unusually very dry seasons :			Total number of seasons :	Number of very dry and unusually very dry seasons :	
Period/Months:	Wet and normal	Dry with months of precipitation less than 30 mm :	Sum-mary:	Very dry with two following months or pre-cipitation less each month :	Unusually very dry with 3 months of precipitation less than 30 mm each month :	Sum-mary:		Yearly Coef-ficient of great droughts:	Approximate intervals:
Fall / X-XII	27	34	61	17	2	19	80	0.24	each 4th year
Winter/I-III	12	34	46	17	17	34	80	0.43	each 2nd year
Spring/IV-VI	41	32	73	7	-	7	80	0.o9	each 11th year
Summer/VII-IX	28	31	59	19	2	21	80	0.26	each 4th year
Summary/X-IX	108	131	239	60	21	81	320	0.2o	each 5th year

In Years 1917 and 1947, they were extremely dry Summer and Fall seasons. They have been catastrophic droughts in those two years which can occur at forty year intervals.

Chart #12:

The Monthly Evaporation (in mm and %) Measured with Evapometer type H. Wild (Data of Hydrological Institute in Bratislava):

			I	II	III	IV	V	VI	VII
Bratislava	(1943-1947)	mm	16.3	21.7	51.2	85.4	90.2	79.8	88.2
		%	2.3	3.1	7.3	12.3	13.o	11.5	12.7
Ilava	(1944-1947)	mm	13.o	26.3	34.7	75.o	88.7	75.1	86.8
		%	2.1	4.2	5.5	11.9	14.1	12.o	13.8
Orav.Podzámok	(1933-1947)	mm	7.3	9.6	19.4	33.7	39.o	40.2	40.3
		%	2.5	3.3	6.6	11.5	13.4	13.7	13.7
Hurbanovo	(1945-1947)	mm	6.7	13.4	40.2	73.o	70.8	67.9	67.1
		%	1.3	2.5	7.6	13.8	13.4	12.8	12.7
Lipt.Hrádok	(1934-1947)	mm	8.2	11.3	29.5	44.1	53.2	54.2	60.6
		%	2.o	2.8	7.3	10.9	13.1	13.5	15.o
Komárno	(1933-1938)	mm	12.7	22.3	47.o	58.6	67.4	75.5	101.8
		%	2.2	3.8	8.o	10.o	11.5	12.9	17.8
Skalnaté pleso in High Tatra	(1945-1947)	mm	17.9	9.6	22.5	41.1	40.3	38.2	43.9
		%	5.o	2.7	6.3	11.4	11.2	10.7	12.2
Košice	(1934-1944)	mm	10.2	15.4	36.5	58.2	73.6	76.4	77.o
		%	1.9	2.8	6.8	10.8	13.9	14.2	14.4

			VIII	IX	X	XI	XII	Yearly
Bratislava	(1943-1947)	mm	91.9	85.o	45.2	26.3	15.o	696.2
		%	13.2	12.2	6.5	3.8	2.1	100
Ilava	(1944-1947)	mm	87.4	68.9	39.9	19.6	13.4	628.8
		%	13.9	11.o	6.3	3.1	2.1	100
Orav.Podzámok	(1933-1947)	mm	34.9	28.3	19.6	12.o	9.5	293.9
		%	11.9	9.3	6.8	4.1	3.2	100
Hurbanovo	(1945-1947)	mm	77.2	59.9	28.6	14.4	9.8	529.o
		%	14.6	11.3	5.4	2.7	1.9	100
Lipt.Hrádok	(1934-1947)	mm	51.2	43.8	25.7	14.4	8.4	404.9
		%	12.6	10.8	6.3	3.6	2.1	100
Komárno	(1933-1938)	mm	77.6	59.8	37.2	17.1	9.1	586.1
		%	13.2	10.2	6.3	2.9	1.6	100
Skalnaté pleso in High Tatra	(1945-1947)	mm	50.3	37.4	22.5	20.8	14.2	358.8
		%	14.o	10.4	6.3	5.8	4.o	100
Košice	(1934-1944)	mm	69.9	59.9	34.3	17.9	11.2	536.5
		%	13.o	10.4	6.4	3.3	2.1	100

Chart #13:

Minimum, Average and Maximum Flow of Rivers in Slovakia for the Years 1871 - 1964 (Data of Hydrometeorological Institute in Prague):

River - Hydrographic Station:	Watershed (km^2):	Flow in m^3/sec :		
		Minimum - Observed	Average for Years 1931 - 1960	Maximum - Observed
Danube - Bratislava	131,338.2	570.00 - 1948	1,992.00	10,870 - 1899
Danube - Komárno	171,660.4	660.00 - 1947	2,291.00	8,500 - 1923
Myjava - Štefanov	542.6	0.18 - 1933	2.61	109 - 1945
Morava - Mor.Ján	24,129.3	7.70 - 1934	111.60	1,600 - 1941
Váh - Lipt. Mikuláš	1,106.6	4.40 - 1921, -1954	21.00	470 - 1958
Váh - Lubochňa	2,140.6	7.80 - 1921	38.70	715 - 1958
Orava - Tvrdošin	1,200.9	0.05 - 1958, -1960	20.30	940 - 1925
Orava - Dierová	1,971.7	3.28 - 1964	33.80	1,120 - 1948
Váh - Kralovany	4,268.3	13.10 - 1921	76.10	1,686 - 1948
Turiec - Martin	827.0	2.20 - 1947	10.90	327 - 1960
Kysuca - K.Nové Mesto	955.0	0.83 - 1930	16.10	850 - 1925, - 1958
Váh - Žilina	5,734.6	16.90 - 1954	97.20	2,010 - 1894
Váh - Šala	10,618.9	21.30 - 1962	152.00	1,912 - 1903

Chart #13 (cont'd.) :

River - Hydrographic Station:	Watershed (km^2):	Flow in m^3/sec : Minimum - Observed	Average for Years 1931 - 1960	Maximum - Observed
Nitra - Chalmová	601.1	0.51 - 1947	6.36	136 - 1946
Nitra - Nové Zámky	3,156.o	2.2o - 1947	18.1o	220 - 1881, - 1920
Hron - Brezno	582.1	1.2o - 1943, -1961	8.o3	180 - 1960
Hron - Banská Bystrica	1,766.5	4.7o - 1921	27.9o	379 - 1960
Hron - Brehy	3,821.4	7.6o - 1928	49.5o	790 - 1960
Ipel (Ipoly) - Ip.Sokolovce	4,838.4	0.44 - 1947	20.62	538 - 1937
Rimava - Rim. Sobota	594.3	0.3o - 1947	4.72	101 - 1960
Slaná (Sajó) - Čoltovo	850.o	0.55 - 1947	8.o3	176 - 1937
Slaná (Sajó) - Lenartovce	1,806.9	0.8o - 1947	13.8o	250 - 1937
Hornád - Sp.Vlachy	809.3	0.46 - 1964	6.26	256 - 1960
Hnilec - Jaklovce	654.5	1.3o - 1947	7.97	200 - 1948
		0.72 - 1961*		
Hornád - Kysak	2,345.7	2.5o - 1947	19.4o	552 - 1960
Torysa - Košické Olšany	1,298.3	0.6o - 1947	7.92	237 - 1952

Chart #13 (cont'd.) :

River - Hydrographic Station:	Watershed (km^2):	Flow in m^3/sec : Minimum - Observed	Average for Years 1931 - 1960	Maximum - Observed
Topla - Hanušovce	1,050.0	0.72 - 1932	8.26	390 - 1932, - 1948, - 1952
Ondava - Trepec	788.3	0.24 - 1961	7.36	455 - 1932
Ondava - Horovec	2,839.o	1.49 - 1961	20.7o	613 - 1952
Laborec - Michalovce	1,401.1	0.49 - 1963	15.4o	418 - 1952
Laborec - Vojany	4,274.o	2.39 - 1961	54.6o	664 - 1964
Uh - Lekárovce	1,995.o	1.31 - 1961	32.9o	799 - 1957
Latorica - V.Kapušany	2,983.1	2.6o - 1954	32.oo	252 - 1964
Bodrog - Streda n/B.	11,311.2	8.39 - 1961	118.oo	1,031 - 1964
Poprad - Matejovce	311.1	0.8o - 1921, - 1932 - 1933, - 1937	4.67	268 - 1960
Poprad - Chmelnica	1,262.4	2.7o - 1964	17.oo	570 - 1958

* After completion of "Hnilec" Dam.
See Figures 1 and 7.

Dividing Slovakia into Seven Regions in Accordance with Monthly Precipitation and Yearly Runoff

Slovakia can be divided into seven zones in accordance with its relief and monthly precipitation as it follows (Data of the Hydrological Institute in Bratislava for the years 1901 - 1940):

Chart #14:

Zones :		% of Yearly Precipitation :					
		I	II	III	IV	V	VI
High mountains :	1	5.6	5.o	6.o	6.5	10.2	12.4
	2	6.5	5.2	5.5	6.4	10.5	11.8
Mountains :	3	5.o	4.6	5.6	6.8	10.8	12.3
	4	6.2	5.6	6.4	7.2	9.7	10.7
Hilly country :	5	5.9	5.3	6.3	7.3	10.3	11.2
Plains :	6	6.1	5.3	6.5	7.6	10.3	11.o
Island of Žitny :	7	6.1	6.o	6.2	8.5	10.7	10.2

		VII	VIII	IX	X	XI	XII	Total
High mountains :	1	13.5	11.8	9.1	7.2	6.6	6.1	100
	2	12.9	12.1	8.9	7.6	6.9	5.8	100
Mountains :	3	12.9	11.9	9.3	7.9	7.1	5.8	100
	4	11.2	11.o	8.9	8.3	7.7	7.1	100
Hilly country :	5	11.o	10.3	8.6	8.4	8.o	7.4	100
Plains :	6	10.2	10.2	8.6	8.2	8.1	7.9	100
Island of Žitny :	7	9.6	8.8	7.9	9.1	8.o	8.6	100

Chart #15:

The Monthly Runoff in % of the Yearly Runoff for Zones in Chart #14 Described and for the Same Period of Years:

Zones :		I	II	III	IV	V	VI
High mountains :	1	1.7	1.1	1.4	6.5	19.8	19.6
	2	5.2	5.o	9.4	9.7	11.9	11.5
Mountains :	3	4.6	4.4	11.2	13.o	12.6	8.6
	4	5.3	5.3	14.2	14.2	11.1	8.1
Hilly country :	5	6.6	7.1	17.8	14.8	9.6	7.7
Plains :	6-7	7.5	9.1	20.2	15.6	9.8	6.9

		VII	VIII	IX	X	XI	XII	Total
High mountains :	1	16.9	10.5	6.9	7.5	6.1	3.o	100
	2	10.1	9.2	7.2	7.1	7.7	6.o	100
Mountains :	3	7.5	7.9	7.2	7.1	9.6	6.3	100
	4	6.4	6.5	6.2	6.2	9.7	6.8	100
Hilly country :	5	5.1	5.1	4.5	4.9	9.4	7.4	100
Plains :	6-7	3.9	3.8	3.1	3.9	8.7	7.5	100

Based on these data the seven runoff zones of Slovakia can be summarized:

High mountains - zone 1 : over 15 l/sec.km^2 runoff with over 950 mm average of yearly rainfall.
- zone 2 : 14 l/sec.km^2 runoff with 900 mm average of yearly rainfall.

Mountains - zone 3 : 10 l/sec.km^2 runoff with 830 mm average of yearly rainfall.

Mountains - zone 4 : 6 l/sec.km^2 runoff with 750 mm average of yearly rainfall.

Hilly country - zone 5 : 2.5 l/sec.km^2 runoff with 700 mm average of yearly rainfall.

Plains - zone 6 : 1.9 l/sec.km^2 runoff with 670 mm average of yearly rainfall.

Žitny Island - zone 7 : 1 l/sec.km^2 runoff 600 mm or less average of yearly rainfall.

Hydrogeologic Characteristics and Division of Slovakia[14]

Division of Slovakia into zones in accordance with monthly precipitation or monthly runoff ratio can not characterize adequately the runoff of smaller watersheds because the division depends mainly on the hydrogeological conditions. The most characteristic are the Flysch-zones, where the greatest values of peak runoff occur regardless of elevation and yearly precipitation. The limestone-zones, on the other hand, have far smaller values in runoff ratio, which phenomenon can be explained by the retardation effect of these geological formations. Finally, the diluvial and alluvial regions, regardless of their elevation (in the South 100 to 200 m, in the High Tatra region 700 to 1000 m) because of the permeability of the subsurface, have a minimum runoff ratio. This proves only that the peak rate of runoff depends almost solely on the geological conditions and the elevation, the slope or the yearly precipitation has very little effect. Furthermore, in the valley of Ipel in South-Slovakia, where the erosion ridden surface creates a permeability of the subsurface similar to diluvial zones of the Plains, the runoff ratio is similar too. (See chart #16 items 48 and 50 and compare with items 36, 49 etc.)

Slovakia could be divided into seven hydrogeological zones as follows (see Figure 2):

1 Tertiary Paleogene formations; Magura-Flysch or Outer Sandstone zone; isolated springs of little capacity with horizontal and vertical layers. (The limestone local formation in the Magura-Flysch in the area of East-Beskides in Figure 2 is marked by a dotted line and "1a".)

2 Pienin-Schist of the Carpathians; without any significant groundwater.

2-3 Subtatric Flysch zone (in Lower Tatra and Central Slovak Mountains);

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14 Hynie, O., Hydrogeologie ČSSR I - Prosté vody (Hydrogeology of ČSSR, Part I - Ordinary Waters), Prague: NČsAV, 1961.

springs and groundwater as in zone 1.

3a-b Earlier Paleozoic and Proterozoic formations: Cambrian, Precambrian zones and Crystalline Complexes of the Carpathians (Region a: Granite; Gneiss, Micaceous Schist; Melaphyry, Diabase; Porphyry, Porphyrite; Trachyte - local springs with a little greater capacity. Region b: Basalt, Andesite - numerous springs of smaller capacity).

4 Limestone zone:
- a Tuffstone group (Rhyolite, Trachyte, Dacite, etc.); numerous springs.
- b Wrappings of Crystalline Complexes created tectonically: mainly Limestones and Dolomites; region of scattered, scarce springs of great capacity.

5 Tertiary Neogene zone or hilly country; groundwater of artesic origin in several layers and of inferior quality. Glacial formations (insignificantly small areas); shallow basins with groundwater of high quality and greater capacity.

6 Diluvium zone (Alluvium and Diluvium mixed):
Diluvium: groundwater of inferior quality and limited capacity;
Alluvium: Shallow groundwater layers with greater capacity, sources of water supply for industry.

7 Alluvium zone; region of shallow groundwater layers with greater capacity, sources of water supply for industry.

Data of Peak Runoff for Smaller Watersheds in Slovakia

A comparison of Figure 2 (Hydrogeological regions of Slovakia) with Figure 4 (Peak Runoff Zones of Watersheds with an Area 300 km^2 or Less) shows a surprising similarity. On the other hand, the watersheds with an area over 300 km^2 are governed less by the hydrogeological conditions than by the elevation, the slope, the vegetation, the shape of the watershed and the yearly precipitation.

The peak runoffs similar to the precipitations have a 9 to 10 year interval. This interval is not accidental but corresponds to the natural rhythm of the rainfall and other climatic conditions. Even more striking phenomenon is the occurrence of the lowest runoffs which have a 40 year frequency. On the other hand, it must be mentioned that the extreme droughts occur also in 40 year intervals.

Based on previous study and on 110 observed and computed runoffs from smaller watersheds, it is suggested to divide Slovakia into seven runoff regions. In establishing equations for these regions, data of runoffs were used which were collected by the author in Slovakia in years 1946 - 1957, and data registered and published by the Hydrometeorological Institutes in Prague and Bratislava, the Office of Water Resources Management in Košice and by various authors indicated in the appropriate chart of this study for the years 1871-1964. (See Figure 3 for location of hydrographic stations.)

Chart #16, Part I:

Data of 100 Year Peak Runoff in Slovakia Observed in Years 1871 - 1964, Collected by the Hydrometeorological Institutes in Prague and Bratislava, and by Several Authors and Sources:

Number:	Hydrographic station - river :	Watershed area in km^2 :	Peak runoff in m^3/sec	and Date of occurrence :	Runoff zone of the author :	Runoff ratio in $m^3/sec.km^2$:
1	Šance - Ostravice[15]	146.4	270	6.29.1958	1	1.85
2	Zdechovka - Creek[16]	4.o4	36.5	6.16.1939	1	8.9
3	Krásno - Bečva[17]	252.2	360		1	1.42
4	Hradlo Čierné - B.Váh[18]	75.o	105	6.29.1958	2-3	1.4
5	Vychodná - Biely Váh[18]	105.2	135	6.29.1958	+3	1.29
5a	Vychodná - Biely Váh[15]	110.2	130	6.29.1958	+3	1.19
5b	Vychodná - Biely Váh[17]	105	105		3	1.o
6	Kr.Lehota - Boca[19]	116.6	95		3-4	0.84
7	Tichá - Creek[18]	52.8	125	6.29.1958	+2	2.38
7a	Tichá - Creek[19]	51	98		-2	1.92
8	Koprová - Creek[18]	31.2	90	6.29.1958	2	2.9
9	Dúbravy - Bystrá[18]	9.5	30	6.29.1958	3	3.15

Chart #16, Part I (Cont'd.):

Number:	Hydrographic station - river :	Watershed area in km^2 :	Peak runoff and in m^3/sec	Date of occurrence :	Runoff zone of the author :	Runoff ratio in $m^3/sec.km^2$:
10	Kamenistý - Creek[18]	10.3	32	6.29.1958	3	3.1
11	Podbansko - Belá[15]	93.5	180	6.29.1958	+2	1.92
12	Račkov - Račková[18]	34.9	80	6.29.1958	2-3	2.3
13	Lipt.Hrádok - Belá[18]	244.4	320	6.29.1958	+2	1.31
14	Žiar - Smrečianka[18]	17.4	50	6.29.1958	2-3	2.86
15	Jalovce - Jalovecký[18]	33.4	85	6.29.1958	-2	2.3

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15 Hydrometeorologicky Ustav, Hydrologická ročenka ČSSR, čast I., Povrchové vody (Hydrological Yearbook of ČSSR, Part I; Surface Water), Prague (Bratislava), 1946-1964 - for items 1, 5a, 11.

16 Vodni Hospodárstvi (Water Management): Čermák, M., "Odtokové pomery malého povodi," (Runoff Conditions of Small Watersheds), IV (1954), pp. 9-12, 41-44 - for item 16.

17 Vodni Hospodárstvi: Čerkašin, A., "Velké vody na malých povodich," (Floods on Small Drainage Areas), VI (1956), pp. 295-299.

18 Sbornik Prác o Tatranskom Národnom Parku (Work Report on Tatra National Park): Pacl, J., "Katastrofálna povoden v oblasti Tatier v júni 1958" (Catastrophic Flood in Region of Tatra in June 1958) III (1959), pp. 17-56.

19 Records in the Waterbook of the Office of Water Resources, Košice, 1953.

Chart #16, Part II:

Data of 100 Year Peak Runoff in Slovakia Observed in Years 1871 - 1964, Collected by the Hydrometeorological Institutes in Prague and Bratislava, and by Several Authors and Sources:

Number:	Hydrographic station - river :	Watershed area in km^2 :	Peak runoff in m^3/sec and	Date of occurrence :	Runoff zone of the author :	Runoff ratio in $m^3/sec.km^2$:
16	Lipt.Sielnica-Kvačianka[20]	75.1	110	6.29.1958	2-3	1.46
17	Podsuchá - Revúca[21]	209.7	180		3-4 = +3	0.86
18	Or.Jesenica-Veselovský[22]	90.1	195	6.29.1958	-1	2.16
19	Zubrohlava - Polhoranka[22]	158.7	425*	6.29.1958	++1	2.7
20	Lokca - Biela Orava[22]	360.o	762*	6.29.1958	++1	2.1
21	Hladovka - Jelešná[20]	19.7	45	6.29.1958	2-5 = 3	2.3
22	Vitanova - Oravica[20]	64.1	115	6.29.1958	2	2.o2
23	Trstená - Oravica[20]	129.9	195	6.29.1958	2	1.31
24	Or.Biely Potok-Studený[20]	118.1	190	6.29.1958	2	1.51
25	Turček - Turiec[21]	45	70		3	1.55
26	Martin - Turiec[22]	827	327	7.26.1960	+3	0.44
27	Stráža - Varinka[22]	139.7	226	6.29.1958	+2	1.62

Chart #16, Part II (Cont'd.) :

Number:	Hydrographic station - river :	Watershed area in km^2 :	Peak runoff in m^3/sec and	Date of occurrence :	Runoff zone of the author :	Runoff ratio in $m^3/sec.km^2$:
28	Rožňov - Rožň.Bečva[22]	156.4	180**	7.26.1960	(1)-2	1.2
29	Gottwaldov - Drevnice[22]	311.7	147	7.20.1949	1+5+6 = 3-4	0.48
30	Petrov - Radejovka[22]	40.5	19.5	8. 9.1955	6	0.48
31	Poluvšie - Rajčanka[22]	230.3	148	7.26.1960	2-4 = -3	0.64
31a	Poluvšie - Rajčanka[21]	230.3	165		2-4 = +3	0.81
32	Zborov - Bystrica[22]	210	400*	6.29.1958	++1	1.9
32a	Zborov - Bystrica[21]	210	275		1	1.33

20 Sbornik Prác o Tatranskom Národnom Parku: Pacl, J., article cit.
21 Vodni Hospodárstvi: Čerkašin, A., article cit.
22 Hydrometeorologicky Ustav, op. cit.
* Probably more than "100 Year Peak Runoff."
** Probably less than "100 Year Peak Runoff."

Chart #16, Part III:

Data of 100 Year Peak Runoff in Slovakia Observed in Years 1871 - 1964, Collected by the Hydrometeorological Institutes in Prague and Bratislava, and by Several Authors and Sources:

Number:	Hydrographic station - river :	Watershed area in km^2 :	Peak runoff in m^3/sec	and Date of occurrence :	Runoff zone of the author :	Runoff ratio in $m^3/sec.km^2$:
33	Vydrňa - Vydrňanka[23]	10.9	100*	6.17.1939	++1	9.3
33a	Vydrňa - Vydrňanka[24]	10.9	80	6.17.1939	+1	7.34
34	Brezno - Hron[25]	582.1	240		3	0.41
35	Hriňová - Slatina[24]	71.5	85		3	1.19
36	Lučenec - Kriváň[29]	265.5	95	3. 6.1962	4-5	0.36
37	Plášťovce - Krupinica[24]	302.8	175		3+4 = 3(-4)	0.65
38	Štitnik - Stitnik[24]	128.1	80		(3)+4 = +4	0.63
39	Rimavica - Rimavica[24]	151.5	90		3+4+5 = +4	0.59
40	Hrabušice - Hornád[26]	288.4	138		3-4	0.48
41	Kudlovce - Kudlovský[27]	5.51	42		1	7.6
42	Choňkovce - Sobranecky[27]	62.5	40		4-5	0.64
43	Lake Poprad - Hincov[28]	8.6	35	6.29.1958	+3	4.o
44	RR.St.Lake Poprad-Poprad[28]	16.9	66	6.29.1958	2	3.9

Chart #16, Part III (Cont'd) :

Number:	Hydrographic station - river :	Watershed area in km^2 :	Peak runoff in m^3/sec and	Date of occurrence :	Runoff zone of the author :	Runoff ratio in $m^3/sec.km^2$
45	Lake Štrbské - Mlynica[28]	7.5	30	6.29.1958	2-3	4.o
46	Matejovce - Slavkovsky[28]	44.9	55	6.29.1958	-3	1.23
47	Matejovce - Poprad[29]	311.1	268	7.26.1960	2-3	0.86
48	Štola - Vel.Rincovy[28]	7	15	6.29.1958	3+5 = 4	2.15
49	Stará Lesná - Studený[28]	18.5	65	6.29.1958	-2	3.5
50	RR.St.Stará Lesná - Studený[28]	29.5	55	6.29.1958	2+3+5 = 3	1.87
51	Tatr.Kotlina - Biela[28]	51.6	80	6.29.1958	2+4 = 3	1.55
52	Javorina - Javorinka[28]	48	125	6.29.1958	+2	2.6
53	Lysa Polana - Bialka[28]	63.8	138	6.29.1958	+2	2.15

23 Dub, O., Režim velkých vod na malých tokoch (Occurrence of Peak Rate Runoff in Small Watersheds), Bratislava, 1947. p. 23.

24 Vodni Hospodárstvi: Čerkašin, A., Article cit.

25 Dub, O., Hydrologia, hydrographia, hydrometria, Bratislava: SVTL, 1963, p. 310.

26 Vodohospodársky Casopis (Water Resources Periodical): Boško, K., "Kyslíková bilancia povodia horného Hornádu" (BOD Values in the Watershed of Upper-Hornád), I (1953), p. 99.

27 Records in the Waterbook of the Office of Water Resources, Košice, 1953.

28 Sbornik Prác o Tatranskom Národnom Parku: Pacl, article cit.

29 Hydrometeorologicky Ustav, op. cit.

* Probably more than "100 Year Peak Runoff."

Chart #17, Part I:

Data of Peak Runoff in Slovakia Observed in Years 1871 - 1964 and Recorded in "Hydrological Yearbook of ČSR, 1946 - 1964" Converted to 100 Year Peak Runoff with the Method of Bratránek:

No.: Hydrographic station - river :	Watershed area in km^2 :	Peak runoff in m^3/sec :	Observed on :	Runoff recorded in "n" years :	Peak runoff (m^3/sec) Bratránek value :	Runoff zone of author :	Runoff ratio $m^3/sec.km^2$:
54 Sobotište - Vrbovec	90.1	16.5	2.12.1961	5	100	1+5 = 3	1.1
55 Studienka - Rudava	270.9	21.5	3.12.1963	4	70	5-7 = 6	0.26
56 Jakubov - Malina	172.6	3.48	10.26.1964	4	34	7	0.2
57 Bernolákovo - Čierna Voda	78.2	6.99	3. 3.1956	15	28	3+7 = -6	0.36
58 Pezinok (Cajla)- Pezinsky	19.1	2.51	4. 1.1963	4	15.7	3+7 = 5	0.82
59 Bohdanovce - Trnávka	116.2	9.2	3.12.1963	4	51	4+6 = 5	0.44
60 Čachtice - Jablonka	163.2	17.5	2.12.1961	4	124	2+4 = 3	0.76
61 Čierny Váh - Čierny Váh	243.3	60.8	6.29.1958	44	110	4	0.45
62 Part.Lupča - Lupčianka	71.4	10.3	11. 7.1961	4	64	3-4	0.91
63 Lubochňa - Lubochňanka	118.2	36.6	7.26.1960	33	61	3+4 = +4	0.7
64 Bytča - Petrovička	63.7	15.9	4. 5.1961	4	128	2	2.o5

Chart #17, Part I (Cont'd):

No.: Hydrographic station - river :	Watershed area in km^2 :	Peak runoff in m^3/sec :	Observed on :	Runoff recorded in "n" years :	Peak runoff (m^3/sec) Bratránek value :	Runoff zone of author :	Runoff ratio $m^3/sec.km^2$:
65 Pov.Bystrica-Domanižanka	100.7	18.4	3. 5.1962	4	147	2	1.5
66 Dohňany - Biela Voda	160.1	60.o	3. 6.1962	4	335	+1	2.1
67 Brumov - Brumovka	65.2	36.3	7.13.1960	6	195	+1	3.o
68 Popov - Vlára	169.2	85.5	7.24 and 25.1960	9	310	1	1.9
69 Nedožery - Nitra	181.2	62.2	7.26.1960	24	103	3+5 = 3-4	0.64
70 Lieštany - Nitrica	143.8	40.8	7.26.1960	8	225	2+3 = +2	1.5
71 Biskupice - Bebrava	312.6	65.8	3. 6.1962	34	87	5	0.28
72 Vieska - Žitava	294.3	59.8	3. 3.1937	34	79	3+4+6=(5-)6	0.27
73 Červená Skala - Hron	84.o	46.6	6.29.1948	34	78	3-4	0.93
74 Hronec - Čierný Hron	237.7	68.8	4.22.1931	33	150	3	0.63
75 Bystrá - Bystrianka	36.o	16.9	6.29.1958	33	41	(3)-4	1.15

Chart #17, Part II:

Data of Peak Runoff in Slovakia Observed in Years 1871 - 1964
and Recorded in "Hydrological Yearbook of ČSR, 1946 - 1964"
Converted to 100 Year Peak Runoff with the Method of Bratránek:

No.: Hydrographic station - river :	Watershed area in km^2 :	Peak runoff in m^3/sec :	Observed on :	Runoff recorded in "n" years :	Peak runoff (m^3/sec) Bratránek value :	Runoff zone of author :	Runoff ratio $m^3/sec.km^2$:
76 Slov.Lupča - Lupčica	38.2	5.55	7.26.1960	9	34.5	3+4+5= -4	0.91
77 Harmanec - Bystrica	55.3	15.2	7.26.1960	10	74	3	1.35
78 Jakub - Bystrica	151.5	60.o	7.26.1960	34	100	3-4	0.66
79 Žarnovica - Klakovsky	131.9	62.o	3. 6.1962	3	116	3+5 = -3	0.88
80 Horné Strháre- Stará Rieka	48.6	9.4	3. 6.1962	3	59	3-4	1.22
81 Plášťovce - Litava	214.4	52.6	3. 6.1962	33	130	3-4	0.61
82 Dobšina - Dobšinsky	31.7	10.3*	4. 5.1962	33	28	--4	0.9
83 Vlachovo - Slaná	123.2	24.3*	6.29.1958	42	66	--4	0.54
84 Jelšava - Murán	275.8	78.5	6.29.1958	41	157	3-4	0.57
85 Tisovec - Tis.Rimava	73.3	8.5	3.20.1947	12	48	-4	0.7
86 Rimavská Seč - Blh	274.1	32.3	3.13.1963	33	85	4+5= 5	0.31

Chart # 17, Part II (Cont'd.):

No.: Hydrographic station - river :	Watershed area in km^2 :	Peak runoff in m^3/sec :	Observed on :	Runoff recorded in "n" years :	Peak runoff (m^3/sec) Bratránek value :	Runoff zone of author :	Runoff ratio $m^3/sec.km^2$:
87 Stratená - Hnilec	68.2	16.4	6.29.1958	11	73	+4	1.o5
88 Obyšovce - Svinka	343.9	132.o	6.29.1958	33	185	3	0.51
89 Solivar - Sekčov	351.3	40.6	3.30.1964	14	155	1+3+5=3-4	0.44
90 Svidnik - Ondava	167.5	81.o	3.31.1964	3	320	1	1.95
91 Jasenovce - Olka	173.8	25.o	2.15.1957	8	155	1+(4)=2-3	0.9o
92 Krásny Brod - Laborec	158.3	81.1	3.12.1963	11	312	1	1.97
93 Snina - Cirocha	256.8	143.o	1962	7	260	1+3 = 2	1.o2
94 Kamenica n/C - Kamenica	60.2	18.6	4.12.1963	4	103	1+3 = 2-3	1.7
95 Rem.Hámre - Cibavka (Okna)	41.8	12.2	12.14.1957	10	60	3+4 = -3	1.45

* The low value of the runoff "--4" instead "4" may be explained by the fact that it is a subsurface mining and natural cave region.

Chart #18:

Peak runoff of smaller watersheds and fish ponds in the plains and in the hilly country of Slovakia:[30]

Watershed (km^2):	No.	Steeper hilly country :	No.	Plains with wavy relief:	No.	Žitny Isl. Plains of Morava :
3	96	3.5	101	1.5	106	0.7
5	97	2.5	102	1.3	107	0.6
10	98	2.o	103	1.o	108	0.5
20	99	1.4	104	0.8	109	0.4
30	100	1.2	105	0.65	110	0.34

Note: "Steeper hilly country" is identical with the runoff zone #4 of the author. Zone "Plains with wavy relief" corresponds to zone #5 and #6 of the author. Zone "Žitny Island and Plains of Morava" is identical with the zone #7 of the author.

The Equations of the Author

For the individual peak runoff zones, the following equations could be used:

Zone 1 : Tertiary Paleogene formations: Magura-Flysch or Sandstone zone with over 940 mm average of yearly rainfall

$$q = \frac{17.5o}{A^{o.44}}$$

Zone 2 : Pienin-Schist of the Carpathians and Flysch-zone mixed with Limestone-zone in Southeast-Orava, North-Liptov and Upper-Torysa river region with 900 mm average of yearly rainfall

$$q = \frac{14.oo}{A^{0.47}}$$

30 Cablik, J., op. cit., p. 94.

Zone 3 : Earlier Paleozoic and Proterozoic formations: Cambrian, Precambrian zones and Crystalline Complexes of the Carpathians with 830 mm average of yearly rainfall

$$q = \frac{10.00}{A^{0.50}}$$

Zone 4 : Mezozoic formations: Limestone, Dolomite and Tuffstone zone with 750 mm average of yearly rainfall

$$q = \frac{6.00}{A^{0.46}}$$

Zone 5 : Tertiary Neogene formations: Zone of the hilly country of Slovakia with 700 mm average of yearly rainfall

$$q = \frac{2.50}{A^{0.37}}$$

Zone 6 : Quarternary Pleistocene formations: Diluvial zone or Diluvium mixed with Alluvium in Southwest- and Southeast-Slovakia with 670 mm average of yearly rainfall

$$q = \frac{1.90}{A^{0.36}}$$

Zone 7 : Quarternary Holocene formations: Alluvial zone in Southwest-Slovakia with less than 600 mm average of yearly rainfall

$$q = \frac{1.00}{A^{0.32}}$$

In equations of all the zones
q = Peak rate of runoff (m^3/sec.km^2)
A = Area of watershed (km^2)
The formulas presuppose a shape of watershed 1 : to 1 : 3 (Width to Length) and that 50 - 60 % of the area is wooded.

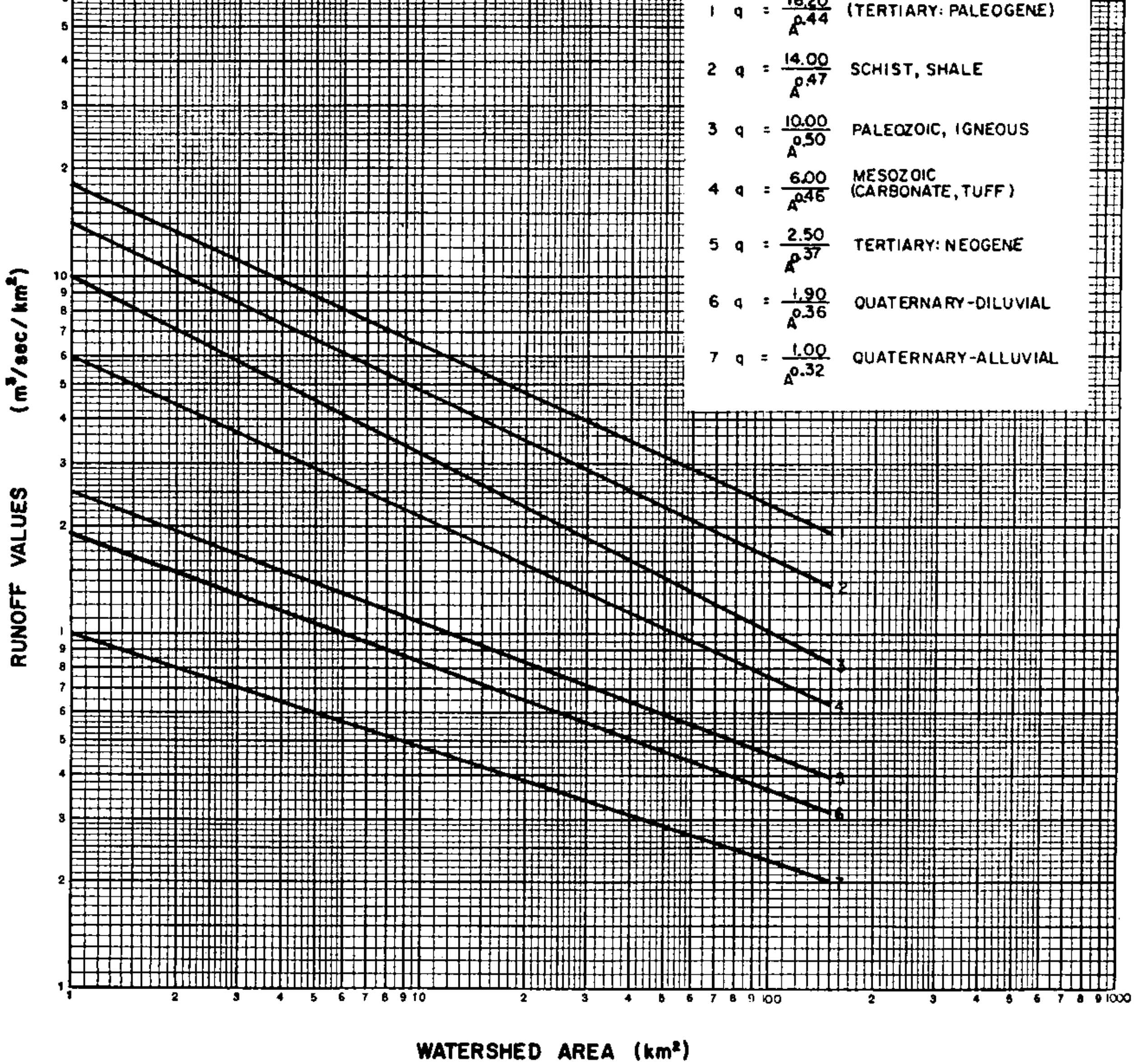
For better comprehension of the equations of the author, there are attached on the next pages:

Chart #19: Peak Runoff Expected Once in 100 Years
Charts #16 - 18: Data of Peak Runoff Compiled from Several Sources. (Runoff zones of the author are also noted.)
Figure 4: Division of Slovakia into Peak Runoff Zones of the Author.

Deviations from the Runoff Zones of Slovakia

Deviations are found only in the following areas: (Compare Figure 2 with Figure 4. In Figure 4 the deviations are already taken into account.)

a) In Southeast-Slovakia the mountains of Milič and Toroňa between the valleys of Bodrog and Hornád are too small in size compared to the adjacent


1 q = A0.44 (TERTIARY: PALEOGENE)
2 q = 14.00 / A0.47 SCHIST, SHALE
3 q = 10.00 / A0.50 PALEOZOIC, IGNEOUS
4 q = 6.00 / A0.46 MESOZOIC (CARBONATE, TUFF)
5 q = 2.50 / A0.37 TERTIARY: NEOGENE
6 q = 1.90 / A0.36 QUATERNARY-DILUVIAL
7 q = 1.00 / A0.32 QUATERNARY-ALLUVIAL
RUNOFF VALUES (m³/sec/km²)
WATERSHED AREA (km²)

c) The Subtatric Flysch belongs to zone 3 if the area is in the windshade of the high mountains: Liptov basin, Spiš basin and the valley of Torysa at Prešov and Sabinov. In all other cases, it should be computed as zone 2.

d) In cases "a" to "c", previously mentioned, the geological conditions should still be applicable when the watershed distinctively belongs only to one zone and the area is 100 km^2 or less.

The Local Corrections of the Formulas

The computed value of runoff should be modified in case the local conditions are different than the supposed ones and if the hydrological and meteorological conditions are changed. The corrections are as follows:

a) Correction of the average yearly rainfall:

$$C_1 = \frac{p-P_c}{p} \qquad \text{, where}$$

C_1 = Correction factor
P_c = Computed average yearly precipitation
p = Observed average yearly precipitation

b) Influence of the wooded area:

50 - 60 % of the watershed areas is forest. If the wooded area of the watershed is greater or smaller than the previously mentioned %, correction is needed:

1) Forest area is less than 50 % :

$$C_2 = 0.5 \; (0.5 - \frac{a}{A})$$

2) Forest area is more than 60 % :

$$C_3 = 0.5 \; (0.6 - \frac{a}{A}) \qquad \text{, where}$$

C_2 and C_3 = Correction factors
a = wooded area of the watershed
A = area of the watershed

c) Influence of the shape of the watershed:

Computed values must be corrected if the watershed has shape different from the assumed one, as has been mentioned. The correction is as follows:

Form of the Watershed:	Ratio of Width to Length:	C_4 (factor):
Fan - shaped	1 : 1	1.o5
Horse shoe - shaped	1 : 4	0.95
Elongated form or at least	1 : 5	0.9o

The calculated correction factors (C_1, C_2, C_3, C_4) have to be multiplied by the computed runoff to obtain the correct peak runoff value.

Finally, in special cases, the hydrogeological factors have to be corrected, as for instance in Horný Slavkov (Šariš, area of Upper-Torysa river), where - because of the local limestone formations (Karst) - the springs have a minimum 112 l/sec capacity. The watershed, geologically, sometimes belongs to two adjacent runoff zones. In such case, the runoff should be computed by interpolation corresponding to the % of the whole watershed area and using the equation which applies to the area in question.

Comparison of Observed Runoff Values with Computed Ones of the Various Authors. (The recorded data were observed in 1871-1964.)

Chart #20:

No.*	Hydrographic station - river :	Watershed area in km^2 :	Wooded in %	Peak runoff in m^3/sec :	Computed peak runoff in accordance with various authors in m^3/sec :					Runoff zone of Author:
					Iszkowski:	Bogdánfy:	Lauterburg:	Dub:	Author:	
2	Zdechovka - Zdechovka	4.o4	5	36.5	20	30	33	37.5	37	1
5a	Vychodná - Biely Váh	110.2	50	130	135	131	-	121	120	3
6	Kr.Lehota - Boca	116.6	90	95	120	81	-	101	94	3-4
8	Vysoké Tatry - Koprová	31.2	65	90	117	98	91	92	89	2
11	Podbansko - Belá	93.5	50	180	170	167	175	160	178	2
17	Podsuchá - Revúca	209.7	90	180	230	145	-	186	175	2-4
25	Turček - Turiec	45.o	90	70	95	90	73	72	70	3
31	Poluvšie - Rajčanka	230.3	60	165	196	160	-	172	164	3
32a	Zborov - Bystrica	210.o	20	275	230	210	-	290	285	1
33a	Vydrňanka - Vydrňanka	10.9	0	80	47	20	76	81	79	1
35	Hriňová - Slatina	71.5	85	85	62	107	87	86	84	3
36	Lučenec - Kriváň	265.5	65	95	117	178	-	108	96	4-5
37	Pláštovce - Krupinica	302.8	15	175	185	195	-	177	174	2
38	Štitnik - Štitnik	128.1	60	80	96	72	-	89	83	4
39	Rimavica - Rimavica	151.5	65	90	97	91	-	115	94	3-4
40	Hrabušice - Hornád	288.4	65	138	172	205	-	146	138	3-4

* The numbers are given in accordance with those in chart #16.

Accuracy of the various methods used in getting peak runoff values in chart #20 can be summarized as follows:

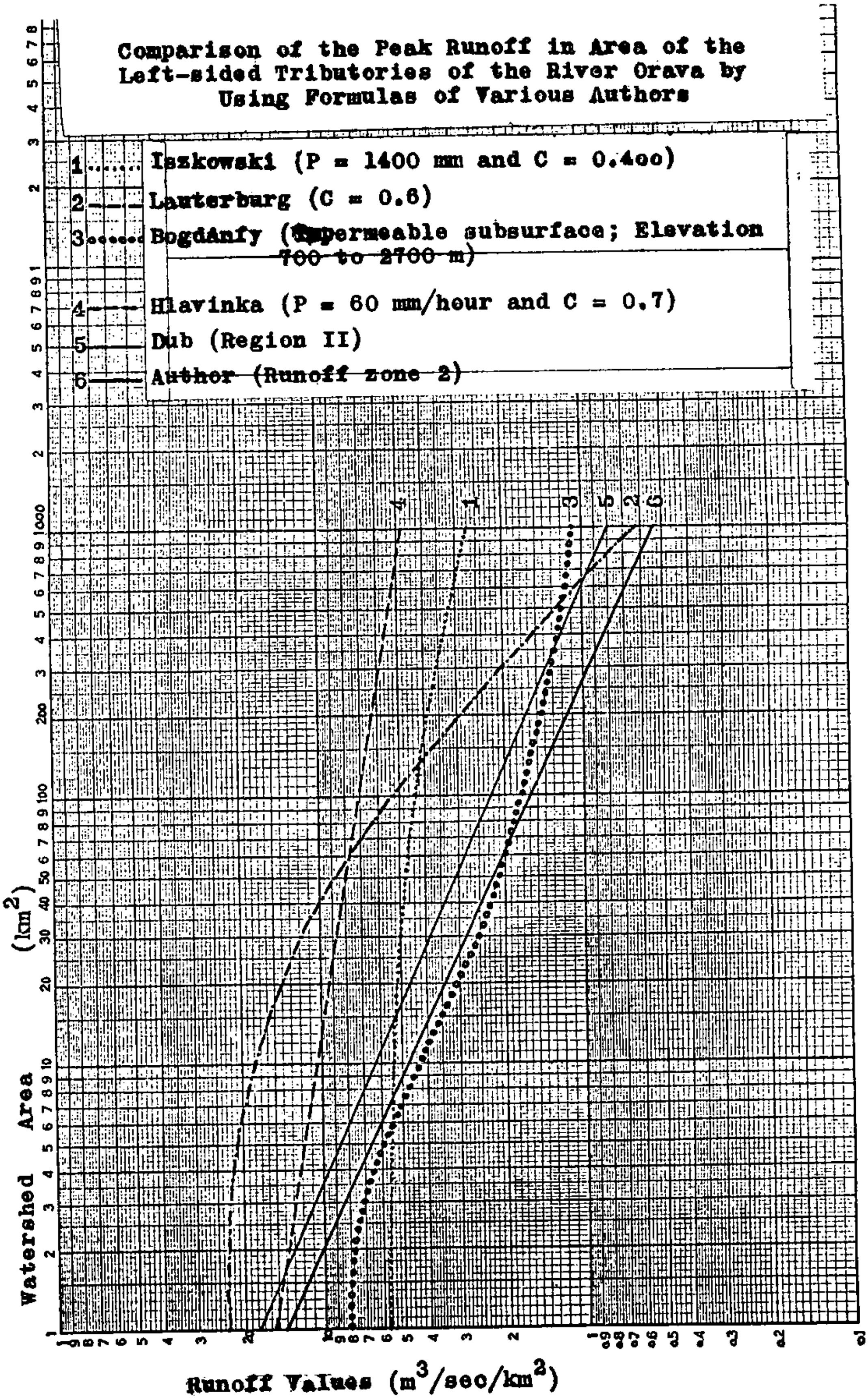
Chart #21:

Computed by the formula of :	Deviations in %: (summarized)	Computed in "n" cases :	Extreme deviation in %	
			+	-
Iszkowski	105.8o	16	35	45
Bogdánfy	103.13	16	94	75
Lauterburg	98.19	6	4	10
Dub	104.77	16	20	11
Author	100.o3	16	4	7.5

In the following chart, #22, the peak runoff values are plotted by methods of Iszkowski, Lauterburg, Bogdánfy, Dub and the author. It is computed for the left sided tributaries of the river Orava and, as is apparent from the chart, the diagrams have no conformity. Each curve has a different pattern, which proves that the equations of Iszkowski, Lauterburg, etc., can not represent the conditions, of course, they are not taking into account enough of the local geological factors. The formula of Lauterburg calculates such a high value that it gives too much safety on one hand and is uneconomical on the other hand because it is highly overdimensioned. Equations of Dub give better results but still too high values and the corrections and interpolations are a further source for error.

It is significant to notice that curves of the formulas of Bogdánfy, Dub and that of the author are running parallel for watersheds with an area 3 to 150 km^2. This can be explained by the circumstances that only these three methods are developed from the local conditions and, therefore, they should show some similarity.

Chart #22 :



Comparison of the Peak Runoff in Area of the Left-sided Tributories of the River Orava by Using Formulas of Various Authors
1 Iszkowski (P = 1400 mm and C = 0.400)
2 Lauterburg (C = 0.6)
3 Bogdánfy (Impermeable subsurface; Elevation 700 to 2700 m)
4 Hlavinka (P = 60 mm/hour and C = 0.7)
5 Dub (Region II)
6 Author (Runoff zone 2)
Watershed Area (km2)
Runoff Values (m3/sec/km2)

Conclusion

This study of peak rates of runoff from smaller watersheds clearly demonstrated the practical applicability of data about the underlain rock formation of the watersheds for computation of 100 year runoff values because, besides the rainfall intensity and duration, the subsurface geological conditions are the only factors which substantially influence these values. The vegetative cover and the shape of the watershed have only a $\pm$ 5 % effect on peak runoffs. The applicability is valid, of course, not only for Eastern Czechoslovakia, but also to Austria as was proved by the Austrian hydrologist Paplham in 1968. By 1970, a further study of the area of West Germany, Switzerland and Austria started to enlarge the applicability of and to refine the described runoff equations for smaller areas. After successful completion, the aim of the research is to achieve more general adaptable runoff formulas and to apply them in the area of the states of New Jersey, New York and of that of New England too.

Figure 1.--Map of Slovakia in 1965--1:750,000.

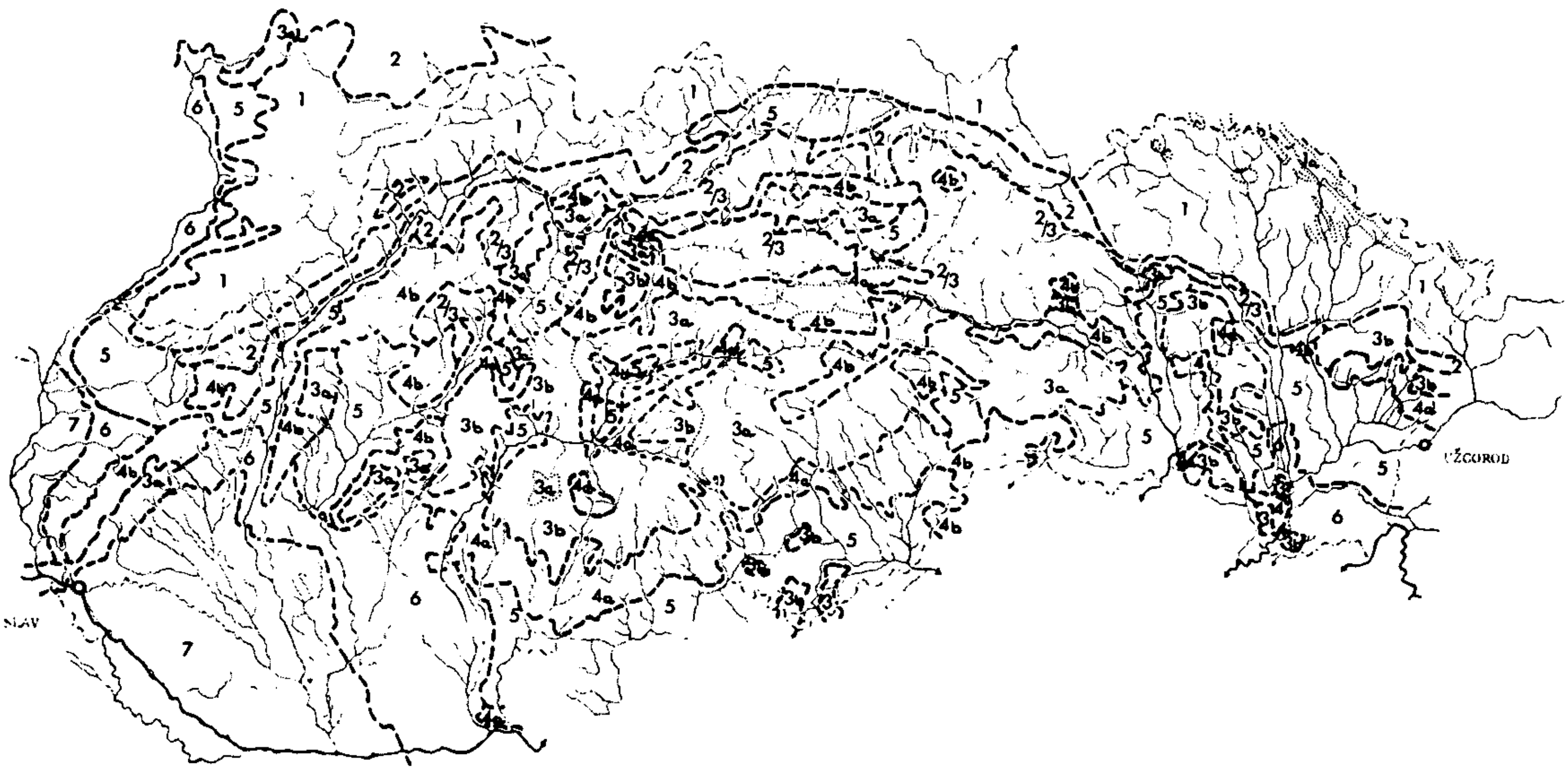


Figure 2.--Division of Slovakia into Seven Hydrogeological Zones, Based on "Hynie: Geological Map of Czechoslovakia - 1 : 500,000 (1954)", and Compiled by the Author.

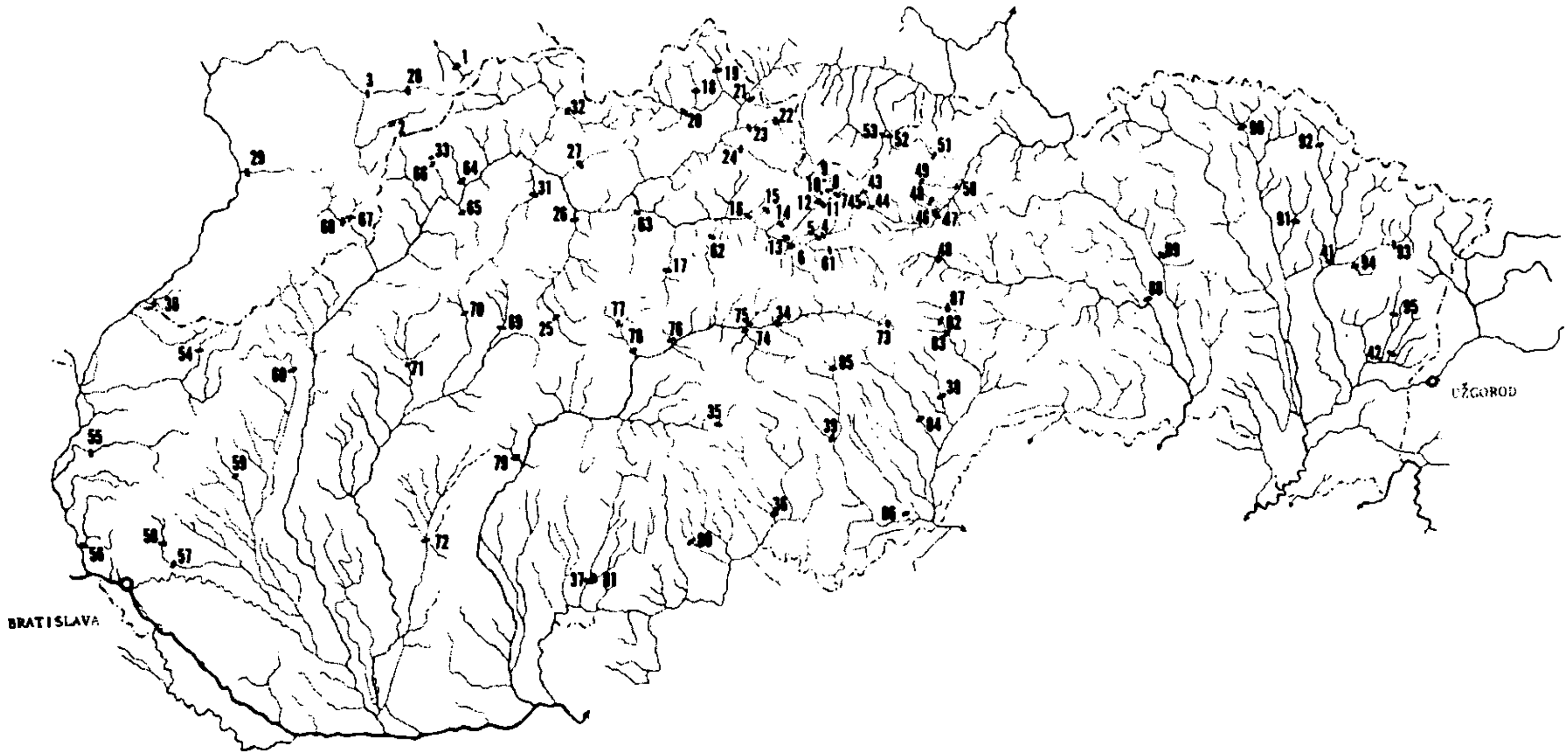


Figure 3. Hydrographic Stations of Slovakia Computed in Charts 16 and 17 for the Years 1871 - 1964. (Stations are numbered identically with those in charts 16-17).

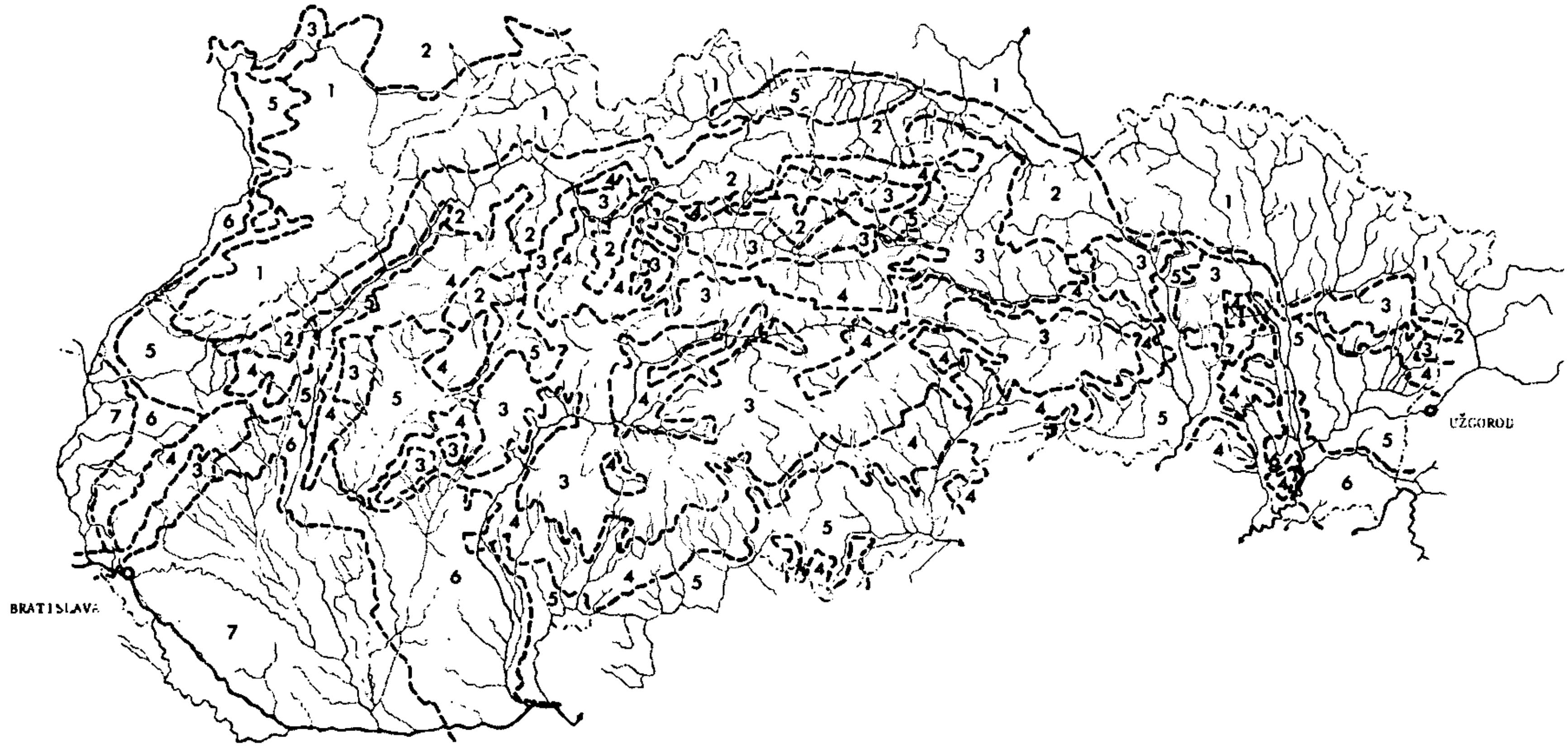


Figure 4.--Division of Slovakia into Peak Runoff Zones of the Author.

GROUND-WATER PROBLEMS ON LONG ISLAND, N.Y.

by

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Someone has said that there are really only three kinds of water problems in the world — factual ones, political ones, and hysterical ones, and that the last two kinds are invented by non-competent people who are observing and misinterpreting real facts. Unfortunately, there is a great deal of truth to this observation, and one doesn't have to probe very deeply to find some excellent examples of "water problems" that have no scientific basis whatsoever. The purpose of this paper is to take a close look at some of the "water problems" on Long Island, New York -- an area commonly referred to as being in serious trouble from a water-supply viewpoint -- and to try to put some of the claims into a more scientific perspective. In this connection, it should be noted that all water pumped for drinking supplies and other purposes on the island comes from wells, and that surface waters are not utilized except as farm ponds and for local irrigation. Thus, the island's so-called water problems all relate to ground-water resources.

Judging from articles and editorials in the press on Long Island, several major problems are threatening the island's ground-water resources. These come under the general headings of salt-water encroachment, declining ground-water levels, pollution by cesspools and by septic tanks, and a growing shortage or depletion of ground-water supplies. The press frequently contains news items and editorials on these subjects, and the whole topic is, of course, a fertile one for public figures looking for new and significant issues.

Before taking a look at the pros and cons of the water problems, however, it will be helpful to first review the hydrogeologic situation of Long Island. Basically, the island is a detached part of the Atlantic Coastal Plain (see Figure 1), and is roughly 120 miles long and 23 miles across at its widest point. It encompasses a land area of 1,400 square miles and is underlain by a series of unconsolidated beds of sand, gravel, silt, and clay whose total thickness ranges from about 200 to 2,000 feet. Below these beds is a relatively impermeable crystalline rock floor. Figure 2 is a generalized geologic cross-section showing the stratigraphy of the island; Table 1 is a summary of information on the principal geologic formations and their water-bearing properties.

From a ground-water standpoint, the bedrock beneath Long Island constitutes an effective bottom for the very thick water-bearing reservoir contained in the various unconsolidated geologic formations. The important water-bearing beds, or aquifers, are the Lloyd Sand, parts of the Magothy Formation, the Jameco Gravel, and the glacial outwash deposits. The clay beds and the glacial till deposits, on the other hand, are units of low permeability which retard the movement of ground water. In addition, the manner in which the different beds were laid down has produced a stratification such that water moves through the materials parallel to the bedding much more easily than it does in other directions. Figure 3 shows by contours the thickness of the unconsolidated materials.

The regional pattern of ground-water movement on Long Island is generally either north or south from the topographic divide located near the center of the island. As shown on the accompanying diagram in Figure 4, the movement of the water is partly downward in this divide area. Farther away from the divide, however, the pattern of flow becomes lateral, and as the shorelines are approached, the movement is generally upward. Most of the water which enters the ground-water reservoir moves laterally at shallow depths toward the sea or toward Long Island Sound, and only a small part penetrates to the deeper formations.

Practically all of the wells used for public water supplies on Long Island are deeper than 100 feet, especially in the densely populated western part of the island. In recent years, the trend has been to construct only deep wells for community systems, and almost all of these wells are screened in the Magothy Formation. However, many private wells do pump water from shallower beds.

Precipitation averages about 44 inches per year, which is equal to approximately two mgd (millions of gallons daily) on each square mile of land surface. Although the exact rate of replenishment of underground water is not known, all studies suggest that it is on the order of 50 percent of the precipitation, the remaining 50 percent being lost through stream runoff and evapotranspiration. Thus, recharge of ground water is probably about one mgd per square mile of land area, which in turn means that total recharge is approximately 1,400 mgd. Some of the land area is beaches and marshes, however, and a more conservative estimate of the amounts of water that theoretically could be relied upon for water supply might be on the order of 1,000 mgd.

Substantial quantities of ground water have been and are being withdrawn from the ground-water reservoir of Long Island. Figure 5 gives a breakdown of ground-water use for each of the four counties that comprise Long Island.

In Kings County, gross pumpage decreased from an average of about 60 mgd in 1940 to about 25 mgd in 1955. In the past 10 years, the average remained virtually constant at about 24 mgd. The decrease in gross pumpage in Kings County is related mainly to (a) the abandonment of many wells owing to salt-water contamination of the ground-water reservoir in the area, (b) closing of nearly all of the numerous ice plants in the county, and (c) the concurrent increase in the use of water from the New York City municipal-supply system that is derived from upstate New York.

In Queens County also, an increasing proportion of the public water supply has been derived from upstate surface-water sources; however, ground-water pumpage in the county has increased progressively in the past 26 years, from an average of about 55 mgd in 1940 to about 78 mgd in 1965. For many years, two privately owned companies have been providing part of the public water-supply requirements in Queens County from wells in the county. Accordingly, as the demand for water within the area serviced by the private companies has increased, new wells have been drilled and ground-water withdrawals have increased.

In Nassau and Suffolk Counties, where ground water has been practically the only source of public-supply water, gross pumpage has increased as the population increased. In Nassau County, it increased from about 75 mgd in 1940 to nearly 210 mgd in 1965. Similarly, gross pumpage in Suffolk County increased from about 30 mgd in 1940 to almost 120 mgd in 1965. Total ground-water pumpage on all of Long Island increased from 220 mgd in 1940 to about 430 mgd in 1965.

The volume of sediments saturated with fresh water is roughly 300 cubic miles, and if the average porosity of these beds is assumed to be 20 percent, then the island is in effect a vast reservoir containing 60 cubic miles of fresh water. This is equal to 66 trillion gallons, which could, for example, supply New York City's needs of one billion gallons daily for roughly 180 years. Since Long Island's own water needs are much less, and are only on the order of 400 mgd, the reservoir could theoretically supply this amount of fresh water for almost 500 years, even if it never rained again on the island. Although it would be impractical to hope to extract all of this water, there is absolutely no reason why a ground-water system couldn't be designed to make much better use of the reserves already stored in the ground.

It should be stressed again that the natural rate of replenishment of ground water on the island is about 1,000 mgd, whereas the total pumpage of ground water is about 400 mgd. It will take a good many years before use is equal to replenishment, and after that, the reserves in storage would last indefinitely, for all practical purposes, since they would only be called upon to make up the deficiency between replenishment and withdrawals. Nevertheless, Long Islanders generally have the impression (gained from reading newspaper articles) that a water shortage is imminent and that the problem is becoming more acute each year.

At this point, the reader may ask himself how anyone could misinterpret a situation in which the use of water is still not even 50 percent of the replenishment, and where the underground reservoirs contain water reserves that could last for centuries. To some extent, the answer is that very few people appreciate the implications of the water already stored in the ground, and moreover, lack the technical knowledge of how it might be utilized and managed. A second additional explanation is that the entire field of water supply is still dominated by the conservationist philosophy that use of a resource should never exceed the rate of its renewal, regardless of the amount already in storage.

This philosophy is clearly a correct one when applied to most surface-water resources. Obviously, one cannot extract from a river more water than flows down the river. But it may be a completely incorrect approach where ground water is involved, because the amounts of water already stored in the earth tend to be incredibly large in contrast with local demands for water. Thus, since the underground reserves alone may contain enough water for centuries of continuous withdrawal, the whole concept of limiting ground-water use to the rate of natural replenishment has no scientific justification in many localities. It is especially inapplicable on Long Island, for the reasons given above.

Recognition of the existence of 66 trillion gallons of fresh water already stored in the ground-water "tank" beneath Long Island should immediately make it clear why some of the other so-called water problems on the island also have little or no basis in fact. For example, salt-water encroachment is widely thought of as an immediate threat to drinking water supplies on Long Island, and considerable amounts of time and money have been invested in studies along these lines. Figure 6 shows areas where one or more water-bearing formations have already experienced some degree of encroachment.

Generally speaking, invasion of the aquifers by salty water has occurred only locally in a few places along the shores. In the first half of the present century, a rather serious amount of encroachment took place in Kings County. There, brackish water invaded several water-bearing zones and caused a gradual rise in the salinity of water being pumped from public-supply and industrial wells. Eventually, in the late 1940's, New York City condemned the public supply wells of the privately owned water company operating in Kings County, and turned to its upstate reservoirs to make up the deficiency. Since then, no ground water has been extracted for drinking purposes in that county.

Kings County has the least favorable hydrogeologic framework on the island, because the sediments are thinnest there (see Figure 3) and practically all of the rainfall is wasted to the sea through a county-wide storm sewer system. Moreover, the amount of ground water that had been pumped can be considered as large, when the relatively small size of the county is taken into account.

The only other reported instance of encroachment that is considered noteworthy is in southwestern Nassau and southeastern Queens Counties. The presence of salty ground water in that area was not known until the early 1950's, when a program of test drilling showed that parts of the Magothy Formation contained brackish to salty water. The first reaction was that encroachment must be taking place, since pumpage from wells tapping the Magothy was rather heavy in the area. Further study by the U.S. Geological Survey, however, revealed that the rate of movement of the salty water was extremely slow, perhaps in the range of only a few tens of feet per year. Moreover, data were uncovered later showing that salty ground water had been detected in the region as far back as the turn of the century, when pumpage from the Magothy was practically non-existent. The current view of hydrogeologists is that the presence of the brackish and salty water can be attributed largely to natural causes and not to human activities.

The other reported instances of encroachment shown on Figure 6 are quite localized, and have in no way harmed the overall ground-water resources of the island. In some cases, the reported contamination is probably due to leaky casings in wells situated close to the shoreline. In at least one other instance, the problem developed when salty water being used in a gravel-washing operation was allowed to soak into the ground close to a well.

Thus, by no stretch of the imagination can Long Island be thought of as having a serious salt-water encroachment problem. Even in the few verified instances of encroachment, the rates of movement are so slow that decades or even centuries may elapse before the salty water encroaches over a distance of one mile. Even then, the invasion may harm only a few wells, and from an economic viewpoint, it would be far cheaper to replace these wells with piped water supplies than to spend vast sums of money on dubious programs aimed at halting the encroachment.

In the writer's view, a number of basic misconceptions concerning the hydraulics and the dangers of salt-water encroachment are rather widely held by laymen. In the first place, salty water in a shoreline area always tends to invade an aquifer when the natural water levels in the fresh ground water are lowered as a result of pumping. In other words, any upsetting of nature's hydraulic balance, even if it be caused by pumping only a small amount of water fom a single well, will theoretically begin to induce encroachment. In artesian aquifers, however, and even in many water-table aquifers, the rate of encroachment is extremely slow, as pointed out previously. The net result is that little or no immediate harm is done to the regional ground-water supply, and decades may elapse with only a few wells being affected.

A second misconception is that once the salty water reaches a pumped well, the well becomes unusable. The fact is that only a tiny stream of contaminated water arrives at the well initially, and this becomes heavily diluted with much larger amounts of fresh ground water that are also arriving at the well. As time goes on, the relative proportion of salty water slowly increases, but it may take many years before the salt content of the water being pumped rises to a point where it can be detected by taste. Figure 7, which is a graph of the increase in chloride content at a well located in the heavily pumped southeastern part of Queens County, shows how slow the process is. The final reported chloride in the well was 160 parts per million, which is only about one percent of the chloride content of seawater. For a further comparison, the U. S. Public Health Service recommends that drinking water contain no more than 250 parts per million of chlorides.

Although the scientific evidence shows that the threat of encroachment is not very great, most laymen, and even some of the people who are thought of as knowledgeable in the field, have an opposite impression. Consequently, in the belief that something had to be done to retard encroachment in southwestern Nassau County, governmental agencies have come up with the idea of reclaiming sewage and injecting it into a profile of wells so as to create a ground-water pressure barrier near the shore. At present, a pilot injection well has been installed and pilot experiments are under way to purify sewage from the Bay Park Sewage Treatment Plant so that it can be returned throught the well. This effort has cost at least $1,000,000 thus far, and if a system of such wells were ever to be constructed, the total cost would be vastly greater.

Ironically, the sewage must be purified completely before it can be discharged into the injection well, so that it actually conforms to drinking-water standards. This, of course, raises the question of why anyone should bother putting water of such high quality into the ground when conceivably it could be used directly after leaving the treatment plant.

To summarize the situation, the salt-water encroachment problem is distinctly localized, and in no sense poses a major threat to water-supply systems. If public authorities would accept the idea that any well close to a salt-water body may eventually be contaminated, simply because the hydraulics of the pumping situation favors localized encroachment, then perhaps the problem could be seen in better perspective. It would also be of help to recognize that a few wells can afford to be lost without jeopardizing the entire water resource of the island, and that perhaps the taxpayer should not be asked to pay for exorbitantly expensive schemes designed to protect a few relatively inexpensive wells.

Another problem that has received a fair share of attention on the island concerns declines of ground-water levels, particularly in western Nassau County. The background of this situation is rather interesting. For many years, Nassau County has been developing at a rapid rate, as part of the suburban region just outside New York City. Water use has of course risen steadily, and is now somewhat more than 200 mgd. Prior to about 1947, however, there were no extensive sewer systems in the county, which meant that all water withdrawn for public supplies went straight back into the earth through cesspools and septic tanks. Thus, despite the statistics showing a steady increase in water use, the fact is that true consumption of water was probably only about five percent of total withdrawal, since practically all of the water went directly back to the place from which it had been taken. As a result, ground-water levels in those years showed no long-term declines -- a situation that puzzled many people who couldn't understand why the water levels could remain stable when so much water was being extracted.

In 1947, Nassau County completed construction of a major sanitary sewer system in the western part of the county, which now dumps into the sea about 50 mgd of water that previously had helped to recharge the county's aquifers. Beginning almost immediately as of that date, ground-water levels started to fall along the extreme western boundary of the county. The total decline is now about 20 feet, at a maximum, and can be thought of as representing a local overdraft. Agencies concerned with water resources are of course concerned about this, but have no control over the agencies that are responsible for sewer construction. The latter groups are proceeding with plans to sewer the rest of the county, which will certainly lead to water-level declines everywhere else in the county.

The basic motive for installing the Nassau County sewer system was to prevent pollution of ground water, since it seemed self-evident that the household wastes being dumped into the ground through cesspools and septic tanks must of necessity be a threat to human health. The facts are quite different, since disease-producing organisms can never move more than a few tens of feet at most away from a cesspool or septic tank in the sands of Long Island. Moreover, practically all public water-supply wells on the island take water from the Magothy aquifer, which is overlain by hundreds of feet of sand, gravel, silt, and clay, having excellent filtration characteristics. An additional fact is that even if germs could be carried along in the water, decades or even centuries must elapse before any water from the shallow glacial beds can penetrate to the depths where the screens of these wells are located.

Since, from a hydraulic viewpoint, disposal of wastes into the glacial beds cannot possibly transmit disease-producing organisms to the deeper public water-supply wells, the principal reason advanced for constructing the sewer system loses its validity. The second purpose of the sewers is to prevent seepage and spillovers from cesspools and septic tanks in low-lying areas where the water table is only a few feet below the land surface. This is a desirable objective, but applies only in shoreline regions and not in the much more extensive inland localities where the water table is at least tens of feet below the land surface.

Suffolk County, which occupies all of the remaining part of Long Island east of Nassau County, also has embarked on a plan to install an extremely costly sewer network. As in Nassau County, the announced justification for the scheme is to overcome pollution, which actually is not a real threat at all. No mention is made that a concurrent result will be a decline of ground-water levels.

The sewer scheme is not the only questionable program in Suffolk County.

A completely different one is a proposal to erect a nuclear-powered salt-water conversion plant to manufacture one mgd of fresh water near the town of Riverhead. This would, of course, be an extremely costly facility, the first of its kind in New York State, and undoubtedly would be the envy of many other nearby states.

Riverhead is located on the Peconic River, which flows right through the town into Peconic Bay, carrying an average flow of fresh water many times greater than one mgd. This available water could of course be utilized at a reasonably low cost -- far lower than the cost for demineralizing sea water. Moreover, the aquifers of the region have hardly been tapped, and there is no doubt that many tens of mgd of excellent fresh water could be developed from inexpensive wells close to the town. The plan to build the demineralization plant, which has received enthusiastic support from many quarters (excluding many water-resource specialists), appears to have no other justification than a political one.

In all fairness, Long Island is not the only place deserving criticism for ill-conceived water schemes. Many other parts of the U. S., and many other countries for that matter, have their own misconceptions, their own unscientific fears, and their own built-in political maneuverings with regard to water resources. Whereas no rational layman would ever dare to meddle in matters like designing a bridge or constructing a space vehicle, many somehow feel fully qualified to assert their personal views where water planning is involved. Consequently, a large percentage of water projects are conceived of and promoted by politicians, who use as justification their own misinterpretations of real facts developed by scientists. A report of a salty well, for example, clearly calls for a new program to halt encroachment. The existence of a cesspool clearly means that water supplies are in danger of pollution. An increase in water demands obviously justifies the building of a salt-water conversion plant. A decline of ground-water levels means that large sums of money must be spent to bring purified sewage back to the places where it used to go before the sewer system was built.

Luckily, in the case of Long Island, there is such a large reserve of excellent fresh water that no serious widespread water problems can arise in the foreseeable future -- despite the confusion and misinterpretations that now prevail. The unfortunate part is that politics and the outspoken views of uninformed people usually override the knowledge of the scientist, with the net result being that the taxpayer must pay for schemes and projects that cannot at all be justified by the minor benefits they achieve.

Table 1 - Water-bearing properties of major geologic units on Long Island, N.Y.

Hydrogeologic unit	Geologic name[2)]	Approximate maximum thickness (feet)	Water-bearing character
Upper glacial aquifer	Upper Pleistocene deposits	400	Mainly sand and gravel of moderate to high permeability; also includes clayey deposits of glacial till of low permeability.[3)]
Gardiners Clay	Gardiners Clay	150	Clay, silty clay, and a little fine sand of low to very low permeability.
Jameco aquifer	Jameco Gravel	200	Mainly medium to coarse sand of moderate to high permeability.
Magothy aquifer	Magothy Formation	1,000	Coarse to fine sand of moderate permeability; locally contains gravel of high permeability, and abundant silt and clay of low to very low permeability.
Raritan Clay	Clay member of the Raritan Formation	300	Clay of very low permeability; some silt and fine sand of low permeability.
Lloyd aquifer	Lloyd Sand member of the Raritan Formation	300	Sand and gravel of moderate permeability; some clayey material of low permeability.

2) Names are those used in reports by the Geological Survey. Perlmutter and Todd (1965, p.9) proposed that the Magothy Formation be divided into the Monmouth Group and the Matawan Group and Magothy Formation undifferentiated.

3) Permeability denotes how readily water can move through porous material.

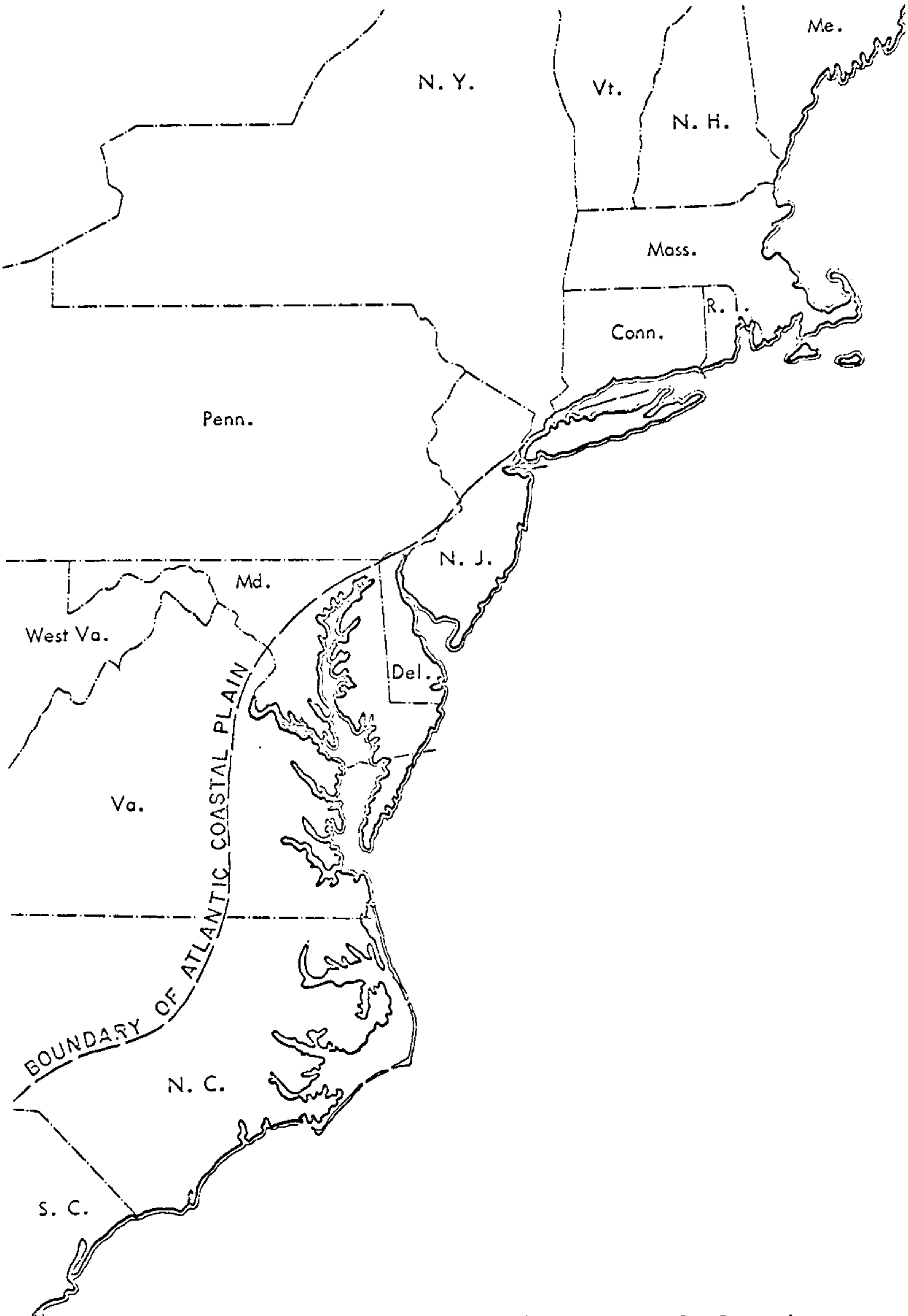


Figure 1. -- Relationship of Long Island to the Atlantic Coastal Plain.

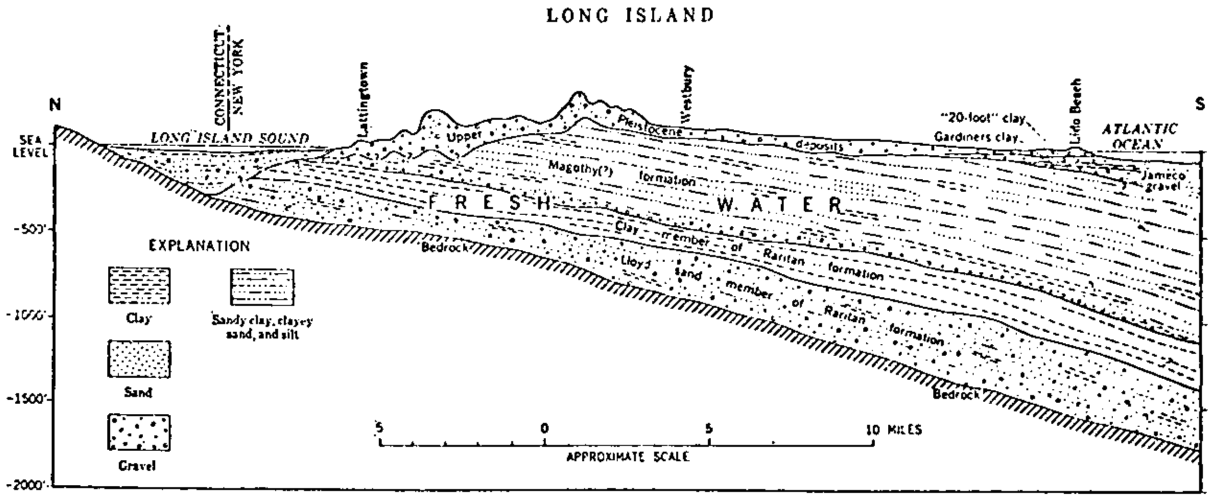


Figure 2. -- Geologic cross-section through central Nassau County, Long Island, N.Y.

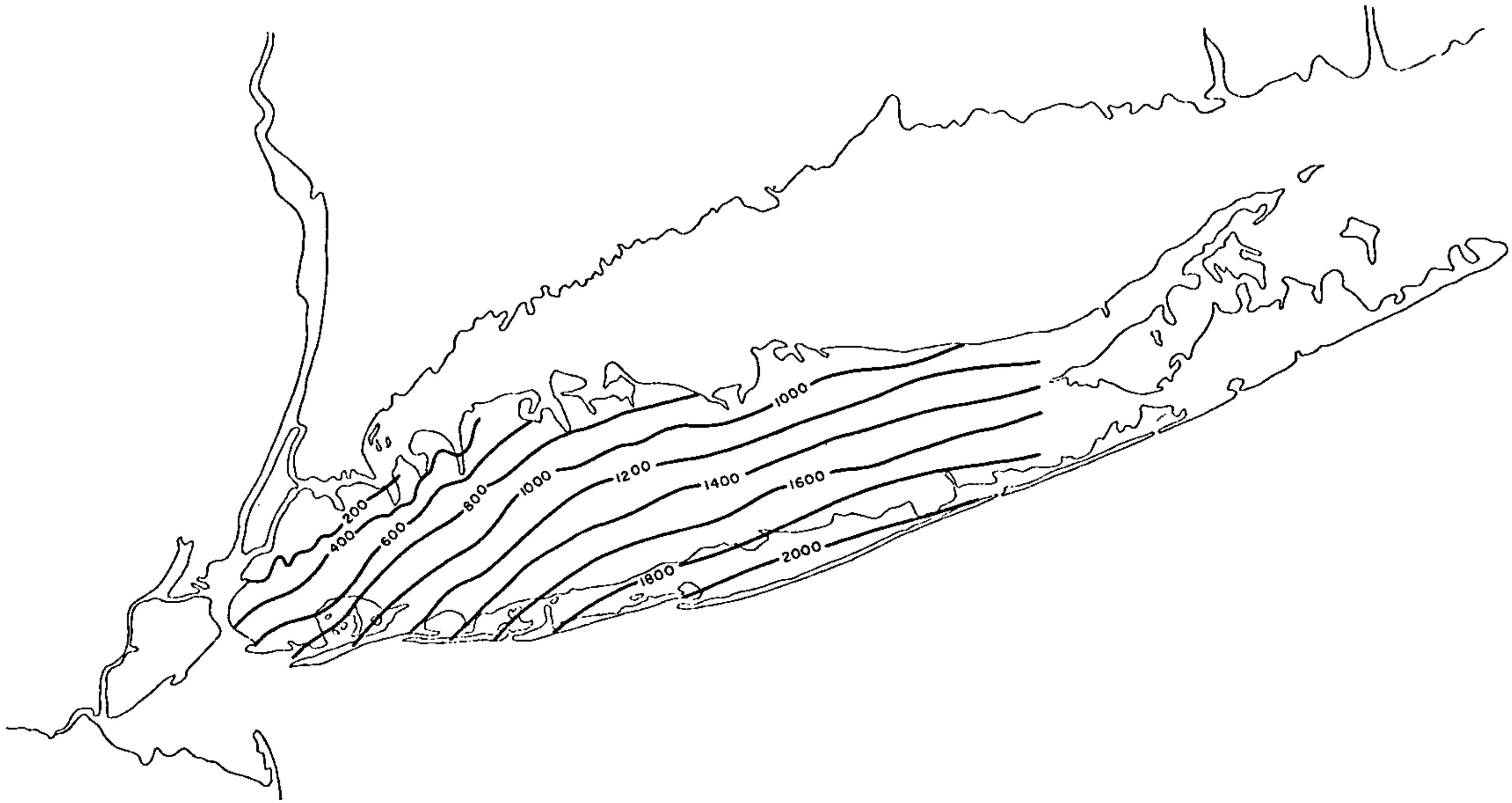
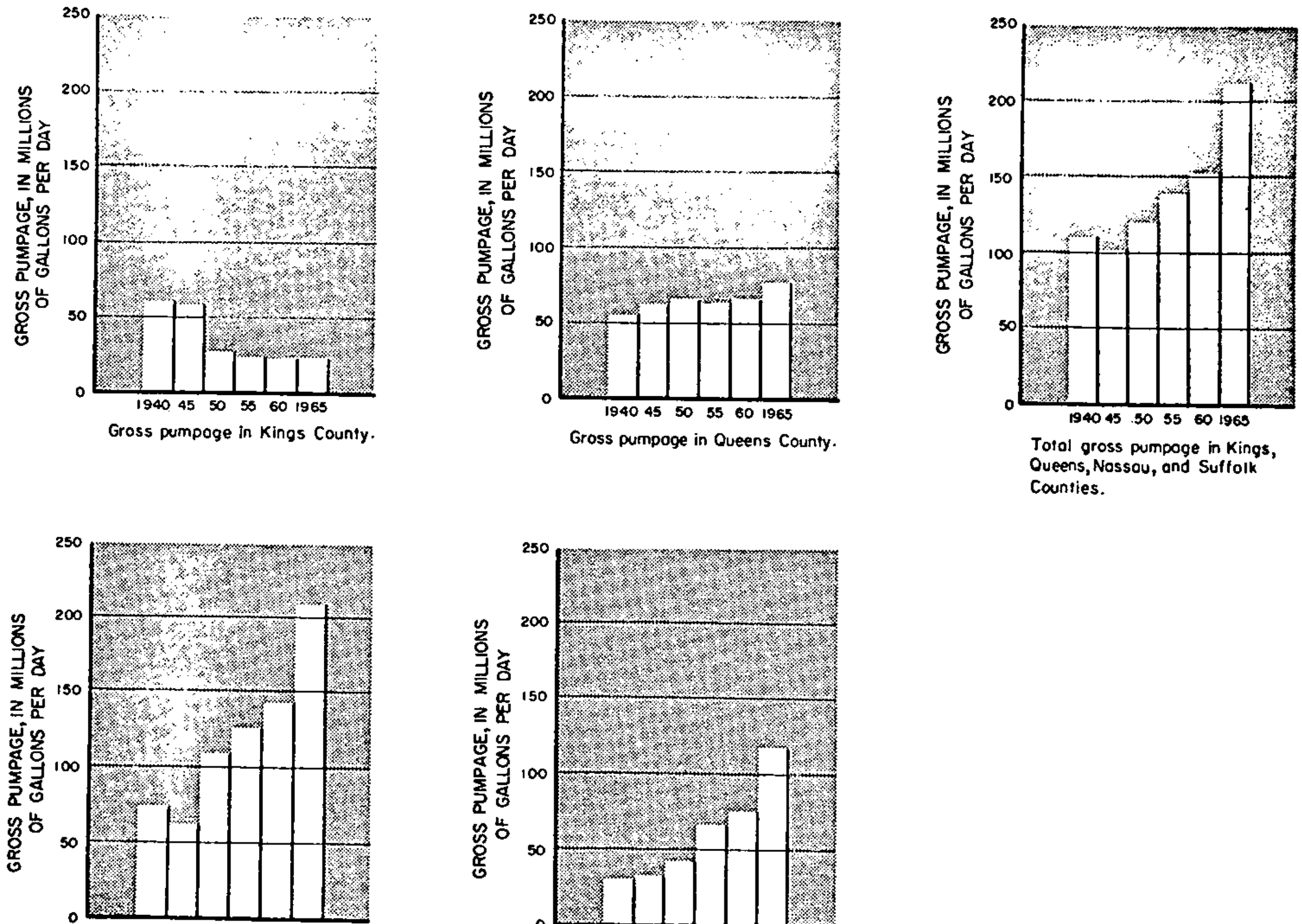


Figure 3. -- Thickness of unconsolidated water-bearing deposits on Long Island, N.Y.



Gross pumpage in Kings County.

Gross pumpage in Queens County.

Total gross pumpage in Kings, Queens, Nassau, and Suffolk Counties.

Gross pumpage in Nassau County.

Gross pumpage in Suffolk County.

Figure 5. -- Ground-water use on Long Island, N.Y., 1940-1965.

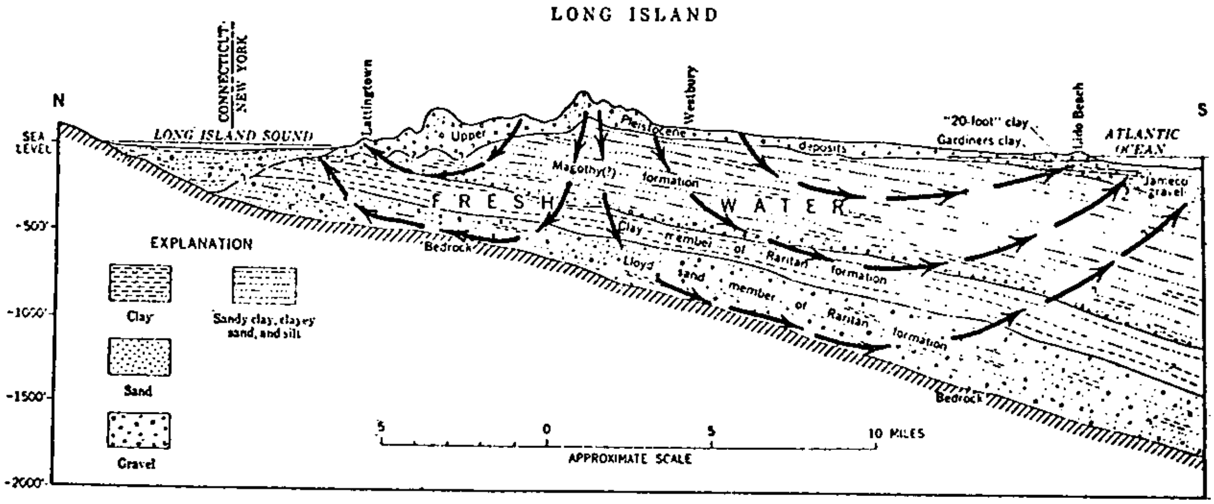


Figure 4. -- Generalized pattern of ground-water movement in Long Island, N.Y.

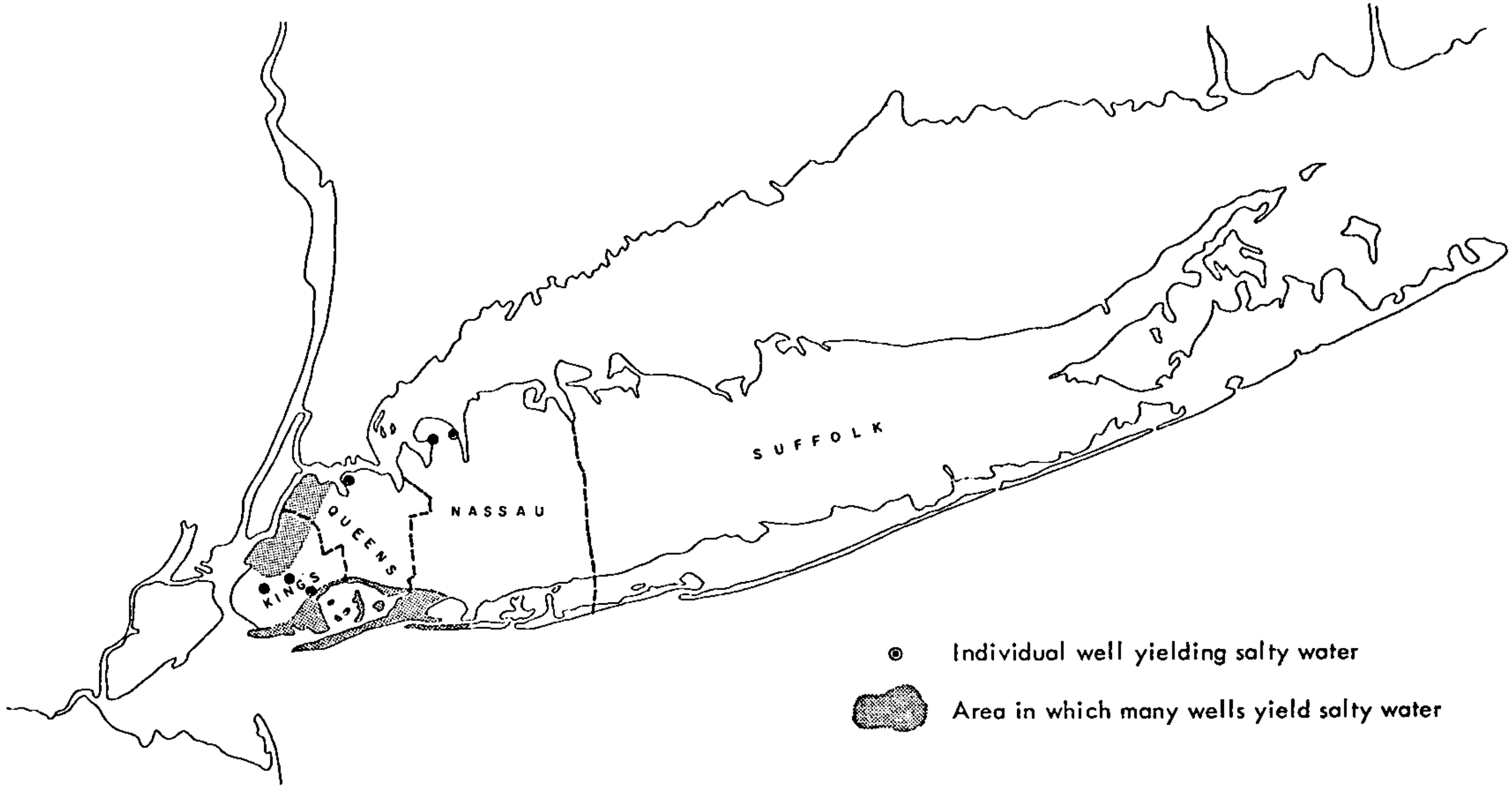


Figure 6. -- Places on Long Island where one or more water-bearing beds contain only salty ground water.

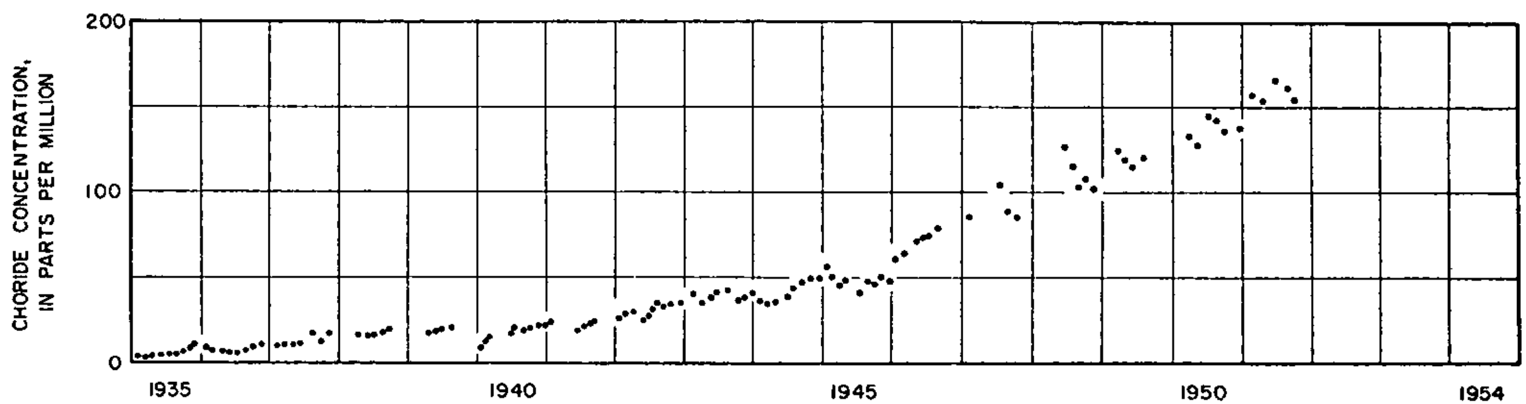


Figure 7. -- Increase in chloride content of Well Q559, Queens County, N.Y.

INSTREAM AERATION OF SMALL POLLUTED RIVERS (PASSAIC RIVER IN NEW JERSEY)

by

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The New Jersey Water Resources Research Institute has conducted field tests of artificial aeration of the Upper Passaic River during two summer seasons, and has found that this method has real possibilities for maintaining water quality where the river pollution problem is serious. Aerators used are the same types which have been commercially developed for activated sludge waste treatment and aeration lagoons. There has been little attention given to the possibilities of artificial river aeration, as a possible supplement to standard waste treatment, and the few studies of this matter have mostly used performance data obtained from waste treatment experience. However, as the Rutgers experience shows, aerators do not perform in moving rivers in the same manner that they do in tanks or lagoons. The Rutgers tests are the first full-scale scientifically-controlled experiments to determine just what the equipment is capable of accomplishing in flowing streams. Aeration equipment previously has been actually used in streams in a number of cases, as an expedient means of dealing with some intolerable situation, but for technical reasons it has not been practicable to evaluate performance with any accuracy.

The New Jersey aeration field tests were conducted under a demonstration project of the Federal Water Pollution Control Administration (FWPCA), entitled, "Oxygen Regeneration of Polluted Rivers," which was approved in 1967 with a first year budget of $102,000. The State of New Jersey Department of Conservation and Economic Development also contributed matching funds of $20,000. Second and third year grants were funded for a somewhat lesser amount. The project was originated by the writer, and several senior faculty members also participated to a major extent in this demonstration project and related research projects, including: Dr. Joseph V. Hunter, Dr. Burton Davidson, Dr. Frank Dittman, Dr. George Mattingly (Princeton University) and Dr. Shaw Yu. Consultants Hazen and Sawyer of New York were employed for design and cost data.

During the first year of the project, a mechanical aerator was installed, demonstrated, and tested on the Upper Passaic River, near Pine Brook. See Figures 1 and 2. This site was chosen because the river at this point is quite heavily polluted, with summer oxygen levels usually not over 25% saturation. The river is small enough so that a single aerator makes a significant change in dissolved oxygen levels. There are over 12 miles of channel without a significant pollution source, or intervening tributary, furnishing an almost ideal river for observing changes. Moreover, the river is used for municipal water supply, the area is developing rapidly, and the Upper Passaic River water quality problems are extremely serious and widely known. Of course it should not be assumed that any actual river maintains a uniform water quality. Figure 3 shows the wide variations of Biological Oxygen Demand (BOD) with relationship to discharge during the three month period July-September 1967, and Figure 4 shows the Biological Oxygen Demand variations at a single point in a 24 hour period. Obviously, any single sequence of data cannot be relied upon for analysis; and a prolonged series of tests were necessary.

The mechanical aerator used was a 75 h.p. electric driven Yeomans Bros. unit. It has an eight foot impeller which throws the water peripherally to the sides, creating an extensive zone of high turbulence. Figure 5 is a schematic diagram illustrating the operation. The impeller has the characteristic of recirculating some of the water, which comes back towards the aerator at low elevation and is drawn into the impeller above. Figure 6 shows the construction of the mechanical aerator at site. After assembly it is lifted into the water with a crane. The aerator is shown in operation in Figure 7. The dissolved oxygen level of such streams is characteristically satisfactory during the colder months, so the aerator in a completed system could be removed from the river to avoid icing during the winter. In the test installation, the mooring cables extended across the river; but at a permanent site, a pile cluster or crib would be provided to allow mooring without interfering with passage of pleasure craft. A typical result of the aerator operation is shown on Figure 8, which shows a dissolved oxygen level of approximately 2 mg/l upstream of the aerator being raised to approximately 5 downstream. This result varied, particularly with the discharge, the temperature and the dissolved oxygen level upstream. Figure 9 shows how the transfer rate varies with the velocity of stream flow, a considerably higher rate being obtained at the greater stream velocities. This is important, because, contrary to first impressions, it requires much more aeration capacity to insure satisfactory dissolved oxygen at medium summer flows than at the very low flows at which dissolved oxygen may otherwise be absent.

The application of oxygen transfer rates to the biochemical dynamics of the stream required extensive computations. In addition to parameters already mentioned, determination had to be made of the BOD absorption coefficients (both carbonaceous and nitrogenous), the effects of benthal action and photosynthesis, and natural aeration coefficient and certain characteristics of the water itself. Figure 10 shows how the reconciliation of various parameters was modelled and checked on an analogue computer. Similar plots were made for river conditions with the aerator in operation.

These analyses produced a surprise. The BOD removal coefficient for several miles downstream of the aerator was much higher than it was without the aerator operating, two or three times as high. One possible explanation would be that this was due to the stirring up of sludge deposits by the aerator which would then exert a greatly accentuated oxygen demand. However, the results persisted after several weeks of testing; moreover, the basin within which the aerator was operating had a sand bottom, so there was probably not a great quantity of readily suspended organic material. Another alternative is that the turbulence itself accentuated biochemical action. However, such an increase would probably have been no more than about 20%, whereas the increase observed was much greater. This striking parameter increase therefore remained for later determination. An interesting hypothesis has been advanced that this effect is due to a stimulation of the nitrogenous BOD action which apparently remains normally quiescent past this point in the stream. But this hypothesis remains for further research, as will be mentioned later.

During the summer of 1968, a diffuser, a bubbler aerator, was installed for test about 2 1/2 miles downstream and operated both separately and in tandem with the mechanical aerator in site. Figure 11 shows the diffuser manifold ready for installation. It consists of two 9" pipes, in each of which 80 Link Belt diffusers are mounted vertically at 1' intervals as shown in Figure 11. These are relatively coarse hole diffusers each having 12 holes, approximately 1/8" diameter. It was known that fine bubble diffusers can provide higher efficiencies, but such diffusers often clog up when used over extended periods of time and it was decided to use equipment representing a conservative design. The manifold was placed in an excavated trench leaving the diffusers 6' - 8' below the surface of the water, depending on the stage.

Figure 12 shows the diffuser aerator starting operation on the first day. Distribution of bubbles was excellent, and a wide band of water was placed in uniform turbulence. However, efficiencies achieved were on the order of 3%, because of the shallow depth and relatively high temperature.

When compared on the basis of pounds per horsepower hour, the mechanical aerator had considerably higher transfer rate at both high and low rates of flow as shown by Figure 13. These transfer rates represent performance at standard conditions, namely, 20°C, pure water, sea level barometric reading, and a starting condition at zero dissolved oxygen. Transfer rates under actual conditions may be quite different; and results in such as these of Figure 13 should never be used without adjustment.

The river aeration work has required cooperation from various agencies. In addition to the state financial support, provided through Commissioner Robert Roe, the United States Geological Survey, under Mr. John E. McCall, made a number of special discharge readings and correlations of the Pine Brook gaging station, and provided major assistance in dispersion studies, using fluorescent dyes; Mr. Robert L. Vannote on behalf of the Morris County Freeholders and the Mosquito Extermination Commission arranged needed rights-of-way without cost, made the necessary channel excavations, and loaned certain equipment. The Essex County Mosquito Extermination Commission also offered help. The Passaic Valley Water Commission, through Mr. W. R. Imhoffer and Mr. Frank J. de Hooge, furnished background data. Meaningful liaison was arranged with the field office of the FWPCA, through Mr. Kenneth Walker. For the Delaware River tests, cooperation was also received from the City of Camden, through Mr. John Frazee.

When the results given above were applied to the Passaic River on a systems basis it was found that, after a certain stage of treatment has been reached, river aeration is by far the most economic means to obtain the desired oxygen levels. Second stage treatment of effluents of all municipal and industrial treatment plants will be required in any event. However, fast-growing, heavily-developed river basins such as the Passaic will need either advanced waste treatment, reservoirs to provide flow augmentation, or instream aeration; and the instream aeration will be by far the least expensive. Since these studies have been initiated, quantitative results have become available of similar aeration work on the Ruhr River in Germany. German analysis has found, as has Rutgers, that instream river aeration is by far the most economical step toward obtaining good water quality, once the second stage treatment has become general. A systems analysis by Resources for the Future on the Potomac River estuary, although not supported by field tests, indicated the same result. There seems little doubt that instream aeration must be considered in planning for water quality systems.

The work described above is not the end of the research and test program. During 1969 tests were carried out on the Delaware estuary at Camden, New Jersey. The analysis of these results is nearing completion and will show the applicability of aeration equipment to deep tidal estuaries. Entirely different types of installations will be optimal, but again river aeration will prove its worth. For the summer of 1970 a further program has been authorized which will bring in for test a new type of oxygen diffusion apparatus, supplied by tanks of liquid oxygen, and provided by a prominent aerospace corporation. The results of these tests and the relative economics of the new method cannot be anticipated, but are being awaited with keen interest. The Water Resources Research Institute has no interest in the success of any particular type of equipment and will evaluate the new apparatus objectively with the old. The cooperation between a university research institute and a major corporation on a problem of mutual interest is certainly interesting, and is indicative of the immediate importance of the problems involved.

Many questions remain to be resolved as to the means by which these new technologies may be introduced into planning for basin-wide water quality control. Now that the practicability and economics of the new method have been demonstrated the matter may be expected to develop fairly rapidly as the new potentialities become more widely known. However, the institutional arrangements at Federal, state, and local level are based upon the older approach of treatment only, without much of a systems analysis; and current analytical and procedures planning must be drastically improved before the new technology can be readily evaluated. One of the problems in using instream aerators is that their use will require a degree of surveillance and a type of basin water-quality management which is quite different than what exists today. The characteristic situation in metropolitan area basins is subdivision of operating functions by area among a number of municipalities and other state subdivisions, with the state and Federal agencies making water quality checks at intervals of months. Only where water supply is directly and seriously involved is it customary to make daily or weekly checks;* and the operating functions are conceived of as centered in the treatment plants and decentralized to individual municipalities, or sometimes to a trunk sewer authority. Now, however, it may be necessary to obtain integrated management of a series of aerators, which may be visualized as being spaced at approximately one mile intervals down the parts of stream systems which are polluted. Moreover, the planning of control measures will need to envisage the entire biochemical regimen of the stream, which has been largely avoided heretofore by means of various approximations and what amount to rough extrapolations of current situations. This problem of an inadequate technical basis for analysis of water quality systems has begun to cause some difficulties; but the consideration of the potentialities of instream aerators will render this problem more acute, and make it imperative that data and methodology for water quality systems analysis be developed much more rapidly than before.

It has not been decided what Federal agency will have cognizance of instream aeration programs, nor who will pay for them. These questions should be resolved, because this is an idea whose time has come. We cannot afford to build uneconomic systems in the field of water quality any more than in any other field; and if the instream aerators will achieve desired water quality standards less expensively than other systems, some way should be found to incorporate them into the total program.

* The developing systems of continuous monitoring by <u>ORSANCO</u> and a few other areas are still quite exceptional.

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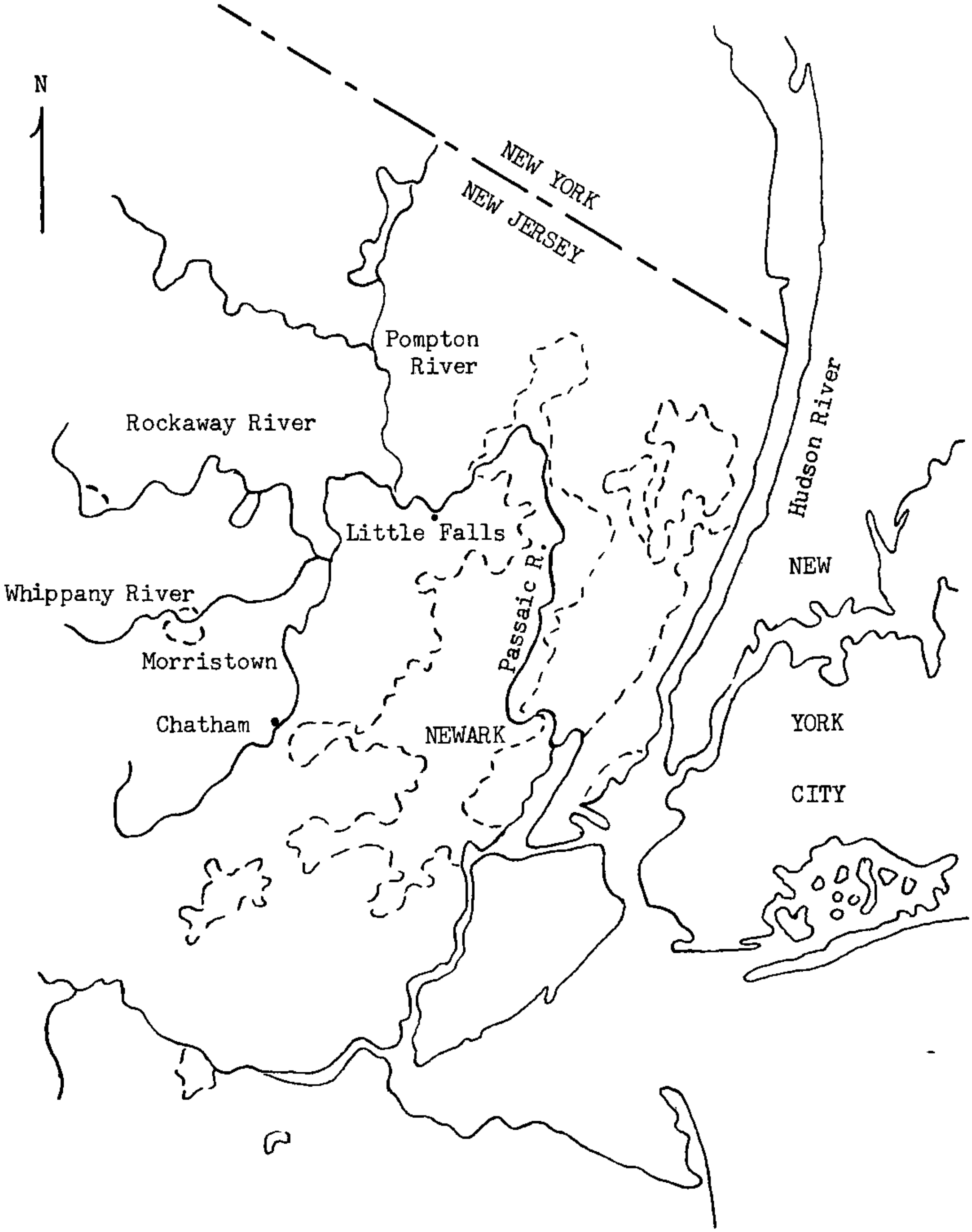

Figure 1. -- Limits of major built-up areas in New Jersey.

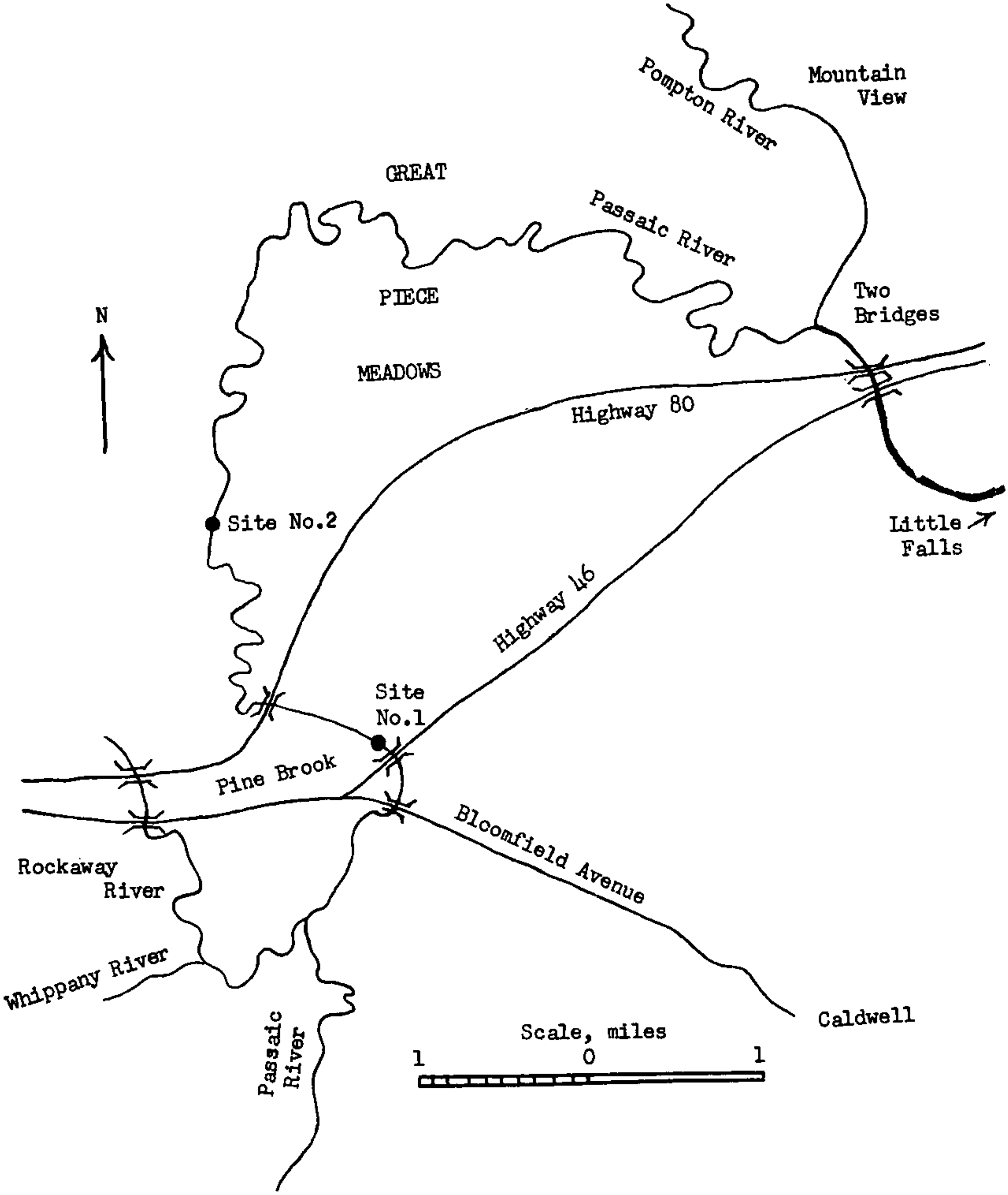

Figure 2 -- Test reach - Passaic River.

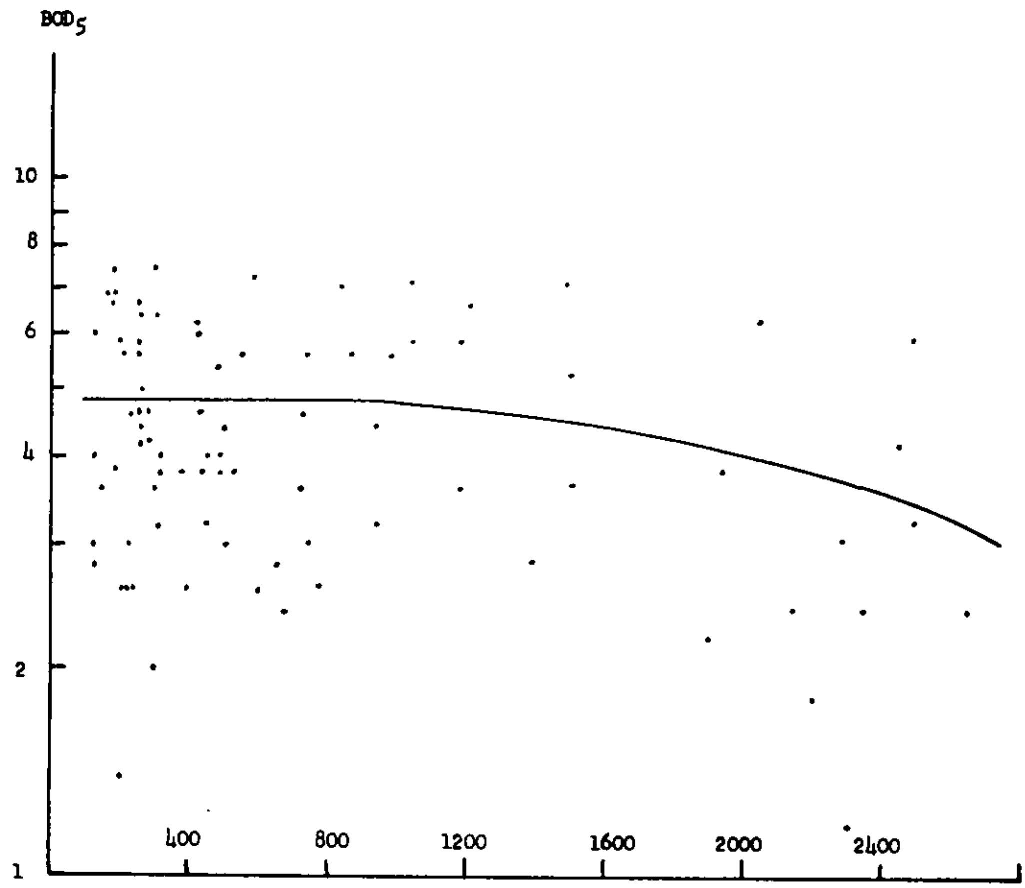

Figure 3 -- BOD discharge relationship Passaic River, Little Falls.
(July, August, September, 1967)

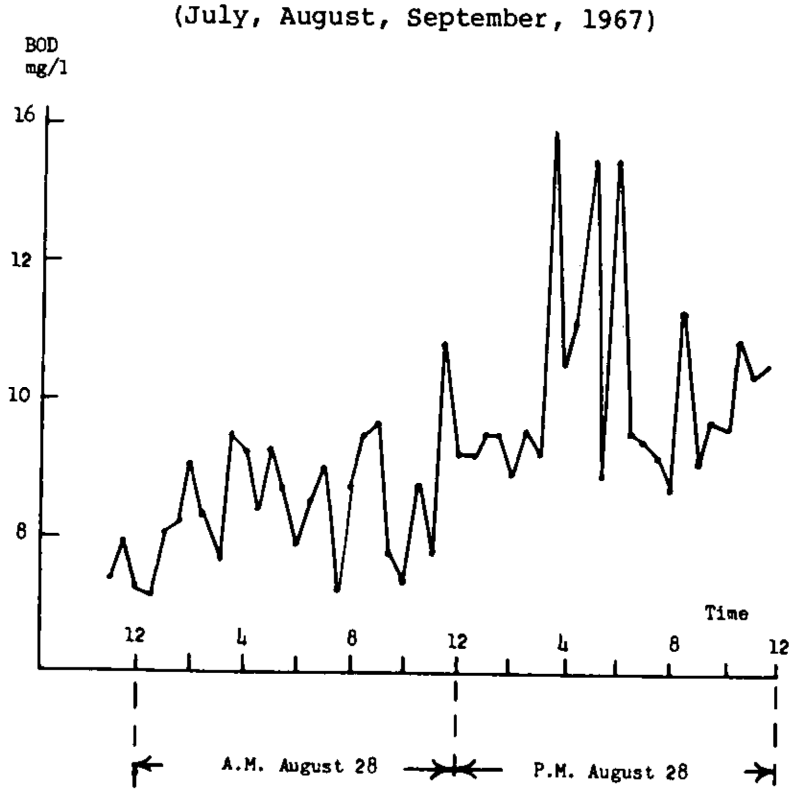

Figure 4 -- 24 hour BOD observations, Station -200.

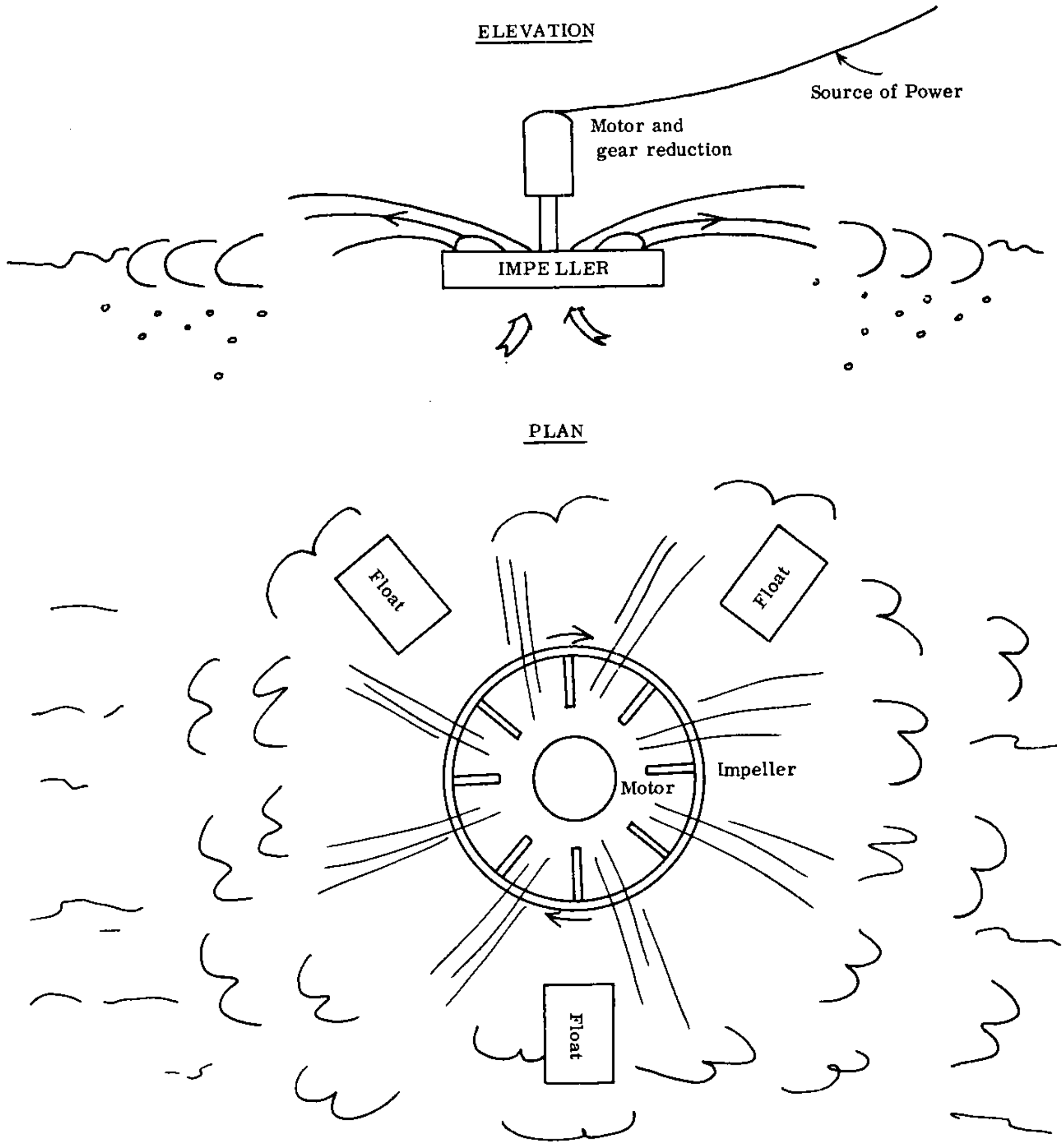

Figure 5 -- Schematic diagram of mechanical aerator.

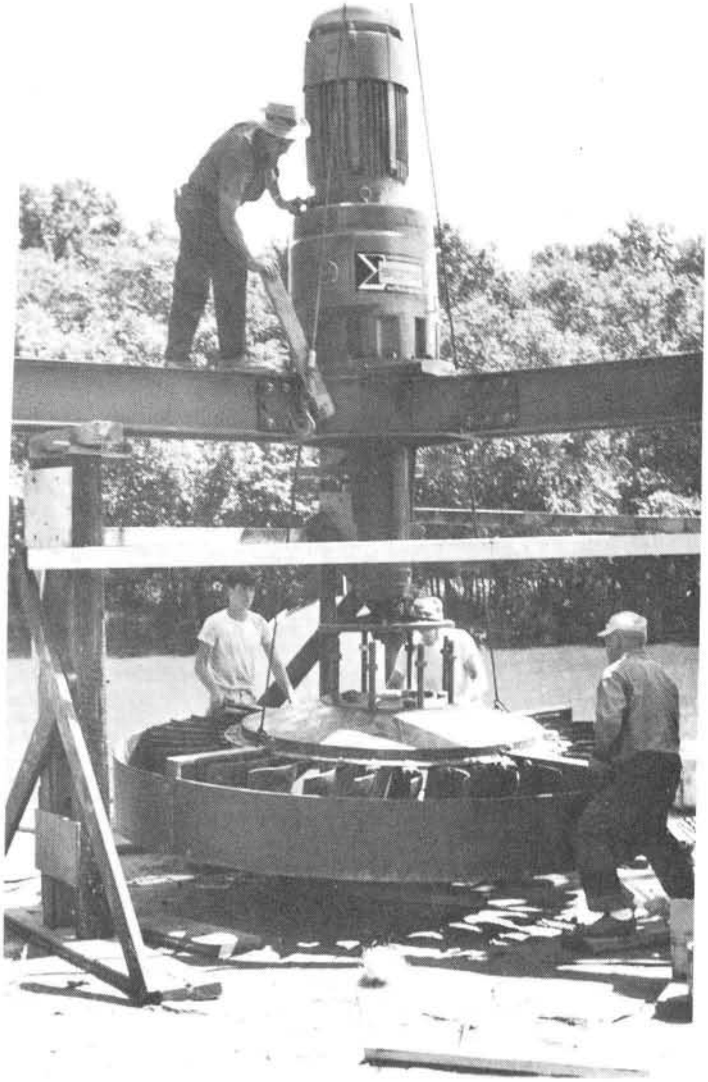

Figure 6. -- Construction of mechanical aerator.

Figure 7 -- Operation of mechanical aerator.

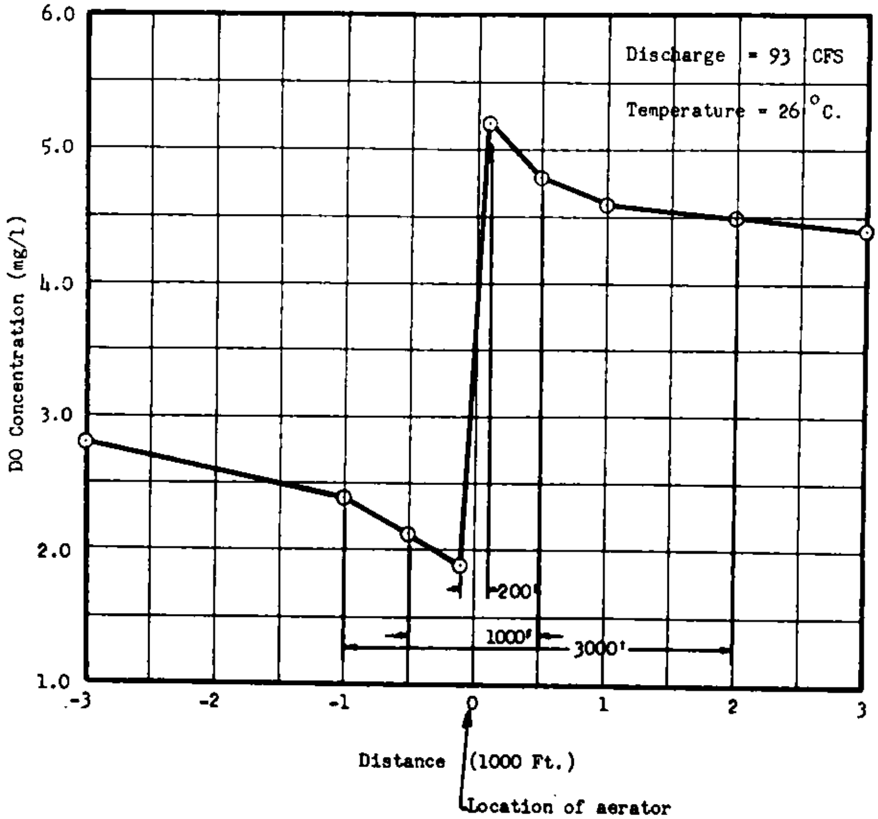

Figure 8. -- A typical longitudinal DO traverse (August 2, 1968).

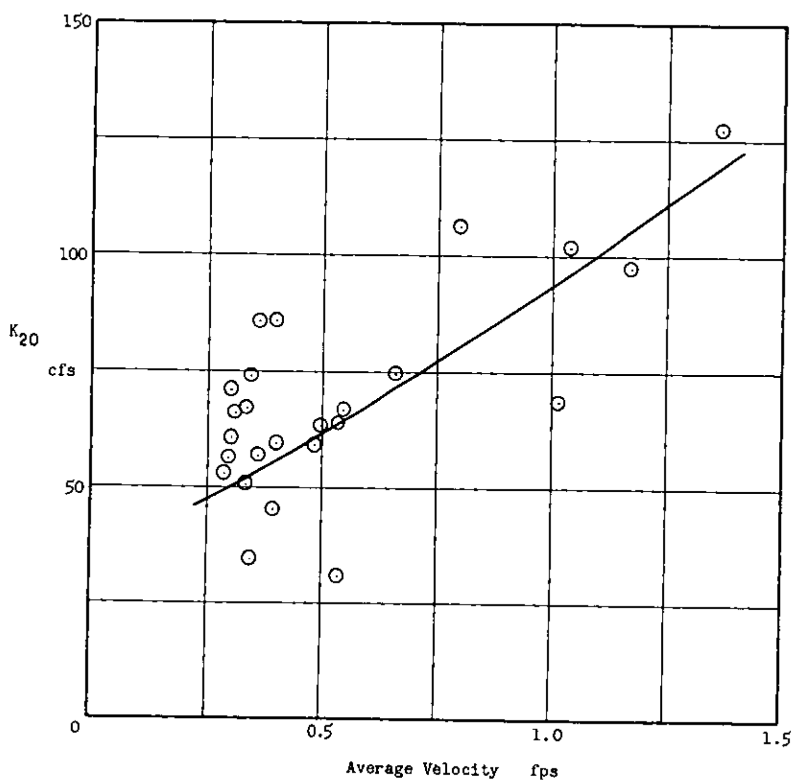

Figure 9. -- Oxygen transfer rate coefficient versus stream velocity mechanical aerator.

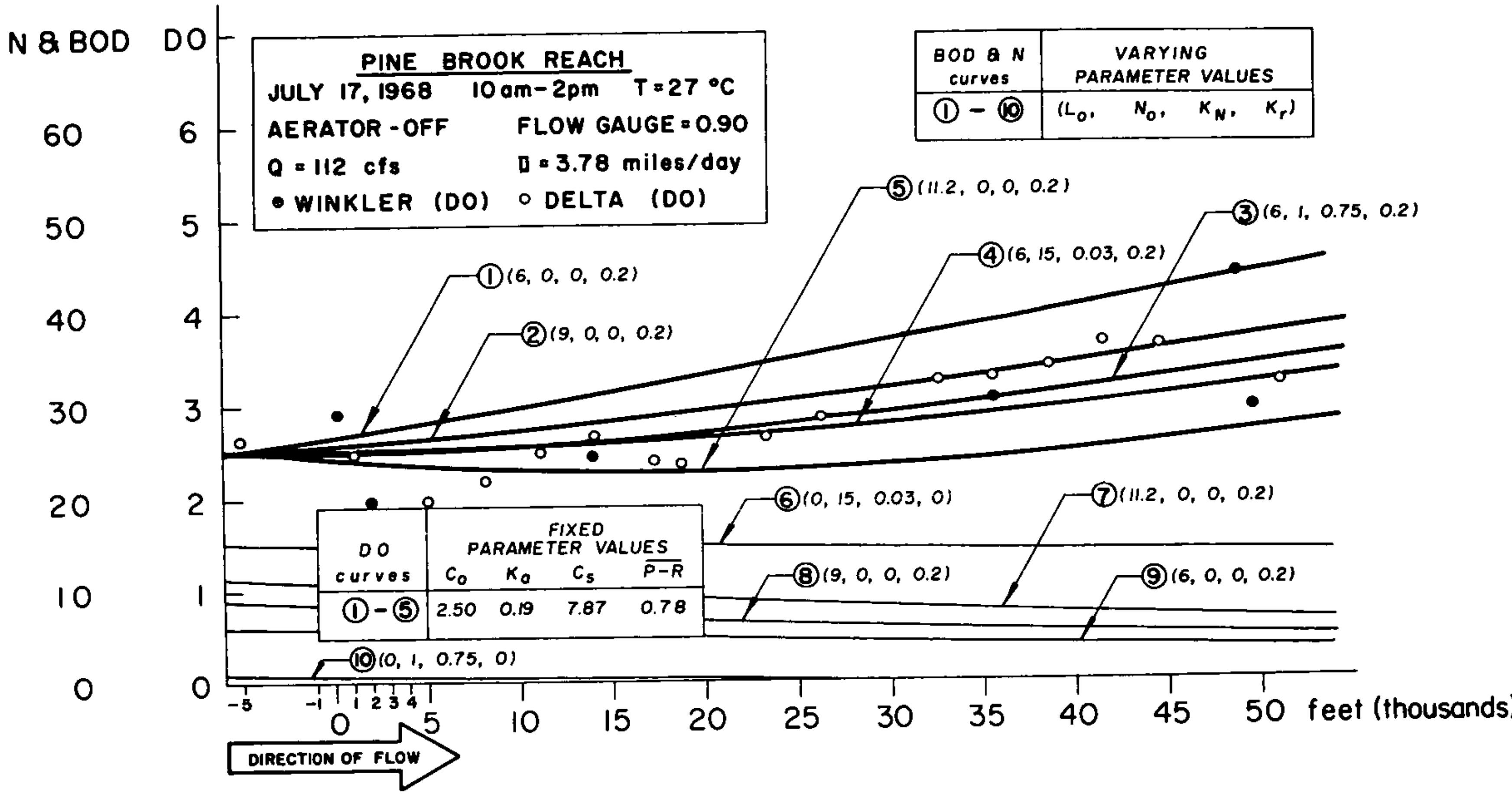

Figure 10 -- Simulated plots of DO, BOD and nitrification for representative sets of parameters.

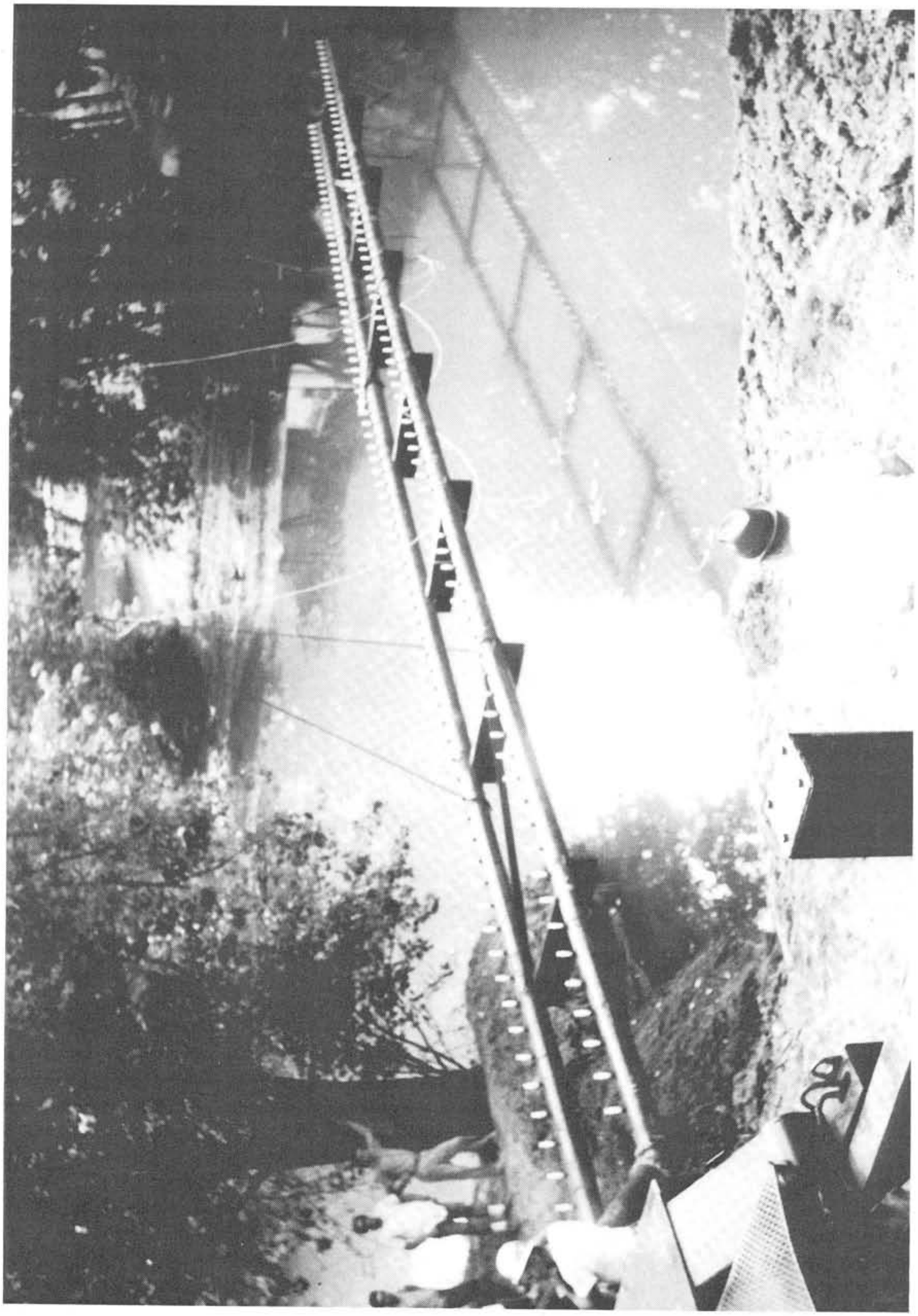

Figure 11. -- Diffuser aerator ready for installation.

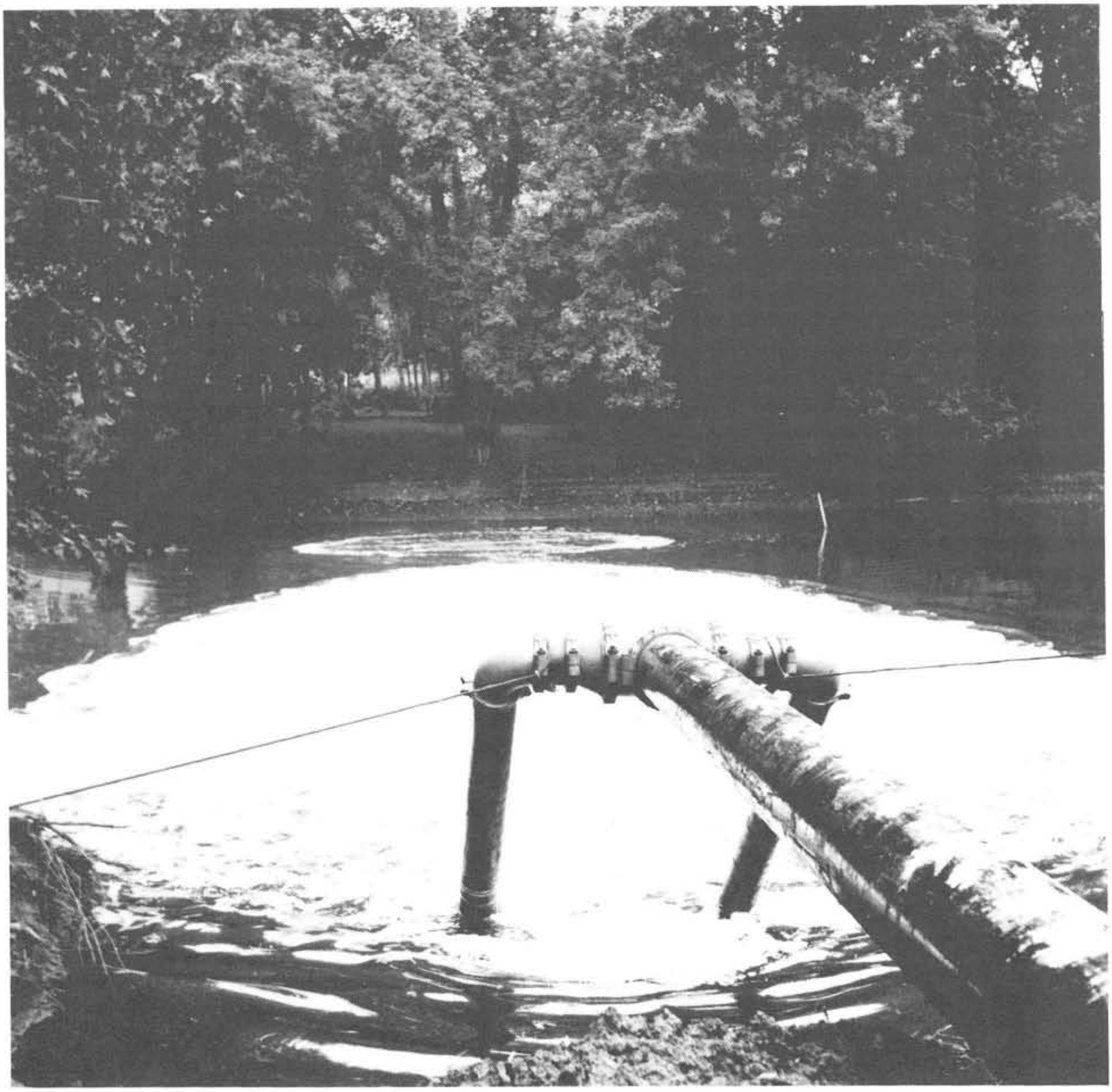

Figure 12. -- Diffuser aerator starting operation.

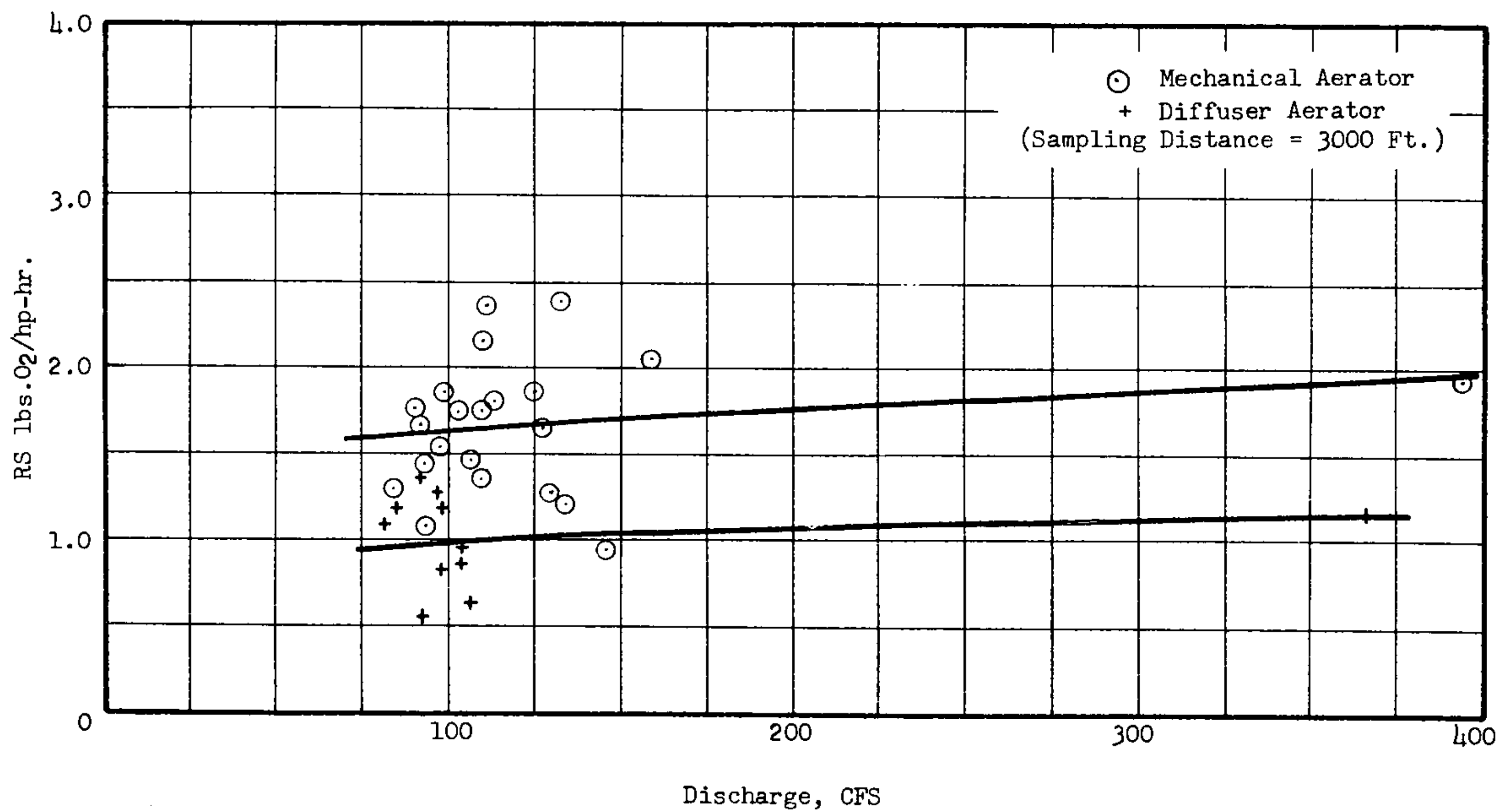

Figure 13. -- Oxygen transfer rate versus flow mechanical and diffuser aerators.

MATHEMATICAL PROGRAMMING, COMPUTERS AND LARGE SCALE WATER RESOURCE SYSTEMS

by

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1. INTRODUCTION

1.1 THE NATURE OF WATER RESOURCE SYSTEMS CONSIDERED

Almost by definition, the concept of a system is subjective depending heavily on the owner of the problem and the resources immediately available for solving the problem. What constitutes a water resource system is even so, more varied since problems requiring different disciplines and people of different backgrounds are usually involved. The variety in concept and content can be illustrated in terms of approaches taken by water resource problem owners living in different civilizations, countries, towns, communities, or geographical belts. The water resource problem as envisaged by people living in humid areas, for example, would be markedly different from that considered by people in arid lands. Even within the context of a given community at a given point in time, a water resource problem represents different things to the engineer, economist, lawyer, politician and to the average citizen. The fact is that the water resource problem is a classic example of an interdisciplinary problem embracing various, and sometimes, different and conflicting viewpoints. An efficient Water Resource Systems project therefore, should consider the physical, economic, political, legal, sociological, geological, agricultural as well as the biological components giving due weight to each one of them. It is not a trivial matter to attempt to bring together professionals in the above disciplines in a joint effort to solve a problem. But so is the pill of any interdisciplinary approach.

It is not the purpose of this effort to consider a complete system that treats all the problems that could foreseeably arise from the inclusion of the details of the components of a water resource system. Such problems are simply too "big" and often unmanageable. While our concern is with large scale water resource systems, as in any systems approach, we will illustrate the power of our tools by considering only components of the total system. That is, we might consider the problem of reservoir design, operation and management as a water resource system problem. We might consider a system that treats only the problems of air and water pollution in a basin. The system of interest may be that of designing an optimal regional water plan. Each of these problems by itself, possesses features that make it quite a formidable task to analyze. Each is a large scale problem. Because of its role in the discussions to follow, it is important to begin with a definition of the concept of a large scale system, as used in this text.

2.1 CHARACTERISTICS OF A COMPLEX LARGE-SCALE SYSTEM

A system can be defined as a set of entities together with their attributes and the relationships between the entities and attributes. A system might be composed, for example, of a reservoir and its operation. Sample attributes of the dam might be the height, active storage, dead storage etc. Sample attribute of the power plant might be its generating capacity. Sample attributes of the operation might be the release rules of the operator. The relationships involved in the system might be the equation that relates the profit from this reservoir operation to the type of reservoir (storage capacity), pumping capacity and the release rules used by the operator over a given planning horizon.

Whether a design or control problem can be considered large or not is rather subjective. However, a problem is generally considered "large scale" or "complex" if the analytical, computational, economical or any combination of factors involved constitute a considerable drain on the resources of the problem owner. This definition is obviously too general and will have to be reduced to a working one in a particular framework. In systems engineering, the following set of guidelines will usually help in classifying a problem as large scale or not.

i) Are the relationships among the variables of the problem, whether explicitly or implicitly defined, such that none of the known analytical or computational approaches can be applied to obtain a solution to the problem?

ii) Is the number of state and policy (design) variables such that the necessary manipulation outstrips the memory and storage capabilities of available computer facilities or the computing time allotted to the problem?

iii) Will the designer or analyst run into constraints in time and resource in an attempt to include essential uncertainties of the variables and attributes of the system in his problem formulation?

Most water resource problems, of multi-purpose and regional nature, are by definition large scale and complex; as a result, they cannot be optimally analyzed or designed by merely using simple, analytical approaches. The tools of systems engineering offer the best available solution methods for handling these problems efficiently.

2.2 SOLUTION AND DESIGN APPROACHES TO COMPLEX, LARGE SCALE SYSTEMS

To obtain solutions to problems posed in large scale water resource systems, the analyst or system designer would consider one or a combination of the following approaches of systems engineering and operations research:

a) Approximation: This is the traditional or classical approach. One begins by approximating the elements which enter specification of a given problem in such a way that it would be possible to employ available analytical and computational tools to solve the problem.

b) Abstraction: This is sometimes referred to as the "feature extraction" approach. Using this method, certain dominant, "key" features of a problem are selected, modelled and solved. In this way, the solution to the original problem is obtained by solving a family of imbedded problems. Several examples of this approach can be found in control problems of dynamic systems, pattern recognition, radiative transfers, and various problems of organizations with centralized functions. This concept and the one to be discussed in the next section are at the very pith of the theory and applications of a tool of analysis known as dynamic programming. Note that using the abstraction approach, the simplification is achieved by the suppression of certain details considered unimportant whereas in the approximation method, this simplification is accomplished by the omission of some parts of the model (variable).

c) The Hierarchical or Multi-level Approach: This approach, pioneered and vigorously explored by a group of scientists at the Systems Research

Center, at Case Western Reserve University, exploits significant concepts of hierarchical organizational structures. It basically involves decomposing an original problem into a family of interrelated sub-problems to be solved separately and then coordinating the problem solving processes so that a solution to the original problem is obtained. The main aspects of this technique, decomposition and coordination, can be performed either in space or in time. For storage and computational purposes, this approach would proceed by decomposing the given problem into sub-problems, then solving sequentially in time and finally achieving coordination via an iterative procedure. In control or operation of complex systems, the operation of each of the sub-system is assumed to take place at the same time and so coordination is best achieved using "on line" intervention procedures. The hierarchical nature of this method is best illustrated using Figure 1. From this figure, the similarities with the philosophy of a typical organizational structure are quite manifest.

Each of the foregoing approaches involves some form of modelling of the system or subsystems. Models usually used are iconic, mathematical and analogue. Mathematical programming and computers (digital, analog and hybrid) play an important role in bringing about the design and solution of models. The next sections will, therefore, discuss these topics and their functions in the design, operation and management of complex water resource systems.

3. MATHEMATICAL PROGRAMMING

The body of techniques developed to solve the problems of optimizing some performance functions within the context of a set of constraining functions belongs to the broad area known as optimization. Mathematical programming, quite interwoven with optimization, originally concerned itself with the problem of allocating scarce resources of a given system in such a way as to optimize a stated objective while meeting all the constraints imposed on the system. In this regard, mathematical programming was most extensively applied to allocation and control problems. It has since been successfully extended to the basic problem areas of design. In the design context, mathematical programming includes those techniques that generate alternate values for the design variables in a constrained domain while at the same time ensuring convergence to an optimal solution. Certain constrained optimization problems can be reduced to unconstrained problems or a sequence of such problems. Thus it is instructive to also study methods of dealing with unconstrained programming problems.

Mathematical programming has been widely used in such fields as operations research, engineering, biomathematics, economics and more recently social and educational systems. Its great potential in water resource systems studies cannot be over-emphasized.

3.1 A MATHEMATICAL STATEMENT OF THE GENERAL MATHEMATICAL PROGRAMMING PROBLEM

The usual statement of the general mathematical program involves the determination of the values of a set of variables, x_j which maximize or minimize an objective function, z,

$$z = f(x_j) \quad , \; j = 1, 2, \ldots, n \qquad (1)$$

subject to a constraining set of inequations or equations

$$g_i(x_j) \left\{ \begin{array}{c} \geq \\ = \\ \leq \end{array} \right\} b_i \ , \ i = 1, 2, \ldots, m \qquad (2)$$

In any given problem, the nature of the functions $f(\cdot)$ and $g_i(\cdot)$ will be known or assumed. In equation (2), the b_i-s are usually known constraints and for each i, only one of the three signs $\{\geq, + , \leq\}$ holds. We shall see how different types of mathematical programming problems result according as the structure of (1) and (2) is varied.

3.2 CLASSIFICATION AND TYPES OF MATHEMATICAL PROGRAMMING

To classify programming problems, we have a choice of approaches. A problem may be thought of as either discrete or continuous, depending on whether the decision, state or stage variables are allowed to take on values in quanta or over all the continuum of space. A problem, whether discrete or continuous, can be thought of as a single stage, one shot problem or as consisting of more than one stage; that is, multi-stage or sequential. Sometimes, single staged and multi-staged problems are used synonymously with dynamic and static (non-dynamic) problems respectively. This synonymity is, in general, not true.

Any of the foregoing problems can be analyzed according as whether it is deterministic or probabilistic implying some uncertainty with regards to outcome of a given action. Deterministic problems involve situations where the result of a given action is known with certainty. Such problems are further studied in the context of linear and nonlinear programs. In a linear program, both the objective function and side constraints are linear in form. If any of or both equations (1) and (2) are nonlinear, a nonlinear programming problem results. Studies in linear and nonlinear programming problems have led to the isolation of certain types of problems with a definite structure to them. This isolation approach makes it easier to exploit certain theorems and algorithms in handling problems with particular structures. In linear programming for example, it is customary to talk of the generalized linear program with a general structure and linear programs with special structures.

Nonlinear programming belongs to the domain of mathematical programming that still has several uncharted jungles in it. Here again we distinguish two types: convex and non convex programs. Exploiting the well studied convexity properties of functions we are able to make definitive studies of the convex programming problem. Studies of non convex programming problems are still in their infancy.

3.3 THE LINEAR PROGRAMMING PROBLEM

3.3.1 AN ALGEBRAIC STATEMENT

An algebraic statement of the linear programming problem is that of finding a set of N decision variables x_j which minimizes a linear objective function

$$z = \sum_{j=1}^{N} c_j \, x_j \tag{3}$$

subject to a set of m linearly independent linear constraints

$$\sum_{j=1}^{N} a_{ij} \, x_j = b_i \qquad i = 1, 2, \ldots, M \qquad j = 1, 2, \ldots, N \tag{4}$$

$$x_j \geq 0 \tag{5}$$

Several variations of this formulation exist. These can be reformulated in the foregoing format, however. Some of these variations and methods for handling them are briefly recounted below.

a) The problem of interest might be that of maximizing z instead of minimizing it as posed in (3). It has been shown that any minimization problem can be easily transformed into a maximization problem, and vice versa.

Thus

$$\min z = -\max (-z) \tag{6}$$

If for example, it is sought to minimize $z = x_1 + 5x_2 + 3x_3 - 9x_4$, the corresponding maximization problem would be to max $z' = -x_1 -5x_2 - 3x_3 + 9x_4$.

b) If the constraints were inequations instead of equations, we can easily convert them to the standard form by adding non-negative <u>slack variables</u> to the smaller side (for less than situations) and by subtracting non-negative <u>surplus variables</u> from the larger side (for greater than conditions).

Thus given the constraints

$$\sum_{j=1}^{N} a_{ij} \, x_j \leq b_i \, , \; i = 1, 2, \ldots, N \tag{7}$$

by adding non-negative slacks x_{N+i} we obtain

$$\sum_{j=1}^{N} a_{ij} \, x_j + x_{N+i} = b_i \, , \; i = 1, 2, \ldots, R \tag{8}$$

and given

$$\sum_{j=1}^{N} a_{ij}\, x_j \geq b_i \;, \; i = N + 1, \ldots, S \tag{9}$$

by subtracting non-negative surpluses x_{N+i} we obtain

$$\sum_{j=1}^{N} a_{ij}\, x_j - x_{N+i} = b_i \;, \; i = N + 1, \ldots, S \tag{10}$$

c) Finally, if some or all of the decision variables are not constrained to be non-negative, we use the mathematical artifice of expressing any number as the difference of two non-negative numbers. Thus

$$x_j = x_j^{+} - x_j^{-} \tag{11}$$

where

$$x_j^{+} \geq 0 \;, \; x_j^{-} \geq 0 \tag{12}$$

3.4.1 BUILDING A LINEAR PROGRAMMING MODEL OF A SYSTEM: WHEN CAN LINEAR PROGRAMMING BE USED?

To decide whether a problem is at all amenable to formulation as a linear programming one requires the verification that the following conditions hold.

a) The proportionality assumption: This requirement implies that the measure of performance and resource usage must be proportional to the activity level. For example, if the activity level were to be increased three fold, the corresponding flows for the unit activity level will have to be increased three fold too.

b) Non-Negativity. Negative solutions are not accepted, that is, only positive quantities or their multiples are of interest.

c) Additivity. This characterization implies a material balance equation with the various terms in it representing inflows and outflows of the activities. In effect, no interactions between the contributions made to the objective functions by the individual activity levels are permitted. In equation form, we have objective function, $z = \sum_{j} c_j\, x_j$, $j = 1, 2, \ldots, N$.

d) Linearity of Objective Function and Constraints.

3.4.2 ASSEMBLING THE MODEL

Having satisfied ourselves that a problem is a Linear Programming problem, how do we put together the problem in a linear programming form? We outline below the following skeletal procedures:

a) Definition of Activity Set - decomposition of problem into elementary levels and units.

b) Determination of Item Set - classification of objects as inputs and outputs.

c) Determination of Input - Output Coefficients - factors of proportionality between activity levels and item flows.

d) Determination of Exogenous Flows:

e) Material Balance Equations:

3.5 LINEAR PROGRAMMING IN WATER RESOURCE SYSTEMS

The complexity and high cost of water resource development projects in recent times have necessitated the invocation of programming methods long proven and applied in industry, military and economic worlds. Reliance on traditional methods which mostly emphasized the "art" became supplanted by quantitative methods involving the "science" of planning. Obviously a happy wedding of both the "art" and "science" of planning, or what is known as systems analysis, is called for. Because of its simplicity, availability and widespread publicity, linear programming has naturally attracted water resource systems scientists. It is instructive to examine the areas of the large domain of water resource systems that analysts have tried to apply linear programming thus far. Based on our expository discussion on linear programming and its extensions, we will better evaluate these attempts to transfer "technology".

Examination of the literature in water resource system planning, design and management shows the bulk of the application of Linear Programming has been in the area of water quality systems management.

3.5.1 LINEAR PROGRAMMING AND WATER QUALITY SYSTEMS

The problem of improving the quality of streams, estuaries or the river basin is a "systems problem" since the pollution of such water resources is not measurably attributable to one user. Several aspects of management problem abound. Some of these are that of maintaining levels of concentration of oxygen dissolved in the water, the concentration of dissolved chlorides, and the turbidity. For some reasons, analysts have used the criterion of maintaining a certain level of dissolved oxygen, (DO), in water. A problem of interest here is the selection from an array of possibilities, those levels of treatment plant efficiency to be achieved by each polluter on the stream that will achieve the desired oxygen levels at minimum cost. Several analysts [Deininger, 1965; Liebman, 1965; Sobel, 1965; Kerri, 1966; ReVelle et. al., 1966; ReVelle et. al., 1968; Graves et. al., 1969] have tried to apply some form of linear programming to various models of this basic problem.

3.5.2 SOBEL'S LINEAR PROGRAMMING MODEL

The following problem formulation was due to Sobel. Consider a water resource system consisting of m interconnected homogeneous segments. Let $\vec{c}$ = $\{c_i\}$ vector of DO improvement sought in segment

$i = 1, 2, \ldots, m$

$x = \{x_j\}$ = vector of decrease at input j of the rate at which its effluent ultimately depletes oxygen from the system, $j = 1, 2, \ldots, n$

$A = [a_{ij}]$ = an m x n matrix of coefficients showing the DO improvement in segment i per unit of x_j ($a_{ij} > 0$)

$\vec{b} = \{b_i\}$ = an m vector of target improvements attained through any one of a convex set of alternative changes to the inputs to the water resource.

$\vec{U} = \{u_j\}$ = an n vector upper bound denoting the present rate at which the j^{th} effluent ultimately depletes oxygen from the system

$\vec{c} = \{c_j\}$ = an n row vector with c_j showing unit cost of x_j

The following linear programming formulation results

$$\text{minimize} \quad \vec{c}\,\vec{x} \tag{13}$$

subject to

$$A\vec{x} \geq \vec{b} \tag{14}$$
$$\vec{x} \leq \vec{u} \tag{15}$$
$$\vec{x} \geq \vec{0} \tag{16}$$

Restriction (14) ensures that the vector of DO changes (improvements) as a result of sediment decrease vector $\vec{x}$, is at least as much as $\vec{b}$, constrained to be non-negative. Restriction (15) puts an upper bound on the rate of oxygen depletion by each effluent. Some of these restrictions can be relaxed depending on the problem environment and characteristics. The interesting point is that these problems can be formulated and in fact solved to completion using Linear Programming. Note that the problem posed in (13) - (16) is better solved by solving the dual. In fact, when the restrictions, $\vec{c} \geq 0$, $\vec{x} \geq 0$ are relaxed, for example, $\vec{x} \geq \vec{L}$, ($L < 0$) the dual solution is easier and more appealing. The dual variables are indicators of the savings in cost to the system for a unit change in stream standard yield.

Sobel also shows how a <u>mixed integer linear programming</u> formulation can be adapted to determine the classical pollution abatement policies referred to as the <u>uniform treatment policy</u>. If

B_j = rate of oxygen depletion by the influent into the j^{th} waste source, if discharged untreated

$q_j = U_j/B_j$ = proportion of B_j present in the j^{th} effluent

$S_j = U_j - x_j$ = oxygen demand of j^{th} effluent after program element x_j has been instituted

$\theta_j = S_j/B_j$ = proportion of B_j present in the j^{th} effluent after instituting program element x_j

a linear programming formulation minimizing the cost of the constrained improvements subject to the additional constraints requiring all θ_j to be identical is possible. Solution of the dual generates a set of shadow prices - the inputed factor prices arising from the optimal solution. Under certain assumptions, a linear programming formulation which minimizes a benefit-cost ratio objective function is also possible. Some of the assumptions are:

i) that the benefits resulting from a DO improvement = Σ benefits from each individual segment's improvements.

ii) that in each segment, benefits are proportional to the DO improvement.

Stochastic analogs of the foregoing models have been proposed in view of the fact that, in reality, the elements of a DO improvement program functions are stochastic.

3.6 REAL LIFE APPLICATIONS

Real life studies of various forms of linear programming approaches to certain water resource systems have been made. ReVelle et. al. (1968) used linear programming to study a simplified version of the Willamette Rivers in Oregon. The objective was to select the efficiencies of treatment plants on a river at minimum system cost subject to constraints of maintaining dissolved oxygen standards. Using dual variables from the linear programming problem, the effects of changes in the dissolved oxygen standards were investigated.

Recently, Graves et. al. (1969) used linear programming to study the problem of water pollution abatement by by-pass piping. Semi-realistic data from the Delaware Estuary were used to test the efficiency of the model. Perhaps the real contribution in their work, from analysis standpoint, was the use of element generation technique in conjunction with a truncated tableau to provide efficient solutions to the linear programming problem formulated. For such a large scale problem with 99 x 10^3 coefficients (75 x 1320) straight forward use of the linear programming package program would have proven very expensive and inefficient. Again, this illustrates the many modifications in approaches that should always be investigated to reduce computational time and storage requirement.

3.7 LINEAR PROGRAMMING, GROUND-WATER AND SURFACE-WATER RESOURCE SYSTEM MANAGEMENT

Ground water, surface water and their conjunctive operations have been analyzed using different methods. The fact that ground water levels in many basins are falling coupled with increasing attention to conjunctive operation of surface and ground water has necessitated the use of more efficient quantitative approaches. Dracup (1967), considers the problem of determining the optimum quantities of water for use in satisfying agricultural, municipal and industrial demands and for ground water recharge. The sources of water for meeting these demands are local surface water, two imported sources, ground water and waste reclamation. Using the optimum quantities of water thus determined, a systems planner and designer can determine the optimal sizing of the underground and surface water facilities. The information generated can be used in developing operational control procedures for utilizing the system. A parametric linear programming model was set up to investigate the response of the system to changes in the objective function cost coefficients - the $\vec{c}$ and the right hand side coefficients, $\vec{b}$, representing the annual change in storage in the ground water basin. Optimal decision rules can be formulated for various situations.

3.8 LINEAR PROGRAMMING, DEVELOPMENT PLANNING AND MULTIPLE PURPOSE OPERATIONS

Developmental planning of a complex water resource system involves a varied number of agencies in an attempt to select from available resources those plans that will best meet their needs and objectives for a given planning horizon. The complexity of the problem therefore rules out any consideration of one method of analysis as "the method" to use. Invariably, a deficiency in one of the known techniques makes it impossible to be used at all phases of the plan. What is therefore needed is a methodology for segregating a complex problem into classes of subproblems where a known method can correctly and efficiently be invoked. The limitations of linear programming were outlined earlier in the form of certain requirements that must be met by a problem before it can be justifiably used.

To solve a large problem using linear programming, one can break it up (decompose it) into several simpler linear programming problems with the ground rules for linear programming satisfied in each of the sub problems. In general linear programming, by itself, is of little help in analyzing the planning and operation of a reservoir system consisting of several dams. The hopelessness of trying to use standard linear programming (without decomposition or dynamic programming) was recorded by Parikh (1965). In a latter effort, Parikh (1966) the operation of a four reservoir system was optimized over a 10 year period with deterministic hydrology, to yield maximal total returns from firm and non firm power and water. Flood control was not included, however. The optimal policy for each reservoir (local operators) was found using dynamic programming (Hall et. al. (1968)); the entire system was then optimized by linear programming (master program).

It would have been impossible to solve the entire problem to completion using only linear programming because of non linearities in the objective function introduced by reservoir head variations. Convergence to the optimum using this method is slow and not monotonic but Parikh (1967), using the approximate cutting plane method, shows that one does not have to solve the problem to completion. If the solution is within an ε-neighborhood of the global optimum, the computation can be terminated with little or no loss in optimality. Convergence to a near-optimal solution in a finite number of steps is thus guaranteed. The method is referred to as convex program with incomplete recourse.

This new fact relaxes one of the objections to using linear programming in large scale problems, usually of stochastic type.

3.9 NON-LINEAR PROGRAMMING

The fact that most real life problems are non-linear suggests the development of a special body of techniques for modelling them. The area of mathematical programming with the capability for including the realities of a problem situation in the formulation is known as nonlinear programming (NLP). Explicit nonlinear programming problems abound in economic, business, government, scientific and engineering areas. Although these problems can be formulated as nonlinear programming problems, their solutions are, in general, not easily obtained. A class of nonlinear programming problems that has been studied in detail includes geometric programming, quadratic programming, convex (concave) programming etc. With the help of Kuhn-Tucker and Lagrange Multiplier theory, it is usually possible to identify solutions to these programming problems.

3.9.1 SOLUTION METHODS OF NON-LINEAR PROGRAMMING

Most of the available methods for obtaining computational solutions to a nonlinear programming problem can be grouped under two distinct approaches:

a) Methods of Feasible Directions

and

b) Penalty Function Techniques

a) Methods of Feasible Directions

Computational approaches classed as methods of feasible direction begin the search for the optimal point by choosing an initial starting point, $\vec{x}_o \varepsilon R$ which satisfies the constraints $g_i(\vec{x}) \geq 0$, $i = 1, 2, \ldots, m$. A direction of movement is then sought such that a small move in that direction does not violate any constraint while at the same time, the objective function $f(\vec{x})$, is improved. Such a direction is then said to be both usable and feasible.

For a minimization problem, a usable and feasible direction is a vector $\vec{s}$ with the property that

$$\text{i)} \quad \frac{d}{d\alpha} g_i (\vec{x}_o + \alpha\vec{s}) \Big|_{\alpha = 0} = \nabla g_i^T (\vec{x}_o) \vec{s} \geq 0, \; i \; \varepsilon \; I \tag{17}$$

and

$$\text{ii)} \quad \frac{d}{d\alpha} f(\vec{x} + \alpha\vec{s}) \Big|_{\alpha = 0} = \nabla f^T(\vec{x}_o) \vec{s} < 0 \tag{18}$$

Here the symbol T is used to indicate the transpose operation. One then moves a certain distance α , in this direction, arriving at a new and better point, x_1. The procedure is repeated until a point is obtained which simultaneously does not violate constraints nor yield an improvement in the value of the objective function. Such a point, when found, is usually a constrained local optimum. Global optimal points can, however, be found under certain assumptions (e.g. convex programs). Since there are many ways of choosing the feasible directions described above, several methods of feasible directions exist. Among the better known, for constrained optimization, are Zoutendijk's procedure [34], and the gradient projection method of Rosen [30].

b) Penalty Function Techniques

A programming technique, with great potential in water resource systems but which has not been used so far, belongs to a class known as penalty function techniques. As the name suggests, the method begins by transforming a given constrained optimization problem using auxiliary functions, into a new unconstrained problem with a penalty term to check constraint violation.

To illustrate, given the following nonlinear programming problem

$$\min \quad f(\vec{x}) \quad , \quad \vec{x} \in E^n \tag{19}$$

$$\text{subject to} \quad g_i(\vec{x}) \geq 0 \; , \; i = 1, 2, \ldots, m$$

a new auxiliary unconstrained problem is defined as

$$\min J(\vec{x}, \; \alpha(t)) = \min_{\vec{x}} \quad f(\vec{x}) + \sum_{i=1}^{m} \alpha_i(t) \; G[g_i(\vec{x})],$$

Here, t is a parameter, $\{\alpha_i(t)\}$ is a sequence of weighting factors and $G[g_i(x)]$ is usually a well behaved monotonic function of $g_i(\vec{x})$ especially in the region $g_i(\vec{x}) = 0$. By properly controlling the choice of $\alpha_i(t)$ and the penalty term, a solution $\hat{x}(\alpha(t))$, of this auxiliary problem converges to the solution, $\hat{x}$, of the original constrained optimization problem. The practical advantages of this approach are that constraints need not be dealt with separately and that any of a number of available efficient algorithms for solving unconstrained optimization problems can therefore, be gainfully employed.

The introduction of this philosophy in devising solution algorithms to nonlinear programming problems is credited to Carroll [7]. He utilizes a function of the following type

$$J(\vec{x}, \alpha(t)) = f(\vec{x}) + t \sum_{i=1}^{m} [g_i(\vec{x})]^{-1} \tag{22}$$

which includes a penalty term that enforces realizability, to obtain solutions to moderately sized problems. Note that the Lagrange Multiplier technique is a classical example of this unconstrained auxiliary function approach. It is clear that a variety of algorithms can be developed depending on the nature and severity of penalties attached to constraint violation. We will present a well developed and rigorously proved penalty function type algorithm due to Fiacco and McCormick [10]. To the author's knowledge, despite its shortcomings, it is to date the most efficient in its class and is currently the favorite of scientists and practitioners in the mathematical and computing fields.

3.9.2 THE FIACCO-McCORMICK METHOD (SUMT)

Given a NLP problem of the form

$$\text{minimize} \quad f(\vec{x}) \tag{23}$$

$$\text{subject to } g_i(\vec{x}) \geq 0 \quad i = 1, 2, \ldots, m \tag{24}$$

define a new auxiliary or P function to be minimized as

$$P(\vec{x}, r) = f(\vec{x}) + r \sum_{i=1}^{m} [g_i(\vec{x})]^{-1} \qquad (25)$$

where $r > 0$ is a strictly monotonic decreasing sequence, $\{r_k\}$. Note: The efficacy of this algorithm depends on a) making a suitable choice of the initial value of r and the factor c by which it is reduced after each minimization of the P function, b) availability of an efficient unconstrained minimization technique, and a good convergence criteria. Briefly the algorithm, in the form of a flow chart, is shown in Figure 2.

3.9.3 POTENTIALS OF THE SUMT TO WATER RESOURCE SYSTEMS ANALYSES

Examination of the literature in mathematical programming applications to water resource systems reveals almost a complete absence of use of nonlinear programming methodologies. Variations of linear programming and dynamic programming (to be treated in the next section) dominate. One might ask why such a void exists in the literature. It would seem that one reason is that very few "ready made" algorithms (like the linear programming packages) for handling the type of problems encountered by the water resource systems analyst are in circulation. Another reason would be the fact that nonlinear methods known to the average analyst, like the Lagrange Multiplier technique, are for computational purposes, simply impractical and so unattractive. Loucks [20] in his critique of Meir and Beightler [21] uses the Lagrange Multiplier approach to counter the dynamic programming method suggested by the author. The fact of the matter though, is that for large real life problems, merely using the Lagrange technique is rather computationally frustrating. Loucks, however, rightly suggests that perhaps "our own limitations rather than those of any particular programming method restrict us from examining and including all we would wish to in our analyses". This is precisely the point.

A method with, perhaps, the greatest potential and applicability belongs to the group known as the Penalty Function Techniques discussed earlier. Of these, the SUMT of Fiacco and McCormick, promises to be most useful. Computational experience with this method [10] is quite encouraging. The main difficulty in using this technique is the use of a good minimization technique for the P-function. Several rapidly convergent unconstrained minimization techniques are now available. See for example, Fletcher and Powell, [11], Fletcher and Reeves [12] , Powell [28] and Zangwill [33] The last two describe methods for the minimization of a function of n variables without calculating derivatives. Use of the SUMT to solve nonlinear problems of 100 variables, 20 constraints plus non-negativities has been reported. (Fiacco-McCormick [10]). The SUMT can profitably be used in combination with other simplicial methods to provide efficient algorithms for solving very large-complex problems.

3.10 DYNAMIC PROGRAMMING

For some reasons, discussions of nonlinear programming conventionally exclude dynamic programming. Perhaps, the most obvious reason is that dynamic programming is such a versatile mathematical tool, almost a problem solving philosophy in itself, that including it in the regular discussions of nonlinear programming will tend to obscure its vast utility.

Dynamic programming is a slightly misleading name for the body of mathematical analysis of multi-stage decision processes in sequential mode. The term was originally used by Bellman [1] to emphasize that programming problems in which time and decision sequence played a major role are involved. Subsequent studies have extended its use to design, optimization and other problems not immediately and obviously categorized in the above set.

In the study, modelling, design and optimization of complex sytems, it is usually desirable to make some changes of variables and transformation so that any of the appropriate mathematical techniques can be invoked. This transformation should ideally not change the properties of the problem but merely transform it such that the same optimal solution is obtained regardless of the method of solution adopted. In essence, dynamic programming is such a transformation. It takes a sequential or multi-stage decision problem containing many interdependent variables and converts (decomposes) it into a series of single-stage problems, each containing only a few variables. This transformation is invariant in the sense that a number of feasible solutions and the value of the objective function associated with each feasible solution are preserved. The methodology involved is referred to as <u>Invariant-Imbedding</u>.

Some of the essential features of dynamic programming processes, are now described.

3.10.1 <u>CHARACTERISTICS OF DYNAMIC PROGRAMMING PROCESSES</u>

a) Multi-stage processes are involved. It should be possible to break up (decompose) the overall problem into identifiable <u>stages</u>, with a policy decision required at each stage.

b) Each <u>stage</u> has a number of <u>states</u> associated with it. There could be theoretically an infinite number of states but given the present computational facilities, we require the state of the process to be described only by a small set of parameters.

c) The effect of the <u>policy decision</u> at each stage is to transform the current state into a state associated with the next stage (possibly according to a probability distribution). This transformation of a set of parameters into a similar set is required.

d) The past history of the system is of no importance in determining future actions. Thus given the current state, an optimal policy for the remaining stages is independent of the policy adopted in the previous stages. This property is usually referred to as the "memoryless" or the "forgetfulness" or the <u>Markovian</u> property.

e) The solution procedure begins by finding the optimal policy for <u>each state</u> of the <u>last stage</u>. The solution of this one-stage problem is usually trivial and generally determined by the initial conditions. The solution technique described here is known as the <u>backward-recursion</u> method. Note that a forward formulation exists for certain problem types.

f) A recursive relationship is available which identifies the optimal policy for each state with n stages remaining, given the optimal policy for each state with (n - 1) stages remaining. This is the basic functional equation of dynamic programming and it is a mathematical transliteration of Bellman's Principle of Optimality which we state:

> "An optimal policy has the property that, whatever the initial state and the inital decision, the remaining decision must constitute an optimal policy with regard to the state resulting from the first decision".

g) Using the recursive functional equation which is a derivative of (f), the solution procedure moves backward, stage by stage - each time finding the optimal policy for each state of that stage - until it finds the optimal policy when starting at the initial stage.

3.10.2 FUNCTIONAL EQUATIONS OF DYNAMIC PROGRAMMING

The functional equation which contains a maximum (or minimum or min max) operator is both mathematically and computationally the kernel of any dynamic programming formulation. Various types of these equations arise depending on the structure of the compositor operator ($\oplus$, $\otimes$, etc.) and whether the problem in hand is discrete or continuous, deterministic or stochastic, finite staged or infinite staged, discounted or undiscounted.

A common functional equation is, for a discounted, discrete, deterministic finite horizon problem

$$V_n(i) = \max_{k \varepsilon K} \beta[a_i^{(k)} + V_{n-1}(j)] \tag{26}$$

with the stochastic version of the foregoing being

$$V_n(i) = \max_{k \varepsilon K} [a_i^{(k)} + \beta \sum_{j \varepsilon J} p_{ij}^{(k)} V_{n-1}(j)] , \ i,\ j \varepsilon J \tag{27}$$

$$V_o(i) = a_i \ , \ a_i \text{ a terminal pay off} \tag{28}$$

In the above formulation,

- $V_n(i)$ = total return (expected return for the stochastic case) over n periods, given the present state i, of the system and using an optimal policy
- n, n - 1 are the stage variable indicators
- i is the current state variable
- j is the transformal state determined by the transformation rule
- $P_{ij}^{(k)}$ is the probability of moving from state i currently to state j at the next time period as a result of action k at current stage
- $a_i^{(k)}$ is the single stage return as a result of decision k at state i

β is the discount factor, an economic consideration

3.10.3 COMPUTATIONAL ALGORITHMS FOR DYNAMIC PROGRAMMING

In its usual form, dynamic programming models are formulated for solution using the backward recursion approach. For initial value problems, however, especially of deterministic nature, where the terminal state and or stage is free, forward dynamic programming formulations are preferable. Solution methods available to date for solving dynamic programming can be classed as one of the following:

a) Direct transformation approach, (Bellman [1])

b) Method of Successive Approximation, (Bellman [1])

c) Approximation in Policy Space, (Howard [17])

d) State Increment Dynamic Programming, (Larson [19])

e) Quasilinearization, (Bellman and Kalaba [3])

f) Iterations about a nominal trajectory by increasingly refining the grid sizes (Nemhauser [23])

g) State Variable Reduction via the Lagrange Multipliers (Bellman and Dreyfus [2])

h) Polynomial Approximation of Cost Function (Bellman [1])

i) Heuristic Procedures (Norman [24])

Which of these computational approaches is used depends on the structure of the given problem. Most of them seek to minimize the effect known as the "curse of dimensionality" usually introduced by the "largeness" of the state vector. In dynamic programming, there is almost a complete absence of general-purpose computer programs like those available in linear programming. This, however, is not a major short-coming since once the functional equation has been derived, programming is a relatively simple matter.

3.10.4 DYNAMIC PROGRAMMING AND WATER RESOURCE SYSTEMS

The application of optimization techniques to the analysis, design and management of water resource systems is comparatively recent. Of these methods, dynamic programming - a comparatively late comer, has fast gained popularity and acceptance by analysts. This claim is supported by the rising number of papers describing application of the technique to various aspects of the gamut of problems arising in the water resource systems field. Even more indicative of this trend is the fact that problems originally treated by simulation, then by linear programming, are now being formulated and solved using dynamic programming.

Most of the problems described in the literature are concerned with the following:

a) Reservoir design and optimum operation for short periods such as 24 hours (deterministic systems) and monthly or yearly operation with system parameters treated as stochastic variables (Keckler and Larson [18]).

b) Optimum operations for long range planning of complex systems (Hall et. al. [14]).

c) Optimization of multi-purpose reservoir systems on separate streams (Hall, Butcher, Esogbue [15]).

d) Optimal control of linked reservoir and branching multi-stage water resource systems (Schweig and Cole [31], Meir and Breightler [21]).

e) Water Use Subsystems - irrigation problems etc., (Hall et. al.).

f) Conjunctive Use of Water and Surface Water Systems, (Buras [4], Burt [5].).

The attraction of dynamic programming to analysts working on the above set of problems rests mainly on the capability of the method for handling nonlinear, non-convex, discontinuous objective and constraint functions. Increase in the number of constraint equations usually reduces rather than increases computational difficulties. It is about the only available technique that explicitly treats stochastic effects in the system. Dendritic branching systems, such as are usually found in water resources development, are easily handled using decomposition and the functional equation.

Of the various problems considered, development of optimal release rules for water and energy supply reservoir systems (with deterministic and stochastic inputs) has, perhaps, received the widest attention. For a detailed application of dynamic programming to the optimization of reservoir operation, see Hall, Butcher, and Esogbue [15] . Assuming that an optimal release has been made by the supplying reservoirs, an interesting problem is that of optimal use of this water for different purposes. Consider an agricultural example where a net impact of supplied water depends on a judicious decision on the quantities and timing of irrigation water. Hall and Butcher [16] considering a hypothetical problem of developing an optimal strategy for irrigation in order to maximize the yield from crops over a given period, develop the following functional equation of dynamic programming:

$$V_n (Q, W, A) = \max [a_n(d_n) \cdot V_{n-1} (Q - q_n, W + q_1 - E_{a1}, A)]$$

$$0 \leq q_n \leq Q$$

$$0 \leq q_n \leq W_f - W = E_{an}$$

where

$W_f = W_{n-1} + x_i \; \phi_i(x_i)$ and $\phi_i(x_i)$ = irrigation efficiency due to addition of water quantity x_i at period i

$a_n(d_n)$ = yield coefficient of plants for period n

q_n = quantity of water allocated for irrigation in period n - the decision variable

E_{an} = actual evapotranspiration in period i. It is a function of the climatic factors and the soil moisture level W_i.

$V_n(\dot{Q}, W, A)$ = maximum n period yield using an optimal timing policy when the state variables Q, W, A represent respectively the quantity of water available for irrigation, the soil moisture levels, and yield coefficient over all periods.

The computation problem involved in this three dimensional Dynamic Programming optimal timing problem can be greatly simplified by explicitly writing out the constraints on the decision and state variables. This sort of procedure and anlaysis can be extended for the optimal timing of irrigation under condition of deficient supply. In this case, it is desirable to uniformly irrigate a given unit area. Other problems of this general nature amenable to this sort of analyis concern the optimal seasonal allocation of an aqueduct system with fixed capacity, optimal allocation of a constant-rate water supply to a geographic district, water conveyance and distribution systems. Another important area is that of conjunctive operation and management of ground water and surface water systems. Several optimization techniques have been proposed but Buras [4] uses dynamic programming procedures to analyze a hypothetical ground and surface water reservoir system with two areas to be irrigated. Two benefit functions for these areas are established under certain assumptions with the complete solution to the problem involving a) determination of the optimum dam size and the recharge facilities in terms of maximum benefits b) finding the optimum quantity of water for the irrigated areas, and c) establishment of an optimal operating policy which specifies the drafts from the reservoir and the pumpage from the aquifer. Dynamic programming has also been used by Burt ([5]) to derive optimal decision rules for resources fixed in supply or only partially renewable in time. An extension of this model to cover temporal allocation of ground water using Dynamic Programming has also been made [6].

3.11 COMPUTERS

The advent of computers is known to have revolutionized science and human understanding of many intricate complex problems of life and outer life. So versatile is this intelligent machine that it has permeated various aspect of modern man's culture. It is difficult to adequately describe computers by their functions alone since they are continually undergoing radical changes both in functions and in types. As a matter of fact, we are going through a second generation of computers.

We distinguish basically three types of computers: digital, analog and hybrid. For the purposes of our discussion, we delineate the following among the varied properties of the computer:

1) it can store and retrieve information

2) it can carry out arithmetic operation on numbers in accordance with stored instructions; it can carry out logical operations according to stored instructions.

Perhaps the most significant aspect of the "wizardry" of the computer is its ability to store and process rapidly sizable amounts of information. The biggest available commercial computers have a storage capacity of between 25 x 10^4 to 10^6 ten digit or twenty digit numbers. This number could be raised to 3 million without too much effort and rapid access time could conceivably go down to 10^{-12} seconds. These points are made here because of their great importance to the size of problems and choice of methods for their solutions.

Digital and analog computers differ mainly in the manner in which the dependent variables are handled within the computer. The hybrid computer represents an effort to combine in one computer system some of the characteristics normally associated with digital computers and analog computers. Some of the characteristics of analog computers include a) treatment of dependent variables within the machine in continuous form, b) high speed or "real time" operation c) ability to perform efficiently such operations as addition, multiplication, integration, generation of nonlinear functions, d) accuracy depending on quality of computer components etc. Digital systems, on the other hand, have the following attributes: a) all data handled in discretized, or quantized form b) accuracy independent of quality of system components c) relatively long solution times usually a function of the complexity of a given problem, d) ability to trade-off accuracy with solutions e) facility of memorizing data indefinitely etc. A variety of hybrid computers exist. Some of these are a) analog computers using digital subunits b) analog computers using digital computers as peripheral equipment c) digital computers using analog subroutines, d) digital computers designed to permit analog-type programming etc. Hybrid computers have a variety of applications, the most widespread area being the simulation of physical systems. They have been used for sampled-data simulation, random processes simulation, optimization of systems, real time auto correlation of EEG recordings etc.

3.11.1 COMPUTERS AND LARGE SCALE WATER RESOURCE SYSTEMS

Early studies in water resource system development relied very heavily on simulation as a tool for optimization. This mode of analysis is almost impossible to perform on a large scale system without the aid of the computer. Thus the computer played a major role in these studies. Such simulation methods, requiring essentially trial and error methods for convergence to solutions, are greatly demanding on the capacities of present day computers.

The introduction of more efficient, programming algorithms such as are found in linear and nonlinear programming, dynamic programming etc., has minimized but has not eradicated exorbitant chewing up of computer memory and time required by simulation approaches. The fact remains that most of the programming techniques are developed with a view to obtaining their computational solutions using the electronic computer. Thus the computer is an indispensable tool in these analyses. A problem of interest is the optimal choice of which computer to use for a given problem.

Most of the studies described in the text that deal with mathematical programming in water resources have utilized the digital computer. This is especially so with regards to problems dealing with optimal design and operation of reservoir systems. For example, a large scale water resources development problem, in excess of 2,400 decision variables, was decomposed and solved using the digital computer. Among the few areas of the vast water resource systems field that have used other types of computers are problems arising in the analysis of ground-water systems. Treating the ground-water problem from the system viewpoint, three problems of the aquifer response system include a) the detection or instrumentation problem (for the input), b) the identification problem involving the determination of aquifer parameters which govern the response of the system, c) the prediction problem.

The identificaton problem has attracted most of the attention of analysts in this area. This usually involves writing and solving a system of differential equations for the behavior of material flow in the media. A mathematical equation describing an asymmetric grid has been solved using the analog, digital and hybrid computers respectively by different analysts.

More realistic water problems of a total system should be considered using hybrid computers. Such computers will appear to have great promise for the analysis of a complex decomposable system. Current efforts have obtained solutions to problems using the digital computer off-line. The need is for the refinement of programs so that on-line computer control procedures can be implemented, for example, in the reservoir management and design field. In short, computer aided design of water resource systems is proposed here.

3.12 A REAL LIFE EXAMPLE

The technique of systems analysis described in the foregoing sections were brought to bear on a recently completed project in water resources systems development in California. The study developed an optimal operating policy which would provide maximum levels of firm power and water deliveries over a critical period of, say, 120 months for a system of a four river-reservoir system plus a regulating reservoir for firm power, firm water, dump power, and dump water. This policy met all contractual requirements and provided for flood control, recreation, fish and wild life enhancement, existing water rights as well as evaporation and other losses indirect in the operation of reservoirs.

This large problem contained at least 2,400 decision variables. However, using decomposition procedures inherent in multi-level concepts, a two level problem involving the reservoir as operators on the first level and a master (2nd level) coordinating their separate activities was created. The master first announces the firm and dump power and water prices he proposes to pay for each of the 120 months of the planning horizon to the operators. With these prices, each of the operator performs a dynamic programming analysis in an attempt to optimize his individual operation subject to his resource and other constraints. The output of this exercise consisting of optimal release rules for firm water, firm power, dump water, is fed back to the master. Using these as inputs to his system, the master performs a linear programming analysis to determine an optimum mix of output power and water for contract sale. The dual variables of this problem generate a new set of prices (shadow prices) which are next fed back to each of the operators.

With the modified price schedule based on the master's analysis, each of the reservoirs can optimize its operations once again reporting its output to the master. His analysis, in turn, provides a new set of prices for a third iteration. The procedure continues in this way until no significant adjustment in prices results, giving both the optimum operating policy for all subsystems and the optimum contract levels for firm water and firm power. The details of these analyses are well documented in Hall et. al. [14] , and in Hall, Butcher and Esogbue [15]. Figure 3 illustrates the above multi-level decomposition concept for the reservoir system. At the first level are the Trinity-Lewiston complex, the Shasta-Kerwick complex, the Oroville-Thermalito complex and the Folsom-Nimbus complex. Each complex is situated on separate streams. An artificial master, on the second level, coordinates these individual operations.

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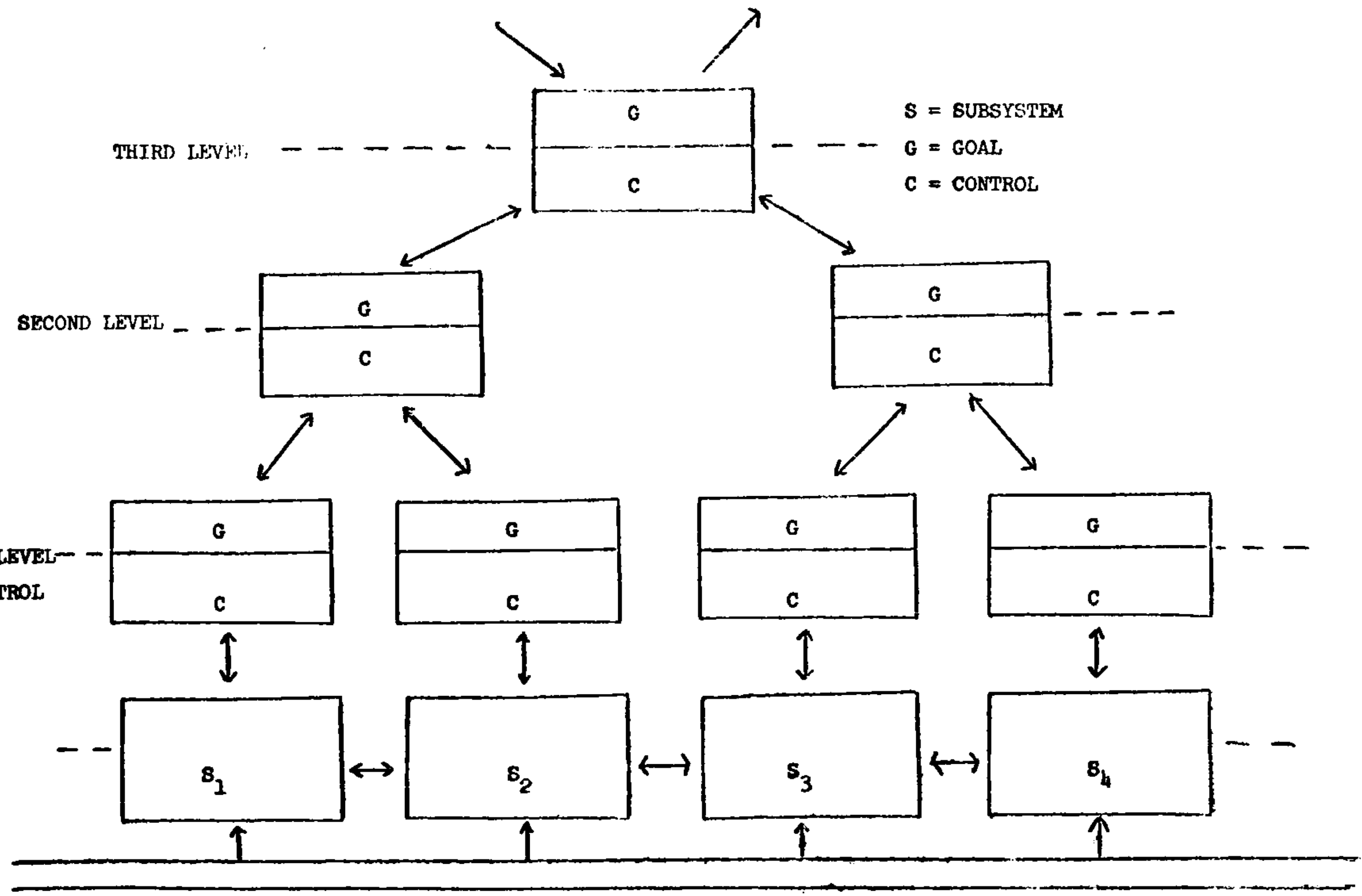

Figure 1. -- Multi-level control structure.

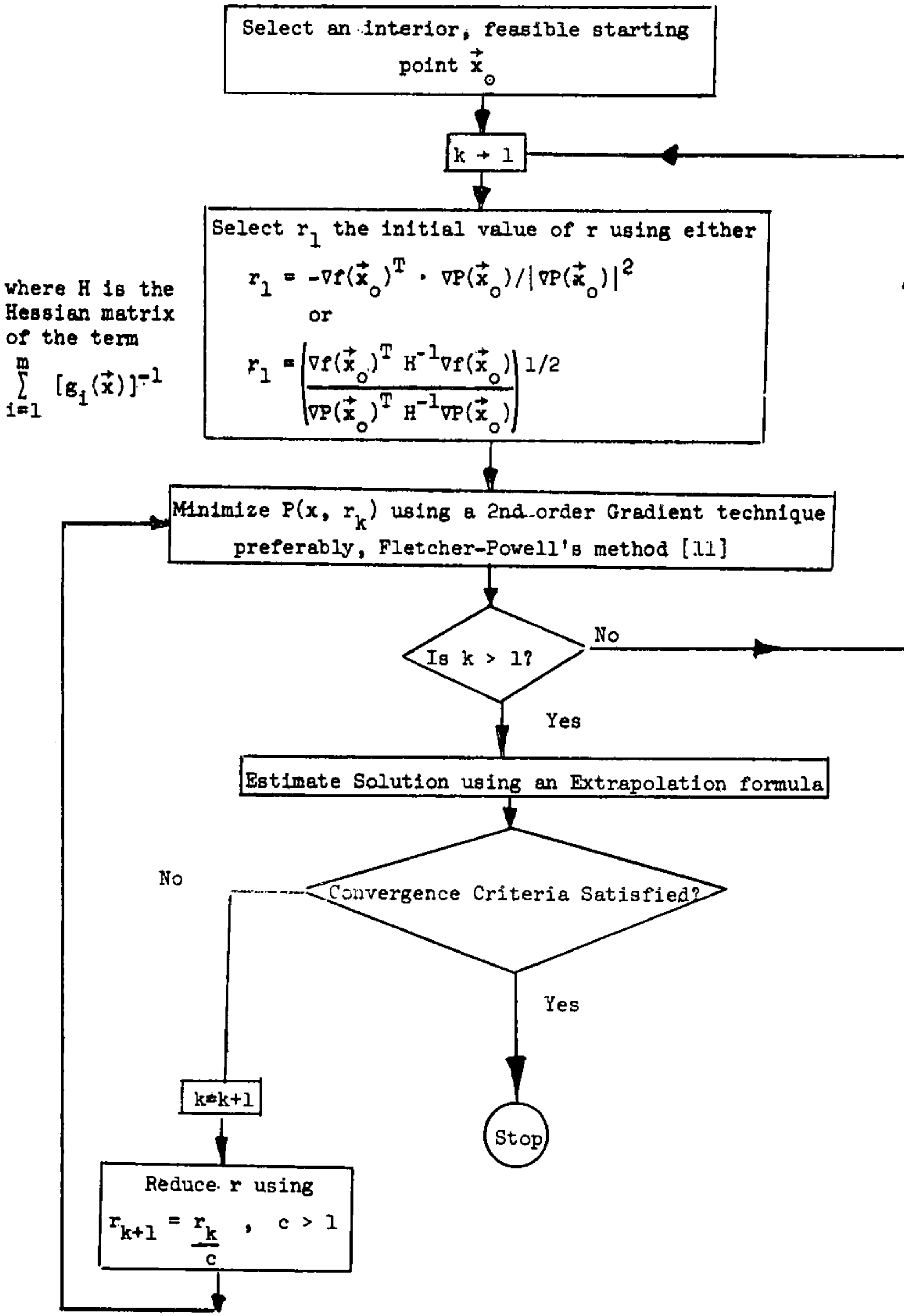

Figure 2. -- Fiacco-McCormick sequential unconstrained minimization technique (SUMT) algorithm.

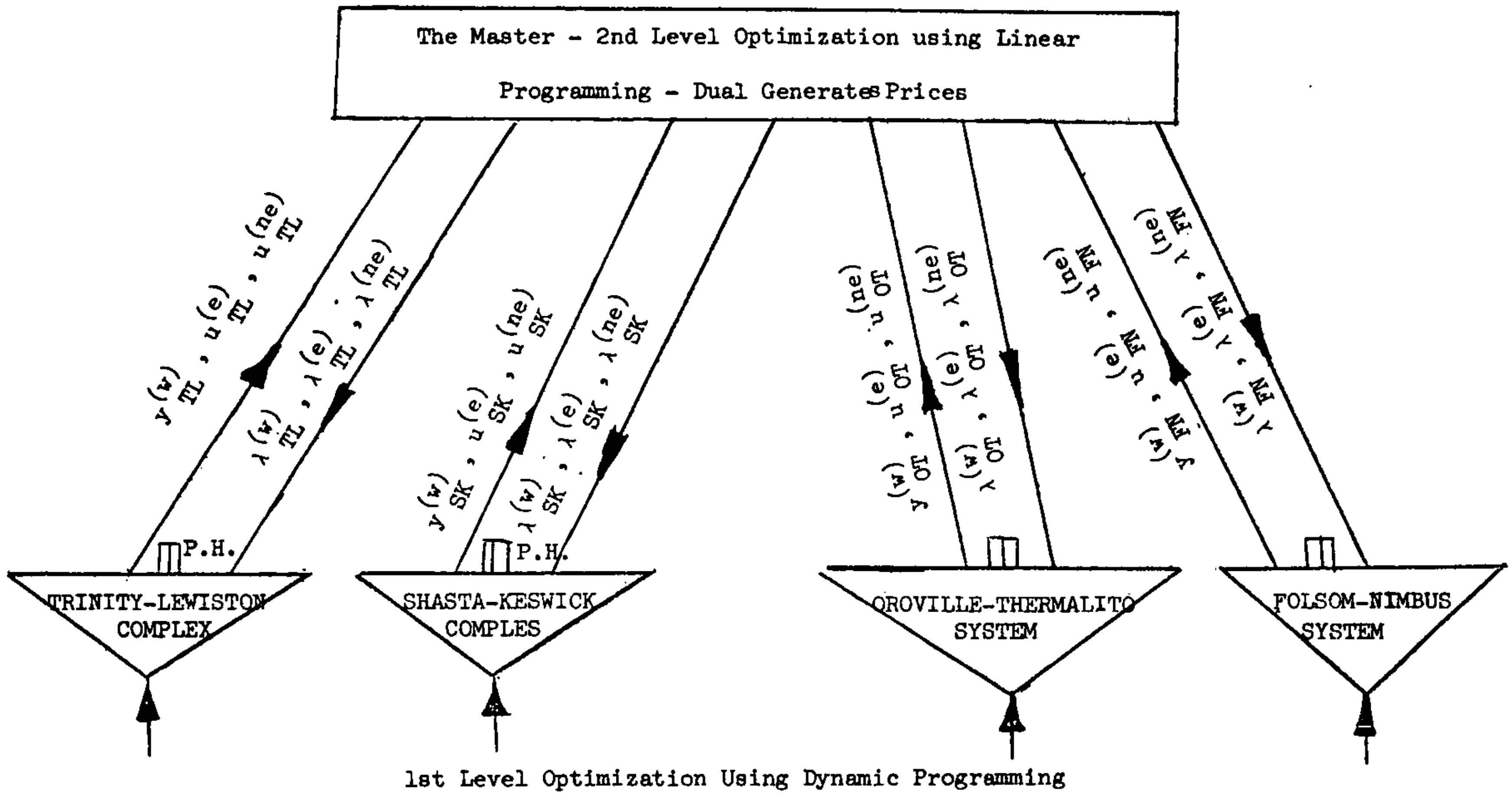

Figure 3. -- Decomposition of the northern California reservoir system into 2 levels for analyses.

SOME EXPERIENCES WITH SYSTEMS ANALYSIS AND THE USE OF MATHEMATICAL MODELS IN RIVER BASIN PLANNING

by

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This paper focuses on several experiences in the World Bank with systems analysis and the use of mathematical models in River Basin Planning. Unlike traditional isolated project analysis, systems analysis, through the use of comprehensive mathematical models, strives to develop an internally consistent scheme of projects simultaneously. Patterns of development are charted on the basis of a system of key economic relations and principles that are found to hold true, as obvious economic relationships within the given area are explored. The system of reactions and principles expressed as a mathematical model is the means by which a multiplicity of data in various combinations can be considered dynamically and simultaneously. The immediate and far-reaching implications of programs and their objectives can be examined, and subsequent alterations tested. The hope of such an approach is twofold: (1) that it will help clarify the objectives and the implications of a development program, and (2) that it will lead to more efficient economic development.

Effective use of this approach to planning was made by the Bank in its Indus Special Study in West Pakistan,1/ in the East Pakistan Water and Power Program, and in the preliminary analysis of the Amplified Basin Plan for the Lower Mekong Basin.2/ Throughout the remainder of this paper, various aspects of systems analysis will be discussed with examples of their practical application drawn from either the Indus or Mekong experience. Though specific mathematical programming techniques are used in developing mathematical models of the system in question, the nature of the analysis is "all-at-onceness": each step of the analysis is so thoroughly interlinked with the next that this introduces an aspect of simultaneity in the planning process.

The first step in understanding the role of the mathematical model is to examine the distinction drawn between the role of the comprehensive study of river basins as such and the part played by systems analysis in such a study. The comprehensive study provides a means of dealing with the overwhelming amount of data inherent in any broad scale effort. Pertinent data are drawn from existing studies, investigations and programs, and synthesized on the basis of obvious economic relationships. This preliminary organization of data helps identify what data are relevant, what data are needed, and what gaps remain to be filled. It also, importantly, helps to raise questions that must be resolved before any substantial project can be evaluated. Mathematical models help in this process. In a model now under study for Mekong, the construction of an economic interrelations model for the Lower Mekong Basin was designed to serve the dual function of organizing data and facilitating communication. Based on available data, for a first effort eight sectors were

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1/ See, Lieftinck, P.; Sadove R.; Creyke, T.; Water and Power Resources of West Pakistan: A Study in Sector Planning (1969). (Cited hereafter as the Indus Study.)

2/ The preliminary analysis is in three unpublished papers by Professor Robert Dorfman: "A Schematic Economic Model for the Lower Mekong Basin"; "Progress with the Economic Interrelations Model"; "Interim Program for Lower Mekong Basin". The sections which follow on the Lower Mekong Basin rely heavily on these papers prepared as part of a summer study for the International Bank for Reconstruction and Development in 1970.

singled out for inclusion in a macroeconomic model, e.g., paddy, rubber, milled rice, fuel, power and water. In collecting data for the model, inconsistencies, unknown derivations and general weaknesses in data were exposed and, as a result, standards for future data acquisition recommended. While the initial computer runs exhibited the need for additional information and confirmed the need for improved existing data, it also showed by general order of magnitude the economic reverberations that would occur from implementation of particular projects in Cambodia. Crude estimates were developed to show the way that various projects would not only affect paddy production tremendously, but would also indirectly stimulate the entire economy of Cambodia, particularly the fuel, water and power sector. Export earnings of milled rice would rise, but the total balance of trade due to the large increase in consumption of imported goods would fall. By exposing the implied results of various tributary irrigation projects on the Cambodian economy as a whole, the model was designed to raise questions about program objectives and to point up areas for more detailed examination.

The fundamental focus of the analysis of interrelated development projects reveals the particular importance of specific objectives and time perspectives. Furthermore, the time perspective itself raises important questions about objectives. For any program to be effective, there will have to be agreement on the consistency between the time framework and the objectives, for example. Objectives that place primary emphasis on the short range must stress quick and visible results. Consider the standard benefit-cost indicators used in project analysis, which are of course, always of substantial economic significance, but cannot tell the economic story in the short run. Rate of return indicators stress the annual flows of project receipts and expenditures, usually in present value terms. However, as Professor Dorfman has expressed in the work of the Summer Task Force, equally relevant indicators would presuppose that the worth of a project in the short run depends on how many families it helps and how significantly it helps them, results that are not fully reflected by a project's contribution to national income in either the short or the long run. In particular the percentage increase in the incomes of families affected by a project will tend to be larger for projects that assist poor localities or poor segments of the population than for equally productive projects whose main impact falls on families or sectors that would be comparatively well off without it. Professor Dorfman has contended that he would attach particular importance to this type of an indicator because it tends to emphasize contributions to the economic welfare of distressed segments of the population. He has stressed that the percentage increase in family income, moreover, is especially attuned to the psychological demands of the problem. Present values and internal rates of return are not very visible to the recipient unless he is well trained in capital accounting. But increases in income are highly visible and strongly motivating psychologically. There is good reason to believe that the amount of stimulus afforded by a change in income is more closely allied to its size in proportion to the pre-existing income than to its absolute magnitude. Both of these features are emphasized in the percentage increase in family income indicator.

Other indicators, such as employment provided by construction, are equally relevant for similar reasons resulting in short-term striking consequences of projects, i.e., via the employment opportunities that they provide. The usual reasoning follows that in normal circumstances, the use of labor is more properly counted as a cost than as a benefit of a project. But when there is substantial unemployment, either chronic or episodic, the use of labor is not an economic cost and the provision of employment is a contribution to welfare.

Professor Dorfman has concluded that having overall different criteria means that the calculation of economic indicators will not provide an unambiguous evaluation of the comparative merits of different projects. Somehow the different criteria must be combined and weighed against each other. This stage of evaluation admits considerable scope for judgment, and this is not feasible without making the objectives themselves clear.

The problems involved in clarifying objectives are not generally thought through. For example, consider the very general objective of a rapid increase in income per head of population. Such an objective may be too general. We have already mentioned maximizing employment, but increasing the output say of foodstuffs or improving the balance of payments position may be equally important, in the short run. Further, maximizing income as an objective is not sufficiently definite or usable without specifying time, space and other dimensions. Is it desired to raise incomes per capita to the highest possible level this year, next year or ten years from now? What is the attitude toward the equitable distribution of income between and within different groups of people? Is it desired to raise incomes most rapidly in the northeastern part of a country, the south or throughout a country? To highlight the issues by exaggeration: Do we say, "Ignore future generations and the countryside, let's concentrate on maximizing income next year in the urban centers that provide politicians most of their electoral support," or do we say, "Ignore this generation, the immediate future and the big cities, let's concentrate everything upon attainment of a target income level ten years hence for all regions of the country"? Clearly, patterns of public expenditure and investment appropriate to each objective differ radically from one another. If time and space perspectives are very short, for instance, as in the first case (and this is not unusual in much of the thinking in the past) we would not be interested in any expenditures that only yielded a return in later years. In fact, this would be the result of the very high discount rate used in our analysis. In such an approach, we would probably not even be interested in planning longer term investment programs.

There's another point, too, that is especially troublesome but intriguing: that is, the high degree of interdependence among investment programs at different times and in different places. Consider the problem of time, for instance: the level of income in one year, its increase over the level attained in previous years and the distribution of that income will have a sizeable effect upon the level of income that may be attained ten years hence because it will affect the level of saving and the pattern of demand. To take an extreme for illustrative purposes, concentration of increases in incomes among higher income groups may mean that most of the added income is spent abroad on imported goods, like cars, that cannot be produced economically at home. But income levels are also interdependent over space. Contact has a great effect upon people. High incomes in one region will tend to pull up incomes in the neighboring region that produces things demanded in the high income center. A region will often desire to imitate its neighbors, whether by its own efforts or with help from outside.

Engineering requirements are hardly more "fixed." As there are many alternative versions to general targets, take any more specific target and I would contend that there will be even more alternatives to it. The difficulty often arises on the project level because the alternatives may not appear clearly from the point of view of the entity carrying out the project. The costs and benefits of a project are not always calculated in such a way as to be comparable with the costs and benefits attributable to other alternative uses of funds available for investment. In the case of public projects the costs and benefits relevant to the decision whether or not to go ahead are usually stated in such a form that they do not clearly indicate the benefit which the project will yield to society as well as to the individual or organization. The prices used in the analysis, especially for important inputs like labor and capital, often do not indicate the cost and benefit of the project to society -- what the economist calls "real" prices. Wages and interest rates for example may need some adjustment.

There is usually a need to delve more deeply into specific alternative projects and programs. In some cases, such alternatives are appraised and evaluated in detail, but in many cases they are treated superficially. Consideration of too many alternatives is, of course, a waste of time and money. The line has to be drawn somewhere. But often it is drawn beneath one single course of action that has been selected as "best."

Similarly, take the technical standards used in designing projects. In numerous instances, they are applied with little change from one country to another. Yet costs of attaining these standards differ considerably from one place to another. Certain extra capital costs are incurred to achieve these standards rather than other lower ones. But for many reasons the extent of these extra costs will vary among countries. Furthermore, the benefits to be derived in the form of reduced risks or lower operating costs will also vary greatly among countries. Cost-benefit ratios are misleading unless they explicitly treat these points. It is poor economics to accept as unavoidable a specific requirement of a system without considering whether another lower or higher standard might not be more appropriate to local cost conditions and time horizons. What is needed is a clear understanding for each standard, of the costs involved in attaining that standard as compared to ones slightly below it and above it. It has been estimated, several years ago, for instance, that to meet fully at all times of the year all the demands placed on one major water syster would involve expansion of storage capacity by 20% -- an enormous investment. No such investment would be required if people could be persuaded to cut down their demand for water by a few percentage points for short periods. This raises the fundamental question of whether consumers would be prepared to pay for the water that they received at these critical periods the full cost of providing that water. If the answer is negative, the fixed "requirements" dissolve. In fact it is not an exaggeration to say that requirements and standards must often be treated flexibly, as variables not as givens.

It is not far from the truth to say that the possible choices in an investment program are often much wider than generally believed. The Bank's work offers numerous illustrations. There have certainly been cases where investments could be altogether avoided by undertaking other less costly courses of development. For example, by using existing facilities more intensively by stimulating off-peak demand, or by avoiding the construction of large-scale infrastructure works in favor of intensifying the use of existing facilities. Capital-intensive water or agricultural projects, readily analyzable but costly, can often be deferred in favor of a crash program to persuade farmers in existing agricultural areas to use improved seeds, better cultivation practices, more fertilizer and more pesticides.

Even when the stage of analyzing specific projects is reached, the broad view must still be kept because it reminds the analyst of effects which each alternative may have on developments over the long run and in other sectors. In economics these effects are called "external" -- to indicate that they do not bring any immediate increase in costs or revenues to the organization undertaking the project, but they do cause increases in costs and returns of other organizations and individuals.

This interdependence among investments -- the way in which one affects the costs or returns on another -- parallels the interdependence among investment programs which I mentioned briefly before. What should be stressed here is that once a specific investment has been selected as the best way to meet some apparent need, it will have a significant effect upon the way in which the pattern of demand subsequently develops. An investment in power, for instance, tends to generate needs for further investments in the same field and in other fields. This is indeed one of the main processes of economic growth. But a careful assessment of the prospective growth has to be made before the initial commitment is made if costly mistakes are to be avoided. Cheap power may be made available but the other disadvantages of the region may be so great as to inhibit the growth of industries which use the power; equally, the prospective growth of demand for power may be so seriously underestimated that within a short time costly additions to the original investment become necessary. In some cases, underestimation of the prospective growth in demand can have even more serious effects, resulting in the adoption of an inappropriate pattern of investment. A conception must be formed of how demand for the services of particular facilities is likely to develop in a region over a long period of time; and in formulating this conception account has to be taken of the effects that the new facilities may themselves have in stimulating greater demand.

The most essential point is that many projects -- especially public works projects -- can often only be adequately assessed as parts of a long-term regional program. You can just go along doing many individual projects, each serving their own purpose, as and when they become obvious. Power is needed and the river is there, so build a dam and install a power house. Later, when other needs appear, raise the dam for greater storage or provide adjuncts to it. But, as often been indicated by my closest colleagues in the World Bank when we discuss large river basin projects, the sum of the results from this type of procedure is usually a good deal less valuable than those obtainable by careful comprehensive analysis with full consideration from the start of the alternative directions possible.

The mathematical model permits us to focus much more clearly, for example on a program's geographical scope by dividing the analysis into a number of development areas. Such a division not only facilitates the accumulation and analysis of data, but enables the planner to respond to the specific as well as the general implications of program development. By relating specific area conditions to broad program objectives, a more realistic analysis of the effect of a development program can be made. This approach to planning entails analyzing the development potentials of each area, as well as the major regional constraints on specific areas. An example of the effect of such detailed and rigorous analysis of area conditions on planning may be illustrated by the Indus Special Study in the case of a public tubewell development for the Dipalpur below BS Link.3/

Such a project for Dipalpur, as well as eleven other public tubewell projects, were selected as priority development projects by the consultants who formulated the first tentative program. The irrigation system was broken up into natural units for analysis in the formulation and selection process, and a comparison of the potential for agricultural production and water resource development was made for each unit. The consultants analyzed specific conditions of each area, and estimated the increase in surface and groundwater (their use is interdependent) for irrigation in each project area by simulating the future likely irrigation system. The relative increase in agricultural production that would be attained by increased irrigation supplies due to installation of each proposed tubewell project was estimated. This increase in production was the criteria used in the project selection process.

A mathematical model related the analysis of development potentials and limitations of each project area more strictly to regional and program concerns. The overriding problem for the Indus program was the regional constraint of limited water resources. This "systems" constraint had to be related to individual project evaluation. Since most public tubewell projects would make use of increased quantities of surface water, it was important to a realistic evaluation of such project that the increase in production due to additional surface supplies be deducted from the increase due to the integrated use of ground and surface water caused by installation of the tubewell project (the consultant's assessment). Computer simulation of surface water deliveries (which included plans for Tarbela dam and reservoir) in each area made it possible to determine the projected use of additional surface water in a particular project area; and to perform the necessary calculations. Largely as a result of this refinement in evaluation, the increase in Net Production Value (present worth at 8%) attributed to Dipalpur by the mathematical program was Rs. 192 million as compared with an assessment of Rs. 982 million, when the calculation is made by more traditional methods.

While the new estimated rate of return on this particular project was still sufficient to support its inclusion in the Indus program, the stricter evaluation of project's benefits could have caused a complete change in the priority of the project. For example, a comparison with continued private tubewell development in the project area indicated that the advantages of public development over private development were only marginal. In the interest of maximizing agricultural production within the Basin as a whole, the inclusion of Dipalpur public tubewell in the Indus program became somewhat tenuous. By analyzing the development potential of each project area through a mathematical model of the system in the light of regional and program

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3/ See, Vol. I. Indus Study, 361-391; Vol. III, Indus Study, 189-193.

concerns, a different conclusion was reached regarding the importance of the Dipalpur project to the Indus program.

What is then the nature of this "different view" resulting from the use of mathematical models? Mathematical modelling of the broader system provides a way for the planner to look at and understand the quantative effect of certain objectives, for example, the careful use of scarce water resources, before selecting actual projects for development. The exercise in model-building (1) requires the planner to focus consciously on objectives in terms of the broad effects of his program (rather than embarking on a broad-scale effort with the hope that objectives will become clearer as the program is executed), (2) allows the planner to think through the immediate and more distant economic effects of his objectives and their possible political and social consequences, and then (3) permits the planner to redefine his objectives by means of the same thorough testing.

The usefulness of this approach to definition is nowhere more evident than in current planning for the Mekong. As noted in our first example, an economic interrelations model of the Lower Mekong Basin is being developed to help the Bank evaluate the Amplified Basin Plan. The model is designed to trace the effects of target output levels anticipated by the Plan, on income levels, consumption levels and output of local commodities and exportable surpluses for each development region. If, as in the case of Cambodia, the effect of one project on exports is unfavorable, the model can also be used to predict output levels required to support a certain minimum level of exports. In any event, the model displays the requirements that program objectives impose throughout the economy, and facilitates evaluation of those objectives.

Most important, the model-building exercise increases the sensitivity of a planner to possible new approaches to programs and to their objectives. Preliminary grasp of the key relationships within a given area, which are revealed by the computer model and supported by a planner's own intuition and experience will, of necessity, provoke rethinking and restructuring of basic objectives. Is, for example, a program designed to increase the rate of change in social conditions a better force for economic development than a program designed to increase national product? The social effects of such a strategy (to achieve the greatest percentage increase in income for the greatest number in relation to foreign exchange costs) in the Mekong are obvious. There would be a distribution of wealth to the areas of the Mekong most impoverished and with highest density in population. The increases in personal income would be highly motivating particularly to those who otherwise would have no reason for commitment. This would enhance political stability, and foster a sense of direction and permanence necessary to long-term development. While systems analysis vastly sharpens a planner's awareness of interrelationships among objectives and even possibilities of objectives, choice of a development strategy cannot be dictated by the mathematical model used to define the system. Choice depends, as always, on the judgment of policy makers.

This type of detailed mathematical analysis gives the planner a tool which permits him to develop a plan which, based on a system of pertinent relationships, is not only consistent within itself, but in the future will be more consistent with its environment. Projects drawn together on this basis, and developed in light of one another will be better designed and more effective as an ongoing system than a number of projects developed in isolation. The program itself will represent a more efficient means of attaining development. As we have seen, mathematical models are ready tools with which to organize and to gain sensitivity to the relative importance of interacting economic, political and social phenomena for a given area and its constituent parts. Given alternative development schemes, linear programming models can reveal simultaneously the effects of the important phenomena (variables and constraints) and determine the best patterns of development. The behavior of projects suggested by these patterns of development can be simulated by computer. Specialized models can themselves be integrated and refined to the extent that the final program becomes fully integrated based on a system of key interrelationships. This final integration allows the effects of any change in the program or system within which it is defined to be fully comprehended. It allows direct reference of specific projects to general objectives, and enhances the opportunity for consistency of environment, strategy, and program.

As noted earlier, systems analysis was used to review and check the internal consistency of the irrigation program proposed in the Indus study. Water development alternatives -- canal improvement, surface water storage, private tubewells for groundwater and public tubewells -- were clear-cut, but their use was interdependent. The problem was to know how the integrated system of irrigation would use limited water resources to best advantage in maximizing the agricultural production potential of any particular area as well as the Basin as whole. Various combinations of water development activities were simulated for each area, and their relative advantages analyzed in terms of estimated irrigation requirements. The combinations had, of necessity, to be considered in light of the possible existence of the large Tarbela dam (due to its overwhelming potential for surface storage) and reservoir proposed for the main stem of the Indus and with reference to suitable time sequence. Project priorities established on the basis of these simulations were reviewed with respect to regional limitations -- foreign exchange, surface water, public development funds, implementation capacity. The linear program revealed simultaneously the effects of alternative levels of these and other regional conditions on investment plans (about 500 projects considered for execution in two different time periods), and facilitated choice of an optimum internally consistent pattern of development.

In summary, the techniques of planning described above are basically a controlled way to order and sharpen a planner's perceptions. The planning task is to understand the nature of the complexities, rather than deny them their place in the planning process. As we have seen, insight into the pertinent interrelationships which define a given area can best be achieved through construction of mathematical models. The models permit the simultaneous as well as the singular effect of particular political, social and economic phenomena to be studied over time, and the models refine the planner's understanding of key interactions and their impact on program development.

In a more specific way, the Appendix outlines the part systems analysis played in the Indus Special Study.

Appendix

THE ROLE OF SYSTEMS ANALYSIS IN THE INDUS RIVER BASIN DEVELOPMENT4/

The Government of Pakistan intended that the World Bank's Study of Water and Power Resources in The Indus Basin provide a basis for development planning for West Pakistan within the context of successive Five Year Plans. The planning task was large and extremely complex: geographically, it covered the Indus River Basin's canal commanded area of 33 million acres; financially, it involved preparation of specific investment plans for water and power development during the period 1965-1975; and economically, it was to guide development of a modern, self-sufficient agricultural economy. Realization of these goals required a highly sophisticated awareness of the exact purposes of the development and the relationships of all investment alternatives. Developing a basis for necessary planning decisions required more than aggregation of enormous amounts of interrelated data; it required so thorough an understanding of the economic system in the Basin at any time that the importance of the total results of any action on, or changes to, the system or any subpart of the system could be realistically comprehended. The basic analytical units of the Study were the natural territorial subdivisions of the Indus River Basin -- the canal command areas. By focusing on these specific areas, planners were required to always relate the costs and benefits from particular projects to the needs and potentials of specific geographical areas of the Basin.

A first task of the Study Group was to analyze the importance of the volume and availability of water as a basic constraint on the Basin and its subdivisions. While annual river flows were substantial, crop yields were limited by the uncertainty and variability of the flows. Eighty percent of annual river flows occurred in the five months, May to September. Water was scarce in the other months because rainfall is short and major groundwater resources were largely untapped. Water was important vis-a-vis other constituents of improved agricultural production, because West Pakistani farmers were familiar with the benefits of its use and there was an apparent correlation between their acceptance of new inputs, such as fertilizer, and the availability of a reliable source of water. Mathematical modeling of the system interactions was used to develop an integrated scheme for the use of available groundwater and surface water in a way which would maximize the development potential of each canal command area and of the Basin as a whole.

Mathematical models were only a part of this analysis, although, I would contend, a most vital part in checking the economic efficiency of the various projects as well as the program as a whole. A computer simulation of canal command operations determined the extent to which irrigation requirements projected for each canal command area would be met by either canal enlargement, private tubewell pumping, public tubewell pumping, or interseasonal surface storage. Computer simulation was also used to analyze potential for the integrated use of groundwater and surface water for each subdivision and

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4/ The appendix has been prepared from the data and evaluation contained in the Indus Study.

the Basin. An important output of the canal command simulation was a series of hypothetical water development "projects" which reflected a particular scheme of water development and indicated the maximum cropping intensity attainable with that type of development. A series of "water projects" were then aggregated and tested within the context of the irrigation system as a whole. As the result of an increasing sensitivity to the implications of alternative water development schemes, criteria were established for making more refined choices among alternative types of water development for each canal command. Through a series of approximations, the proposals for irrigation development were reduced to a single program consistent with implementation capacities, internally consistent in terms of availability and usage of surface water and groundwater and phased in terms of project priorities. Linear programming permitted the Study Group to test and compare the economic efficiency of the project priorities proposed in a Basin-wide context. The linear program's special advantage was its ability for any time period to simultaneously take account of a wide variety of resource constraints -- not only, for example, the availability of surface water, but also the availability of technical expertise to implement public tubewell projects and canal remodelling projects. Furthermore, by varying the levels of resource availabilities, the linear program could test the relative severities of each constraint with respect to investment plans. About 500 "water projects", produced in the canal command simulation, were considered. Alternative "projects" were considered for execution in each of the two time periods, 1965-1975 and 1975-1985, either singly, or where appropriate, in combination with one another.

Tentative program projections for integrated use of water showed the provision of surface storage water to be extremely excessive in relation to other means of irrigation. This seemingly excessive cost was a critical factor in the analysis which justified the construction of Tarbela Dam and Reservoir. The cost of water from Tarbela Reservoir delivered at watercourse would be about Rs. 93 per acre-foot, about three times as much as the cost of water supplied by tubewells. Cost was critical because the existence of Tarbela Dam and Reservoir was fundamental to every other project involved in development of the Indus Basin and to Pakistan's national plans. Tarbela's size and potential dictated its predominance. Its construction cost represented about 17 percent of total public expenditures for water, agriculture and power through 1975. Its 50 mile long reservoir would be able to store nearly 100 square miles of water, sufficient to increase winter flows on the Indus in an average year by about one-third and those in a dry year by about one-half. It was estimated by means of computer simulation that in 1985 Tarbela Reservoir would supply more than 10 percent of total winter supplies of irrigation water. In addition, it was estimated that the power potential of Tarbela could supply more than one-third of West Pakistan's total annual requirements of electric energy for the years 1975-1985. As a result, if the very high cost of Tarbela's stored surface water could not be justified in light of all the demonstrable direct and indirect benefits which would flow from the project, the focal point of the Indus Basin development might have been jeopardized.

These mathematical exercises in fact, permitted just such accurate estimates of the direct and indirect economic benefits of Tarbela. Estimated by conventional means, the cost of stored water was high relative to other means of supplying water. But judged in a Basin-wide context and as part of a plan for the optimum use of water, the long run benefits of Tarbela justified its high costs. Computer simulation had fully tested the capacities of each proposed alternative for the integrated use of groundwater and surface water in each subdivision and in the Basin as a whole. These computer simulations, as we have seen, were related to linear programs which had explored the combinations and permutations of resource availabilities to describe schemes for maximized agricultural production.

The power benefits of Tarbela were a second and very important consideration in the preliminary analysis of the cost of Tarbela's stored water. Power system simulation of Tarbela operation indicated that with an installed capacity of 2100 MW, Tarbela could contribute 10 billion kwh during the period 1975-1985, over a third of West Pakistan's projected requirements for electric power. Tarbela's power benefits could easily be estimated to be about one-quarter of the cost of its total benefits. By attributing three-quarters of the cost of Tarabela to its irrigation function, the average cost of stored water delivered to the watercourse could effectively be reduced to about Rs. 70 per acre-foot, rather than Rs. 93. A further adjustment, made for the addition to useful recharge by conveyance losses on water released from storage at Tarbela, would reduce the cost of stored surface still further to about Rs. 61 per acre-foot.

Tarbela's power benefits could be considered even higher if allowance were made for the saving in natural gas that the hydroelectric plant would make possible. This saving was of special importance because natural gas was in demand not only as a major source of fuel, but as an ingredient of fertilizer. The scarcity value of natural gas, therefore, was dependent in part on the fertilizer requirement of the proposed agricultural program. Calculation of the economic price of natural gas was made on the basis of the foreign exchange costs of the imports which would have to substitute for the natural gas consumed to produce power. Through computer simulation of the operations of Tarbela for each subdivision of the Basin, it was determined that the completion of Tarbela by 1975 would provide a saving of ten percent of West Pakistan's known reserves of natural gas. This saving was a substantial indirect benefit of Tarbela.

Calculations with the model permitted a fuller estimate of the benefits of Tarbela than was the case with conventional project analysis. Quantifiable power benefits, long run benefits and economic benefits stemming from the optimum use of scarce resources, as well as non-quantifiable benefits such as the facility of using stored water for irrigation were all part of the final analysis justifying the construction of Tarbela. Tarbela was studied with respect to both irrigation and power systems, and its impact on each canal command and on the entire Basin was measured systematically.

Development of a scheme for analyzing water and power systems simultaneously and in light of one another, required the most sophosticated programming techniques. For example, the power load forecasts developed by computer simulation were related to agricultural needs (specifically tubewell pumping load, industrial power load and rural electrification) which had been defined for each year by the irrigation study. Another test for systems planning was the development of a reservoir operating policy. Reservoir releases made in early winter for irrigation might help increase crop production, but would decrease hydroelectric capability at a time when power supplies are short. This would create a greater need for a thermal plant fueled by natural gas. As a result, questions about the valuation of gas, as well as the effect of various operating policies on the optimal allocation of water became relevant. Systems analysis techniques provided a bridge between water and power systems, and permitted a schedule for reservoir operations to be devised that was consistent with plans for both.

Unlike the irrigation programs, the long term program for power was not developed on the basis of a fully integrated systems analysis. Rather, the power consultant relied on judicious partial analyses related to his judgment of the overall character of the power system and the relative attractiveness of individual investments. Systems analysis techniques were used to help define the range of alternatives and to test the economic efficiency of the power consultant's proposals. More specifically, and to quantify the results, the Study Group using a computer developed a simulation model of the electric power system of West Pakistan down to, but not including, the distribution stage. By providing a means for comparing the costs and operational characteristics of alternative proposals, this simulation technique played a central role in the formulation of the Study Group's conclusions and recommendations.

By programming comprehensively, the Study Group recognized above all the interdependencies among investments at any given time and cumulatively over a period of years. The completion of Tarbela, for example, was not considered in isolation, but as the major element of a strategy to meet West Pakistan's need for additional supplies of irrigation water and electric power. Systems Analysis thus provided a guide to the timing of these investments and the ultimate formulation of an action program.

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SURFACE TEMPERATURE PROFILE OF NEW YORK STATE WATERS

by

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Background

In 1968, the New York State Atomic and Space Development Authority (ASDA) initiated a series of analyses to determine the effects of thermal discharges on the waters of the State. Early in this study it became apparent that only limited information was available on the thermal characteristics of the water bodies, themselves. Data was spotty, at best, consisting of periodic observations at several water sampling stations around the State and a few intensive local studies at industrial or sewage waste facilities.

About this time, mid-1968, infrared sensing devices previously used for military purposes were declassified by the Department of Defense and were made available for non-military use. This technology, which had been developed as an intelligence-gathering tool in military reconnaisance, appeared to be well suited to assist in the solution of environmental problems related to temperature distributions in water bodies.

In the summer of 1968, ASDA acquired an aerial infrared scanning system, developed by the Aerospace Systems Division of the Bendix Corporation, and mounted it in a World War II TBM Torpedo Bomber, which the State Department of Environmental Conservation uses to make water drops on forest fires. This aircraft was chosen because it could be rapidly adapted for the installation of the infrared sensing equipment and because it is an unusually stable aircraft, thus minimizing data distortion due to aircraft pitch or roll.

Using the Bendix Thermal Mapper, data on surface water (0.1 - 0.02 mm) temperature variations was obtained over large areas and in all seasons to an accuracy of $\pm$ 1^{o} F. This data was then used to study the natural thermal variations of water bodies, to identify existing artificial and natural heat sources, and to provide a means to correlate and verify the analytical models of heat dispersion.

The thermal mapper is an airborne instrument which scans the surface of the ground and water below, senses the temperature-related infrared emissions and translates them into visible light signals which it records on continuously unrolling 70-millimeter photographic film. The result appears to be a light-sensitive "picture" but is, in fact, a recording of temperatures. The warmer areas appear in prints made from this film as areas of light tone, approaching white; the cooler areas are indicated by the darker tones.

For areas of special interest, additional airborne temperature-sensing equipment and high speed magnetic tape data storage equipment were utilized in conjunction with the thermal mapper. Data recorded by this equipment was reduced to graphic form by computer data sorting and visual display techniques at Pacific Northwest Laboratories. The resulting isothermal charts were then used as a supplement to the photographic images of these areas.

A full discussion of the thermal mapper and its operation is contained in the Appendix.

Survey Schedule

After a series of test flights to calibrate the equipment and train the aircraft crew in its use, the thermal survey began on August 21, 1968, with a flight down the Hudson River from Glens Falls to the George Washington Bridge, a distance of approximately 190 miles.

There was an important reason for beginning the survey at this time. During late August and early September, water temperatures reached their yearly peak and water flow is at its annual minimum. For many watercourses, this can be the most critical time of the year with respect to the ecological significance of thermal discharges. Subsequent flights explored the characteristics of the cooling-off period during the late summer and early autumn as well as the warming period of spring and early summer of 1969. A second complete year (1969-1970) of data collection has been completed.

The flight routes and dates of flights accomplished in the survey are tabulated in Table 1.

As can be seen, the areas surveyed include the portion of the St. Lawrence River bounded by New York, the New York shores of Lakes Champlain, Erie and Ontario, all of Lakes Cayuga, Seneca and Chautauqua, portions of the Genesee and Susquehanna Rivers, the Niagara River, the Hudson River below Glens Falls, the Mohawk River east of Utica, the East River, the Harlem River, the shore of Long Island Sound and the Atlantic Ocean within New York State, and New York Harbor. Out-of-state areas of interest have also been covered.

As the flight record indicates, the same areas have been overflown a number of times to obtain data over a broad range of conditions. In estuaries an even more intensive program has been undertaken in order to assess complex tidal influences.

Discussion and Results

A "thermal discharge" may be defined as the flow of warmer water into a cooler body of water. The thermal survey has graphically indicated that thermal discharges have always occurred naturally in our water bodies.

These natural thermal discharges occur because shallow rivers and streams respond more rapidly to solar heating and local climatic conditions than do the larger, deeper water bodies into which they flow. Thus in the spring, tributaries heat more rapidly than the larger bodies of water into which they discharge and constitute a naturally occurring thermal discharge at their mouths. Conversely, in the fall, these same tributaries cool more quickly than the larger body of water. The tributary that was the source of a natural thermal discharge in the spring may discharge water that is several degrees cooler than the receiving water body in the fall. In the winter and summer, both large water body and tributary attain equilibrium with the environment so that there is little temperature difference at their confluence.

Images 1 and 2 present an example of seasonal influences on the relationship between a small stream and the lake into which it flows. Image 1, recorded in May, 1969, shows that the waters of Oak Orchard Creek have experienced a more rapid temperature rise than the waters of Lake Ontario and thus constitute a naturally occurring thermal discharge. The warmer waters of the creek (gray area) enter the lake and tend to move eastward, mixing rapidly with the lake water. Image 2 shows the same area during October, 1969. Little or no temperature difference is detectable as both creek and lake have attained equilibrium.

A striking example of seasonal temperature differences is presented in Image 3 and 4, which show the confluences of the north and south branches of the Ausable River and the bay formed at the mouth of the Little Ausable River with Lake Champlain. Image 3, recorded during a May 1969 flight, shows these tributaries as contributors of heat to the lake. Image 4, recorded in October, 1969, demonstrates that the discharges into Lake Champlain were at that time cooler than the lake's water. The cooler water can be followed only for a short distance into Lake Champlain as compared to the warmer water discharges because the cooler water, which is less buoyant, quickly sinks below the surface.

Naturally occurring temperature gradients in a shallow cove on the east bank of the Hudson River are shown in Image 5, which was recorded in August, 1968. In this area the river widens and a series of shallows and small coves are formed on the eastern shoreline. Water flow in these areas is relatively slow so the sun tends to warm the water to higher temperatures than are found in the main river channel. The water depth of the cove shown in this image is approximately three feet as compared to fifty feet, the depth of the main river channel.

In recent years, man-made thermal discharges have added to the heat loads of these water bodies as well as others throughout the State. In many cases, the scale of such heat additions is much larger than any naturally occurring condition observed by the survey.

The thermal survey provides base line data on the existing temperature patterns of a water body for use in assessing its capability to accept future thermal discharges. Additionally, these data will be utilized to correlate actual diffusion and dispersion patterns with theoretical calculational models in order to improve the capability to predict accurately, in advance, the effects of thermal discharges on the receiving waters.

The following images constitute a representative sampling of the man-made thermal discharges released to the State's waters.

The heated water discharge from the Albany Power Station of the Niagara Mohawk Power Corporation is shown in Image 6 and Figure 1 recorded during June, 1969. The station, which is located on the west bank of the Hudson River, consists of four fossil-fueled units with a total rated capacity of 437,000 kilowatts.

The image of the heated discharge from the plant, as detected by the thermal mapper is shown in Image 6. Figure 1 presents an isothermal chart of this discharge produced by the computer from the magnetic tape record of this flight. The imagery indicates that the discharge is directed downstream by the flow in the river. The contour lines of the isothermal chart represent the changing temperature patterns within the discharge and river. The discharge, through mixing and heat transfer to the atmosphere, dissipates rapidly, so the surface water has essentially returned to its original temperature about a mile downstream.

On the Hudson River south of Peekskill, which is approximately 40 miles north of the Battery, there are two thermal discharges, the 265,000 kilowatt Indian Point Nuclear Power Station of Consolidated Edison Company of New York, Inc., and the Lovett Power Station of Orange and Rockland Utilities, Inc. The Lovett Power Station, a fossil-fueled plant with a rated capacity of 518,200 kilowatts, is on the river's western shore one and a half miles south of Indian Point.

A strong tidal influence is present in this section of the river. The tide and the river current act together to influence the configuration of the heated water discharge, as well as its diffusion and dispersion patterns. In view of the complexity of these patterns, a number of flights have been flown to study, in depth, the influence of tidal variation on these discharges.

An all-day intensive flight series to record tidal variations in this area was undertaken in December, 1968. The aircraft made longitudinal traverses of the river from Stony Point on the south to Dunderberg Mountain on the north at intervals of approximately eight minutes, flying at 2,000 feet. The first flight sequence of the day, from 7:30 a.m. to 9:10 a.m., provided data on dispersion under flood tide conditions.

Images 7, 8, and 9, selected from among those taken during this flight, show the dispersion patterns of the discharges from the Indian Point Nuclear Power Station (upper left) and the Lovett Station (lower right) just before maximum flood tide, at maximum flood tide, and late in the flood tide, respectively. Note how the thermal patterns are altered by changes in tidal conditions -- changing in the course of this period from a strongly up-river driven pattern to one beginning to extend out into the river.

In sequential comparison of these and subsequent images in this flight it is interesting to observe that the ships anchored in the river produced a lighter image with each pass of the aircraft, as their temperature rose due to the warming action of the sun.

From 10:40 a.m. to 12:10 p.m., the aircraft made another series of passes every eight minutes to record surface water temperature distributions during the slack tide. Images 10, 11, and 12, selected from among those taken during this flight, show the thermal patterns in the very late stages of flood tide, at slack tide, and during the first stages of the ebb tide, respectively. The complete reversal of the thermal pattern was accomplished quite quickly.

The next series of passes was flown from 2:06 p.m. to 3:42 p.m. to record the temperature distributions of these discharges during ebb tide. Images 13 and 14 respectively show the dispersion patterns during maximum ebb tide and late ebb tide.

During ebb tide, the tide and the river current combine to direct the thermal discharge downstream. When the tide and the river current are in equilibrium, during slack tide, the thermal discharge moves out perpendicular to the river bank on either side of the point of discharge, dissipating before it reaches midstream. With the onset of the flood tide, the tidal waters begin to move back upstream, against the river current, carrying the thermal discharge with them.

In addition to power plants, there are other industries which release thermal discharges to the State's waters. For example, Image 15 and Figure 2 show the heated water discharge from New York City's Hunts Point Pollution Control Project, a sewage treatment plant on the East River, located north of Riker's Island. Both the image and the isothermal chart were recorded on a flight during April, 1969.

The preceding images and figures illustrate the dissipation patterns of thermal discharges as they are affected by the flow rates and tidal cycles or a combination of both in the rivers on estuarine waters on which they are located. The dissipation patterns of heated discharges in natural lakes are not influenced by these factors but are determined primarily by wind speed and direction, the velocity, volume and direction of the discharge, the configuration of the shoreline and the contours of the lake botton in the area.

An example of the heated discharge from an industrial plant located on a large lake is presented in Image 16. This image of the Bethlehem Steel Corporation mills south of Buffalo on the eastern shore of Lake Erie was recorded in October, 1969. The steel mill uses lake water in its production processes, and releases its heated discharges to Lake Erie via canals and creeks in the area.

Image 17 shows the heated water release of the Georgia Pacific Corporation to the waters of Lake Champlain just north of Plattsburgh. The plant uses lake water in its pulping and papermaking processes and releases the heated water into a boggy delta at Cumberland Bay. The heated discharge forms channels in the delta and then flows into Lake Champlain. This image, recorded in October, 1969, also shows the confluence of the Saranac River and Lake Champlain to the right of the plant. At this time of the year the river's waters are indicated to be cooler than the lake.

To date, over 4,000 feet of film data has been recorded containing a multi-year record of the thermal characteristics of New York State's major water bodies. As this program continues, these data are expected to become increasingly valuable to ecologists and environmentalists studying the temperature characteristics and assessing the temperature sensitivity of the State's waters.

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APPENDIX

The Thermal Mapper

Heat or thermal energy is a form of electromagnetic radiation similar to, but not as energetic as, visible light or X-rays. The frequency and intensity of this thermal radiation vary with the temperature of the radiating object. In fact, when an object becomes hot enough, the emitted electromagnetic radiation is visible. For example, the incandescent light is, in actuality, a tungsten filament that has been heated to a temperature at which it radiates white light.

The temperatures of interest in the thermal survey do not radiate energy in the visible light spectrum but in the less energetic infrared range.

Although air is essentially transparent to visible light, it is opaque to most infrared radiation. However, a significant amount of electromagnetic radiation in the infrared range is transmitted through the atmosphere at three energy bands -- so called "atmospheric windows."

The energy detected in the first window, which passes radiation having a wavelength of less than 2.7 microns, is primarily reflected solar radiation. Thus it does not represent the actual temperature of the radiating object. The two remaining atmospheric windows, which pass radiation with wavelengths between 3 to 5.5 microns and 8 to 14 microns, include portions of the middle and far infrared electomagnetic spectrum in which the emitted radiation from objects corresponds to their absolute temperature. The thermal mapper utilized in the survey detects radiation which passes through the 3 to 5.5 micron window.

Aerial films are not sensitive to infrared radiation, so this radiation cannot be directly recorded by ordinary photography. The thermal mapper transforms the infrared radiation patterns into electrical signals which modulate a light source. Thus, variations in infrared radiation are converted into luminous variations in the visible wavelength region which can then be photographically recorded.

The thermal mapper consists of three basic components:

- A detector that senses the infrared radiation and generates an electrical signal proportional to the radiation's intensity (temperature),

- An amplifier which, quite simply, increases the strength of the electrical signal, and

- An intensity modulated film recorder in which a light beam, having an intensity corresponding to that of the incoming infrared radiation, exposes a continuously unrolling photographic film to record the radiation intensity, thus creating a continuous image of surface temperature variations on the film.

Using a mirror rotating at 6000 rpm and a simple optical system, the thermal mapper rapidly scans the earth along a line perpendicular to the flight path of the aircraft. As the aircraft flies over an area, this repetitive scan covers the strip of land and water beneath the plane.

The film recorder duplicates this process, projecting the modulated light beam at the scanning rate of 6000 lines per minute onto a roll of 70-mm film. The method of image production is similar to that used in a TV set, that is, by rapid scanning of a modulated beam.

The film moves at a rate proportional to aircraft speed and altitude. As the aircraft flies over an area, a complete and continuous image of surface temperature variations is recorded on the film.

Isothermal Charts

For areas of special interest, a Barnes PRT-5 radiometer and an Ampex FR-1300 tape recorder were used in conjunction with the thermal mapper to provide input for a computer program to develop isothermal contours.

The radiometer detects infrared radiation levels in the 8 to 14 micron window along a line parallel to the flight path. It produces an electrical signal, proportional to the input of infrared radiation, that may be directly converted into absolute temperature. This signal and that of the detector in the thermal mapper are simultaneously recorded on magnetic tape. Thus, the absolute temperature calibration, which is derived from the radiometer data, is directly correlated with the relative temperature data recorded by the mapper.

Utilizing a computer program developed by Pacific Northwest Laboratories, a single contour line connecting points having the same surface temperature is produced as a computer-generated display on an oscilloscope. Contour lines for several different temperatures are similarly produced. The final isothermal chart is prepared by superimposing the individual contour lines on a map of the area.

TABLE 1 - FLIGHT RECORD

Date of Flight		Area Covered
August 21, 1968	Flight 1	Hudson River (Glens Falls to George Washington Bridge)
	Flight 2	Hudson River (George Washington Bridge to Governor's Island) East River North shore of Long Island
August 26, 1968		West shore of Lake Champlain (Whitehall to Canadian Border)
August 28, 1968		Mohawk River (Troy to Utica) Lake Cayuga and Lake Seneca
August 29, 1968	Flight 1	St. Lawrence River, the east and south shore of Lake Ontario and Niagara River (Massena to Tonowanda)
	Flight 2	Lake Erie (Buffalo to Pennsylvania Border)
September 12, 1968		Hudson River (Albany to Arthur Kill, Staten Island) East River and return up Hudson River to Kingston
September 13, 1968		Hudson River (Albany to Arthur Kill, Staten Island and return to Albany)
September 30, 1968	Flight 1	St. Lawrence River and the east and south shore of Lake Ontario (Massena to Rochester)
	Flight 2	South shore of Lake Ontario and Niagara River (Rochester to Buffalo)
October 2, 1968		Lake Cayuga and Lake Seneca South shore of Lake Ontario, Niagara River and Lake Erie (Rochester to Pennsylvania Border)
October 10, 1968		West shore of Lake Champlain (Plattsburgh to Ticonderoga)
October 22, 1968		Susquehanna River (Oneonta to Waverly)

TABLE 1 - FLIGHT RECORD (Cont'd)

Date of Flight		Area Covered
December 18, 1968	Flight 1	Hudson River (Indian Point Area) Flood Tide
	Flight 2	Hudson River (Indian Point Area) Slack Tide
	Flight 3	Hudson River (Indian Point Area) Ebb Tide
April 4, 1969		Hudson River (Roseton to Poughkeepsie)
April 11, 1969		Arthur Kill East River
May 15, 1969		West shore of Lake Champlain (Whitehall to Canadian Border)
May 16, 1969		South and north shore of Long Island
May 27, 1969		St. Lawrence River and the east and south shore of Lake Ontario (Massena to Rochester)
May 28, 1969		South shore of Lake Ontario, Niagara River and Lake Erie (Rochester to Pennsylvania Border)
June 2, 1969		Lake Cayuga and Lake Seneca
June 25, 1969		East shore of Lake Ontario (Pulaski to Stony Point)
June 26, 1969		Hudson River (Glens Falls to Tappan Zee Bridge)
June 30, 1969		Hudson River (Tappan Zee Bridge to Harlem River) North shore of Long Island Harlem River
October 9, 1969		Lake Cayuga (Ludlowville)
October 10, 1969		Ticonderoga Creek West shore of Lake Champlain (Plattsburgh Area) Grass River and Raquette River (Massena Area)

TABLE 1 - FLIGHT RECORD (Cont'd)

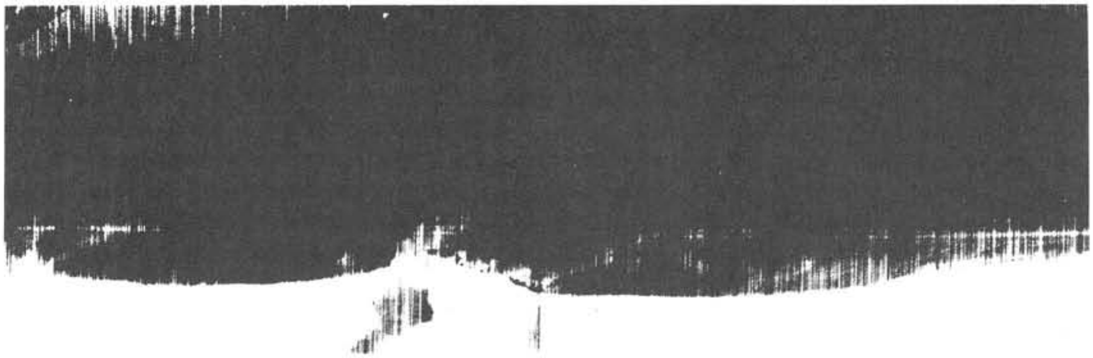
Date of Flight		Area Covered
October 15, 1969		Hudson River (Albany to Governor's Island) Portions of the north and south shore of Long Island
October 30, 1969	Flight 1	Hudson River (Albany to Glens Falls) West shore of Lake Champlain (Ticonderoga to Plattsburgh) St. Lawrence River
	Flight 2	Black River East and south shore of Lake Ontario (Watertown to Oswego) Mohawk River (Utica to Troy)
October 31, 1969	Flight 1	South shore of Lake Ontario, Niagara River and the east shore of Lake Erie (Oswego to Woodlawn)
	Flight 2	Lake Chautauqua Chemung River (Corning to Elmira)
March 20, 1970		Arthur Kill Staten Island (all shores) South shore of Long Island (Verrazano Bridge to Islip) North shore of Long Island (Port Jefferson to LaGuardia Airport) North shore of Long Island Sound (Westchester to Old Saybrook, Connecticut) Connecticut River (Old Saybrook to Middletown, Connecticut)
May 12, 1970		Susquehanna River (Oneonta to to Waverly) Chemung River (Waverly to Addison) Genesee River (Mount Morris to Rochester) South and east shore of Lake Ontario (Rochester to Watertown)

TABLE 1 - FLIGHT RECORD (Cont'd)

Date of Flight		Area Covered
May 21, 1970		East River Harlem River Hudson River (George Washington Bridge to Governor's Island) Arthur Kill Staten Island (all shores) South and north shore of Long Island
May 28, 1970	Flight 1	West shore of Lake Champlain (Whitehall to Canadian Border) St. Lawrence River and the east and south shore of Lake Ontario (Massena to Sodus Bay)
	Flight 2	South shore of Lake Ontario, Niagara River and Lake Erie (Sodus Bay to Pennsylvania Border)
June 8, 1970		Lake Cayuga and Lake Seneca
June 15, 1970		Susquehanna River (Oneonta to Waverly) Mohawk River (Utica to Troy) Hudson river (Albany to Governor's Island) East River Harlem River
June 19, 1970		Lake Seneca Chemung River (Waverly to Addison) Lake Ontario (Rochester to Niagara River) Lake Erie (Buffalo to Dunkirk) Lake Chautauqua
June 23, 1970		West shore of Lake Champlain (Whitehall to Canadian Border) St. Lawrence River and the east and south shore of Lake Ontario (Massena to Sodus Bay)

TABLE 1 - FLIGHT RECORD (Cont'd)

Date of Flight	Area Covered
June 24, 1970	Mohawk River (Troy to Utica) Hudson River (Glens Falls to Troy)
June 30, 1970	Hudson River (George Washington Bridge to Governor's Island) South and north shore of Long Island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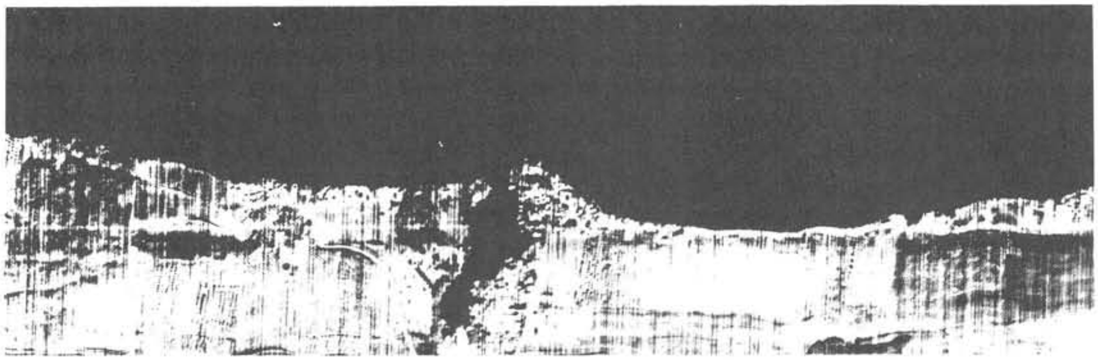
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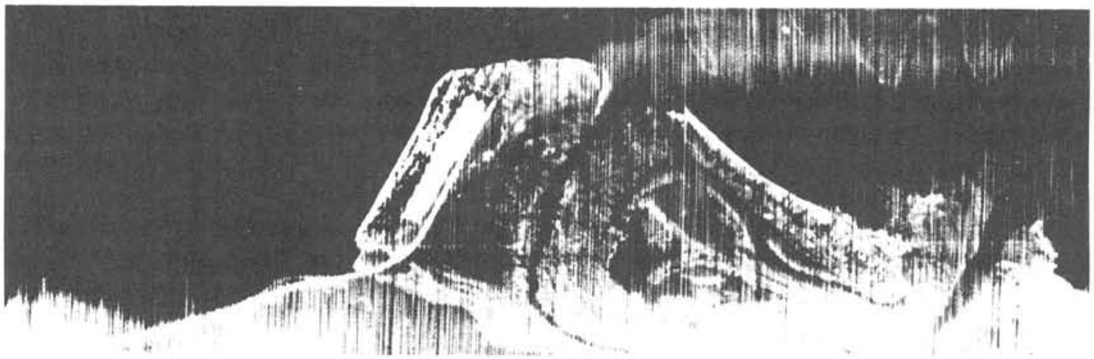
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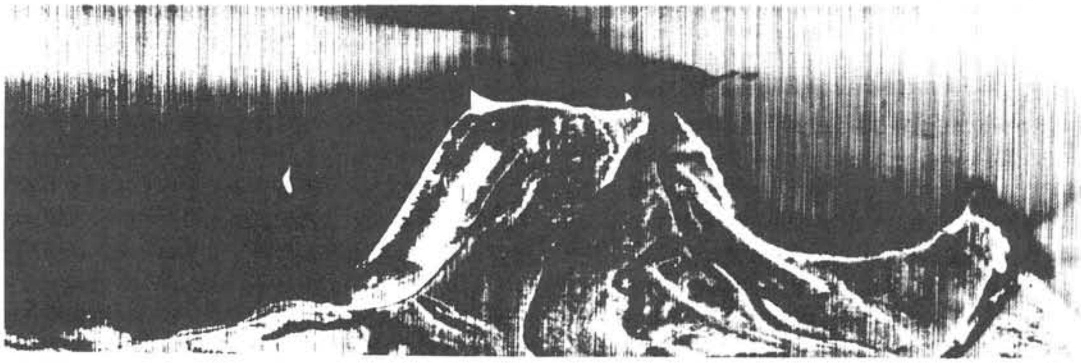
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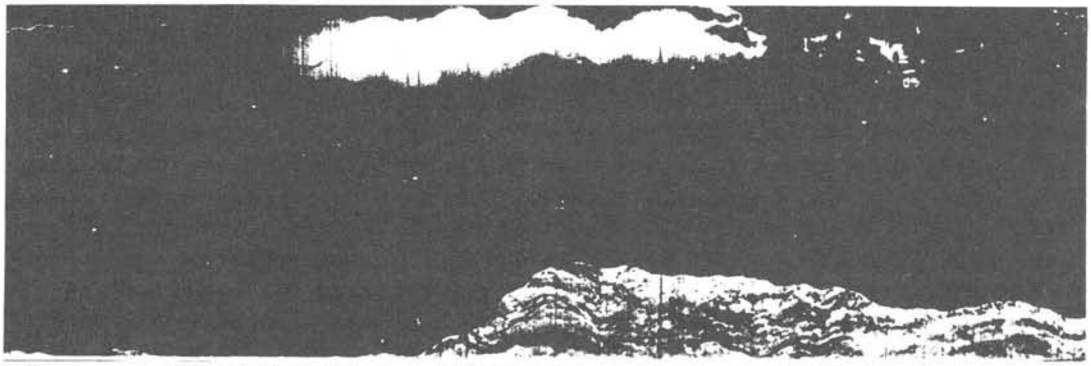

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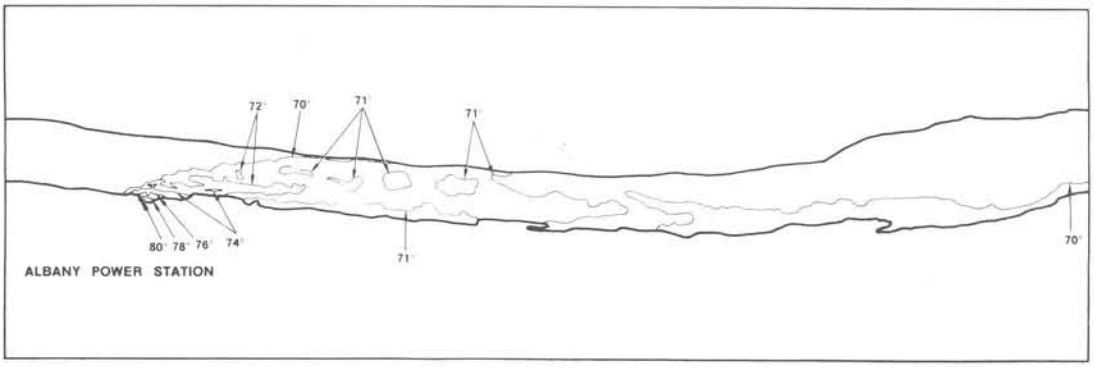
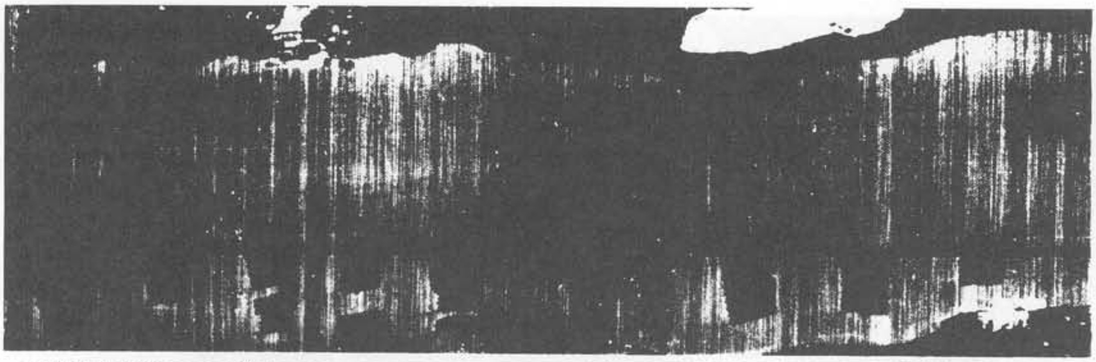


Figure 1



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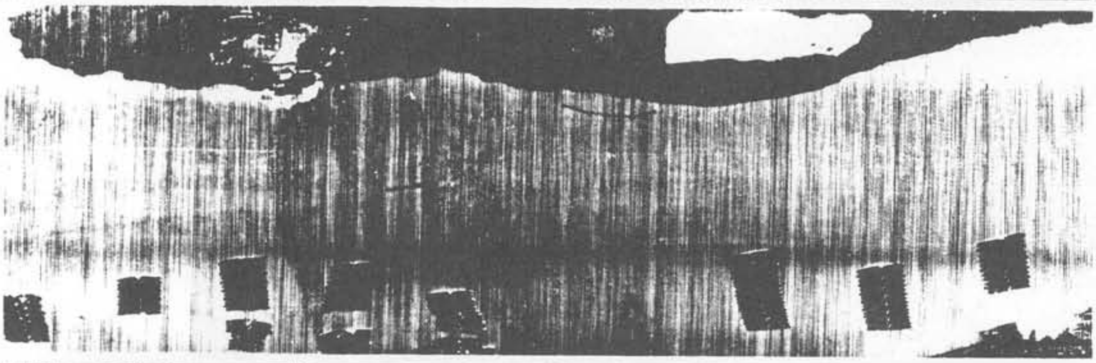

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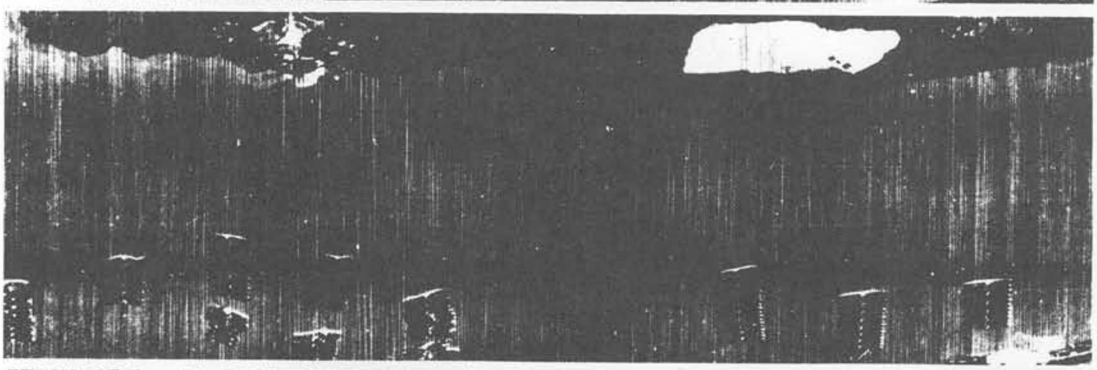

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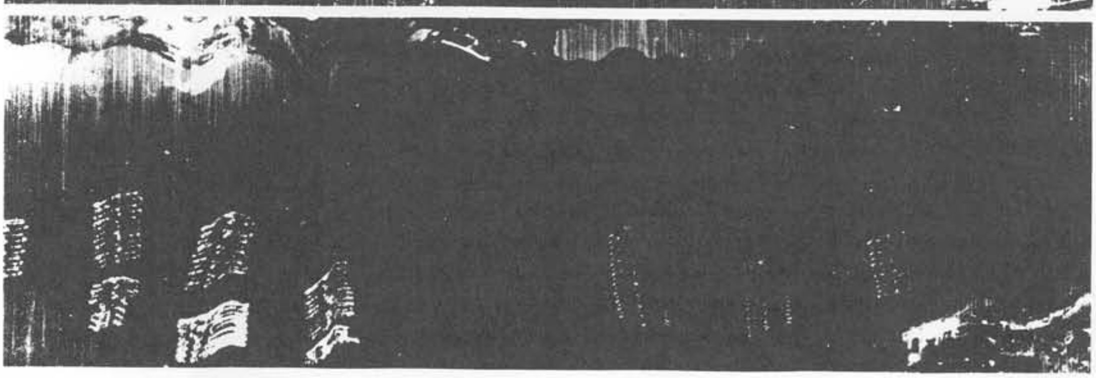

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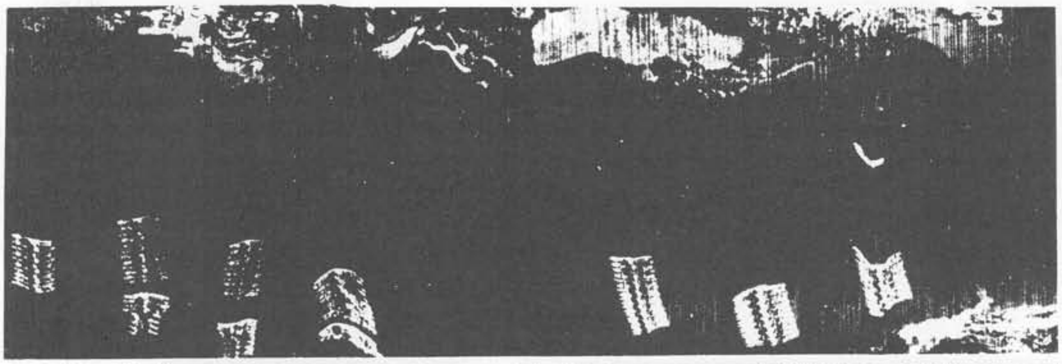

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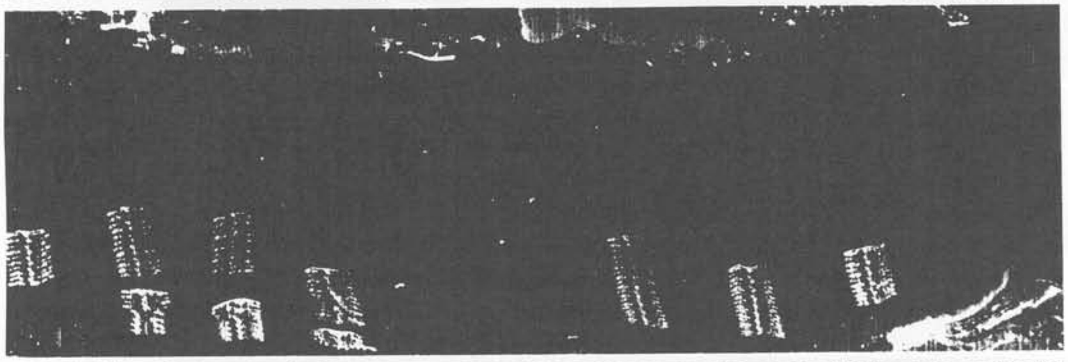

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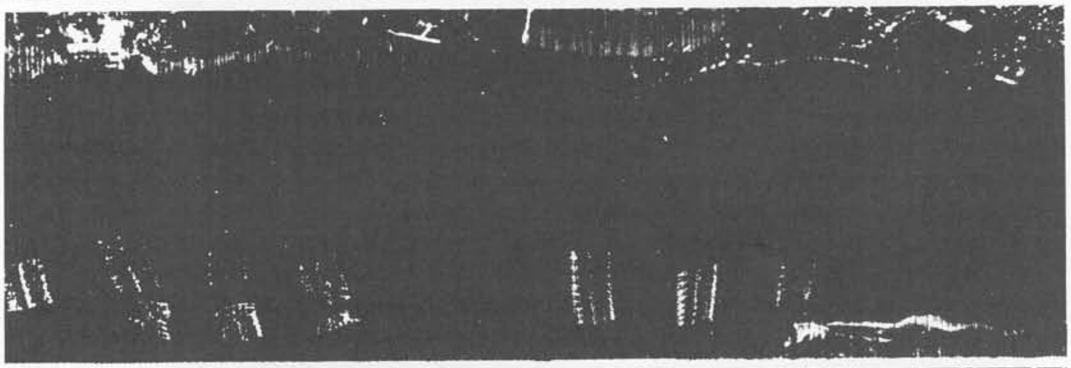

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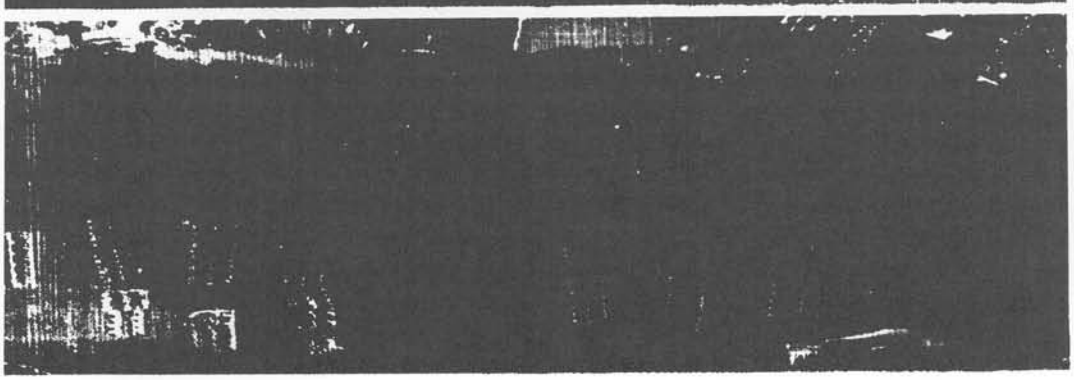

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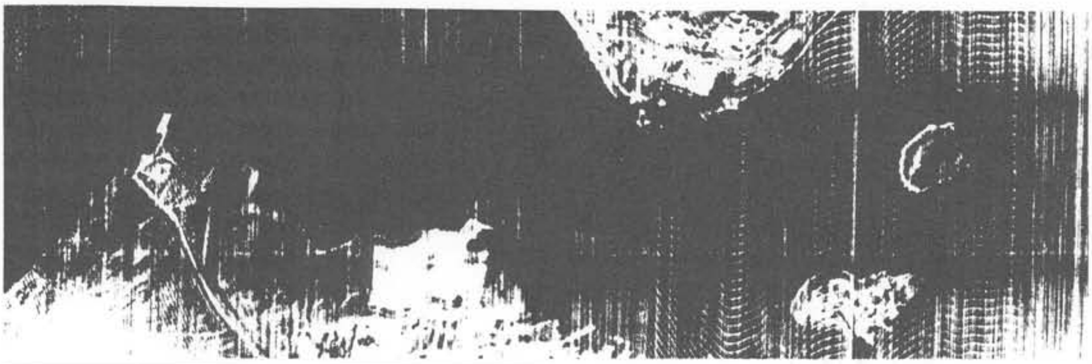

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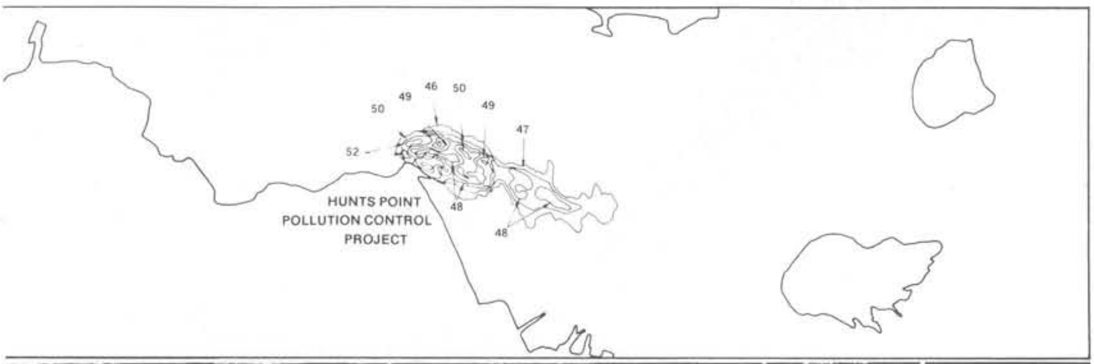
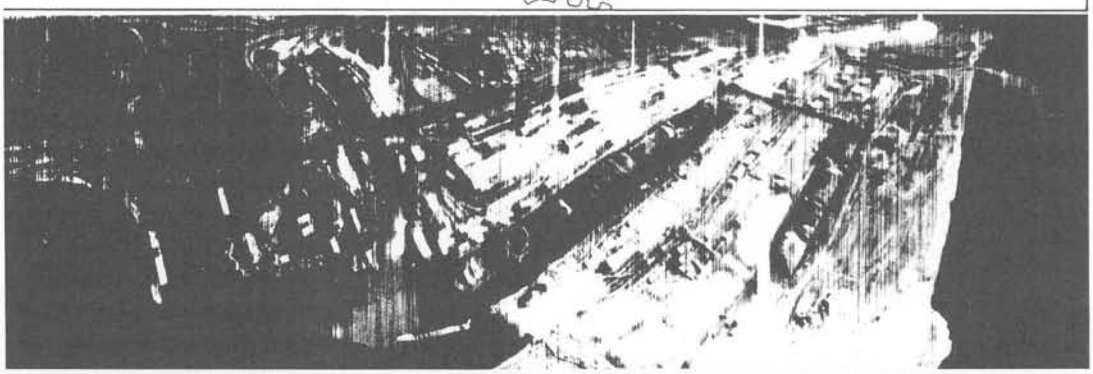


Figure 2



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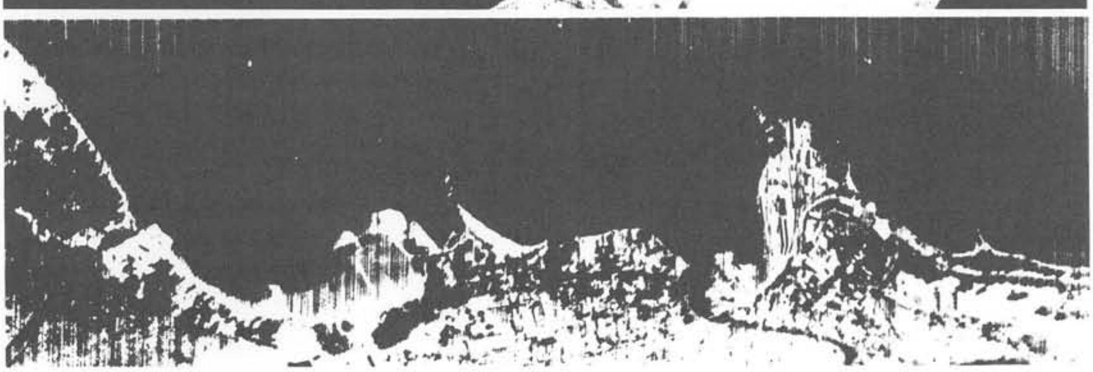

Image 17

SPACE STATION MILIEU

by

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INTRODUCTION

From the time man first made his appearance on earth, he has faced a continuing need to solve the problems raised by his environment. Whether we are discussing prehistoric man's basic struggle to stay alive, or twentieth century homo sapiens' far more subtle and complex requirements to survive the rigors of space flight, we are constantly concerned with man's reaction to his environment--natural and otherwise.

The major reason for putting man into space is to make best use of his unique qualities. His primary contributions are judgment, resourcefulness, memory, and perseverance, all mental qualities. His physical contributions are secondary. This premise dictates that manned vehicles be designed around man and his requirements, rather than that man be modified to fulfill the needs of the vehicle.

The useful human payload is therefore the brain, approximately 2.5 percent of the body weight which necessarily must accompany the brain as far as we know now. By the same token, a disproportionately large mechanical weight must be carried along to serve the sustenance, comfort, protection and safety of the body, namely the entire life support subsystem, and additional weight necessitated by the design philosophy of redundancy, fail-safe equipment, and rescue equipment.

Thus, the problem of manned space flight is one of engineering habitability in a sealed cabin that will ensure an environment in which the occupants can maintain their technical proficiency. In terms of space station design, habitability is defined as the resultant state accomplished through the interaction of all of the provisions furnished to aid the crew in the achievement of a successful mission in a hostile environment. This definition implies a state of being at the completion of the interaction of all the factors rather than a discrete entity unto itself.

There are countless factors influencing man's ability to withstand, endure, or function in space flight. Perhaps the most significant factor is time. The time factor becomes important because there is literally nothing in space of a material nature which can be used to sustain life. Thus, the spacecraft is uniquely different from any other vehicle previously designed, in that it must carry everything needed for man's existence.

The environment of a closed space such as that encountered in a manned space vehicle will be rendered unsafe by man's occupancy. The main attempt of this paper will be to discuss those areas where space travel poses problems (man-made as distinguished from natural) involving closed space ecology. That is, the processes of living requiring an intake of air, food and water and the disposal of the waste products produced by man, that will maintain life without endangering the health of those occupying the closed space. Such problems of living in the encapsulated area of a space vehicle are more difficult than the layman realizes.

HABITABILITY FOR LONG DURATION MISSIONS

Man has evolved as a product of the variables which constitute the earth's surface environment. Since he has evolved in a very specific medium, it is necessary, when operating within an environment different from his natural one, that he be protected from these differences and that his natural environment be maintained and controlled within the vehicle. At present, we have demonstrated that we can encapsulate men in a can of earth-like environment, launch them into the near reaches of space, and bring them back alive. This represents a triumph for a manned space flight of several weeks. But, can we maintain the environment inside the cabin for years, so that man can go beyond the near space we have explored and take a close-up look at the planets?

Environmental Temperatures

Man exists on this earth in ambient temperature extremes from -90°F to +136.4°F. But man's own body temperature range is much smaller - from about 86°F to 110°F. Below 86°F, he loses consciousness; above 110°F, he dies.

Temperature control, for example, will be very important. Closely related to the temperature of the air is the humidity. It is known that humans can endure higher temperatures if the relative humidity is kept low, and also that low temperatures are much more easily tolerated in a "dry" rather than "wet" climate. In the sealed space cabin, however, where utmost physical comfort is mandatory, the temperature must lie within the stipulated range, and the humidity must be stabilized at about 40%.

There are, of course, a number of heat generators other than the sun that must be dealt with if comfortable atmospheres are to be created for future space crews. Among them: electronic equipment; the crew itself; and less important, cooking facilities, lighting, and the like.

In space, the problem is strictly one of thermal radiation balance - balancing the incoming radiation from the sun with the radiation from the spaceship on the side away from the sun.

In theory, this does not appear to be an overwhelming task. After all, we have had experience in air-conditioning many types of vehicles. But in practice we will run into trouble, because the precise environmental conditions are not known, and because the weight, bulk, and power requirements are formidable. The magnitude of the problem is apparent when one considers that extreme temperatures will probably be encountered during escape and more so on re-entry of the ship, and that even if these extremes are survivable, the ambient temperature within the vehicle must be kept between 60°F and 80°F, during the "cruising" part of the voyage to assure that the crew can perform at its best. As important as studies of that tolerance under extreme conditions are, it should always be remembered that the upkeep of normal temperature-humidity conditions is the basis for full mental and physical performance.

The Encapsulated Environment

Man can adapt himself in many ways to his surroundings. However, he displays very little natural flexibility in meeting his respiratory needs. Man can survive for weeks without food, days without water, but only minutes without oxygen. The oxygen is required by the body cells to support the combustion of food into energy in metabolism. An adequate amount of oxygen must be made available, and it must be fed to him under adequate pressure. Lack of oxygen results in hypoxia. The respiratory quotient (RQ) that is the ratio between the volume of exhaled carbon dioxide and the volume of consumed oxygen is on the average, close to 0.85. That is, referring to volume, about 15 percent more oxygen is absorbed from the surrounding environment than is replaced by exhaled carbon dioxide. If there were no oxygen supply or absorption of carbon dioxide, the carbon dioxide levels would rise to the danger point long before the oxygen content would drop to its critical level. It becomes obvious that the rise in carbon dioxide concentration is the decisive factor in survival under such conditions. It therefore becomes imperative that the carbon dioxide which the body produces as metabolic waste must be removed from the environment. Although maximum concentrations of 3 percent at atmospheric pressure may be tolerated as an upper limit for prolonged exposure as shown from experience gained from submarine practice, one percent is a more conservative value to preclude any performance decrement.

Oxygen at sea level has a partial pressure of 160 mm Hg. (103 mm Hg. in the lungs) and saturates about 95 percent of the arterial blood. Any increase in altitude (decrease in oxygen pressure) results in a lower oxygen saturation of the blood. It is theoretically possible to breathe 100 percent oxygen at 160 mm Hg. (3.1 psi). Although this pressure is sufficient for some humans, it is probably not high enough for all. A pressure of about 5 psi (1/3 atmosphere) seems to be the minimum sufficient, but is questionable whether such an atmosphere is physiologically adequate for a longer period. But, since oxygen partial pressure must be maintained at about sea-level values, an increment of 1.9 psi must be supplied by another gas. In air, this gas is nitrogen. The role of nitrogen as a diluent in the atmosphere man breathes has received little attention. Whether or not it must be included is not known for certain, but it is believed that over long periods of time man will run the risk of oxygen poisoning if oxygen constitutes more than 80 percent of the atmosphere in the presence of pressures above 425 mm Hg.

By adding an inert component to the oxygen, a suitable atmosphere can be obtained. Nitrogen is unsuitable for this purpose because it is quite soluble in the blood and tissues. Sudden decompression will release nitrogen bubbles in the body tissues causing "bends" or "chokes." Helium is only about 1/5 as soluble as nitrogen, and would probably be used as the diluent. Gases such as argon have been suggested but not enough is known about them. A wealth of material on this subject can probably be obtained from the Navy on their work in synthetic atmospheres for "frogmen." Aside from the pure physiological aspects of this problem, another factor has to be reckoned with and that is the fire hazard, which becomes greater as the concentration of oxygen is increased.

Physiologically, normal atmospheric pressure and composition of gases would be most desirable. The pressure of the encapsulated environment is a compromise between human requirements and structural consideration. Lower pressures will reduce the total weight of air and the required skin thickness of the sealed cabin. Secondly, if uncontrollable leakage of cabin air should occur, the best situation to minimize such leakage would be a pressure differential as small as possible.

The one way in which we may sustain the human organism in a state of omeostasis for prolonged periods without a severe weight penalty for such items as water, food, oxygen storage and carbon dioxide absorbers, will be to reutilize all bodily wastes - solid, liquids and gases. The critical point is the flight duration beyond which weight penalties of cycling equipment are less than weight penalties of extra food, water, and waste storage containers.

MAN - THE POWERPLANT

The problem of sustaining the human organism in a closed ecological system is fundamentally dependent on man as a powerplant. The engineer defines a powerplant as an arrangment of apparatus, the function of which is to change energy stored in some fuel into another form of energy called useful work. From a physiological point of view, the apparatus is the human organism, the fuel in this case is food and oxygen and the output from the man is useful work such as movement and application of force. Physiologically, the conversion of the chemical energy in food to useful work with the concomitant production of waste products is known as metabolism. The process of living requires an intake of food, air, and water and in the course of normal activity and body metabolic functions, wastes are created which if not treated, will endanger the health of those occupying the closed space.

The main problem here is to define the requirements for an ecological space for man. A starting point for studying the problems of an ecological space of this kind is to determine the input requirements which will allow a man to live, in the full sense of the word, for one day. The implicit assumption is that the sealed cabin will be a closed ecology with an environment not too different from that of our own earth.

Basically, the task to keep a person alive in a hermetically sealed cabin seems to consist of providing enough food, water, and oxygen on the one hand, and to remove feces and urine and to absorb carbon dioxide, water vapor, potentially harmful gases and odors so that they will not reach harmful levels, on the other hand.

Under normal conditions the body weight of the adult remains constant, observed over a long period of time. Consequently, the weight of input has to be exactly the same as that of the output. The only change is a chemical one from a higher energy level compound to lower energy compound, the difference of energy eventually emerging as heat.

It is well known that in man the daily metabolic turnover undergoes considerable fluctuation. Especially the oxygen consumption may vary between anything from 360 liters per day at complete body rest (sleep) to more than five times as much at heavy muscular work. Next to muscular work, eating and the subsequent processes of digestion exert a powerful stimulation upon metabolism, as does environmental temperature.

For a pilot of an airplane a consumption of 535 liters of oxygen per day has been reported. While it is impossible to arrive at a single figure for oxygen consumption without showing correspondingly accurate figures for all the factors mentioned above, an oxygen consumption of 603 liters per day has been tentatively set for our standard man of about 154 pounds body weight with a respiratory quotient of 0.82. This assumes that the activity of a "space pilot" may be close to that of his counterpart in the troposphere. In reality the pilot of a large spacecraft with more room and occasion for muscular exercise may consume considerably more oxygen. In any case for determining fairly accurate figures on oxygen consumption, a previous complete duplication of all conditions that may occur in the cabin of a spacecraft has to be arranged and tested. For our standard man performing normal routine work the following assumed diet would be adequate.

Protein	80 grams	
Carbohydrate	270 grams	
Fat	150 grams	
Minerals	23 grams	
Total	523 grams	(total dry weight)

With this diet the standard man performing routine work would have the following daily metabolic turnover.

INTAKE			OUTPUT		
Oxygen	603 liter	862 grams	Carbon Dioxide	496 liter	982 grams
Water	2.20 liter	2200 grams	Water	2.542 liter	2542 grams
Food		523 grams	Solids		61 grams
	Total	3585 grams		Total	3585 grams

Again, while these values may fluctuate, over a length of time they remain fairly constant. Especially the mass of all input substances must equal exactly the output mass, since the body mass of a healthy individual (adult) remains practically constant and no substance can vanish. The only loss consists of heat energy (2830 Kilocalories per day).

Consequently all the water consumed on the intake side has to reappear quantitatively at the output, whether as part of the urine, feces, perspiration or exhaled water vapor. But added to this output is the water produced metabolically by the oxidation of food. With the diet chosen for our standard man, the metabolic water surplus will amount to about 10 percent of the total water turnover. The percentage of metabolic water depends on the type of food. Fat, for instance, would produce more metabolic water than carbohydrates or proteins.

To keep the weight and space requirements of equipment to a minimum, it is necessary to consider carefully all the factors involved in maintaining a proper balance. An increase of fat in the diet with its high caloric value would save weight on the food supply, but it would also increase the amount of oxygen and water absorbents needed. In any case, the amount of water in a space cabin will constantly increase with time.

The food needed for a long flight will amount to at least 2500 calories per day per man, which in terms of pure carbohydrate, protein, and fat weighs about one pound. Such concentrated food is not tolerable because of its low palatability. To make it appealing, its weight must be increased to well over two pounds per day.

The results of nutritional studies aboard submarines has shown that the mere provision of enough calories, bulk, vitamins and minerals, and other essentials is not enough to keep a man physically and mentally healthy. It is very important that some of these needs be supplied in the form of fresh food, that the types of foods and the cooking techniques be varied, that the diet be balanced, and that the food be tasty as that served at home. If it is not, the crew's performance and morale will not be at their best.

Basically, for a self-sustaining ecological system as constituted by the sealed cabin, everything needed to support life has to be stored before departure of the spacecraft. With increasing duration of a flight into space, the necessary storage will increase correspondingly. To minimize the mass of storage, the ideal process would be the complete recycling of the metabolic waste products back to food; carbon dioxide and water back to sugar and starch, nitrogen compounds (for instance urea in urine) back to proteins.

TOXICOLOGY OF HUMAN OCCUPANCY

There is no parallel in present living which can be compared with the environment of closed space. The condition of space in which all elements of environment are self contained, in which there is neither loss nor addition over a matter of months, and in which space and weight limitations are severe, raises issues of waste handling that demand solution before man can occupy closed space safely.

Some of the wastes that may result from closed space occupancy would cause no problem in normal environment. However, it is in fact necessary to consider many questions of seemingly trivial nature in order to be reasonably certain that nothing has been overlooked in establishing what may be expected as the environment of closed space.

The probable wastes may be classified to include feces, urine, sebaceous gland excretions, perspiration, respiratory end products, washings containing soil from clothing, and washings containing food particles as a result of food preparation and service. In addition, there probably is a series of wastes that will result from working operations. The actual working operations that force the closed space circumstances may produce waste materials that require controls.

A further complication is introduced by the fact that physiological data applicable to individuals living in normal environment at normal atmospheric pressure, eating the usual food, and consuming the usual amount of liquid may be altered by changes in the closed space mechanism for living.

All living things including man have developed methods of attaining virtual freedom from their excreta. Through an intricate chain of reactions many end products of metabolism are rendered physiologically inert, some useful, some ornamental, and others merely tolerable; e.g. the lignin in trees, the calcereous sheath of the plant chara, the urate bearing wing scales of butterflies, the shells of molluscs, the skeletal structure in corals, and for that matter in man, and countless other examples. Those which cannot be made inert are diffused into an environment which luckily is still vast enough to make this procedure practical. If, by chance, this is not the case, a series of detriments are incurred beginning with discomfort and, if continued, culminating in death. This spendthrift method of diluting the end products of metabolism is not available on the same scale in a sealed cabin.

At this point it is apropos to mention that the toxic effect of a substance is proportional to concentration times the duration of the exposure. This is, however, a misleading simplification since the substances with which we shall deal may have a threshold or be so chemically active as progressively to inactivate living material as it becomes exposed.

What are the toxic factors which one must consider? Let us first dicuss those arising from the over-all metabolic activities. In this regard there are four general media of excretion: air, urine, perspiration, and feces.

In the breath, carbon dioxide, water, and oxygen are the major metabolic constituents. There are, however, other volatile components which are diffused from the blood. Among these, even in health, are small amounts of nitrogen, acetoacetic acid, characteristic volatile oils of certain foods, and gases formed in the fermentation by bacteria in the bowel; i.e., methane and hydrogen.

In the urine, the greater part of the excretion products are non-volatile but small amounts of ammonia, ethereal sulphates, and the substances also cited above to be in the exhaled air normally may be found.

In the glandular excretion of the skin perspiration may include a variety of components including urea, uric acid, creatinine, lactic acid, ethereal sulphates of phenol and skatole, amino acids, sugar in traces, and albumin. The sebaceous gland excretions are known to contain small quantities of cholesterol, simple fatty acids, fatty acid esters, albumins, and inorganic salts.

In the feces, however, there is a variety of substances formed through the action of the symbiotic or commensal bacteria, some of which are quite toxic. These are normally detoxified by the liver in the limited concentration in which they gain access to the blood stream. Since these substances are low in concentration, as excreted, they pose no health problem unless allowed to accumulate. The volatile substances, largely contained in the flatus, are carbon dioxide, nitrogen, oxygen, hydrogen, and methane, as well as such minor constituents as mentioned earlier. In the feces proper the toxic substances indole, skatole, hydrogen sulphide, phenol, and various amines range to high potency.

It is apparent that the body defends itself against most excreta in the normal site of exposure. For substances such as carbon dioxide which are readily invasive, there is a relatively high tolerance. For others, such as the amines, while in the bowel there is perhaps some mechanism of exclusion, but if they were inhaled, the lung surface would be defenseless.

If excreta are retained in the sealed cabin for, let us say, the conservation of water, then a problem is posed regarding the accumulation of such substances as are normally excluded from contact with the body. Thus dried feces may constitute a hazard through toxicants borne as dusts, or from waste distilled from such sources leaving residues as toxicants, such as amines, which may be hazardous to handle unless the expense of oxidative treatment is feasible for detoxification. The same sort of problem exists with the recovery of potable water from urine.

This leads to consideration of externally generated toxic agents. The variety here is much greater than with toxicants produced by the body, and the toxicities, of course, range over a comparable gamut.

Since there are many sources of externally generated toxic agents, I shall confine this discussion to those toxic agents generated in relation to living in the closed space.

Consider first certain practices which may obtain in everyday living, as, for instance, in the preparation of foods. Cooking odors can be unpleasant and constitute a nuisance in air conditioning but are not ordinarily hazardous. However, there are certain substances which are highly toxic, such as acrolein, which is reproduced in the destructive heating of fats. Under normal circumstances this rarely rises to concentrations greater than will produce pulmonary irritation, but deaths have been attributed to this occupational hazard of chefs. In this connection also it is more or less obvious that any form of combustion which may be incidentally produced is at the expense of oxygen in the cabin. Though smoking is tolerated on commercial planes at this time, a space craft designer would need to think carefully about the effect of carbon monoxide thus generated would have on the crew. Carbon monoxide, like other chemicals of high affinity or solubility, is avidly accumulated in the blood to the detriment of the normal function of oxygen transport.

Another important source of toxic agents is to be found in the ordinary common household cleaning substances such as floor and furniture waxes. While the use of these products under normal living conditions produces no hazardous effects, the use of such items in sealed cabins may produce harmful effects due to the aromatic constituents of the waxes and polishes which will be continuously produced and whose concentration will reach critical levels. In order to circumvent this hazard all cleaning operations will probably be made with soap and water or some inert detergent.

It becomes necessary to scrutinize the routine procedures of normal living on the surface of the earth in order to prevent the introduction of harmful toxic agents into the sealed cabin environment.

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