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INTRODUCTION

Development in the next decade and continued misuse of our water resources can cause irreparable damage to our environment if the pollution is not regulated. An alarming indication of the mismanagement and of each regulation of water pollution presently in force is the shameful devastation of the Great Lakes where many of the finest beaches became health hazards. The recent oil leaks from drilling on the seabed in Santa Barbara, California and oil spills in the English Channel and off the Louisiana coast are constant reminders that time is running out for regulating these problems. Even in the New York metropolitan area, the tragic condition of the Hudson River, the Passaic River and Jamaica Bay are sufficient to demonstrate the necessity of control of water pollution by legal and technical means.

The fourth academic year of the Seminar is devoted again to explore the essential aspects of the above problems. The great progress of mankind in industrialization and urbanization together with the high demand for water makes essential the redistribution of available water resources. The "Annual December Meeting in Washington, D.C." with the World Bank as host was again a review of the world situation in water resources distribution.

On November 29-December 4, 1970 in New York City, the Seminar organized together with Columbia University School of Journalism-American Press Institute a symposium for journalists: "Seminar on Environmental Problems." Thirty-one journalists from the United States and abroad participated in the symposium and our Seminar members delivered six half-day lectures.

Finally, the editors of the Proceedings wish to express their appreciation to all members contributing articles and lectures to foster the Seminar. The publication was made possible only by the generous help and cooperation of the U.S. Department of the Interior and the State of New Jersey, Department of Environmental Protection in the series of U.S. Geological Survey - Water Resources Investigations.

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WATER PROBLEMS OF THE MINING INDUSTRY IN THE UNITED STATES

by

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Water use in the mining industry is only a small percentage of the total water use of the Nation, but an adequate supply of water is essential to the future growth of the industry. My remarks for the most part will be addressed to the relation of water to mining and to the beneficiation of coal, metallic, and non-metallic minerals. I will also discuss briefly the major pollution problem of acid mine water.

The terms used to differentiate between the types of water used in the mineral industry may be defined as follows. Intake water, withdrawal water, or makeup water, is defined as new water introduced from an external source for the first time into a given mining or milling operation. Recirculated water is water that is used more than once in the mining or milling process. It may be treated before being reused, or it may be recycled without treatment. Recirculated water, plus new water, equals total water used in the process. Consumed water includes water lost by evaporation as well as water lost in the operation or in the product and discharge water is water eliminated from the processing cycle. The latter usually is conveyed to a lake, stream, well, or some other area foreign to the process.

In some mines water is used to keep down dust, flush drill holes and for other similar functions. However, most of the water used by the mineral industry is for mineral beneficiation processes such as washing, gravity separation, flotation and as a vehicle for the transportation of some materials as slurries.

In the processing of minerals, ample intake water is necessary for the operation. If water is in short supply, it may be necessary to increase the amount of recirculated water to a maximum which, in turn, may require treatment of the water to remove deleterious substances. Sediment is the most common constituent of mill discharge water. It is the practice in most operations to allow the sediment to settle in tailings ponds and reuse the clear water. The water discharged from the operation, in some instances, may contain excessive amounts of dissolved metals or compounds. If such contamination exists, it may be necessary to remove these chemical impurities by precipitation or other methods before release into a lake or stream.

According to the Water Resources Council total water withdrawals for all purposes in 1965 were 269.6 billion gallons daily. The water withdrawals of the mineral industry for the same period were 3.2 billion gallons daily. The mineral industry, therefore, only accounts for slightly over one percent of total water withdrawals. It also is estimated by the Water Resources Council that by the year 2000 total withdrawals for all requirements will be 804.6 billion gallons a day and water withdrawals for the mineral industry will be 4.7 billion gallons a day. Consumption of water by the mining industry, as projected, consequently will play only a minor role in the overall water resources picture during the next 30 years. One reason why projected withdrawals of the mineral industry do not increase at as rapid a rate as total withdrawals is because the recirculation ratio of the mineral industry is expected to increase more rapidly than the recirculation ratio of total water usage.

The greater portion of the water used in the mining industry is confined to the production of a relatively few commodities. The comparisons that follow are based on figures from the U.S. Bureau of Mines publication, Mineral

Facts and Problems. Water usage of the petroleum and natural gas industries has not been included.

The processing of non-metallic minerals and coal accounts for approximately two-thirds of the mineral industry's water withdrawal. The non-metallics include construction materials, phosphate rock, sulfur, barite, and the evaporites. The largest single use of water in the mining industry is for construction-materials processing, which accounts for over one-third of the total withdrawals. Although only about 500 pounds of water are used to process one ton of sand and gravel, the magnitude of the industry is responsible for the high water usage. The water used in the processing of phosphate rock constitutes about one-sixth of the total withdrawal. It takes about three tons of water to produce one ton of phosphate rock. In Florida the practice of producing slurries from the ore and pumping them to washing plants accounts for the large quantity of water used per ton of phosphate produced. Although an estimated 64 percent of the bituminous coal produced in the United States is washed and processed before delivery, water requirements for the coal industry are only about 7 percent of the total mineral industry withdrawal. It is estimated that about 500 pounds of water are used per ton of coal processed. Water used in mining and processing the evaporites--salt, potash, soda, and borates--accounts for about 5 percent of the industry's withdrawal. Water required for the remaining non-metallic minerals, which include sulfur and barite among others, is estimated at another 5 percent.

Slightly less than one-third of the mining industry's withdrawal demand for water is in the mining and processing of metal-bearing ores. In metal mining the largest water usage, about 15 percent of the total withdrawal, is in the mining and beneficiation of iron ores. Copper mining uses about 10 percent of the total withdrawal, and about 5 percent is used in the processing of lead, zinc, uranium, and other metallic ores.[1]

Separate statistics on the amount of water used for transporting mineral materials are not available but are included in total industry consumption. Compared to the total water withdrawal of the mineral industry, the quantity utilized for slurries is believed to be relatively insignificant but may increase in the future due to technological advances and a gradual shift to the mining of coal in the western United States where distances are great and over 60 percent of our coal reserves now lie.

For instance, the new Black Mesa coal mine in northern Arizona which goes into operation this summer will supply coal for the 1.5 million kilowatt Mohave power project in southern Nevada. The power plant will receive a coal slurry, force-fed at 660 tons of coal per hour through a 273-mile long, 18-inch diameter pipeline, believed to be the longest and largest coal slurry line ever built. Over a period of 35 years, it is estimated that at least 117 million tons of coal will be pushed through the pipeline in a solution of 50 percent pulverized coal to 50 percent water by weight.

[1]Water used for smelting and refining is not included.

The Nation has made heavy withdrawals of minerals from the wealth of resources with which it has been endowed. In the process, we have mined as cheaply as possible those deposits which were most accessible and provided the greatest profit to the producer. The preoccupation with short term gains too frequently ignored the long term costs of silted streams, acid laden waters, and wastelands left by surface mining. Today we face the necessity of maintaining the balance between the requirements of a rapidly increasing population and a shrinking natural resource base while at the same time preserving our natural environment and its ecology.

The return of mined land to recreational or productive use is fast becoming a way of life in this day and age. Mining operations, like all industry, are under close scrutiny and it is popular to ferret out something vulnerable to attack and emphasize its impact on the environment. Most of the progressive mining companies today have facilities for, or are making plans for combatting water pollution and for reclamation of the land following mining operations.

In the past the mining industry, particularly in sand and gravel operations, has contributed greatly to the sediment pollution of surface waters. Now, however, general practice is to allow the solids to settle out before the effluent is discharged. This control of sediment pollution is simple and is relatively inexpensive. The Federal Water Pollution Control Act of 1965 (P.L. 89-234) now requires the States to set up water quality standards, and industry is required to take whatever steps are necessary so that discharge water will conform to these standards.

The sand and gravel industry is noteworthy for taking land reclamation measures that leave the mined-out site with more land value than it had before. In many cases the pits of depleted deposits have been turned into lakes and the areas around them have been landscaped to provide recreational areas for thousands of people, for game and wildlife sanctuaries, or have been converted for other productive land uses.

A modern metal-mine beneficiation plant requires a tailings dam and settling pond that covers several acres. The operation may also require a series of dams, reservoirs, and ditches to assure an adequate water supply for the operation. An outstanding example of environmental planning in the development of a large western underground metal mine is that of the Climax Molybdenum Company's Henderson operation in Colorado. The operation is big, $200 million is being invested, and the beneficiation plant will have a capacity for treating 30,000 tons of ore per day.

The Henderson operation is unique in that Colorado conservationists and the Company are working together on water purity, recreation, land planning, and other esthetic considerations. The effort is truly an experiment in industry-conservationist cooperation. The willingness of both groups to work together in a constructive way to achieve maximum benefit from the mineralization with minimal effect on the environment is most encouraging.

In order to minimize water pollution, four criteria were used in Climax's extensive studies of tailing disposal sites: (1) The sites should be economical, (2) they should be located off stream, (3) they should be located so that flood dangers would be minimal, and (4) their location should allow minimum exposure to the general public. Thirty-six potential sites were studied in detail, and it was determined that only two areas, located on the western slope of the Continental Divide, could meet the four criteria. It was further determined that the best location for the ore-milling plant was the western slope in conjunction with the tailings pond.

The decision was made to haul mine ore through a 10-mile tunnel to the western side of the Continental Divide and thence 4.6 miles to the mill. Much of the latter portion of the electric rail system will be hidden from view. Mill water will be conserved with a tailings settling pond and recycle system, and a series of diversionary canals planned above the mill and tailings pond will provide flood protection. These measures, and many others will be undertaken primarily for conservation purposes, and with the blessings of the Colorado conservationists.

The program at Henderson has received an award from the Sports Foundation and recognition from the United States Forest Service. The project was honored by Business Week Magazine as one of the best examples of corporate effort to protect and improve the country's physical environment.

David Ackerman, Vice President, Amax Copper Division, made the following statement at the October 1969 American Mining Congress meeting in San Francisco:

> "First we are evaluating all development plans on the basis of their impact on the environment. Second, we have discovered that even more effective environmental protection results if we keep governmental and private conservation groups informed as planning proceeds. While the capacity of designing a good tailings disposal system, of making efficient use of water resources and of protecting watersheds from contamination is part of a mining engineer's trade, we have also discovered that we can communicate this ability and intent to those most concerned with a mine's environmental impact. This process of communication can build mutual understanding and confidence that, in itself, provides for even better environmental planning."

An operation that has not fared as well is the Silver Bay Taconite plant of the Reserve Mining Company on the shore of Lake Superior. In 1947, after extensive public hearings before both Federal and State agencies, the Reserve Mining Company was granted a permit to deposit its mine tailings in Lake Superior. The appropriate State conservation agencies in Wisconsin and Michigan advised the Minnesota Commissioner of Conservation that they had no objection to issuance of the permit. It was brought out at the hearings that the large quantities of water and the land areas necessary for the deposition of tailings were not available near the ore deposits. The area near the mine is valued as a resort and recreation area, and the ore deposits abut on the Superior National Forest.

The tailings are being deposited in an area where the lake is 600 to 900 feet deep. The shoreline in the Silver Bay area is unusually steep and rocky, and has never been considered a substantial fish spawning area. The greatest thickness of tailings deposited on the lake bottom after 14 years of operation is 6-3/4 inches and it is estimated that after 50 years of operation at the present rate the tailings layer will be less than five feet thick.

Reserve Mining has a big stake in this venture. Its capital investments to date are about $350 million. In April 1969, the company had 3,200 employees with an annual payroll of $29.4 million, and $3.7 million in State and local taxes were paid in 1968.

Now, however, after 14 years of operation, the question of pollution of the lake has become an issue. The Lake Superior Enforcement Conference has concluded that there is "presumptive evidence that discharges from the Reserve Mining Company endanger the health or welfare of persons in States other than the one in which the discharges originate, and that they also have a deleterious effect on the economy of a portion of the lake by reducing the organisms necessary to support fish life."

Interior Secretary Walter J. Hickel last month requested the U.S. Army Engineers to issue a "conditional" three-year revalidation of the Company's permit. He also recommended that Reserve modify its discharge system so that tailings are not carried beyond the three mile square limit specified in the first permit issued twenty years ago, and that the Company adhere to the requirements and recommendations of the 1969 Lake Superior Enforcement Conference and the pollution abatement compliance schedule outlined in the Water Pollution Control Act.

Another case history illustrates the problems and scope of water use in metal mining in the arid southwest. The Phelps Dodge Corporation has recently completed a 25,000 ton-per-day mine-mill complex at its Tyrone property in southwestern New Mexico. In the early planning, the Company was faced with the problem of an adequate water supply. The Gila River some 23 miles from Tyrone was the logical source, although the distance and an adverse head of nearly 1,500 feet between the river and the plant site were major obstacles.

Having acquired water rights on the Gila River, through purchase of farm and ranch land, the company went ahead with the formidable job of getting the water to Tyrone. First, to control the seasonal fluctuations in water volume, an off-steam, earth-filled dam was built. The reservoir created by the dam will contain, when full, 2,100 acre-feet of water with a surface area of 65 acres. From a pumping station on the Gila River, four 5,400-gpm pumps lift the water to the reservoir a mile distant and 275 feet above the river. Phelps Dodge has optioned the newly formed lake and the land around the reservoir to the New Mexico State Game Commission for recreation purposes. The lake has been stocked with trout, and fishing prospects for this coming summer are excellent. From the reservoir the water is pumped through a 27-inch pipeline for 12.8 miles to a booster pumping station, then another 9.7 miles to two 4-million-gallon-capacity supply tanks at the mill.

The Tyrone plant, like all other copper operations in the west, conserves as much fresh water as possible. Approximately half of the water used in ore processing is reclaimed and recycled. Water from the mill is clarified in four 325-foot diameter thickeners, and the overflow water with some solids flows to the first of three tailings ponds. At present only one pond is in service; eventually all three will be used in sequence. The tailings dams are two, five and seven miles respectively, from the concentrator. In addition, a diversion dam for surface water is being built with 800,000 cubic yards of earth fill. This dam will be used solely to contain drainage between the No. 2 and 3 tailings ponds and to prevent any washing of tailings onto neighboring land. Without this elaborate and costly system for water use and conservation, the Tyrone property would not be operating.

Not only water pollution, but also air pollution is involved in tailings disposal. Wind blown tailings can create a serious air pollution problem. Mining companies must give consideration to the stabilization of tailings dams during the operation, and the stabilization of both the dam and the tailings after the operation has been completed. This can usually be accomplished by planting grasses, trees or other vegetation native to the surrounding area.

Salts of metals such as zinc, lead, arsenic, copper, and aluminum, if present in other than minute amounts, are toxic to fish, wildlife, plants, and aquatic insects. Indirectly associated with acid drainage are the undesirable slimy red or yellow iron precipitates, known as "yellow boy," in streams that drain sulfide-bearing coal or metal mines.

Although the water discharged after use by the mining industry is a relatively small portion of the total water in the processing cycle, most of the liquid effluents from the mineral processing plants are treated or are of satisfactory quality to be discharged directly to the stream. However, there is a large inflow of water, exclusive of intake water, into mines (active and abandoned) through the unconsolidated materials resulting from surface strip mining operations, breaks in the surface, and from seepage through broken strata caused by underground mining. This of course, is a form of withdrawal that cannot be measured. This water, because of the intimate contact with sulphide-bearing coal or metal deposits often becomes a major source of acid mine water when returned to the surface by pumping or gravity discharge. It is known that chemical pollution of ground water flowing or pumped from both surface and underground mines and spoil banks contributes more than 4 million tons of equivalent sulphuric acid annually to thousands of miles of streams in the United States.

Although a few of the metal mining regions of the west have had a significant effect on local water quality, the major stream pollution resulting from this cause is in the midwestern and eastern coal areas.

The greatest concentration of coal mining in the country is in the Appalachian Coal Region. The Federal Water Pollution Control Administration reports that thousands of miles of streams are adversely affected from the drainage of 66,500 surface and underground coal mines and coal mine waste sources in this area. In May of 1965 the United States Geological Survey conducted a stream-quality reconnaissance in the region. Field measurements were made at 318 major stream sites throughout an area of 160,000 square miles. Results indicated that 194 stream sites (61 percent of the total sites examined) were measurably influenced by surface and underground coal mine drainage. One Appalachian State agency estimates that it will take many years and more than one billion dollars to correct the acid mine water pollution problem associated with coal mining in that State.

Progress is being made toward control of acid mine drainage, particularly in Pennsylvania, although the size of the area affected causes the results to date to appear rather insignificant. Several measures are being employed to combat the problem:

1. Stripping of overburden from drift and other shallow cover mines and backfilling to completely fill the old mine.

2. Lime treatment to neutralize the acid water discharge from mines and that carried by small streams.

3. Planting of vegetation on spoil used to backfill strip mine pits.

4. Covering refuse piles with clay and planting vegetation.

With adequate funding, the application of existing measures, and the development of new techniques to effectively combat acid mine water, the problem can be brought under control, but it will take time.

The main aspects of water problems in the mining industry can be summarized as follows: First, water supplies for the mining industry on the whole will not be a major problem in the foreseeable future; second, water pollution by sediment from mining operations can be well controlled, with existing technology and equipment; third, chemical water pollution, especially from acid mine waters, presents a problem for which methods of control have been developed and which must be employed regardless of cost if the ecology of the environment is to be preserved.

The mining industry as a whole should be praised for instituting mining practices in recent years that are directed toward reclamation. The industry is making great strides toward keeping our waters clean, and it puts water to good use in its reclamation planning. We look forward to increased reclamation efforts in the industry in future years. Finally, multiple use of our resources, both land and water, is the key to the preservation of the Nation and the continued well-being of our society, and one of our basic needs is the production of mineral commodities. We must work for a happy middle ground; mining has an important place here along with all other land and water uses and the mining industry must strive to make the most of it.

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CONTRASTS AND CONVERGENCE IN ENGINEERING AND ECONOMIC APPROACHES TO WATER DEVELOPMENT AND POLLUTION CONTROL

by

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For more than 5,000 years large-scale water resource development has been carried on by engineers. It was only two decades ago, however, that academic economists in the United States began applying in an important way the powerful abstractions of economic theory to decision-making in the water field. While the civil engineer and the theoretical economist interested in water resources address themselves essentially to the same problem, their intellectual history has given them distinctive viewpoints and, in some cases, seemingly incompatible planning criteria.

This paper will treat five aspects of the question. First, some evidence of the contrasts in the engineering and economic approaches to water resource decision-making will be offered; next, the dissimilar backgrounds of the two fields will be explored from the historical perspective; some important implications of the contrasts will be discussed; significant ways in which engineering practice and economic theory are converging will be mentioned. And lastly, it will be suggested that the experience gained from the confrontation of the two fields over the past 20 years in the United States can be used in meeting the broader problems of the total environment in what is rapidly becoming a struggle for ecological survival.

Some Evidence of the Contrasts

The experience of the writer in the early post-war period--both as an undergraduate civil engineering student, and later in a firm planning hydroelectric projects for developing countries--seemed to be consistent with economist Kenneth Boulding's observation that water resources are developed by engineers and that economists usually have very little to do or say about it.[1] In 1949, Hollis Chenery wrote about the "conventional division between economics and engineering which has been assumed by economists."[2] By the early 1950's, however, a few notable works on water by theoretically-inclined economists began to appear, such as those of Ciriacy-Wantrup at Berkeley. By the late 1950's, a substantial literature on water had appeared in economics and in some of the other social sciences.

However, this important development remained, for the most part, unheeded or ignored by many influential members of the engineering profession engaged in water development. In 1961, for example, a prominent engineer wrote this:

> Just this week I received an announcement of a new publication. . . . Inasmuch as water supply is my business it was a surprise to see a new book authored by unfamiliar names. Upon closer check it developed that two of these men are economists. . . . What is the state of our science that geographers, economists and lawyers can speak for us these days.[3]

At about the time this was being written, economist Robert Dorfman was writing in a very different vein, averring that the design of a water resource project depends on "an intimate interplay of economic and engineering considerations."[4]

Anyone in the field of water resources can be excused for being confused by this. If the influential professionals in the field disagree on such fundamentals, consider the plight of the layman decision maker--the congressman, the government administrator, the water board commissioner, etc.,--who plays such a key role in public water investment and management. Among other things, both laymen and professionals involved in water resource decision-making are very often not aware of the lack of integration of engineering design and economic theory in the undergraduate civil engineering programs. Political scientist Hubert Marshall cites several studies showing that "engineering economy" courses are required by only about half of the leading civil engineering departments in the country.[5] Marshall's discussion, however, omits an even more fundamental point: the fact that "engineering economy," as taught in engineering departments, and theoretical economics, are vastly different fields.

Engineering economy developed into a formalized body of knowledge and techniques for the analysis of engineering projects. The first textbook in the field was published in 1915, following a long application of the principles to bridge and highway design. Engineering students for decades have been using successive editions of E. L. Grant's book on engineering economy.

However, as Vernon Smith observes in a perceptive discussion in his book Investment and Production,[6] these developments in engineering economy seem to have taken place completely outside the main stream of economic analysis, and without any connection with the body of economic thought or on the theory of production. Instead of using the terminology and conceptual framework of the Walras-Wicksteed theory of production, Smith argues, engineering economy has evolved its own jargon and conceptual framework. At the same time, Smith points out that a similar paucity of attention has been given to the subject of engineering economics by economists. Smith sums up the contrasts in the two subjects as follows:

1. Engineering economics is almost entirely problem--or "case"--oriented with little recourse to the use of theoretical constructs or the development of general principles.

2. The economic theory of production (until very recently) has evolved in an empirical vacuum except for the outstanding contributions of agricultural economists to applied production theory.[7]

An Historical Perspective on Water Resource Engineering

In order to see the decision-making process in its conceptual essence, as it applies to water development, it is helpful to view it in a primitive form. Let us, therefore, look back in history and view the decision-making process as it worked before calculus or algebra, or even before the decimal notation; before, too, power driven machinery, electric energy and reinforced concrete. Let us go back 5,000 years. The Bronze Age had not yet reached northern Europe, and its inhabitants were still wandering bands of hunters and gatherers. This was, remember, a thousand years before Abraham and the origin of the world's three great monotheistic religions. By 3,000 B.C. men were standing on the banks of the Tigris, Euphrates, Nile and Indus, making decisions and taking actions to bring these great rivers under their control.

Actually, the beginning of water control engineering predates the first civilizations by many millenia, going back to the beginnings of irrigated agriculture, perhaps as early as 7-10,000 B.C. By 3,000 B.C., the great river systems mentioned above, and innumerable smaller streams as well, had been harnessed for flood control, irrigation, water supply, and in the case of the Harrapan civilization on the Indus, for an intricate sewerage system.

The engineers who brought this about were in a very real sense participating directly in the civilizing process. When one studies the world's first civilizations--those in Mesopotamia, Egypt, the Indus Valley, and China--it is evident why they are called "riverine cultures." The engineers who created these enormously productive man-environment systems were men who worked without electronic computers, slide rules, or handbooks. They were working with little more than their own minds. The important thing here is that the creativeness of the human mind was, 5,000 years ago, and remains today, at the heart of the water decision-making process.

The so-called "benefit-cost" concept has been employed for the past 5,000 years. Anthropologist Karl Wittfogel, discussing early man's decision-making for arid land development through water projects, expresses the process in its most general form:

> Man pursues recognized advantage. Whenever internal or external causes suggest a change in technology, material productions, or social relations, he compares the merits of the existing situation with advantages--and disadvantages--that may accrue from the contemplated change. When the sum total of the accruing benefits clearly and convincingly exceeds the required sacrifices, man is willing to make the change. . . .[8]

Notice how this resembles the statement of a modern economist describing the heart of marginal decision-making: "an action merits performance if and only if, as a result, the actor can expect to be better off than he was before."[9] (The fact that the modern economist uses differential calculus to express this mathematically does not change the fundamental conceptual resemblance.)

The important point here is that there seems to be nothing logically or philosophically intrinsic in the water decision-making process which accounts for--or indeed which makes necessary--the contrasts between water engineering practice and theoretical economics. What is suggested, rather, is that the contrasting approaches have evolved as variants growing out of very different historical backgrounds.

In examining the history of water engineering over the past five millenia, we can observe a set of professional attributes which have remained surprisingly consistent over time. The water development engineer has always been involved in large-scale public works requiring government funding and subject to government control. The water engineer has thus always had to have an intimate working relationship with government officials, administrators, planners, political decision makers and the government bureaucracy in general. He has always had to work with, not isolated from, the political power structure. The criterion of success for the engineer has always been concrete results. He succeeded or failed on his ability to get things done--to provide irrigation water, control floods, or drain swamps. His productive efforts were, and are, clearly tangible and measurable. If destruction and death resulted from a project failure, the engineer alone was held responsible. Without hydraulic laboratories or a grounding in fluid mechanics, the engineer through most of this period had to rely upon trial and error. Above all, the engineer was an empiricist, relying primarily upon observation and experience in the real world. Water control has always required creativity; the engineer has combined his assets and resources into workable projects and system designs.

These attributes remain a part of the water engineer's professional heritage. In the past century, they have been formalized and institutionalized in the schools and departments of civil engineering. Despite a postwar trend toward more scientific content in civil engineering curricula, the attributes developed and reinforced for over 5,000 years still constitute an important element of water engineering education and philosophy.

One trait which has characterized many civil engineering minds has been referred to as a "monument syndrome." This trait accounts for a great deal of disagreement between the engineer and the economist. The engineer whose pulse quickens at the sight and sound of a Hoover Dam can be excused, perhaps, for wondering how anything so patently efficient and majestic could be questioned as a public investment. An example of the tendency toward inspirational hyperbole is seen in the following quotation from a paper given several years ago by a representative of the Corps of Engineers:

> Wise King Solomon was inspired to write in Proverbs 25:2-3, 'It is the glory of God to conceal a thing but the honor of kings is to search out a matter. The heaven for height, and the earth for depth and the heart of kings is unsearchable.' This the Corps of Engineers is doing in the 'Land of Opportunity' and all throughout this great land of ours.[10]

Today in the United States most of the federally sponsored water resource projects are designed by three "construction agencies": The Corps of Engineers (Department of Defense), The Bureau of Reclamation (Department of the Interior), and The Soil Conservation Service (Department of Agriculture). The first two agencies, and to a lesser degree the third, are heavily engineering oriented. Projects sponsored by these agencies are subject to what some writers term "heavy political involvement," (or to what others refer to as "pork barrel" legislation.) All three agencies have their work authorized on a project by project basis. A succinct description of the decision-making process associated with these agencies has been written by Clawson and Fox of Resources for the Future:

> Water development projects typically start in field offices of federal agencies, flow upward through the central bureau offices, outward to other federal and state agencies for review, and finally upward again for approval by the President and Congress. Only rarely is more than one alternative presented for consideration; thus, no real choice is possible at any stage. Various projects, particularly if not in the same river basin, typically move forward separately, without reference to similar projects elsewhere. Projects often acquire active local political support during their development stage, thus making impartial decisions on their merits more difficult at later stages.[11]

An Historical Perspective on Economics

Economics as a modern discipline may be said to have started with the writings of Adam Smith in the latter part of the 18th century. Unlike water control engineering, it is a product of modern times. Following Smith, a number of other economists--men like Ricardo, Mill, and Marshall--developed the "classical school" of western economic thought during the 19th century. These men acquired great and lasting intellectual stature because they were essentially model builders, deducing general theoretical structures from certain assumptions. Their models gave the world insights into economic processes and made economics pre-eminent among the social sciences as a theoretically based and highly structured discipline.

From the very beginning economics--in contrast to water engineering--was theoretical, deductive and highly abstract. By making certain simplifying assumptions about the real world, the classical economist conceptualized powerful logical constructs, such as the "market." He was willing, as all model builders must be, to simplify reality when necessary in order to make deductions about it.

A fundamental characteristic of economics from the very beginning has been its concern with maximizing behavior, both in business and government. With the generation of the first national income data and the development of macro-economics in the 1920's, the field broadened and added more empirical elements--not replacing, but supplementing and refining the earlier deductive mode. After the depression struck in the 1930's, the Keynesian "revolution" followed, and economics became an important decision-making tool for social and political policy at the national level. The tenets of welfare economic theory were involved with a policy objective of maximizing national income or national welfare. Until the Second World War there were few applications of economic theory to water resource problems. These few exceptions were mainly in agricultural economics. Then, during the 1950's and 1960's, a small but highly regarded group of academic economists began to take an interest in water resources. (During this period, certain geographers, political scientists and other social scientists also began to turn their attention to water problems.)

The economic theoretician approached the problem of benefit-cost analysis from the standpoint of welfare theory. The cardinal question for him was how to approach an efficient allocation of water resource investment, given the objective of increasing or maximizing national income. Other objectives were not overlooked, but economic evaluations for public agencies have always involved national income as a criterion. (This criterion, of course, is far different from the earlier project formulae used by the construction agencies and based on engineering economy practices.)

During the 1950's and 1960's, a growing literature appeared by a number of prominent economists and other social scientists. The economists did not always agree among themselves on fundamental questions, and a healthy controversy arose over such things as the use of indirect (secondary) benefits, and the degree to which intangible benefits might be considered. However, the literature and the ideas of these social scientists did not easily penetrate the water engineering profession. (Recall the engineer's 1961 statement, quoted above.)[12] Partly due to this disinterest or resistance, but also partly due to the economist's reluctance to speak to the engineer in his terms, the approaches of water engineering and economic theory have remained essentially different over the past two decades. (A notable exception here has been the use for 20 years of teams of engineers and economists by the World Bank for the evaluation of potential water projects overseas.)[13]

Generally speaking, then, water engineering remains today--after 5,000 years--empirical, project oriented and pragmatic; on the other hand, theoretical economics in its approach to water problems remains highly abstract and oriented in terms of national efficiency criteria. Engineers still very largely address themselves to questions of project or system efficiency, while the economist asks the question "Is the project worthwhile in relation to alternative national investments?" The economists' question is a vitally important one to society, but it is a question which is very difficult to answer. Since we are a highly pragmatic people, the question goes largely unanswered, while engineers and politicians go on collaborating on water planning and project development.

Having said this, it must also be said that some limited but very significant areas of convergence have been developing between the two fields over the past two decades. These will be discussed later. But first, I want to mention some important influences which economists have been having on the water field in general, in recent years.

Some Fundamental Contributions of Economics

A question naturally arises from the discussion so far: if the language, concepts, and methodology of economic theory appear in many ways to be incompatible with engineering economy, what value is it in water decision-making? This can best be answered briefly, I think, by pointing out what I consider to be several fundamental contributions. A great source of confusion in the water field in recent years has been the difference in the concepts of benefit-cost analysis and the role of interest held by practicing engineers and economists. I mentioned before that the accounting or engineering economy procedure called "benefit-cost analysis" for project evaluation had been employed by Federal construction agencies for several decades prior to 1950. The Corps of Engineers led the way with these procedures. However, as R. J. Hammond points out, this was really an administrator's device which owed nothing in its origins to economic theory.[14]

To the theoretical economist, benefit-cost analysis must fall within the overall theoretical structure of welfare economics. At the heart of this structure lies the notion of discounting benefits and costs over time; and this notion involves the highly subtle concept of the role of interest rates within the national economy. The very different concepts of interest and prices in engineering economy and economic theory is partly responsible for the deep intellectual separation between the two fields. The engineer treats prices as a constant, while the economist considers them a variable. The engineer traditionally is interested in low interest rates because this will depress future costs and thus gives his project a higher benefit-cost ratio. The economist wants to use an interest rate which will reflect the possible alternative uses of the resources for the project; invariably this is much higher than the interest rate charged on federally sponsored water projects.

Professor Boulding has described this elegantly in one of his poetic expressions:

> Around the mysteries of finance
> We must perform a ritual dance
> Because the long-term interest rate
> Determines any project's fate:
> At two percent the case is clear,
> At three some sneaking doubts appear,
> At four, it draws its final breath
> While five percent is certain death.[15]

Economic theory then, can be used in a constructively critical way to point out questionable public investment policies in water. An example is R. H. Haveman's study of Corps of Engineers projects in 10 Southern States. Haveman concluded that a great number of projects were constructed which, if economic efficiency had been the sole objective, would not have been built.

> According to this approach then, 63 of 147 projects representing \$1,169,000,000 of committed Federal funds or about 44 percent of the total are devoted to projects which should not have been undertaken; the construction of these projects has led to a misallocation of national resources and economic waste.[16]

An example of a critical economic analysis of public water investment at the State level is the study by Hirshleifer, DeHaven and Milliman, of the multi-billion dollar Feather River project in California.[17]

These men are representative of the small but growing number of economists and other social scientists who have been making contributions in the water field over the past two decades. They have been quiet voices of reason speaking on matters relating to planning and investment in public water projects. Writing primarily in scholarly journals, they have largely gone unheard by the public--and by many of the professionals making decisions on water as well. They are hard to discern over the clamor and polemic on the water question very often generated by the mass media from non-scholarly sources.

Virtually all of these voices of reason aver that water in the United States is priced too low, and is therefore used extravagantly and not in the best interest of the general public. It has been a primary role of economics to call attention to the extravagant heritage of water use in the nation and to attempt to persuade both planners and the public to view water as an economic good to be used, insofar as possible, in maximizing the national welfare.

Growing Areas of Convergence between the Fields

Despite the persistence of a general separation between water engineering practice and academic economics in this country, a number of areas of convergence have been developing between the two fields over the past two decades. Much of this convergence is the result of research efforts and teaching at the graduate level in universities.

Academic departments in both engineering and economics have adopted a number of post-war mathematical and scientific tools and methodologies, such as mathematical modeling, systems analysis, operations research, linear and dynamic programming, and simulation studies and other computer applications. In learning to use these powerful new methods of analysis and design, the modern students in engineering and economics have attained a common language, a common set of analytical concepts, and a common set of methodological strategies. A notion of this convergence in its strongest form can be gained from the description of a course taught by Myron Fiering at Harvard. The description reads, in part:

> This course will treat the construction of linear and non-linear mathematical representations of decisionmaking processes. The interdependencies between engineering and economic analysis will be developed. . . . The use of computing machinery will be demonstrated. . . . The algebra of linear programming will be discussed in detail.[18]

Over a decade ago Walter Lynn of the School of Civil Engineering at Cornell began a productive research collaboration with mathematician-economist A. Charnes of Northwestern on system studies in sanitary engineering. Other examples of the convergence can be drawn from our own seminar. Several members, including Professor Loucks of Cornell and Professor Major of M.I.T., have been making important contributions in this direction. A number of universities have important programs--mostly of recent origin--also leading in this direction. However, in my opinion, the most original and fundamental contributions to the convergence have come from the Harvard Water Research Program, which has been operating since the mid-fifties. The Harvard Program, conceived by Arthur Maass, Professor of Government, brought together academic economists and engineers from Harvard and elsewhere for a multi-disciplinary intellectual confrontation. But the effectiveness of the Harvard Program was greatly enhanced by inviting engineering practitioners with long experience from the Corps of Engineers and other government agencies to participate. Over the past decade, academicians educated in the Harvard Water Research Program have gone to a number of other schools where they are establishing new centers of influence. In addition to the universities, the convergence between academic economics and water engineering is being stimulated in other places. The research organization, Resources for the Future, in Washington, is one.

Implications of the Convergence for Broader Environmental Problems

This nation appears to be crossing a threshold with respect to its perception of, and political commitment to, a growing set of environmental problems. This healthy new national concern for ecological survival, however, moves us into largely uncharted waters. The complexity of the water resource decision-making process is described by Dr. Luna Leopold of the Geological Survey:

> The best that can be hoped for is an unending series of decisions on water problems, each complicated by the cultural pattern and water development practices inherited from the past, and each tending to complicate the decisions that will have to be made in the future.[19]

But this relates to water alone; the decision processes required for future approaches to the man-environment problems, which are growing more critical year by year as we accelerate into the future, appear to be formidable indeed. Not a small part of the task ahead will be the introduction into the decision process of factors which have been difficult or impossible to measure quantitatively. The growing mood of society, especially the younger people who will be most affected, is to place heavier emphasis on esthetic and other intangible factors in evaluating the total quality of the environment. Criticism has begun to mount from within the federal government itself to relax strict economic efficiency criteria. Last year, for example, a report from the General Accounting Office strongly criticized one of the water construction agencies for violating new Federal fish and wildlife proscriptions.

There have even been arguments put forward by some persons to eliminate the benefit-cost analysis altogether. In my opinion, this is very bad advice. It is in the best interests of society to retain the concept of the benefit-cost analysis, but to study ways in which it can be extended and modified to include indirect (secondary) benefits and intangible benefits wherever possible and appropriate. Probably the most important thing to be done is to find ways to measure, or at least to better express the possible long-range indirect and intangible <u>costs</u> involved in resource development. Much of the soul searching we are doing now as a nation results from past ignorance of, or indifference to these costs and to their long-range effect on man's physical environment. It is not generally recognized that--as incom- plete and imperfect as it may be--modern benefit-cost analysis has been more extensively applied in the field of water resources than in any other area of public investment in this country. In recent years two conferences have been held in Washington for the purpose of using the experience from the water field to design decision-making procedures for other areas of public investment.

It is my suggestion that the experience gained over the past 20 years from the confrontation of engineering practice and academic economics in the water field can be used as a guide in establishing decision-making procedures to better cope with the growing complexity of the evolving man-environment problems. Inspiration and intellectual leadership for this movement must come from the universities. The Harvard Water Research Group can, I feel, be used as a model for a similar but broader-gauged study of decision-making for ecological survival. For the cost of one guided missile cruiser, a program of this kind could be started and amply supported for years.

The associated problems of resource development and environmental pollution control seem formidable as we accelerate into the future. We can only trust that the creative mind of man, which for over 5,000 years has met and overcome an increasingly complex set of water resource needs, will continue to find solutions to man's growing planetary challenges.

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POLLUTION OF THE RHINE RIVER AND ENVIRONMENTAL PROTECTION PROBLEMS IN THE RUHR AREA

by

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Introduction

This is a survey of the pollution problems in the Rhine basin, and especially in the densely populated and highly industrialized section of North-Rhine-Westfalia. The problems involved indicate that both the population and industry will have to spend more money, both in coping with pollution and in using the natural water resources in a more economic way.

The section of the River Rhine in the Ruhr area is relatively short compared to the total length of this water course. Therefore, a brief survey of the complete Rhine watershed area will be given before the pollution of the Rhine River in the Ruhr district is treated. Then, another restriction has to be made. The Ruhr area represents a zone of extreme agglomeration and there are no fixed boundaries. Originally, the name of the Ruhr district was confined to a region of dense industrialization in the area of the city of Essen on the Ruhr River. Today, however, the industries have been expanded further to the North and West, and a further development of the chemical industries has resulted in additional zones of agglomeration in the region of Köln-Leverkusen merging into the Ruhr area.

It is interesting that the Rhine River is only, to a negligible extent polluted by the Ruhr River itself.

For a better analysis of the pollution of the Rhine River, the examination should include the whole state of North-Rhine-Westfalia, which includes the cities of Duisburg, Düsseldorf, Köln, Bonn, and Leverkusen. The pollution caused in this federal state, which is the last state of Germany along the Rhine River before entering into the Netherlands, may not be considered exclusively; it is necessary to make a survey about the rest of the Rhine watershed area, especially, the Moselle and the Saar Rivers; area of Frankfurt with the mouth of the Main River; the Mannheim-Ludwigshafen area with the mouth of the Neckar River; the Basel-Strassbourg area and the Alsace.

The Rhine River is the most important waterway and has the largest water resources in Europe. Seven European countries share the valley of the Rhine River: Belgium, France, Luxembourg, the Netherlands, Switzerland, Austria, and Germany. The problems related to the international water law will be discussed only to that extent which is necessary for the understanding of the technical and economic structure.

Geographical and Hydrological Conditions

The River Rhine as "Alpenrhein," rises from the St. Gotthardt glaciers in the Western Alps and reaches Lake Constance, which has an area of 210 square miles (545 km^2) and has an equalizing effect on the discharge for the lower section of the River Rhine. Lake Constance is also important as a source water supply to Southern Germany, Switzerland, and Austria.

A comprehensive research program was started in 1960 to develop protective measures for the lake, because the water quality was steadily worsening due to the increased inflow of waste water. This program calls for draining waste water through pipelines around the lake to a water treatment plant before the water is discharged into the lake.

Downstream of Lake Constance, the River Rhine runs, as "Hochrhein," through the narrow and deep valley between the Jura Mountains and the Black Forest, to Basle and is mainly used to generate hydropower. In the midway of the reach, tributary Aare River doubles its discharge.

Between the cities of Basle and Breisach, the river flows through the Rhine-Lateral-Canal in French territory. In the natural river bed, only a small discharge is left. Receiving some smaller tributaries from the Western part of the Black Forest and from the Eastern slopes of the Vosges Mountains, the River Rhine is joined by two major tributaries: the Neckar River at the city of Mannheim and the Main River between the Odenwald and Taunus Mountains.

Until it reaches Karlsruhe, the River Rhine marks the border between France and Germany. From this point to Emmerich, it passes completely through German territory and then runs through Dutch territory into the North Sea. The River Rhine has a length of 700 miles (1070 km) and covers a watershed of 62,000 square miles (160,000 km^2), with the exception of its estuary. From Rheinfelden in Switzerland to its mouth, the Rhine is navigable over a length of 560 miles (900 km).

Leaving the Alsacian basin, the River Rhine crosses a range of slaty mountains receiving the Nahe, Lahn, and Moselle Rivers. Below Köln, the confluenting rivers are the Sieg, Wupper, Ruhr, Emscher, and Lippe Rivers.

The average precipitation in the whole watershed is 35 inches (900 mm) per year in comparison to 27 inches (690 mm) for Germany as a whole. The highest values in precipitation occur in the Alpine region, amounting to 100 inches (2500 mm). The main part of the yearly precipitation is retained in firns and glaciers during winter. There melts during summer and cause floods. On the other hand, the flow from the subalpine mountains and the lowlands is low during summer, as most of the precipitation is transpirated by the vegetation; however, the major part of the precipitation flows as surface run-off from the wet and frozen soil.

These factors equalize the hydrograph of the River Rhine over the year. Furthermore, the change of discharge is reduced because it flows through several large lakes, such as Lake Constance, in the foothills of the Alps. The discharge ratio between lowest lowflow and highest flood is 1 : 16 at Basle, and 1 : 20 at Emerich, near the Dutch border. Leaving Switzerland, the Rhine River has an average discharge of 37,100 cfs (1,050 m^3/s); at the Dutch border it is 76,100 cfs (2,150 m^3/s), the lowest discharge being 9,600 cfs (272 m^3/s) at the Swiss border and 21,200 cfs (600 m^3/s) when it enters the Netherlands.

The rich and steady discharge of this stream created very favorable conditions and resulted in the Rhine's becoming an important traffic route, with prospering towns on its bank even in early times.

The average density of population is 635 inhabitants per square mile (250 inh./km^2), in the total Rhine area. Its lowest value is 330 inhabitants per square mile (130 inh./km^2) in North-Rhine-Westfalia, which will be of special interest to us later.

The Importance of the River Rhine

The River Rhine is the most important inland waterway in Europe. In 1967, the total goods traffic on the River Rhine and on its navigable tributaries reached the figure of 200 million tons. From this amount, 49 million tons were shipped upstream and 24 million tons downstream at Emerich, near the Dutch-German border. The intensity of traffic on the River Rhine is 18,000 boats per year.

In 1817, improvement works downstream of Basle had been started by Tulla in order to make the stream navigable up to Basle. These correction works took 50 years. Because of this comprehensive river-training work, the plains became protected against floods and so could be used for agricultural purposes. At the same time, the malaria disappeared in this region. Bed erosions up to 23 feet (7 m) deep were a resulting disadvantage of this measure, because the average sediment of 15.5 million cubic feet (440,000 m^3) now no longer was eroded from a 2.5 miles (4 km) wide bed of the meandering natural stream, but from a deepened new river bed having a width of only 650 feet (200 m). Today, several barrages have been constructed to prevent further erosion.

The increasing sizes of vessels made it evident that the water depth was not sufficient in other places. In order to improve the condition of navigation, several rocks were blasted away from the river bed between the towns of Oberwesel and St. Goar; the reefs that had to be removed from the waterway were situated near the Lorelei Rock.

The conditions in the Lower Rhine are similar to those in the Upper Rhine. The erosion was gradually shifted from the embankments to the bottom in this region, too.

The River Rhine, furthermore, is a large water resource. In the Rhine area, the drinking water supply comes from surface water or filtrated river water to an increasing extent. The total demand for drinking water amounts to 506 billion gallons (1,600 million m^3) per year in the basin of the River Rhine in Germany.

In North-Rhine-Westfalia, for the 12.1 million inhabitants living in the watershed of the River Rhine, 175 billion gallons (660 million m^3) of drinking water per year are needed, which are taken from the surface flow to a great extent.

Sixty-four billion gallons (240 million m^3) water per year are taken from the River Rhine, the Lippe and the Ruhr Rivers and are treated for drinking purposes; the rest of the needed water is secured by direct filtration from the water of the Ruhr River, before it reaches the Rhine.

In addition to the previously mentioned demand, industry needs large quantities of surface water for processes of cooling and production. The total water demand of industry amounts to 4,000 billion gallons (15,000 million m^3) per year in the watershed of the River Rhine on German territory. North-Rhine-Westfalia shares nearly 1/3 or 1,450 billion gallons (5,500 million m^3) per year from which the largest portion, 930 billion gallons (3,500 million m^3) per year, are withdrawn from the River Rhine.

The remaining part is taken from the Ruhr and Lippe Rivers and the canal system in West Germany.

Water is needed for irrigation in addition to drinking water. Although the Rhine area generally is rich in precipitation, there are regions, especially in the fertile plains of the Upper Rhine, with an average precipitation of 24 inches (600 mm) and in parts even less then 20 inches (500 mm) per year. Irrigation is essential for an intensive agriculture in these regions. The total demand for irrigation is 64 billion gallons (240 million m^3) per year for the German part of the River Rhine. In North-Rhine-Westfalia 5 billion gallons (19.5 million m^3) are needed to irrigate 37,000 acres (15,000 ha.). The demand will be increased to 12 billion gallons (45.5 million m^3) in the next decades.

The water quality of the River Rhine is considerably affected by the inflow of waste water. A survey of the waste-water treatment is given in Figure No. 2. Main areas of concentration of industry and population are the Saar district, the Ludwigshafen-Mannheim region, Rhine-Main area and the highly industrialized districts in North-Rhine-Westfalia, just to mention a few.

The River Rhine, thanks to its high discharges, is capable of coping with organic pollution. In case of excess concentration of pollution, the oxygen content in the water decreases and the quality of the water deteriorates to a threshold value that can result in the death of fish and initiation of the process of putrefaction.

The process of purification is often negatively influenced by noxious or toxic substances. Furthermore, chemical reactions that consume oxygen have a harmful effect on the water quality. Other substances as with a high concentration of ferrum fallout, in the form of bottom sludge impairing or making impossible the growth of organisms, are responsible for the lack of purity. There are chemical compounds affecting or killing the organisms useful for purification and thus almost completely stop the purification process.

Finally, the River Rhine has attractive scenery and serves recreation and public health purposes. The River Rhine and its confluences are attractive regions for vacation. But another social problem arises because the number of fishermen has increased considerably in recent years though the fisheries steadily declined due to increased pollution.

The Water Quality of the River Rhine

Leaving Lake Constance, the water of the River Rhine is relatively pure. The inflow of waste water coming from villages and towns and smaller industries has hardly any effect on the pollution of the "Hochrhein." The stronger organic load of the Aare River is largely decomposed and the water quality in the "Hochrhein" can be regarded as good.

The mapping is based on the biological status of river waters at medium low flow, using the saprobic stages. In addition to that, chemical data are taken into consideration--especially the oxygen balance of water resulting in four classifications of pollution.

The Upper Rhine, at the beginning, is charged with the inflow of industrial wastes and domestic sewage from the Basle-area. Further downstream, the waste water from the cities of Karlsruhe and Strassbourg and several smaller inflows are added. These loads are mainly decomposed in the reach of the Upper Rhine up to Mannheim. In the Mannheim-Ludwigshafen-area, the sewage of these two cities, the waste of the chemical industry and wood pulp factories impose a strong pollution on the river. The inflow of the Neckar River and the sewage water of the Darmstadt area are an additional pollution. The strong pollution of the River Rhine near Frankfurt is mainly caused by the Main River.

In the following mountainous reach, the water of the River Rhine is slightly purified because of the faster and more turbulent streamflow. Entering North-Rhine-Westfalia, the River Rhine is again polluted by industrial waste water and domestic sewage.

The River Rhine is charged with mineral oils originating mainly from the navigation. Cities such as Bonn, Köln, Düsseldorf, Krefeld, and Duisburg and a series of large industrial plants bring into the River Rhine sewage only partially treated or not treated at all. Leaving the industrial area, the stream is heavily polluted. In the reach downstream of Wesel, there is a slight improvement, but the waste-load with its dissolved or undissolved substances has still high oxygen consumption (BOD) and is quite noticeable at the Dutch border.

In contrast to the organic pollution, which is reduced by the purification of the river, the inorganic load is maintained.

The chloride concentration is significant for the inorganic pollution. It is very low in the Upper Rhine, up to Kembs. A strong increase in the chloride concentration is caused by the inflow of waste water originating from the potash factories of the Alsace. From Mannheim to the mouth of the Mosel River, the chloride concentration is slightly increased by the inflow of industrial waste water and domestic sewage.

From this point to Köln, the growth of chloride can be explained mainly by the inflow of the Moselle River, and industrial waste water.

In the reach from Cologne to the Ruhr River, several inflows of industrial origin cause additional charge to the stream. The sudden worsening in the reach of the mouth of the Ruhr River and the German border is exclusively caused by the inflow of the Emscher and Lippe Rivers.

This high concentration of chlorides has a very bad effect on the drinking water which is secured only by filtration. The Netherlands and many German cities on the River Rhine can only filter the water for their supply system. Sources for this pollution are the natural loads caused by erosion of soil and mud however, and have no permanent detrimental effect on the water quality.

A more serious influence is the load caused by the industries, especially by steel and coal mills and potash and chemical industry. Recently, attention has had to be given to the waste of nuclear energy plants: thermal and radioactive pollution.

The Sieg River forms the first main confluence in North-Rhine-Westfalia. The pollution of the Sieg River charges over its entire length. At the mouth, the water is moderately polluted and has little effect on the water quality of the River Rhine.

The situation is worse in the neighboring watershed of the Wupper River, with a population of some one million people. Numerous metalworking and chemical industries and some textile and food industries, as well as individual wood pulp and cellulose factories, are located in this district. Besides industrial waste, larger quantities of domestic sewage (approximately 0.5 million population equivalents), insufficiently treated, flow into the Wupper River. Therefore, the Wupper River is excessively polluted downstream of the city of Wuppertal. Thus, the water quality of the River Rhine is badly affected. A similar increase in pollution is caused by the inflow of the Erft River, which is also excessively contaminated.

Approximately 2.1 million inhabitants live in the watershed of the Ruhr River. The domestic sewage of 1.44 million inhabitants is biologically treated. A quantity of 530 million gallons (2 million m^3) of sewage is pumped into the watershed of the Emscher River. The sewage of the three cities Mühlheim, Oberhausen, and Duisburg is collected and directly piped through a regional sewage treatment plant to the Rhine River.

In the upper reach, the Ruhr River is moderately polluted. At some points, in the middle course, the water quality decreases and the river is intensely polluted. The water is purified by artificial lakes with the result that in the downstream reach, the pollution is only moderate; and improves the water quality of the Rhine River.

Approximately 2.8 million inhabitants live in the watershed of the Emscher River. The domestic sewage of 2.7 million inhabitants is treated by mechanical and biological sewage treatment plants. From the municipal area of the city of Dortmund, 1.5 billion gallons (5.8 million m^3) of domestic sewage is treated on sewage farms and then pumped to the Lippe River. Approximately 6,000 tons of phenols are produced in 19 phenol extraction plants.

The largest portion of pollution in the Emscher River comes from the numerous mines and coke factories discharging waste water with a high content of chlorides. The strong inflow of waste-water brings about an excessive pollution in the Emscher River, making fishlife impossible. The influence on the water quality of the River Rhine is very detrimental and it can be traced, still, at Wesel on the right bank at a distance of about 10 km from the Rhine.

Approximately 1.1 million inhabitants live in the area north of the Lippe River, there, surface water almost completely is treated in sewage treatment plants. Seven phenol extraction plants produce approximately 2,000 tons of phenol per year. An amount of 32 billion gallons (12 million m^3) of sewage water are pumped into the watershed of the Emscher River each year. Moreover, an average quantity of 580 billion gallons (220 million m^3) of river water of the Lippe River are pumped into the West German canal system and thus are lost for the watershed of the Rhine.

Besides the mining industries, several other industries and quite a number of large power plants are situated on the Lippe River, the latter discharging large quantities of heated water. The high concentration of chlorides originates partly from salt-bearing sources; however, it comes largely from the waste-water of the mining industries. The upper and middle reach are moderately polluted, and farther downstream the pollution is intense to extreme, being disadvantageous to the water quality of the River Rhine and raising the high concentration of chlorides in the river.

The effects of pollution are manifold. So far, about 30 different substances of organic pollution have been found. Moreover, the riverbed at some places is covered with a thick layer of oil which penetrates the natural filter of the bottom and is transported into the ground water. Recourse of this, the efficiency of the infiltration at the banks is affected. The high content of ferrum and manganese is disadvantageous to the operating of the filtration installations. So, quite expensive activated charcoal filters had to be used in order to avoid a bad taste in the water.

Detergents from washing and cleansing media, which reduce the surface tension, also have disadvantageous effects on the treatment of drinking water. A federal law on the decomposition of active detergent substances in washing and cleansing agents ensures that detergents are not a danger to the preservation of the purity of the water.

Examinations show that cancerogenic substances in the water of the River Rhine have not yet reached alarming proportions. However, the pollution of the water of the River Rhine is detrimental to public health, and at many places swimming is not permitted. Also, the navigation is affected to some extent, where the water in the River Rhine is used for cooling purposes (creating also thermal pollution) and where a high demand for boiler water occurs along the river. The increasing hardness of the water makes a frequent removal of boiler scale necessary.

The pollution of the water of the River Rhine is disadvantageous to agriculture insofar as dry areas affected by muddy matter of floods cannot be used for agriculture because the mud interferes with irrigation. Sensitive plants such as tobacco cannot sustain the high concentration of chlorides of the irrigation water. Downstream of the inflows of the Emscher and Lippe Rivers, the concentration of chlorides is raised to a degree which makes impossible any agricultural use.

Measures for the Preservation of Purity of the Water of the River Rhine

It is necessary to have a complete knowledge of all factors influencing the water quality to take measures for improvements. Therefore, after the water quality of the River Rhine and its tributaries gradually worsened, the authorities insisted on a detailed control. Today, at regular time intervals, water samples are taken from the River Rhine at specified places and tested according to standards. The date of sampling applies to the whole Rhine area and is specified by the International Commission for the Protection of the River Rhine against Pollution. In particular, the tests are mainly confined to pH-value, dissolved oxygen, BOD_5, ammonium, nitrate, chlorides, phenols, and radioactivity.

From the lower Rhine, large quantities of water are withdrawn for the supply of drinking water and fresh water. The authorities carry out an extensive test for mineral oil wastes and fuels which, in addition to the phenols, give a bad taste and smell to the water. Furthermore, the authorities investigate whether the inflows are in accordance with the rules and permits.

For planning on a large scale, the riparian states have joined the International Commission for the Protection of the River Rhine against Pollution, founded in 1963. The commission takes samples of water according to uniform rules and at regular time intervals. For this purpose, eleven stations have been established on the River Rhine and thirteen stations on the main tribu- taries and eight further stations at the mouth of smaller confluences. At each station, the samples are taken in the whole cross section of the river at intervals of roughly 6 feet.

Pollutions which cannot be avoided by individual waste-water treatment plants, such as harmful salt concentrations and oil-spill from navigation, have to be prevented by international measures. These means are applied in order to diminish the salt concentration. The salt originates 41% from the potash industries of the Alsace, France, 13% from the industries at the Moselle and Saar Rivers, 17% from German coal mines and 26% from the sodium and other industries. The remaining part comes from the cities and other communities.

Those parts of the salt concentrations originating from the potash factories can be retained in a relatively economical way. A research program started to test whether the salt can be stored on a waste heap. The total costs of the experiment amounting to over $250,000 have to be covered by all riparian states.

The water quality of the River Rhine must be improved to such a degree that:

a) It can be used for drinking water, after filtration

b) It can be used for industrial purposes without any purification, and

c) Conditions favorable for irrigation, fisheries and public health are provided

For the improvement of the River Rhine and its tributaries on German territory, $650 million (2,500 million DM) were invested in 1950-1959. In each of the following years, a proportional amount has been invested. For the period of 1963 to 1967, a second plan for the purification of the River Rhine was drafted, providing expenditures of $1.5 billion (5,500 million DM).

In the Upper Rhine area, the construction of waste water treatment plants is planned in larger cities. In North-Rhine-Westfalia, the cities of Koblenz, Cologne, Düsseldorf, Krefeld, and Duisburg biological treatment plants will be provided. The industries situated in this area take great effort to treat their waste water. Detailed plans for new biological treatment plants are being worked out.

So, in the Cologne area, a fairly large regional waste treatment plant of the "Farbwerke Bayer," the "Wupper River Authority," and other local authorities is under construction in order to eliminate solid and liquid wastes from the River Rhine and the Wupper River. The layout of the treatment plant is 35.5 million gallons (135,000 m^3) of waste water each day. The fresh sludge is deposited together with other solid waste of the factory on a waste heap. Furthermore, harmful waste of the industries is incinerated.

Simultaneously, the gradual completion of a biological treatment plant is being achieved, reaching a capacity of 9 million gallons (34,000 m^3) of waste water each day in the year 1970. After completion, the total waste water of the Bayer factories and the sewage of 375,000 people consuming 110 metric tons of BOD_5 per day will be treated. The cost for the regional treatment plant amounts to $55 million (200 million DM), the share of the Bayer factories being $40 million (145 million DM).

The cost of operation is expected to be $3 million (12 million DM).

Similar large treatment plants are under construction in the Ludwigshafen area by BASF and in the Frankfurt area by the "Farbwerke Hoechst".

A further decisive improvement of the Rhine water is expected with the construction of a large biological treatment plant at the mouth of the Emscher River. In the Emscher basin, with 2.8 million inhabitants, the production rate of coal and coke is 60% of the total German production. Furthermore, important steel mills and chemical industries are located in this area forming the focal point of the Ruhr district. The waste water situation is extremely delicate and critical. In addition to that, especially noxious industrial waste water is pumped from the neighboring Ruhr and Lippe basins into the Emscher River in order to relieve the Ruhr and Lippe Rivers. Formerly, in this area treatment plants were constructed in order to purify the whole discharge of creeks and smaller rivers. The largest plant of this type is the Emscher River Treatment Plant north of Essen, having been taken into operation in 1928 and controlling 75% of the Emscher basin.

The sewage treatment is closely related to the compositing or removal of sludge. Today, 700,000 metric tons of spadeable sludge have to be disposed of in the Emscher basin each year. Due to its composition, the sludge cannot be used for agricultural purposes. Today, there are scarcely any places for waste heaps in this densely populated area. Therefore, ways had to be found to withdraw water from the fresh sludge. Ninety percent of the sludge retained in the Emscher basin is artificially dried. The dried sludge, with a water content of forty percent, is immediately burned in a neighboring thermal power plant.

To improve the water quality, a large treatment plant is under construction. The capacity of the plant is a waste-water discharge of 720 cfs (20 m^3/s), which is adequate for 270 days operation each year.

A maximum of 1,060 cfs (30 m^3/s) of waste water can be biologically treated, which means that on fifty days per year, the total discharge of the Emscher River will not be biologically purified in the future. On the other hand, by this measure, it is guaranteed that in all cases where with respect to the flow in the River Rhine biological treatment is necessary, this will be done. The cost of the large treatment plant amounts to $40 million (145 million DM). After this plant goes into operation, the load will be reduced by the factor of 10.

The problems in the neighboring Ruhr district are different with respect to water supply and waste treatment. In the Ruhr basin with 6 million people, the demand for water is 450 billion gallons (1,700 million m^3) per year. In contrast to the Emscher and Lippe Rivers, which are used in their lower reaches for the transport of sewage, the water demand is covered to seventy percent by the Ruhr River, though the River Rhine is not very far away.

The reasons are: the quality of the water of the Ruhr River is better and the hardness is less than that of the River Rhine. The Ruhr River runs along the southern border of the industrial area. From the water resources to the centers of consumption, generally the minimum available flow is sufficient.

Today, 110 billion gallons (410 million m^3) of drinking water flow from the Ruhr basin to the basins of the Emscher, Lippe, Wupper, and Ems Rivers. For the purpose of water supply, numerous dams were constructed. Altogether, 320 billion gallons (1,200 million m^3) of water are withdrawn each year, of which forty percent are diverted to other basins.

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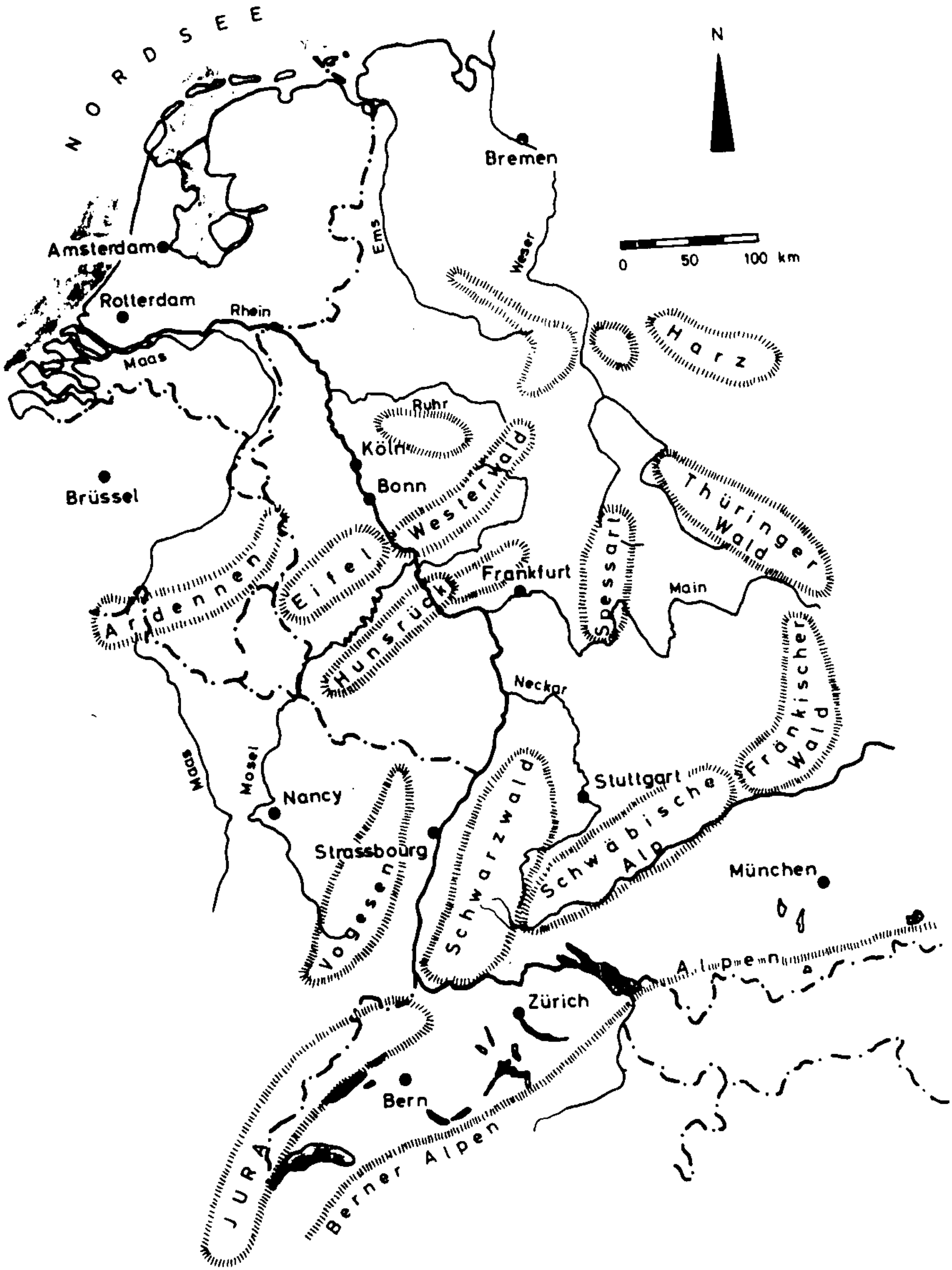

Figure 1. The River Rhine and its Tributaries.

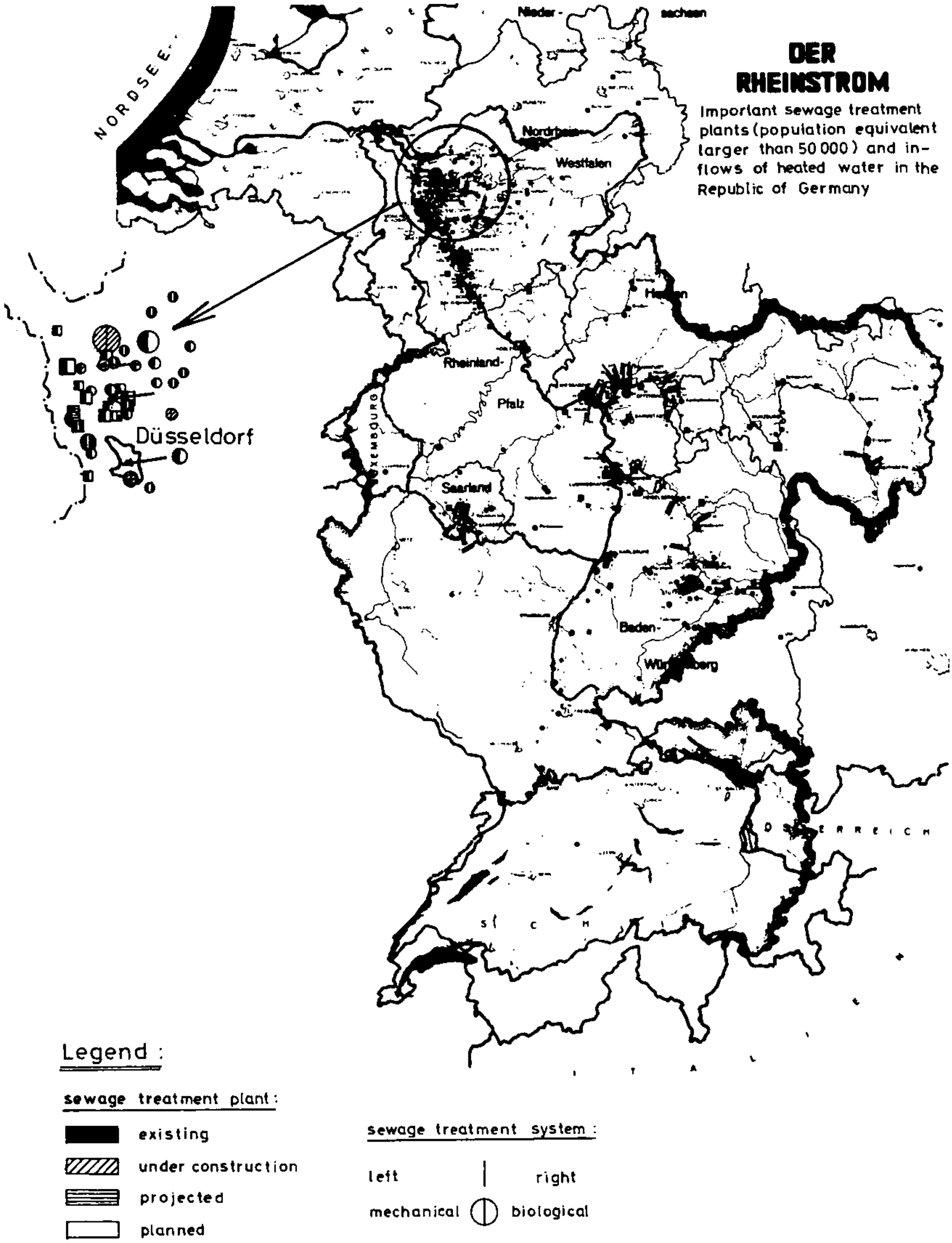

Figure 2. Important Sewage Treatment Plants of the Rhine.

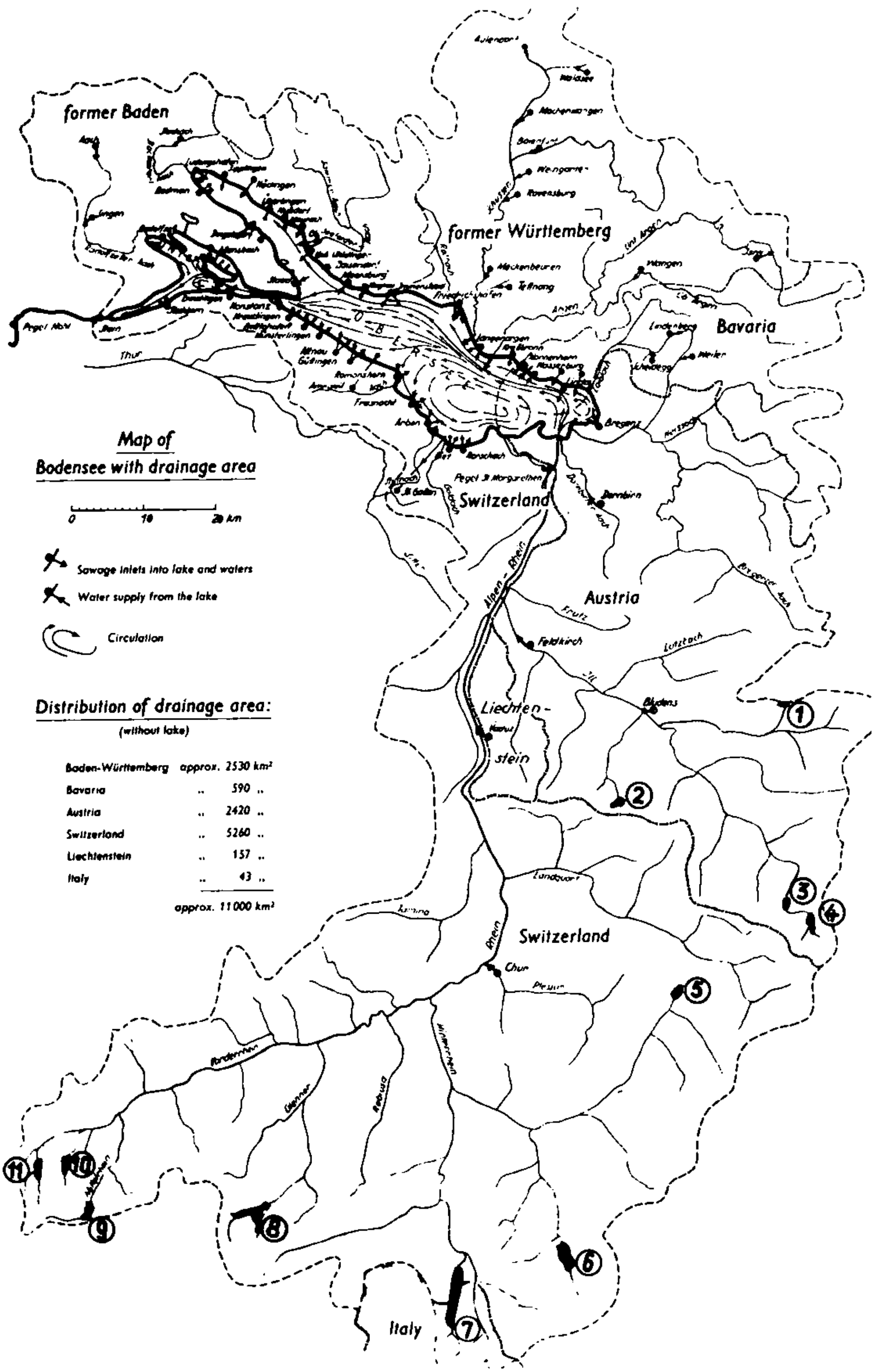

Figure 3. Map of Lake Constance (Bodensee) with Drainage Area.

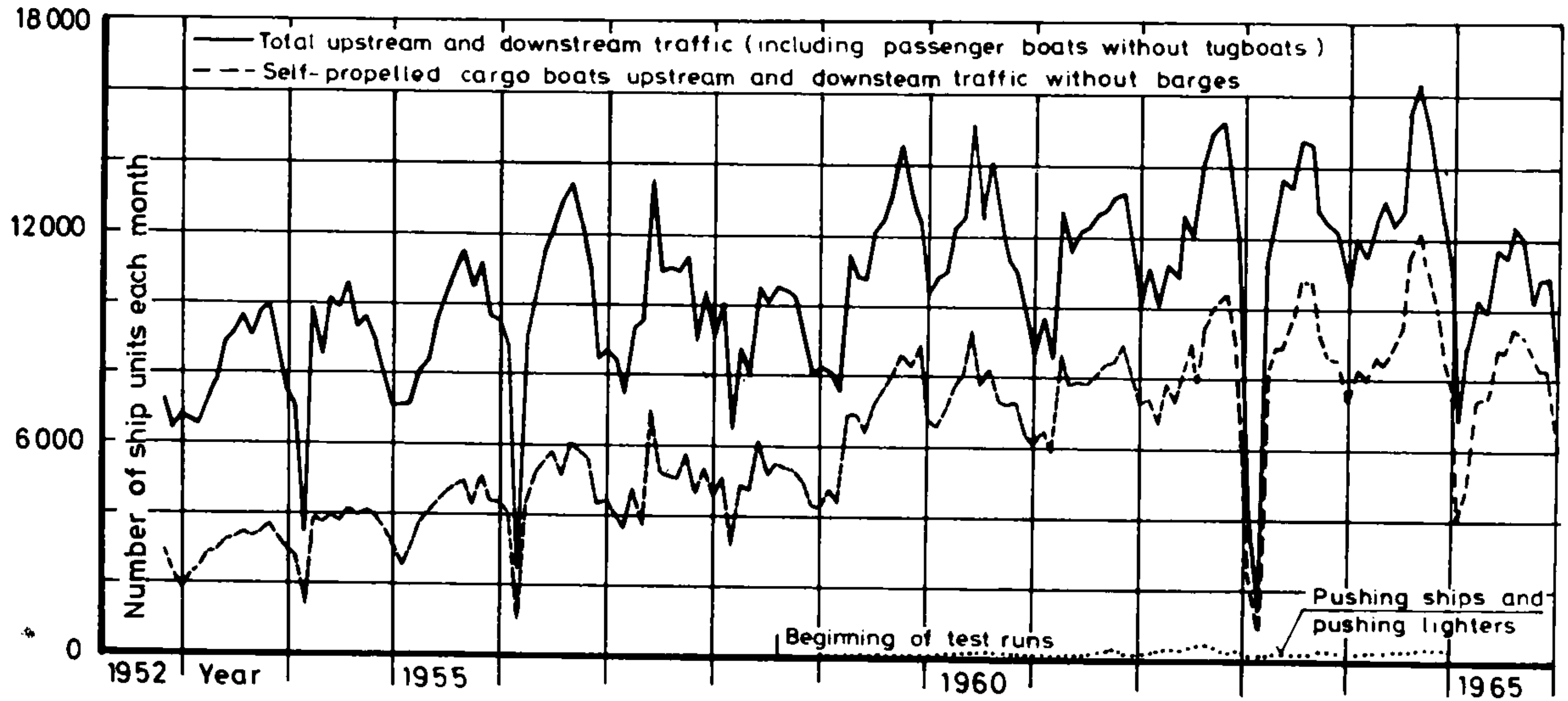

Figure 4. Ship Traffic on the River Rhine near Kaub.

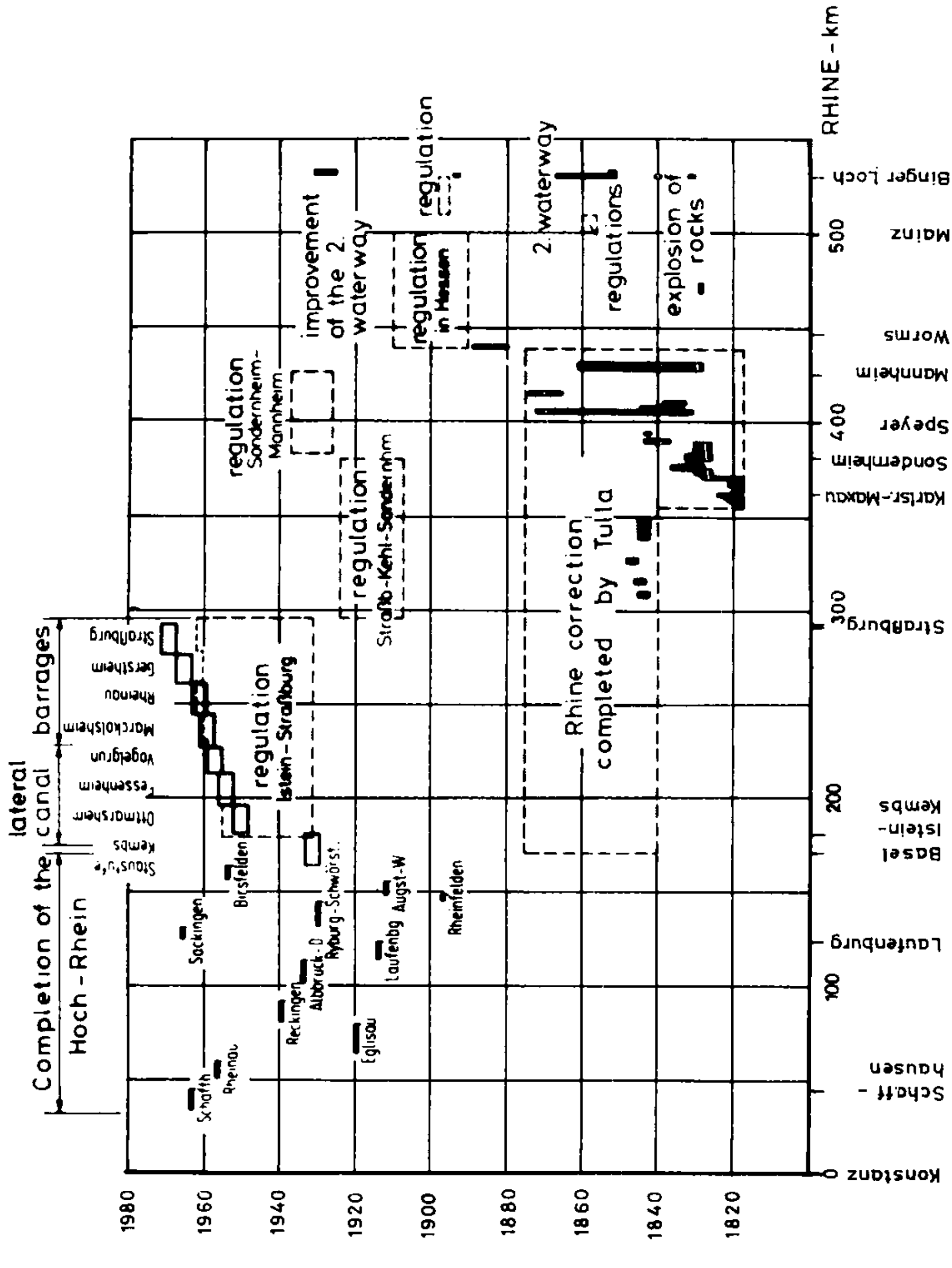

Figure 5. River Training Works.

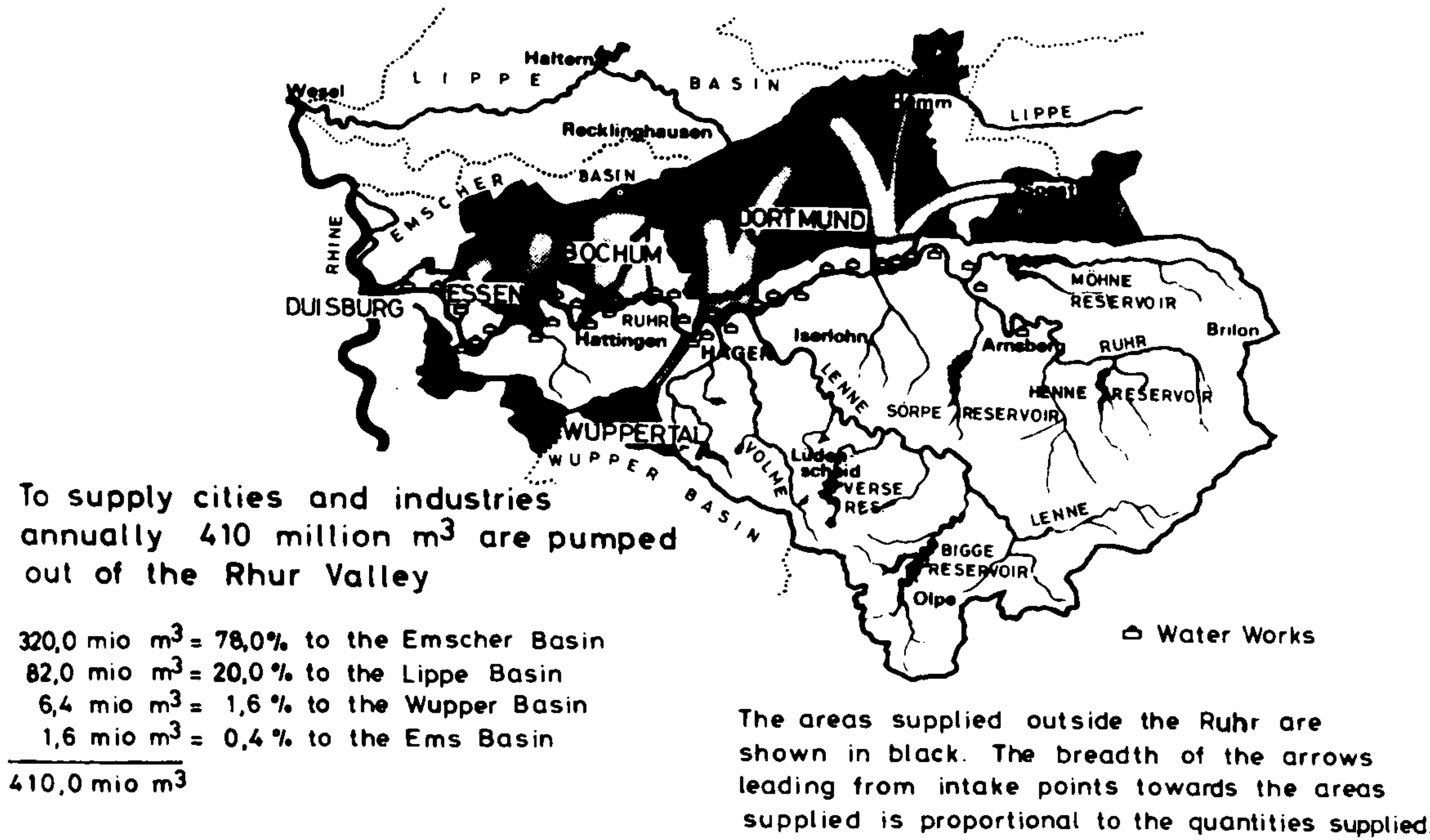

Figure 6. Water Supplied from the Ruhr.

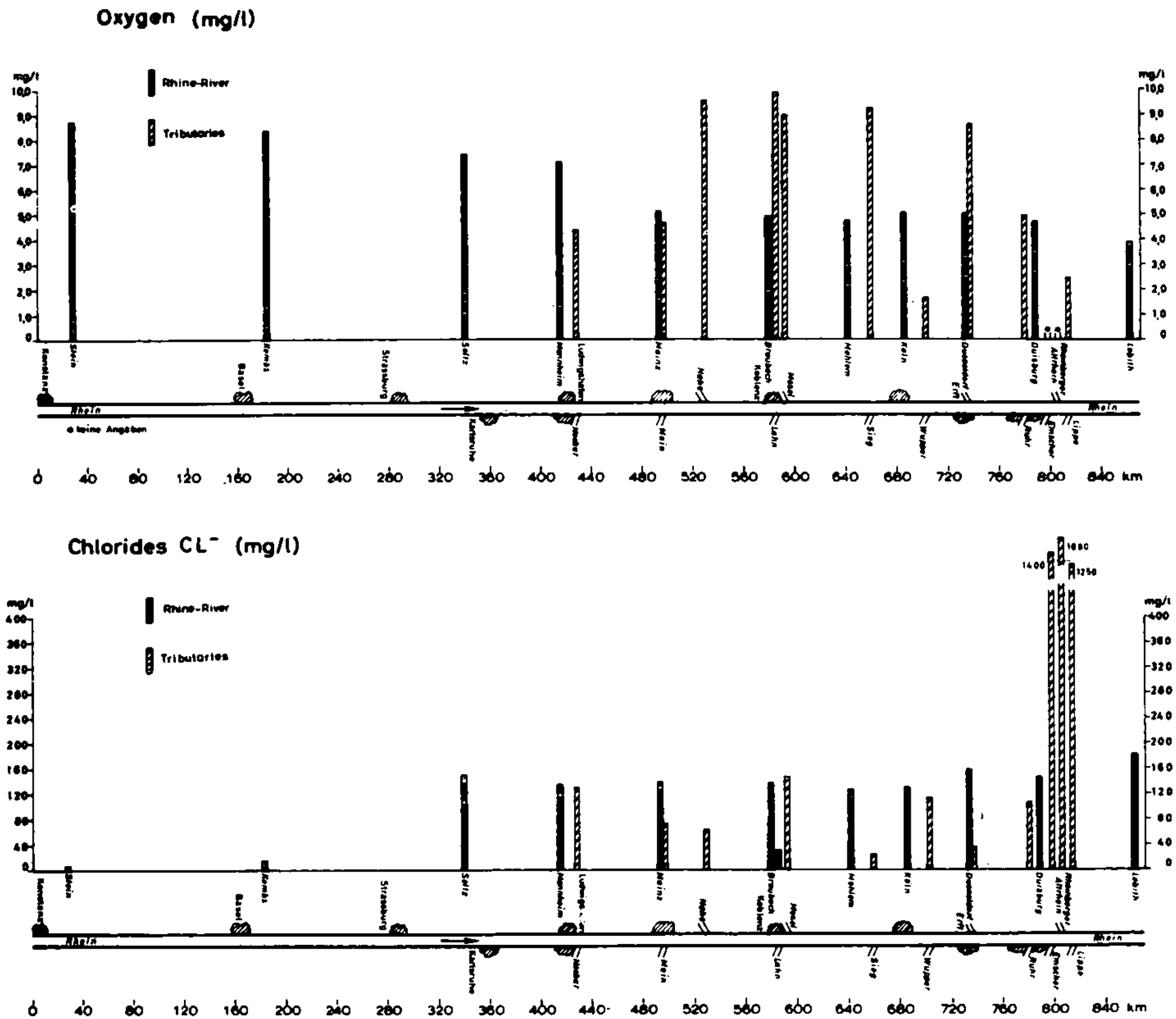

Figure 7. Oxygen (mg/c) and Chlorides CL(mg/c) Content of the Rhine and its Tributaries.

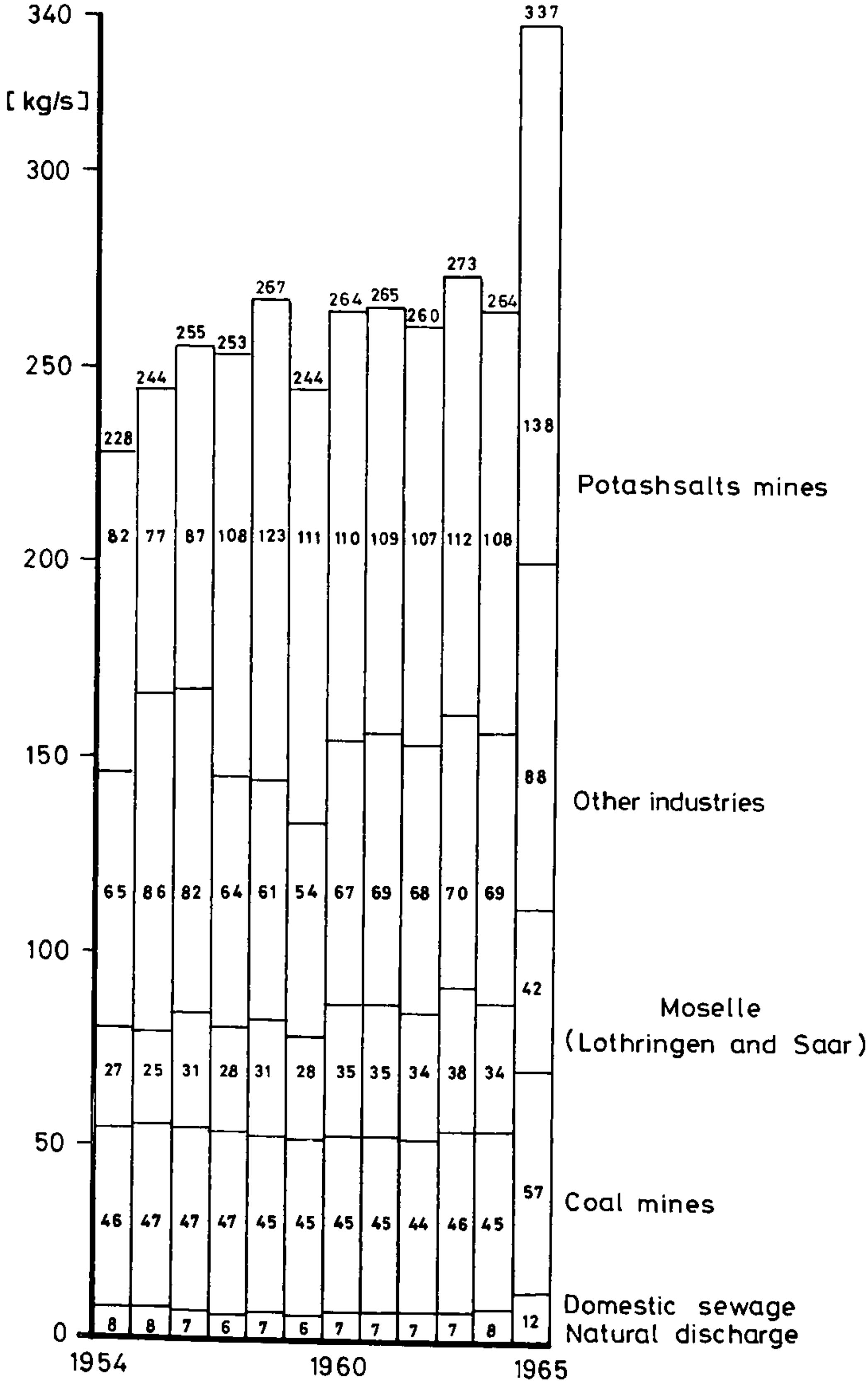

Figure 8. Concentration of Chlorides at Emerich.

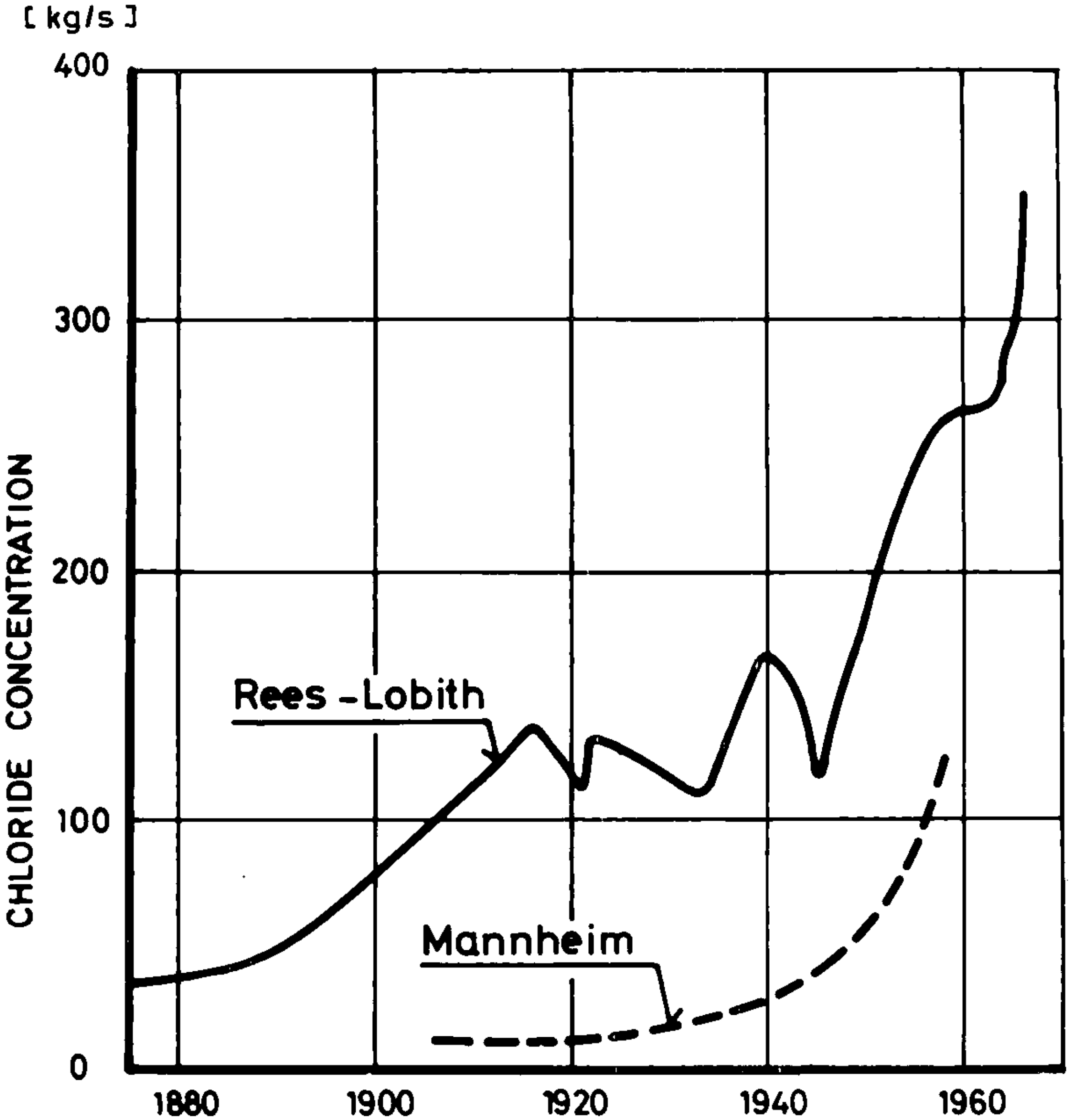

Figure 9. Annual Averages of the Amount of Chloride - Ions at Lobith.

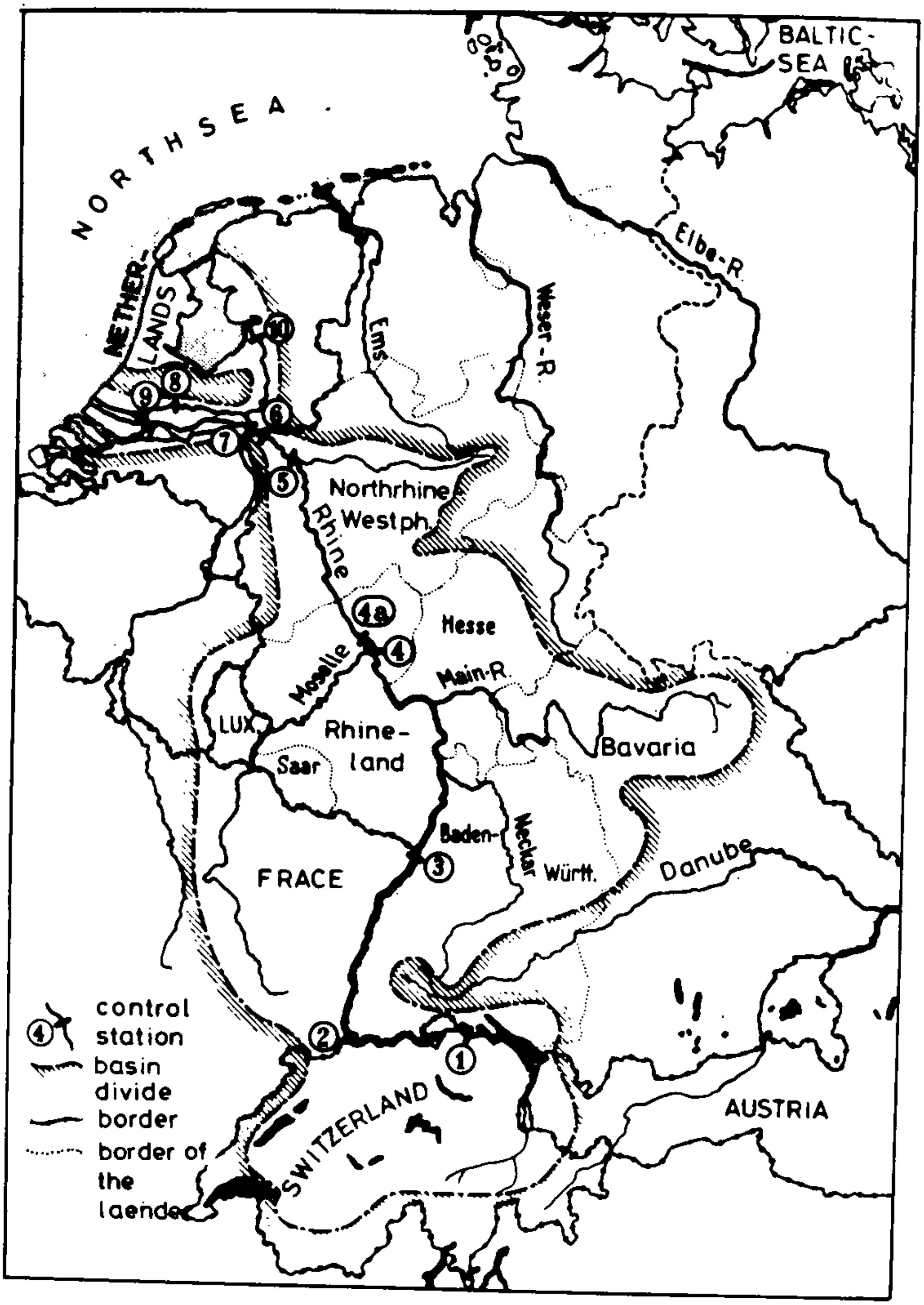

Figure 10. Control Stations of the International Commission.

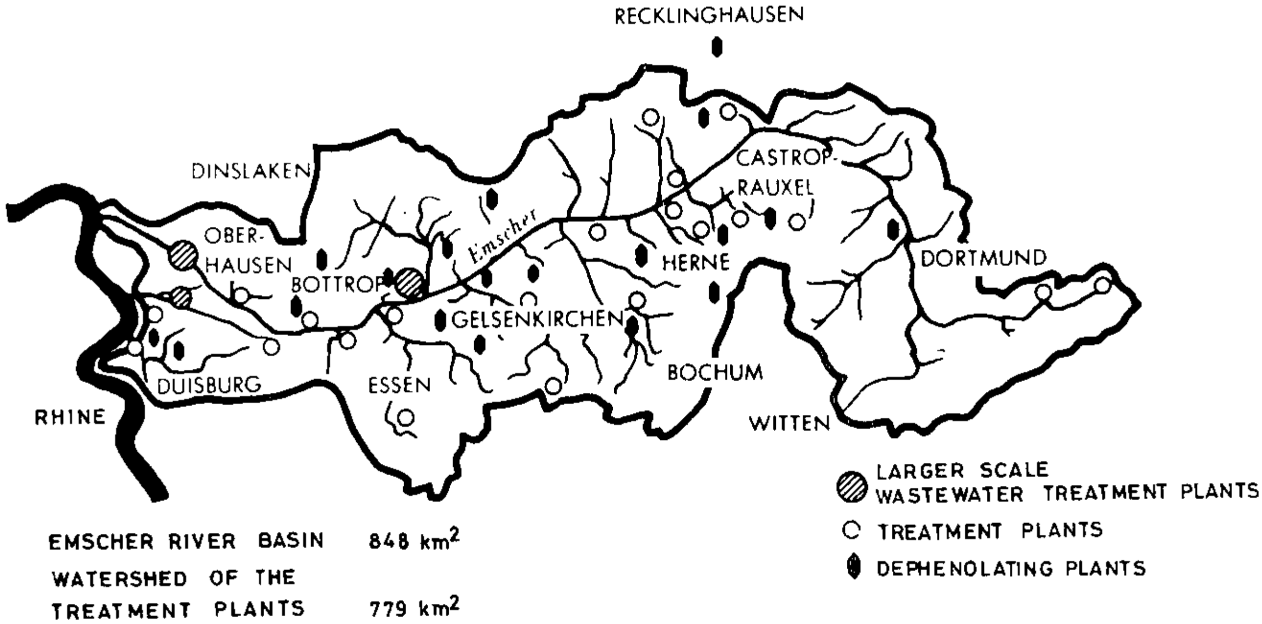

Figure 11. The Emscher River Basin and its Treatment Plants.

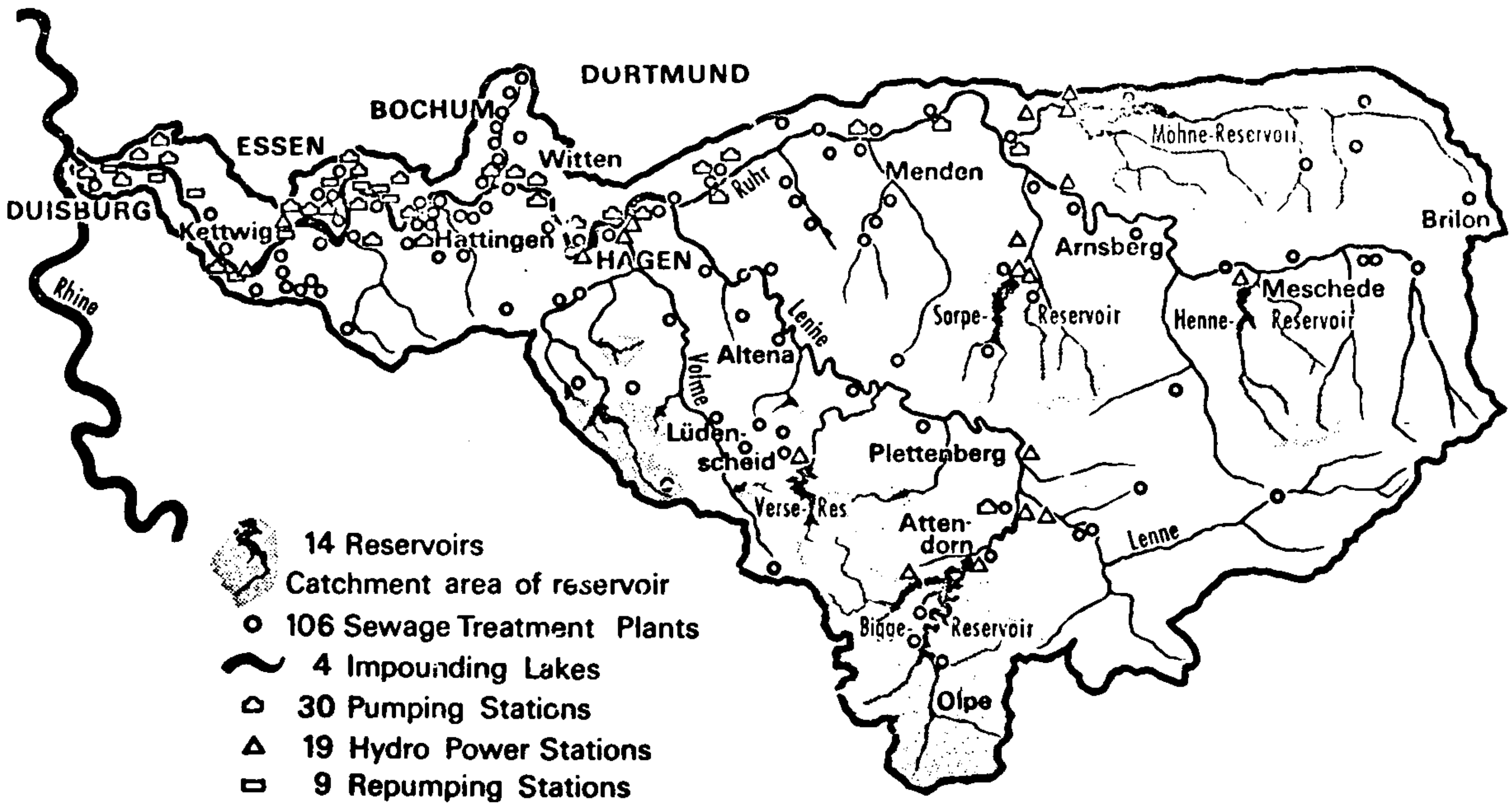

Figure 12. Ruhr River Basin and its Treatment Plants.

THE ROLE OF ECONOMICS IN MUNICIPAL WATER SUPPLY: THEORY AND PRACTICE

by

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Introduction

The objects of this paper are to point to ways in which economic pricing and investment theory might be adapted to suit the practical requirements of decision-making in the water supply field, and to define the role of the economist in this process. The paper is restricted to economic aspects of municipal water supply, and does not deal with multipurpose water resource development, an area in which economic analysis has been much more prominent. Furthermore, the paper does not discuss the important area commonly referred to as "engineering economics," which basically consists of the application of financial criteria in project planning, design, and execution.[1]

Although the principles discussed here are of general application, frequent reference is made to the particular problems encountered in developing countries. It is paradoxical but true that the wider the gulf between real-life conditions and those described in economic textbooks, the more significant is the role of the economist. Thus it is probable that economic analysis is relatively more important with respect to water supply in the developing than in the developed countries, for the characteristics of backlogs in supply, unemployment, poverty, and administrative delays and inefficiency, all of which add to the complexity of economic analysis of pricing and investment policies--are normally exhibited most clearly in the developing world.

Economic Efficiency and Marginal Cost Pricing

An important benchmark by which policies relating to water supply may be judged is the contribution that those policies make toward economic efficiency. A course of action may be roughly defined as improving economic efficiency if those who gain from that action could, if they wished, compensate the losers to the full extent of their loss and still remain better off. An efficient policy is therefore one which maximizes real income benefits accruing to a community, no consideration being paid to the way in which those benefits are distributed within the community. A proposition stemming from this definition is that the price of any service or commodity supplied by a public body should be equated to the cost of producing an additional unit of it or, in other words, to its marginal cost. If consumers are willing to pay a price that exceeds marginal cost, it means that they place a value on the marginal unit consumed that is at least as great as the cost to the rest of society of producing that unit; output and consumption should therefore be expanded. If on the other hand the market clearing price is less than marginal cost, it can be assumed that there is oversupply of the commodity: the cost of additional output exceeds the benefits.

Whether or not a policy is considered to contribute toward efficiency will, of course, depend upon the community whose benefits the analyst is interested in increasing. Having determined the relevant group of people, it is necessary to distinguish between purely accounting costs and real (or economic) costs incurred by that group. The former, which could include re-

[1]See E. L. Grant and W. G. Ireson, Principles of Engineering Economy, Ronald Press Co., New York, 1970.

payment of past loans, simply represent a transfer of income within the community. Efficiency in resource allocation dictates that these "sunk costs" be ignored for pricing purposes for they represent no net loss, or avoidable cost, to society as a whole. On the other hand, the resources employed in the construction and operation of a particular project represent, at the time of employment, real costs, in terms of opportunities foregone elsewhere. The price charged for the good or service concerned should clearly incorporate recovery of such costs if they are incurred as a result of additional consumption.

Capital Indivisibility

The foregoing suggests that it is necessary to make a distinction for pricing purposes between those costs which are a function of consumption, and those which are not. Ambiguity in the definition of marginal cost arises where capital indivisibility (or "lumpiness") is present, for, with respect to consumption, costs will be marginal at some times and non-marginal at others.[2] For example, if the safe yield of a reservoir is less than fully utilized, the only costs immediately attributable to additional consumption are certain additional operating and maintenance costs. These may be called short-run marginal costs. Long-run marginal costs, on the other hand, refer to the sum of short-run marginal costs and marginal capacity costs, the latter being defined as the cost of extending capacity--for example, building a new reservoir--to accommodate an additional unit of consumption.

Since we now have two definitions of marginal cost, one applicable in the short run and the other in the long run, what happens to the rule that price should equal marginal cost? Strictly interpreted, the rule requires that price should equal short-run marginal cost when capacity is less than fully utilized, but if demand increases so that existing capacity becomes fully utilized, price should be raised to ration existing capacity. This should continue up to the point that consumers reveal their willingness to pay a price equal to short-run marginal cost plus the annual equivalent[3] of marginal capacity cost. At this point, i.e., where price equals annual-equivalent long-run marginal cost, investment in capacity is justified. Once the investment has been carried out, however, price should fall again to short-run marginal cost, for the only real costs (or opportunity costs, in

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[2]Certain other costs are not marginal with respect to consumption. These may include "consumer costs" such as meter reading and billing, certain managerial overheads, and so on.

[3]The annual equivalent (A) of a lump sum expenditure (E) is here defined as:

$$A = (Ei\ (1+i)^n)/((1+i)^n-1),$$

where i is the rate of interest and n years is the expected useful life of the project. Any demand period could be chosen, but an annual one is obviously convenient. Note that if demand is expected to continue growing, willingness to pay a price equal to annual equivalent long-run marginal cost by consumers in the first year would imply willingness to do so over the rest of the designated useful life of the asset. If not, it simply means that the useful life has been estimated incorrectly.

terms of alternative benefits which will be foregone) are then operating costs. Price therefore plays the roles of (a) obtaining efficient utilization of resources when operating at less than full capacity, and (b) providing a signal to invest.

Problems associated with strict marginal cost pricing, as just described, are particularly apparent where capital indivisibility is present. This is typically true of municipal water supply, where productive capacity is often installed to meet demands for a number of years hence. Initial costs of constructing reservoirs and laying connecting mains are usually very high in relation to operating and maintenance costs. Strict marginal cost pricing in these circumstances would involve significant fluctuations in price, which would be the source of considerable uncertainty for consumers, creating particular problems for planning long-term investment in facilities that are complementary to or competitive with water consumption.[4] Exploitation of ground water sources often gives rise to less difficulty in this respect: in the economist's jargon, the long-run marginal cost curve is frequently relatively "smooth." However, even where it is <u>technologically</u> possible to extend capacity in fairly small increments, fluctuations in the availability of finance may mean that capacity is extended in large lumps. This applies particularly with respect to underdeveloped countries, where large backlogs in supply may be remedied and excess capacity created at one go.

One solution--necessarily an imperfect one--to this problem is to define marginal cost more broadly, and set price equal to the average unit cost of incremental output.[5] This policy may be called <u>average incremental cost pricing</u>. In practice, any version of marginal cost pricing has to be approximate, and ultimately some averaging of costs over a range of output is always required. Average incremental cost pricing will be theoretically less respectable the greater the degree of capital indivisibility and the slower the rate of increase in demand, for while capacity remains idle, price will be in excess of the currently relevant marginal cost.

This still has not dealt with the ambiguity in the definition of marginal, or incremental output. Generally speaking, marginal outputs should refer to production associated with the next investment in capacity when such investment is immediately due, and with the most recent investment when investment has just taken place. Between these two extremes, price should be adjusted periodically to conform to the long-term trend in costs. Average incremental cost pricing corresponds to long-term marginal cost pricing at capacity points, and it provides an indication that investment is justified, although as noted, short-run inefficiency is necessarily involved. This appears to be the best practicable approximation to optimal pricing that can be achieved in the water supply field.

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[4]For a defense of this policy see J. Hirshleifer, J. C. deHaven and J. W. Milliman, <u>Water Supply, Economics, Technology and Policy</u>, University of Chicago Press, 1960. Hirshleifer <u>et al.</u>, dismiss opposition to price fluctuations as "prejudice."

[5]This has been recommended for electricity supply. See R. Turvey, <u>Optimal Pricing and Investment in Electricity Supply</u>, George Allen and Unwin, London, 1968.

The characteristic of capital indivisibility is demonstrated in an extreme form by a distribution network: at its inception it is by definition a marginal cost and is, presumably, a function of the expected consumption of those benefiting from it. It is, however, normally designed to meet demands placed upon it for many years hence, during which time additional consumption by existing consumers is responsible for negligible additional capacity costs. The pure marginalist approach would suggest that the price charged for this element of a water undertaking's services should also be negligible. It has to be financed somehow, though, and the case is illustrative of the often-encountered conflict between economic efficiency and financial requirements.

Financial Viability and Economic Efficiency

Marginal cost pricing will result in an enterprise making financial losses when average costs are falling, i.e., when marginal cost is less than average cost. This could be a temporary situation, arising, for example where there is excess capacity and price is equated to short-run marginal cost. It could also be a situation of some permanence, even if there is perfect capital divisibility, if long-run average costs continue to decline and price is equated to long-run marginal cost. If there is lumpiness, a price equal to average incremental cost would in these circumstances also result in loss making. On the other hand, if long-run average costs are rising, profits would be made.

Any surplus that results from the application of marginal cost pricing could conceivably be used to defray other public expenditure, or avoid taxation, and few distributional or resource allocation problems arise. Loss-making, on the other hand, can be attacked on the grounds that those who benefit should pay for a service, even though the expenditure of real resources might have taken place in the past. Indeed, the possibility that efficient resource allocation could require subsidy from the remainder of society, who do not benefit from the supply of the good concerned, should lead the analyst to examine with care the often multiple objectives of a pricing policy. Thus, if there is a clear conflict of interest among various groups in society, one has to weigh the benefits of overall economic efficiency against those of income redistribution.

Loss-making may also involve certain drawbacks from an efficiency standpoint. First, the accounting losses have to be made good somehow, and it will often be difficult to achieve the necessary transfer of real income without creating distortions of consumer or producer's choice which may be as severe as those encountered in departing from marginal cost pricing. Secondly, it is often argued that the financial discipline and organizational autonomy resulting from financial viability are necessary to ensure efficient operation of the undertaking concerned.

Several solutions to this dilemma have been proposed by economists, who have usually tried to obtain the best of both worlds: the advantages of marginal cost pricing on the one hand and of avoiding loss-making on the other. There are in fact many variations on a common theme, the simplest of which being a two-part tariff where a water consumer would pay a sum per thousand gallons consumed equal to marginal cost, plus a lump sum that covers non-marginal "sunk costs" and consumer costs. In this way, as long as liability to the lump sum payment does not deter anyone from consuming municipal water altogether, optimal allocation can be achieved. Similarly, efficient allocation can theoretically result from the activities of the economist's imaginary "perfectly discriminating monopolist" who charges each consumer a price equal to the maximum that the consumer would pay, right on down to the consumer who places a value on water that is equal to marginal cost. Although such omniscience is rare, this general approach, popularly known as charging "what the traffic will bear" is often employed to finance water supply: for example, industrial consumers may be charged higher prices than domestic consumers for this reason. A general problem with these methods is that even if they succeed in achieving efficiency in the short run, the investment decision still cannot be signaled without price fluctuations if capital indivisibility is present.

The "Second Best" Problem

Another difficulty that is encountered in applying marginal cost pricing in the real world is what economists call the "second best" problem. The problem is that what may appear at first sight to be a step in the direction of economic efficiency (e.g., setting a price equal to marginal cost, or indeed, of introducing a pricing mechanism where none hitherto existed) may not be an improvement at all if nonefficient conditions prevail elsewhere in the economy. Optimality in any one sector might require a price greater to or less than marginal cost to counter inefficiencies elsewhere.

In practice, however, in any economy in which there is a good deal of competition, it has to be assumed, as a rough and ready principle, that elsewhere goods and services are sold at prices that in general approximate longrun marginal cost. If not, the difficulties of adjusting for all imperfections would lead us to the nihilistic conclusion that there are, after all, no empirical grounds for preferring any one set of pricing rules over any other.[6] But where goods or services that are in direct competition with (or are complementary to) the service in question are priced in a way that diverges blatantly from the standard that we have set for the industry in isolation, it may be feasible to make some adjustment. If prices of resources employed in developing water supplies diverge sharply from their long-run marginal cost to society, "shadow" prices should ideally be placed upon them in evaluating the real cost to society of the development expenditure. Thus, labor that would otherwise be unemployed should be valued at zero (i.e., at its opportunity cost) even though, due to market imperfection, it is able to command a wage rate that is well in excess of the minimum amount needed to attract it; foreign exchange costs should be valued at their natural market rate; interest rates should reflect the social opportunity cost of capital; and so on. The theoretical and empirical problems involved in making these adjustments are of course immensely complex.

[6]The informational problems emerge quite clearly in R. Rees, "Second-Best Rules for Public Enterprise Pricing," Economica, 1968. This difficulty has prompted some people to allege that price is a meaningless indicator of social benefits/costs, and attempts at achieving efficiency by this means are futile. However, those same people often stress the importance to society of achieving least-cost solutions, an attitude which is inconsistent with their professed skepticism of the role of price.

"External" Effects and "Intangibles"

Closely related to the above are the "external" effects that may accrue from expenditures on water supply. "Externalities" can be defined as those benefits (or costs) that accrue to parties other than the buyers or sellers of a commodity and from whom (or by whom) compensation is not for some reason exacted. Economists make a distinction between technological and pecuniary externalities: the former are those which add to (economies) or subtract from (diseconomies) society's physical production possibilities or the net satisfaction consumers can obtain from society's resources.[7] A classic case of a technological externality is man-made water pollution, the costs of which are not borne by the parties responsible for it. A technological external benefit of municipal water supply would arise if water consumption by an individual is of benefit (perhaps because of his more attractive garden, his improved health or his reduced fire risk) to his neighbor.

Economic efficiency, being concerned with activities which show a net gain or net loss to society, requires that technological externalities be taken into account in pricing and investment decisions. The optimal pricing rule is therefore redefined to be that price should equal marginal social cost, which may be greater or less than marginal cost, depending upon whether technological external diseconomies or economies prevail. Where strict adherence to marginal cost pricing is not feasible, the policy might therefore be to set price equal to average incremental social cost.

Efficiency in pricing and investment is unaffected by the presence of pecuniary economies or diseconomies, for these merely represent transfers in income or in kind between members of the same society. Thus, a water supply project that stimulates industrial development, thereby yielding net gains to a particular region of a country will not be justified on grounds of economic efficiency if the effect is merely to attract industry and reduce gains by an identical amount in other regions. However, purely pecuniary, or transfer effects such as these, which form the most important element of "intangible" effects, which cannot be evaluated in monetary terms, may sometimes be accepted as overriding the losses in efficiency that might result from a given policy.

It is of course not inevitable that efficiency and other criteria should be at odds with each other. Thus if subsidization of water consumption encourages industrial development in a depressed region, income distributional arguments for such a policy may be reinforced by efficiency considerations. This might be so, for example, where the subsidy results in a transfer of activity, without undue loss of productivity, from a region of high employment where the marginal opportunity cost of labor is high, to an area where it may be zero. The general conclusion therefore is that rational decisionmaking will be facilitated by making explicit to policy-makers either the cost of achieving "intangible" objectives, in terms of efficiency benefits foregone, or the "intangible" benefits or costs that may be associated with a particular course of action. Having said this, it must be pointed out that the task of disentangling these effects is by no means an easy one. Externalities (al-

[7] For a brilliant popular discussion of this whole subject, see E. Mishan, The Costs of Economic Growth, Staples Press, London 1967.

most by definition) are difficult to estimate, and any decision involving the redeployment of resources or the redistribution of income will invariably set off a complex chain reaction involving further technological and pecuniary repercussions in other parts of the economy.

A particularly important problem that arises in developing countries is that water undertakings are often called upon to supply water to people who are too poor to pay for it. Where this is so, a clear distinction should be drawn between the two roles that a water authority plays. In part, it is a "public enterprise," preferably financed according to the principles outlined earlier, and, in part it performs a "social service," which can be defined here as a service that is subsidized or supplied free of charge for the benefit of those unable to pay for it. Most standpipe consumption, subsidized supplies for poorer metered or unmetered domestic consumers, or rural consumers[8] generally, might come into this category.

It is commonly the case that these consumers are subsidized at least in part by other water consumers. Where this is so, it is important that the subsidy should be explicitly recognized. There may often be grounds for financing the "social service" aspect of a water undertaking's activity, as in the case of other social services, out of general tax revenue. Indeed, this illustrates the general point that estimation of the need for divergences from efficient pricing on grounds of "intangible" effects or technological externalities are outside of the competence of water authorities, and should clearly be recognized as such.

The Benefits and Costs of Pricing

The theoretically ideal pricing and investment rules discussed earlier are far removed from the methods actually used to finance and determine the value of municipal water supplies in this country and abroad. Water authorities are normally reluctant to make positive use of price as a means of achieving an efficient allocation of resources, and it is by no means obvious that this results from a decision that has consciously weighed potential efficiency gains versus potential "intangible" losses.

The inefficiencies generated by failure to adhere to marginal cost pricing rules are extreme when the means of implementation (i.e., metering) do not exist. Where price at the margin of use is zero, a consumer will continue to use water at least up to the point that the value to him of the last unit consumed is also zero. At this point the net social "economic" loss will be the relevant marginal cost, which, when capacity is less than fully utilized, will equal short-run marginal cost. Inefficiency is, however, particularly evident when, at the current rate of use, existing capacity is on the verge of full utilization, and for some reason rationing by price is not feasible. In these circumstances, the decisionmaker is faced with the choice betweeen permitting shortages and allocating water by non-price means, or extending capacity. Rationing by physical or administrative means is generally accepted as being unsatisfactory as a permanent policy, although it is the norm in many developing countries. However, as a public utility, a water

[8]Note that even in developed countries rural consumption is frequently heavily subsidized.

undertaking should be able to supply water to those willing to pay for it: moreover, health hazards arise from intermittent supplies. There are also theoretical objections: non-price rationing is a necessarily arbitrary device and can rarely be administered in accordance with the value of the benefits derived from the services rendered. It is therefore inefficient in allocating resources in the short run and offers absolutely no guidance for the investment decision.

However, the policy that is usually preferred by decision-makers in this, and in many other public utility areas, is automatically to increase capacity when existing capacity approaches full utilization. In other words, at this point, more capacity is deemed to be "required." Clearly, in the absence of a signal to invest of the kind described in the section on Capital Indivisibility, it can rarely be certain that the value of the additional consumption--or usage--made possible by the investment will exceed the costs thereby incurred.

Of course, the absence of a market for a particular commodity cr service does not <u>necessarily</u> preclude the achievement of efficient investment decisions. The current popularity of the technique of social cost-benefit analysis is partly based on the assumption that this is so, for schemes that are made the subject of such studies are usually characterized by the absence of a market for the services they are designed to provide. Indeed, it is often because of this that provision of the services concerned is a public responsibility and therefore most relevant for cost-benefit analysis, in which all costs and benefits, whoever bears or receives them, should be included.

Nevertheless, the literature of cost-benefit analysis,[9] which is merely a simple extension of the pricing and investment rules outlined above, abounds with examples of the extreme difficulties encountered in measuring benefits when no market exists. In the absence of pricing, the demand function for project outputs has to be imputed by indirect methods, and this is often a hazardous process.[10]

In the absence of any budgetary constraints, a failure to employ pricing, and reliance upon the "requirements" approach, will almost certainly result in over-investment. But when the guidance of the pricing mechanism is not available, budgetary constraints may prevail when, in terms of the costs and benefits of a given project, they should not. Either way, inefficiency is likely to result.

Unfortunately, implementation of pricing (i.e., metering) for water supply is a costly exercise, and its introduction or continuation should ideally be subject to cost-benefit analysis. Roughly speaking, the benefits of metering are the cost-savings brought about by reducing consumption. Savings may be achieved by deferring investment as well as reducing annual operating

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[9]For a summary of the literature, and a useful bibliography, see A. R. Prest and Ralph Turvey, "Cost-Benefit Analysis, a Survey," <u>Economic Journal</u>, December 1965.

[10]An important exception to this general rule may exist where the services provided (i.e., the benefits) are <u>cost savings</u>, in which circumstances, cost-benefit studies may be most useful.

and maintenance costs. The present worth of these savings should be compared with the present worth of initial and annual costs of metering, plus the reduction in the value of water consumed,[11] in order to determine whether or not the investment in metering is worthwhile.[12] Since the reduction in consumption likely to result from metering is normally highly conjectural, one way to approach the problem is to ask the question: what percentage reduction in consumption would justify its introduction? If we are lucky, extreme values necessary for outright acceptance or rejection of metering will result from such a calculation: if not, at least the worst excesses of installing or not installing meters can be avoided.

Prima facie, the case for metering industrial and commercial consumers is not a matter for serious dispute. The real question is whether or not to meter domestic consumption,[13] and here the need for some sort of cost-benefit calculation is clear. Although hard empirical evidence is lacking, the water supply industry generally argues that while metering will reduce recorded per capita consumption, changes in price once meters have been installed appear to have an insignificant effect. This apparent paradox is conventionally explained by two things. First, universal metering reveals discrepancies between the quantity of water going into supply and that actually received on consumers' premises; such discrepancies, which reveal the extent of leakage from the mains and illegal connections at once facilitate and provide an incentive for the improvement of waste prevention methods of water authorities, for waste outside registered consumers' premises would, prior to metering, have been recorded as domestic consumption.[14] Metering would therefore result in a permanent reduction in annual wastage and this would obviously be unaffected by subsequent price changes. Second is the argument that metering will encourage individuals to reduce the amount of water wasted on their premises, but once having made this adjustment, their demand for water use remains inelastic. The elasticity of demand for water is of extreme importance for the foregoing discussion, and this is a subject to which we now turn.

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[11]This would be roughly equal to the product of half the expected reduction in consumption and the price per gallon that is charged when metering is introduced.

[12]For details see J. J. Warford, Water "Requirements," The Investment Decision in the Water Supply Industry, in R. Turvey (ed) "Public Enterprise," Penguin Modern Economics Series, 1968. Note that our concern should be with real as opposed to purely monetary costs, so it is irrelevant to argue that, because of expected inflation, to delay investment is to incur greater costs in the future. Real costs are defined solely in terms of resources used up: general price increases therefore should not affect the calculation, although if they were known, relative price changes should.

[13]Since a utility manager will be more concerned with financial viability than with optimal resource allocation, he may prefer that metering should be extended to poorer properties rather than more valuable ones. This attitude would typically be encountered where a progressive property tax is currently employed to finance water supply.

[14]Such savings would be realized if every major outlet (e.g., an apartment block as opposed to individual apartments) is metered.

The Elasticity of Demand for Water

As noted, it is normally argued that the demand for water, at current price ranges, is inelastic--i.e., price changes have no apparent effect on consumption. To the extent that this is true, the case for marginal cost pricing is weakened. If price can be ignored as a determinant of consumption, the "requirements" approach now employed would quite adequately signal the need for further investment whatever pricing principle is employed, and there would be no short-run misallocation either.

The case for marginal cost pricing (or some variant thereof) and against the "requirements" method is therefore dependent for its validity upon the presence of some degree of price elasticity. The nature of the problem is such that little empirical work has adequately substantiated or denied the presence of elasticity of demand for water supply. There are, however, several good reasons for adhering to the principle of marginal cost pricing, and thereby implicitly accepting the notion of elasticity.

First, even though evidence has not been able to provide any clear picture, it is generally true that water supply will in real terms become more and more expensive as more distant or lower quality sources have to be exploited. Already we are warned of impending water "crises" that may overtake us in the U.S., unless we do something rather dramatic about obtaining fresh supplies. If water charges rise to correspond to long-term trends, it is reasonable to assume that consumers will become more inclined to exercise care in water use: in other words, historical evidence or intuition based upon observed reactions to price changes will not invariably be relevant for the future.

Second, there is some evidence that although the demand for most domestic use of water has in the past been fairly price inelastic, "luxury" uses have been less so. A study of water use in Southern California has demonstrated an elasticity of unity (i.e., a given percentage increase in price is associated with an equal percentage reduction in consumption) for lawn sprinkling.[15] As per capita incomes grow and water is used for less and less "necessary" purposes, the presumption that elasticity will increase seems a reasonable one, particularly when allied to the assumption about increasing costs.

Third, is the argument that because statistical evidence is lacking, and because the circumstances of town to town, country to country vary so considerably, it is much better to stick to a principle which is theoretically correct rather than one which will only give the right results if the particular condition of demand inelasticity happens to be present.

Fourth, is the apparent importance of metering in reducing consumption plus waste on the consumer's premises. The usual distinction between consumption and waste is an uneasy one; there must presumably be some price of water at which some waste (such as a leaking tap) will be permitted to continue indefinitely, just as there must be some other price at which some con-

[15] See F. P. Linaweaver, Residential Water Use, Department of Environmental Engineering Science, Johns Hopkins, Baltimore, 1966.

sumption (such as washing) is curtailed.[16] If raising the price from zero to some positive figure reduces consumption, plus waste, this seems to be evidence as to the existence of price elasticity.

Finally, a word about demand forecasting, which to an economist involves the use of "economic" variables--including price--in determining production targets. Sophistication in this area is perhaps developed to a higher degree in electricity supply than any other field of activity. Even in the case of electricity, however, econometric demand analysis has been conspicuously unsuccessful. Demand forecasting in the sense that econometricians use the term is virtually unknown in the water supply field.[17] Nor is there much hope that it will ever be very accurate: this is largely because of the high cost of transmitting water between communities. Unlike electricity supply, which is normally a large grid system, linking up consumers all over a particular country or region, water supply is usually a municipal affair. Stochastic effects, including large industrial location or output decisions are likely to throw all calculations out of order, whereas on a national basis they can be absorbed much easier. This however does not constitute an argument against domestic and existing industrial econometric demand forecasting, because ideally significant industrial locational or output decisions should not be made without knowledge of the local water supply (and other infrastructural) situation.

Maringal Cost Pricing in Practice

It is clear from the foregoing that it is no simple matter to translate the textbook theory of marginal cost pricing into practice. Nevertheless it remains an incontrovertible fact that real costs are marginal ones, that decisions should be based on comparison of marginal costs and marginal benefits, and that properly operated, a pricing policy will aid this process. It also remains true that something can be done to take care of the most critical problems created by such things as lumpiness, financial constraints, "second best" considerations, externalities, income distributional effects, and the physical difficulties of operating a pricing mechanism.

As a general principle, it is suggested that some approximation to average incremental cost pricing, as described earlier, is appropriate for water supply. If long-run marginal costs of water supply are rising, this may be sufficient to cover all financial requirements of the undertaking. If not, a two-part tariff may be necessary, the lump sum being used to recover consumer and other non-marginal costs, and to cover production costs in cases where long-run marginal costs are falling.

The marginal cost pricing principle requires that distinctions should not be made between consumers on any basis other than the costs their consumption involves. This implies that unless economies of scale in the supply of water

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[16]In economic efficiency terms, and ignoring any income distributional connotations, wasteful use can be defined as any use for which the beneficiary would be unwilling to pay the true marginal cost.

[17]For a useful summary of the current state of the art and recommendations for improvement, see P. W. Whitford, Forecasting Demand for Urban Water Supply, Stanford University Research Report EEP-36, September 1970.

to an individual consumer are present, price should be the same for all consumers, and that distinctions, for example, between industrial and domestic use should not be made. It also implies that variation in the unit price of water according to quantity consumed is also incorrect: if I am consuming water at a price of 30 cents per thousand gallons, and you are paying 40 cents, it means that at the margin of use I am using water for less valuable purposes than you, and that transferring water from me to you should increase economic efficiency. Often, however, a decision is made to diverge from this principle on grounds of income distribution, or equity. A free initial allowance of water to ensure a basic minimum amount of consumption for poorer people is commonly practiced, as is discrimination against industrial or commercial users. The effects of these devices are often insufficiently analyzed: it is the job of the economist to trace their impact not only in order that the costs of achieving certain ends be made explicit, but also to check whether the income-distributional results are likely to be in the desired direction.[18]

The marginal cost pricing principle also implies that price should reflect variations in the cost of supplying water to different consumers. While water undertakings often vary price when on efficiency grounds, they should not, they often fail to do so when they should. This is most clearly observed with respect to seasonal variations. Whether pressure on capacity is due to demand peaks or supply troughs or both, there is a case for raising the price of water to achieve rational allocation of supplies:[19] for example, the additional storage capacity necessary to satisfy the excess of summer over winter demand should be constructed as a result of a revealed willingness of consumers to pay the extra amount for water during the summer months. In practice, it would only be possible to introduce differential pricing for seasonal peaks: the need to meet diurnal peaks is reflected in the design of distribution systems, and differential pricing here would, prima facie, be too expensive to administer.

As just described, the implementation of a practical version of marginal cost pricing for water supply in a developed country is a fairly mechanical process. However, even here, further analysis is required to determine the reactions of consumers to price changes (including metering, which raises price from zero to some positive figure), to assist in demand forecasting, and to deal with the problems caused by externalities, intangibles, and divergences between price and long-run marginal cost elsewhere in the economy. These tasks fall firmly within the domain of the economist.

Because these problems are normally exhibited to an extreme degree in developing countries, it is sometimes difficult to use price as a means of achieving efficient resource allocation in the water supply field. A particularly important reason for this is to be found in the huge backlogs in

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[18]For example, where industry pays a relatively high price for water it would be instructive to examine market conditions to determine the extent to which costs might be passed on to poorer sections of the community.

[19]The case for seasonal peak pricing for municipal water supply in the Washington, D.C. area is clearly stated by S. H. Hanke and R. Davis, "Demand Management Through Responsive Pricing," Journal of the American Water Works Association, September 1971.

supply that are normally encountered. Fluctuations in the availability of capital and the fact that much investment is to provide piped supplies to people who have never had them before or had them only intermittently, mean that lumpiness is an overwhelmingly important characteristic, and the revealed willingness of consumers to pay for water often cannot be employed to determine willingness to pay for further capacity extension. Moreover, public health benefits loom much larger in justification of water supply projects in these circumstances than when relatively small increments are made available. Note, however, that health benefits should only be counted as additional to monetary benefits (as revealed by revenues) if (a) they are external to individual consumers or (b) consumers lack the knowledge that would permit health benefits to be recognized accurately in their market decisions.

Where pricing breaks down entirely as a method of signaling the justification for investment (e.g., where metering is not practicable), an independent evaluation of the merits of a project must be made. The problem is to measure the total amount that beneficiaries would be willing to pay for the project rather than do without it altogether. This represents the economic benefits stemming from the project and consists of two elements: the amount that people actually do pay, plus the "consumers' surplus" that is obtained. Because of the assumed inelasticity of demand for water, the unknown--the consumers' surplus--is generally to be the dominant element of benefits, and it is likely that water supply is grossly undervalued by the revenues obtained from sales.

Attempts are sometimes made to demonstrate the economic justification of municipal water supply investments in terms of health benefits. Unfortunately, while everyone pays lip service to the benefits to health of piped water supply, it is never possible to call upon firm statistical evidence to demonstrate what will happen in any particular town or region when supplies are laid on. Since health is commonly cited as being a fundamental justification for devoting large sums of money to investment in water supply it would seem that extensive research into this area is long overdue: not only are we unaware of the effects of improved water supply on health in monetary terms, we do not even know what the <u>physical</u> benefits are.

Cost-benefit analysis has had some success in areas in which benefits can be equated with cost-savings. The theoretical position is that where services provided by a public project are identical in quality and quantity to those that would otherwise be privately supplied, the cost saving is an adequate measure of benefit.[20] This is particularly applicable to the replacement of private industrial abstraction of water by a public supply.

Normally, of course, there are qualitative differences (e.g., a piped water supply may be preferable to purchase from a street trader on grounds of convenience and bacteriological quality) as well as differences in the quantity of water supplied. However, even where such differences are present, alternative cost calculation may be of some help in providing information to rank projects by need.

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[20] See P. O. Steiner, "The Role of Alternative Cost in Project Design and Selection," <u>Quarterly Journal of Economics</u>, August 1965.

Possibly the most hopeful attempt to get to grips with the measurement of water supply benefits is a research project at present being undertaken on behalf of the World Bank. Recognizing that measurement of water supply benefits is frustrated by the imperfection of the water supply "market," a somewhat more perfect market--that of housing--is being used as a proxy. It is observed that property values increase when piped water is supplied, and this, very roughly, can be seen as a measure of the consumers' surplus arising from the investment.[21] It is hoped to derive a shorthand method of using observations of property transactions as an indicator of benefits of water supply, in order that evaluation of projects may be assisted on this basis as a matter of routine.

The Role of the Economist

In writing this paper, I am continually struck by the fact that those areas in which the tools of the economist are specially relevant are also areas which are of little concern to the practicing water engineer. As long as the water supply authority can meet its financial goals and not be unduly criticized for failing to supply enough water, there is no reason at all why the water engineer should concern himself with economically efficient pricing and investment rules. The reason for this is, quite simply, that the interests of the water undertaking and those of the remainder of society do not necessarily coincide. This is highlighted by the case in which economically efficient pricing would require loss-making, but is also easy to see in cases where shadow pricing ought ideally to be employed (why should the engineer be concerned with the opportunity cost of labor or foreign exchange?), or where externalities or intangibles are present.

In general, it is fairly clear that, if they had their way, managers of water undertakings would have no time for economic analysis unless, of course, it can be employed to demonstrate that water supply is really much more valuable to society than is suggested by financial data. The attitude of managers of other public utilities, power and telecommunications and so on, can reasonably be expected to be the same. Yet if economists are to do anything useful, the cooperation of the relevant industry is essential. For example, it is all very well to talk about marginal cost pricing, but the estimation of what marginal cost actually is in any particular instance is something that only the engineer can provide. So, not only do economists try to evaluate pricing and investment policies on a basis for which the utility can have little sympathy, they also have to take up the valuable time of engineers in helping this to be achieved. And if the utility permits this, it is usually because the economist has been imposed upon it by some higher authority, determined to reconcile the conflict of interest between the utility and the rest of society: insult is therefore added to injury.

I have sympathy for the utility manager in such a case, but hope that the preceding pages have done something to indicate to him certain areas in which

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[21]The theoretical basis for the study is analyzed in R. Bahl, S. Coelen and J. Warford, Land Value Increments as a Measure of the Net Benefits of Water Supply Projects in Developing Countries: Theory and Measurement, paper presented to the committee on Taxation, Resources and Economic Development, University of Madison, Wisconsin, October 1971.

his competence should not reasonably be expected to extend. If it is accepted that these are socially important areas, and that pricing and investment decisions should not simply be the mechanistic outcome of certain accounting and engineering rules, and that the necessary economic analysis is more than a straightforward application of simple textbook theory, the role of the economist is plain to see.

The final word, however, often rests with the politicians. Water supply has acquired a mystique that other commodities do not have: probably the thing that a water engineer dreads most is having to announce a rate increase, and politicians can acquire instant popularity by decrying such action. In an age in which water costs are rising rapidly, political necessity and the requirements of economically efficient pricing must often be in conflict. This is especially clear when efficiency requires price to be raised in advance of the installation of new capacity, so that people are asked to pay for water before they get it. Once more, the role of the economist is not to make any judgment about what price ought to be, but merely to identify the efficiency implications of accepting the appropriate constraints.

PROBLEMS ON POLLUTION AND WATER RESOURCES IN THE NEW YORK CITY METROPOLITAN AREA

by

MARTIN LANG, P. E.

Commissioner of New York City Department of Water Resources

The magnitude of the New York City water pollution control program can be demonstrated by this statement: "Flow was recently started through the newest of our 14 plants, 'Newtown Creek', a single plant with the capability of affording secondary treatment for the wastewater of an equivalent of 2,500,000 people."

The City of New York has made substantial progress toward its goal of high-degree secondary treatment for all dry weather wastewater. Of the 1.5 billion gallons per day of sewage generated within the City, about 75% currently receives secondary treatment. Twenty-five percent, or about 375 million gallons per day, are discharged untreated into the estuary, mainly from the west side of Manhattan, where about 200 MGD of raw sewage still spills into the Hudson River. The City is now engaged in a $1.9 billion construction program to construct two new sewage treatment plants and upgrade and enlarge 12 existing plants.

This is the largest water pollution control program ever undertaken in the City's history or in the country's. In order to be eligible for state aid, all of this construction must start by March 1972. To meet this deadline has required a re-examination of the methods traditionally used by the City to design and construct its public works. With cooperation from the Bureau of the Budget and the Comptroller's Office, EPA has greatly streamlined the entire process from concept to construction (see Table 1).

In addition to high-degree secondary treatment for all the dry weather flow, some system had to be developed for the treatment of storm waters which flow into our combined sewers in much of the City and overwhelm the capacity of the treatment plants. A demonstration storm water treatment plant is being built on the shore of Jamaica Bay (Fig. 1) which will serve as the prototype for a ring of several plants that ultimately will encircle Jamaica Bay. Additional storm water treatment plants will be constructed along the Upper East River. The construction of these plants will be long and expensive and will not be completed until the mid-1980's. By that time there should be a dramatic change in the quality of our estuary. Beaches that have been closed for decades should then be suitable for bathing.

Intelligent management of the estuary requires a better understanding of its biology and hydrology than we now have. Toward this end, the ecological study of Jamaica Bay discussed below should properly be extended to the entire estuary. Only when data from these studies are obtained will it be possible to predict the effect of changes in plant design or operation on the water quality of the estuary. When one considers that the total capital investment for water pollution control alone will result in expenditures of about $2 billion on behalf of New York City and another $2 or $3 billion on behalf of the other communities in the area, a major research effort can be more than justified in the decades ahead. A $100,000 project for the determination of Harbor water quality has just begun, called the New York Bight Study, Phase I.

<u>The Jamaica Bay Ecological Study</u>

Estuarine waters, where fresh water from the land mass meets the salt water, are the cradle of much marine life. They are the spawning and feeding grounds for a wide variety of fish and the nesting and resting grounds for sea birds and migratory land birds. Jamaica Bay is even more important for human beings. This 25-square-mile bay is ringed by parkland, nearly all as yet undeveloped. If the quality of the water of the bay could meet recreational standards, the bay would be one of the City's greatest parks. Recovery of the bay depends heavily on the construction of a series of auxiliary water pollution control plants on its periphery. The first of these, at Spring Creek, is already under construction, and it is the central element of a three-year before-and-after study of the bay and its waters. Despite the bay's importance, not much is known of its physical quality. The study, undertaken by EPA's Bureau of Water Pollution Control operating on a federal grant, covers the bay's hydrology, the tidal exchange and flow patterns, water quality including bacteria types and sources, levels of phosphates, nitrates and synthetic detergents; a quantitative assessment of all marine biota, and the relationship of algae to bacterial flora. The information developed will govern the design of all future auxiliary plants around the City. This study is the most important ecological investigation of estuarine water under way in the country, and the implications of the results will have nationwide application. Jamaica Bay falls within the boundaries of the recently-designated Gateway National Seashore, and we can expect coordinated conservation efforts for the entire area to grow out of the steps now being taken.

The basic program is, of course, the prime concern of the City. In 1931, long before Federal participation was envisaged, this City, with its wastewater discharging not into a potable watershed but into a salt water estuarine complex, began its construction program. Despite the stringencies of the Depression, the City taxpayers built 500 million gallons per day of treatment capacity between 1931 and 1945 at a cost of $67 million. In the post-war era, between 1948 and 1957, another 400 million gallons per day capacity was added for $117 million. At this time, massive population shifts to the periphery of the City compelled us to give precedence to increasing the capacity and improving the treatment at certain plants. For example, the Rockaway plant, which was completed in the fall of 1952, had to be doubled in capacity by 1961. Between 1958 and up to the time when the influent sluice gates opened at Newtown Creek, a further 450 million gallons per day capacity was added at a cost of $223 million. The present facilities, with a treatment potential of 1.35 <u>billion</u> gallons per day, would cost three-quarters of a <u>billion</u> dollars at current cost indices.

The average daily flow to the present plants is 1.15 billion gallons per day, of which 98 percent receives secondary treatment.

Of the 14 plants, six are designed for the step aeration process, which New York City experience shows has consistently yielded 90% removal of the biochemical oxygen demand (BOD). These plants are Wards Island, Hunts Point, 26th Ward, Jamaica, Tallman Island, and Bowery Bay, with a present total flow of 730 MGD. Of the other eight, five plants operate on the "Modified Aeration" process which gives removals in the 70 percent range, two are primary treatment plants and one is an obsolete screening plant which will be replaced by a new plant.

The last frontier of New York City, Staten Island, already has the Port Richmond and Oakwood Beach plants, both in their first stages. The City's original master plan envisaged four other small plants in still lightlypopulated areas, Bloomfield, Tottenville, Fresh Kills and Eltingville. We now find it feasible to proceed with the interception and treatment of flows from these areas by an economical system of pumping stations and force mains to the two "super plants". Port Richmond will be expanded from a 10 MGD primary plant to a 60 MGD step aeration plant, and the Oakwood Beach plant will be increased to 40 MGD and ultimately to 60 MGD, also on step aeration. Particular emphasis is being placed on effluent dispersion patterns at Oakwood Beach for the restoration and upgrading of the Staten Island beaches. The new Red Hook plant will be built as a step aeration facility on the Brooklyn shorefront on the Lower East River.

When the designs of the North River plant on the Hudson River, and much of its interceptors were virtually complete, Mayor Lindsay ordered an independent engineering and architectural appraisal of this project. This was done to insure its complete esthetic compatibility with the neighboring community and to insure making a positive contribution to the offshore vista of the Hudson shoreline. The engineering analysis affirmed the adequacy of the design and even predicted that this plant, <u>by itself</u>, would improve the Hudson water quality within the City line, to the State standard. The changes in design to incorporate urban beautification have somewhat delayed initial plant construction, but one of the four sections of the interceptor has been constructed and two are near completion, and construction of the foundation will begin in January 1972.

In November 1965, New York City entered into a stipulation with the State incorporating a timetable for completion of all components. Since then, certain changes have become appropriate. The necessary lead time for the Federal Environmental Protection Agency review of projects, a prerequisite before even advertising contracts, had to be built into this schedule. In addition, the new commitments of the City for step aeration at Port Richmond, Oakwood Beach and Red Hook called for revisions of target dates. Also, the City acceded to recent State requests for the provision of effluent hypochlorination facilities in all plants, even those not directly affecting recreational waters.

The second phase of our effort to cope with combined sewer overflows will serve the country as a whole because of the close scientific scrutiny of the prototype Spring Creek storm water plant. New York City conducts surveys of the harbor each summer, with data going back to 1909. This unprecedented fund of data enabled us to discern an abnormal increase in coliform population in the 1950's, and an exhaustive statistical study by New York University confirmed the validity of our observations. Since this is of significance to all coastal cities, and since this phenomenon manifested itself at a time when the eutrophication of inland waters became apparent, Federal aid was sought to investigate all factors related to estuarine coliform populations. This would include the effects of synthetic detergents, the need for control of phosphates and other nutrients, nonfecal coliform sources, the possible role of algae in a symbiotic relationship with bacterial flora and a quantitative assessment of all marine biota in the zone of influence of the Spring Creek plant, before and after construction. Other parameters, such as the effect of leaching from sanitary landfills and the effect of dredging and filling on tidal exchanges in Jamaica Bay, have been included in the Spring Creek study. All this, of course, is in addition to the basic monitoring of the auxiliary plant's reduction of coliform contributions from impounded and treated combined storm overflows.

The foresight of the City engineers has made possible the new third phase. At a time when all regulatory agencies, including the Interstate Sanitation Commission and the New York State Department of Environmental Conservation, approved plain sedimentation plants, our designers, like the fellow in the poker game who said, "I'll see you, and raise you" insisted on providing secondary biological treatment. In many instances, back in the 1930's and 1940's, they also fought for and obtained sites with the capability of expansion to treat more flows and treat to a higher degree. Their foresight made it possible to not only increase the Jamaica plant from 65 MGD to 100 MGD, but to add primary tanks and additional aerators and upgrade its treatment capacity from modified aeration to full step aeration. They made it possible to convert Coney Island from a primary treatment plant with chemical coagulation to a modified aeration plant; and they made possible the second stage of Wards Island, now under design, increasing its step aeration capacity from 220 MGD to 290 MGD and adding modern thickening and digestion facilities. Similar extensions were made to Bowery Bay, tripling its capacity, as well as to Hunts Point, Tallman Island and Rockaway.

The 160 MGD Owls Head and the 110 MGD Coney Island Plants will be converted to step aeration by maximum utilization of existing sites, even though our original commitment did not call for this at all. Recently we began a vigorous development and pilot program to improve the removal efficiency of the Newtown Creek plant, including a $2.5 million study of "oxygenated aeration." Thus, we are now preparing for the undoubtedly more rigorous requirements that will come in the future.

I think it appropriate to consider the impact of Federal and State aid programs. In 1965, the Mayor of New York City pointed out that in the years from 1957 to 1965, $223 million were spent by the City of New York, with reimbursements of only $3.4 million from the State, limited to design and boring costs. Approximately $2.3 million in Federal aid was obtained for construction, under such limited authorizations as existed prior to the Water Quality Act of 1965. Reimbursement for the remainder of the basic and auxiliary water pollution abatement programs will be under the New York State Pure Water Bond Act of 1965 and the Federal Clean Waters Restoration Act of 1966. It is anticipated that eligible design and construction costs will be approximately one billion dollars by 1972. Under the New York State Act, 30 percent of the eligible cost of construction and design is reimbursable, with provisions for pre-financing up to an additional 30 percent of the Federal share, in the event that Federal funds for this program are unavailable. It is anticipated that under a reimbursement clause in the Federal Act this money will be returned to New York State.

Editor's Note

Current information (Lang et al., 1975) demonstrates an accelerating and increasingly effective water pollution control effort by the City of New York.

With effective welding of borough organizations into a city-wide unit and innovative management of this unit, productivity was increased sixfold in five years, sewage line construction reaching sixty miles per year in 1972.

By 1973, major outlets for sewer networks had been committed to construction. In 1973 and 1974, numerous sewer installations were consolidated into sewer-highway and sewer-highway-water supply projects. An additional impetus in 1974 was the signing of Local Law #7, which mandates storm drainage for new developments. Design and construction of sewage of facilities and study of the estuaries of the metropolitan area are continuing at a rapid pace into 1975.

This record of accomplishment is even more impressive in view of economic readjustments, a personnel reduction of 30%, and defeat of a referendum to extend the exemption of sewer construction from the debt limit imposed on city projects.

Reference

Lang, M.; Samowitz, C.; Jethwani, M.; and Novotny, F., "Sewering the City of New York", The Municipal Engineers Journal, v. 61 (1975), p. 39-54.

NEW YORK CITY'S WATER POLLUTION CONTROL PLANTS - Table 1.

Plant	1969 Date in Operation	Design Capacity MGD	Percentage of City's Liquid Waste Treated	Ultimate Capacity MGD	Percentage of City's Liquid Waste Treated
Coney Island	1935	110	6.0	110	5.6
Wards Island	1937	220	17.3	310	16.2
Tallman Island	1939	60	4.2	80	4.1
Bowery Bay	1939	120	8.1	150	7.6
Hart Island	1942	1.5	0.07	0	0
Jamaica	1943	100	6.2	100	5.1
26th Ward	1944	60	4.4	85	4.3
Owls Head	1952	160	7.2	160	8.2
Hunts Point	1952	150	10.4	200	10.1
Rockaway	1952	30	1.3	45	2.3
Port Richmond	1953	10	0.6	60	3.0
Oakwood Beach	1956	15	1.0	80	4.1
Newtown Creek	1967	310	10.2	310	15.2
Future Plants					
East River Environmental Protection Center	1976	60	0	60	3.0
North River	1976	220	0	220	11.2
Total			77.0	1970	100.0

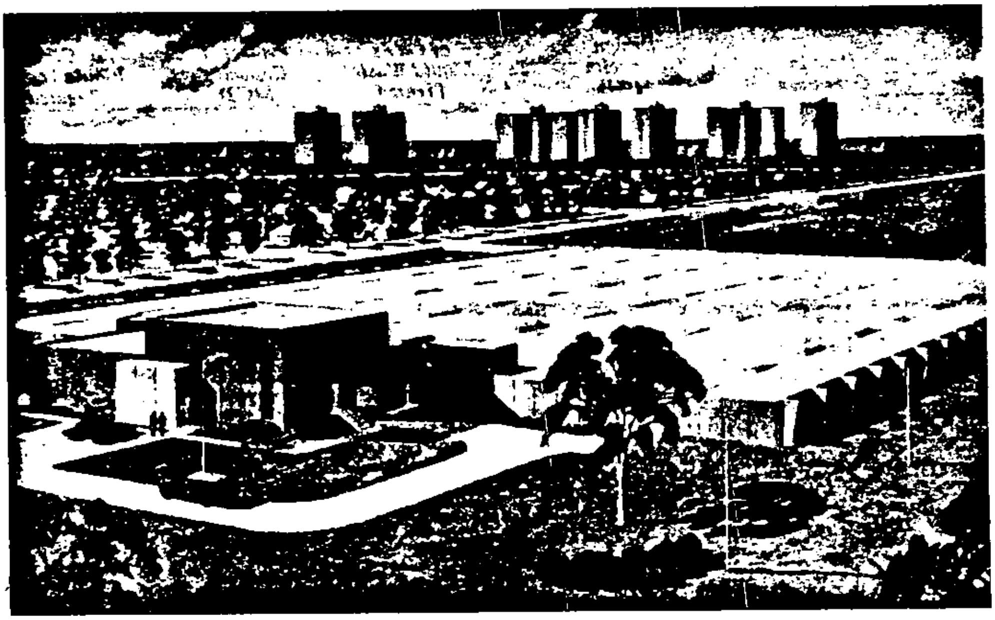

Figure 1. The Spring Creek Auxiliary Water Pollution Control plant now being built in Jamaica Bay is the first long step towards recreational use of the bay waters. The plant is the central element of the ecological study of the bay.

WATER RESOURCES MANAGEMENT IN THE TISZA VALLEY IN HUNGARY

by

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Introduction

Half of Hungary's 92,000 sq. km. territory and about 40 percent of its approximately 10 million inhabitants and national wealth is situated in the Tisza Valley. The Tisza River is the biggest left-handed tributary of the Danube.

The 157,000 sq. km. catchment area of the 761 km long Tisza River includes the central, northeastern, and eastern part of the Carpathian range.

Mountains (belonging to Czechoslovakia, the Soviet Union, and Rumania) cover half of the catchment area. Fifty-six percent of the plains area of the basin is in Hungarian territory. The river's confluence with the Danube River lies in Yugoslavia, 272 km upstream of Iron Gate Barrage (Figure 1).

Water resources management in the Tisza Valley has been a purposeful development activity for 150 years. Flood control and reclamation works finished by the end of the nineteenth century are comparable in size to those of Holland, the Po Valley, or the Mississippi Valley.

Water management experiences gained in 150 years in the Tisza Valley are numerous and diverse. The distance in time enables objective evaluation of completed civil engineering works apart from changing socioeconomic conditions and different aims and stages of economic development.

Writers taking part in recent development work draw upon the published articles of civil engineers of different periods dealing with the same problems.

Prospective development plans are based on a study under the direction of Imre Dégen, Secretary of State, President of the National Water Authority, presented at the inauguration of the Kisköre Barrage--second on the Tisza River--in May 1973.[1]

The authors deal with questions of general interest in the following chapters:

1. Geological, geographical, and hydrometeorological conditions of the Tisza Valley.

2. Damage prevention:

   - Flood control and regulation between 1846 and 1973,

   - Problems of reclamation and drainage works,

   - The hydrometeorological data and the main experience of the record flood of 1970.

3. The development of multipurpose water resources management:

   - Irrigation works built in the preceding 30 years,

- The canalization of the Tisza River. The conception of present-day water resources management in this region,

- The Kisköre and Csongrád Barrages and Irrigation Sytems,

- The complexity and problems of implementing these systems and priorities among investments.

4. International aspects of prospective development schemes:

   - The C.M.E.A. cooperation plans in the Tisza Valley,

   - Description of the Yugoslavian Danube-Tisza Danube canal and the Hungarian Danube-Tisza canal schemes,

   - The development of international navigation.

1. Geological, geographical, and hydrometeorological conditions of the Tisza Valley

The Great Hungarian Plain was formed by alluvial deposition from the surrounding mountains into the Pannonian Sea of the Tertiary period, which was filled by the beginning of the Pleistocene Era, about a million years ago. In the absence of definite slope, rivers meandered across the plains changing their beds frequently and sustained vast marshes.[1,2]

The climate, basically continental in character, is occasionally influenced by Mediterranean and Atlantic air currents. A regular alternation of wet and dry seasons is attributed to this. In general, two to three arid and two to three wet years have been recorded per decade. Temperatures range from -30° to $+40^{\circ}$ centigrade. There are nearly 2,000 hours of sunshine per annum.

Average annual precipitation and runoff volumes of the Tisza River are shown in Table 1. Characteristic data of river regimes in the Tisza catchment area are summarized in Table 2.

The aridity factor (the ratio of potential evaporation to precipitation), which is of fundamental importance for plant growth, ranges from 1:2 to 1:4. The seasonal distribution of precipitation also is adverse to agricultural production, in that lowest discharges occur in summer at the peak of the irrigation season.

The alluvial soils in the plains are not uniform in quality. Meadows, marshes, and so-called "top silt" layers (soils with rather low permeability found along the rivers) can be found on the 1.2 million hectares of drained land.

Saline soils cover 350,000 hectares. Water circulation in these soils is poor. Their vegetation suffers badly from droughts.

The border of the plains is sandy, in the vicinity of Debrecen there are loess soils.

The depth of the water table below the ground surface is, in general, from 2 to 6 m.

The characteristic cross-sectioned shape of the riverbed is a so-called "alluvial cone" (Figure 2) which may rise to 2 m over the surrounding terrain.

Before regulation, vast areas remained flooded for months, or even years, because of slow return of water to the riverbed.

In this region, a primitive method of flood-plain farming, adapted to the prevailing natural conditions, had evolved before regulation work started and played an important role in medieval economic life. Flood-plain farming was practiced as early as the thirteenth and fourteenth centuries as part of an economy that included fishing, stock breeding based on storage irrigation of meadows, the operation of water mills, and the production of building materials and basic materials for domestic industries. This economy was destroyed by warfare and the Turkish rule of 1450-1678. After the resettling of the plains, started in the eighteenth century, development was frequently interrupted by famines and losses of stock after droughts or floods.

2. Damage prevention

2.1 The beginning of water management

By the beginning of the nineteenth century, it was generally recognized that even under the existing feudal circumstances the regulation of the Tisza River was a prerequisite of economic development. Reclamation of arable lands for the landowner class became technically and economically possible.

Major projects were begun around the middle of the nineteenth century and consisted of simple flood control and regulation works coordinated with the marshland drainage projects.

After 1895, riverbed protection, reclamation and drainage works were emphasized. At present, a complex water resources management view prevails.

2.2 Flood control and regulation works

The Tisza Valley Flood Control Association, a union of several local associations, was founded in 1846 as the result of incessant efforts by Count István Széchenyi. A master plan of regulation was elaborated by Pál Vásárhelyi, an eminent engineer, who advised the cutting off of meanders to shorten the course of the stream and increase its gradient and the construction of flood levees, first along the river's upper reaches, then downstream.

Owing to a lack of capital, important aspects of his plan did not near completion for 100 years.[2,3]

Table 3 shows the development of flood control works, the extension of protected areas, and important data of later floods.

Accompanying this flood-control work, 112 meanders were cut by 1890, shortening the course of the river by 37 percent. At present, 1.8 million hectares of land along the Tisza River and her tributaries are protected by main-line levees extending over a length of 2,800 km. (Figure 3). The river's length decreased from 1,214 km to 761 km. Her average slope increased from 3.8 cm/km to 5.89 cm/km. The regulation work may be divided into two periods.

In the first period, owing to local interests, lack of capital and lack of experience

- Meander cutoffs, generally reducing the length of the affected section by two-thirds, were carried out simultaneously on the upper and lower stretches of the river,

- Reaches protected by levees were not uniform and were not sufficiently long,

- Cutoffs along the upper stretches of the river became riverbed; those along the lower stretches did not develop without dredging,

- In the original plan, the width between the levees varied from 550 m to 1,800 m. In the implementation, narrow stretches were left, as at the Szeged fortress where the width is 150 m. Moreover, in an effort to economize, additional bottlenecks were created, as at the new Szeged railway bridge with 352 m spans,

- Owing to local technical difficulties, or special interests, levees were often arbitrarily placed and poorly constructed.

Despite circumstances adverse to the starting of regulation works, 1,243 km of levees requiring 5.6 million cu. m. of earth were completed between 1846 and 1872.

A regulated section of the river with levees and cutoff meanders is shown in Figure 4.

A section of the flood bed, the area of flood plain between levees, is shown in Figure 5. In some such areas where levees were too widely spaced, it has been possible to turn previous errors to advantage in water-storage projects. The cross-sectional development of an individual levee is shown in Figure 6.

The 1879 flood disaster at Szeged completely destroyed the city of 60,000 inhabitants, brought a turning-point in the history of regulation, and led finally to a suitable solution of flood-control problems. These later works were carried out under the direction of the Hydrographical Department, founded in 1886 as a service of the Ministry of Transport and operated according to modern scientific principles of Jeno Kvassay.[4]

After appropriate river bed and width corrections, facilitated by a new national dredger fleet, the dikes were strengthened and obligatory cross-section dimensions were specified for all associations. The success of flood control work is shown by the fact that no dike breaches have occurred since 1895.

Further development in river training at the beginning of this century is characterized by mean and low-water bed regulation and river bed protection works and the construction of the Körös River barrage at Bökény--one of the first reinforced concrete structures in Europe.

Summarizing: increased river slope, reclamation of marshes and clearing of wood-lands changed the runoff coefficients. Floods became more destructive--stage fluctuations on the Tisza River increased by as much as 4-6 m, compared with pre-regulation conditions. The increase of peak height of floods can be ascribed to human activity and the spread of agriculture.

With the completion of flood control works, agriculture became safe and floods ceased. The population of the area increased tenfold within a century from 400,000 to its present value. The cessation of floods was not, however, entirely beneficial. A change of water balance of the soils decreased their fertility.

2.3 Reclamation and drainage

Reclamation of marshland and drainage of large, flat plains were begun in the early 1860's. The importance of a solution to the drainage problem is shown by the fact that, for much of 1942 an area of 560,000 hectares was covered by excess surface waters. 3.2 million hectares are now drained by 24,000 km of canals and a series of pumping stations with a total capacity of 500 cu. m/sec.

The specific drainage capacity of this system is 26 lit/sec. sq. km, capable of draining spring inundations in 21 days on the average. A specific drainage capacity at 37 lit/sec. sq. km. is envisaged for 1985. This would reduce the average drainage time to 12-14 days.[1]

2.4 The flood of 1970

A disastrous flood, with record crest heights, occurred between May 13 and July 4, 1970, in the catchment area of the Tisza River and in neighboring Soviet and Rumanian territories.[5] The Szamos, Kraszna, Tur Rivers, the three branches of the Körös System, and the Maros River were involved.

The extraordinary crest--exceeding the previous record by 107 cm on the Tisza River, 159 cm on the Szamos River, and 367 on the Maros River--must be attributed to an exceptional combination of meteorological conditions. The total rainfall in six weeks for the 138,000 sq. km drainage area above the Szeged section was 112 mm on the average, corresponding to a volume of 15.5 cu. km.

Superimposition of flood crests of confluent streams resulted in a crest at Szeged which exceeded the previous record by 38 cm. Hydrographs of the height and discharge of the flood are shown in Figures 7 and 8.

After inundation of 60,000 hectares on the first day of the flood, succesful emergency measures were organized on the 2,800-km long levee system (see Figure 3). Ten to forty-five thousand people worked to protect an area of 1.8 million hectares and hundreds of villages and towns. Economic assets, estimated at about $7,000 million, were involved in Hungary. Estimates of flood damage are as high as $150 million. The costs of flood fighting and reconstruction amount to $105 million.

3. The development of multipurpose water resources management

3.1 Barrages and irrigation systems built in the last 30 years 6

Large-scale multi-purpose water resources management began only in the 1950's with the canalization of the river. Systematic irrigation farming was legislated in the Irrigation Act of 1937. The 14,000 ha Tiszafured and 10,000 ha Hódmesövásárhely irrigation systems began operation in the early 1940's. Stable pumping stations, with 6 and 4 cu. m/sec. capacity, respectively, supplied river water. At roughly the summertime, the Békésszentandras Barrage was built on the Körös River, a tributary of the Tisza. The area irrigated by this project was 10,000 ha before and 25,000 ha after World War II.

The preliminary stages of the Tiszalök Barrages and Irrigation System were begun before World War II, and the first section of the Eastern Main Canal constructed. The Tiszalök Barrage was inaugurated in 1954 and the development of the entire 120,000 hectares irrigated area was finished in the late sixties.

It should be mentioned that the barrage was constructed at the same site and with the same retention level as was envisaged 90 years ago. The site of the barrage maximized the intake of the system's main canal, the 97-km long, navigable Eastern Main Canal, carrying 60 cu. m/sec of irrigation water. The Main Canal is divided into two branches near the Tiszavasvár combined regulation weir and navigation lock--the second branch was called Western Main Canal. It is 42 km long and carries 12 cu. m/sec of water. By means of the Eastern Main Canal, 10 cu. m/sec of water can be supplied to the Körös Valley Irrigation System.

The view of the Hortobágy "Puszte" (plains) was entirely transformed by fish ponds covering 6,000 hectares of alkaline soils. Drainage of the Tiszalök Irrigation System is ensured by the small Hortobágy River.

An interesting feature of the Tiszalök Barrage was its construction on a dry site as part of a meander cutoff.

Completion of the entire irrigation system required 15 to 18 years. Results included the spread of irrigation farming, fish ponds, water fowl breeding, and rice growing. The area of paddy fields reached 55,000 hectares and stabilized around 35,000 hectares. This is adequate to supply the total domestic rice consumption. Irrigated farms experienced major difficulties resulting from local conditions.

The following experiences based on the first years of the Tiszalök Barrage should be mentioned: Damming of flood water must be discontinued. The annual sediment load in the Tisza River at Tiszalök and Szeged amounts to 9 and 22 million tons respectively. Bedload transport is insignificant. During the first seven years of operation, approximately 5 million cu. m of sediment were deposited in backwaters. This has not interfered with the maintenance of the riverbed or in the runoff of flood waters. Before completion of the second Tisza Barrage at Kisköre in 1973, the water supply for irrigation was ensured by inexpensive floating pumping stations. From 2 to 12 electrical or diesel pumping units with capacities of 0.5 cu. m/sec. were used at each station (Figure 10). Altogether, 26 floating stations temporarily supplied about 80 cu. m/sec irrigation water.

3.2 The canalization of the Tisza River, the present concept of water resource management

The barrages needed for the complete canalization of the Tisza River were first indicated in the "20-year Master plan for water management" in 1952. Present irrigation systems on Hungarian territory (Figure 11) are:

- In the Tisza Valley:
  - The Tiszalök Irrigation System (fully functioning),
  - The Kisköre Irrigation System (partly functioning),
  - The Csongrád Irrigation System (design in preparation).
- In the Körös Valley:
  - The Békés Irrigation System (fully functioning),
  - The Körösladany Irrigation System (design in preparation).
- In the Maros Valley:
  - The Csanád Irrigation System (in preparation).

Over an important area of the irrigation systems under development, irrigation farming by pumped water supply is already in operation.

In Rumania, the border of the Great Plains between the Szamos and Maros rivers is suitable for irrigation and development is in preparation. In Yugoslavia, in the Bácska and Banat territories, multi-purpose water resources management is partially developed.

Development of the Tisza River and tributaries area must take the following circumstances into account. Monthly discharge values (Figure 12) are smallest in the second half of the irrigation season when water demand is greatest. The extremely variable regime of the Tisza River is unsuited for meeting, in any reliable manner, the growing demands of agriculture and industry, as long as it is left in its natural condition.

Shortage in water supply is hindering the development of irrigation and industry. This shortage is increased by conflicting water utilization schemes of neighboring countries. Variable management techniques must be standardized.

For meeting the demand for water in the Tisza Valley, several alternatives have been considered but none by itself would overcome the deficiencies. A low-cost water supply in summer can only be provided by a combination of canalization and storage in the flood plain and channel. For complete canalization, five barrages are contemplated in Hungary and one in

Yugoslavia. In addition to existing barrages at Tiszalök and Kisköre, an additional barrage at Csongrad appears to be necessary.

The Novi-Bechey Barrage in Yugoslavia will be completed in 1974. After 1985, two additional barrages upstream of Tiszalök will economically justified (Figure 13).

For meeting water demands, the following alternatives have been investigated: storage in mountain, hillside, plains-land, or channel areas, pumped storage, and the importation of water from the Danube River. It should be understood that the regime of the river and the topography of the valley do not permit the total equalization of discharges; only 10-15 percent of the annual runoff value can be stored.

Mountain storage is possible only beyond the frontiers of Hungary. It cannot be considered economically equally advantageous because water storage for flood control, irrigation, and power purposes are partly contradictory, and the cost of irrigation storage is high. (See Section 4.1).

Hillside storage possibilities are small and the cost of reservoirs is high. Three hundred million cu. m of storage volume has to be provided for the industry and communal water supply.

Plains-land storage occupies large farming areas, are subject to evaporation losses and is expensive. Possible plains-land storage amounts to 760 million cu. m.

Pumped storage was left out of consideration in the case of Kisköre because of high construction and operations costs, but seems reasonable at the Csongrád Barrage.

Importation of water from the Danube River is economically feasible as a multi-purpose navigation, water transport and peak power generating project. The construction of this complex canal takes into account the major Danube River discharges and is an important potential water supply for the Tisza Valley. Because of topographic conditions, the most advantageous route joins the Tisza Valley in the backwater reaches of the Csongrád Barrage. Impoundment storage in the flood plain and channel has proved to be the most economical solution for the Kisköre Barrage. Water storage in the river channel and the area of flood plain confined by levees, rather than in far-off plains-land reservoirs, raises the retention level above maximum flood stages and may be considered a novelty.

The filling of this reservoir will take place this spring in approximately a week when falling-stage flood waters are trapped. In the first stage of development, 300 million cu. m of storage capacity is procured by very economical means. Capacity can be increased to 400 million cu. m.

The Kisköre Barrage[1,7] has been in operation since 1973. The completion of its reservoir and canals will ensure the extension of communal and industrial water supplies and irrigation farming in the central parts of the Tisza Basin.

The Kisköre Barrage and irrigation developments constitute a multi-purpose system, ensuring:

- In eight years out of ten, an agricultural water supply, corresponding to a continuous flow of 179 cu. m/sec,

- The distribution of this discharge, mainly by gravity, across 300,000 hectares where irrigation farming is already practiced or is to be introduced--and to 12,000 hectares of fish ponds,

- The generation of a 103 million KWh of hydro-power annually at a 28 MW capacity power station,

- The creation of a 120 km long waterway to Tiszalök permitting passage of 1,350-ton vessels,

- Flood protection by enlarged embankments along a 65 km long section of river,

- Improvement of flow conditions, balanced sediment and ice movement and slope protection, by modernization of regulation works along 120 km of river,

- Drainage and underseepage control by reclamation work and intercepting canals parallel to flood levees,

- Decrease of the pumping operational costs by raising the retention level and abolishing most floating pumping stations,

- Recreation opportunities on the 127 sq. km lake. The reservoir lake and the canals are eminently suited for fishing. Reed and forest belts border the reservoir and sections of the main canals. New opportunities in the area contribute to the elimination of isolated farms, to the development of new settlements and farm centers. Completed or partially completed water supply facilities in 25 villages will bring about fundamental changes in life in the Tisza Valley.

An industrial development program, including a 2,000 MW capacity thermal power station, an oil refinery, and several chemical plants, may be started.

3.3.1 Description of the Kisköre Barrage, development of its irrigation system

The Kisköre Barrage raises the retention level in the reservoir to 0.5 m above the highest recorded flood stage.--The flow is regulated by five radial gates with flap in the weir (Figure 14). The backwater reach, created by the Kisköre Barrage, extends to the tailwater of Tiszalok Barrage 120 km upstream. The 40-km long lake, created by the barrage and confined by the strengthened original flood levees, has a surface area of 127 sq. km, a maximum width of 6 km, a 3.2 m average depth and a final 400 million cu. m storage capacity. Figure 15 shows the arrangement of the reservoir's levee, forest belt, and intercepting canal.

Construction scheduling is based on the anticipated increase of irrigation water demands in the following 16 years. The headwater level is at present fixed at El. 87.50 m A.O.D. It will be raised in stage II to El. 88.50 m A.O.D; in stage III to El. 90.50 m A.O.D., and finally to El. 91.50 m A.O.D.

Two 240,000-hectare irrigation systems on the two sides of the river receive their water through two main canals--the Jászság Main Canal, carrying 48 cu. m/sec, and the Nagykunsag Main Canal, carrying 80 cu. m/sec. Water is distributed from these by 14 section principal and lateral canals. The latter, when completed will carry 25 cu. m/sec of irrigation water to the Körös Valley. Sixty thousand hectares along the reservoir are irrigated directly from the river. The total length of the main canals is 190 km. Their slope is 2 cm per km. The number of major structures along the canals, including small pumping stations, is over 100. Construction cost of the barrage amounted to about 100 million dollars. Cost of works on the reservoir and irrigation system will be about 300 million dollars. The principle of distribution entitles the user to divert, within certain limitations, an ensured quantity of water at the desired time for irrigating his crops. This downstream-controlled system with control applied in the main and section principal canals, is solving distribution by automatic regulation of falling-stage flood water.[1,7] The water volumes diverted by consumers will be metered continuously.

Sprinkler irrigation sections are contemplated on two-thirds of the newly irrigated area. These would be fed and controlled by high-pressure automatic pumping stations, served by a buried main distribution pipeline system, with mechanically-transported 240-m long final local connections. This more expensive irrigation method is widely employed because of lack of manpower. Irrigated sections are covered in one operation by a water volume equivalent to 60 mm of precipitation. Recent utilization shows a trend towards the use of water volumes equivalent to 80 mm precipitation.

3.3.2 Water consumption

By the completion of the Kisköre reservoir, discharge conditions in the river will become more uniform and the available supplies for irrigation will increase. In preliminary studies, hydrological records for the past 60 years were analysed and variations in expected demand was represented by 10-day averages. This demand was compared with natural and modified supplies (Figure 16).

Storage capacities were assessed for meeting demands with 70, 80 and 90 percent probability analysed in several combinations (Figure 17).

The natural rate of flow in the critical period is 220 cu. m/sec of water. From this quantity, 157 cu. m/sec is obligatory flow discharge (minimum acceptable flow) to be left in the riverbed or reserved for other uses, e.g., communal and industrial water supply, international obligation. Agricultural consumption is considered only 63 cu. m/sec. By storing 300 million cu. m, the discharge available for irrigation may be increased by 179 cu. m/sec. The surplus from the prospective live storage capacity of 400 million cu. m is necessary to meet the growth of other demands. The reservoir, as mentioned before, will provide irrigation water for about 300,000 hectares and will supply 12,000 ha of fish ponds.

Existing irrigation plots also will be supplied from the newly constructed main canals. In this way, the practice of conveying irrigation water in drainage canals can be discontinued. The water needed for large-scale stock breeding and diluting water permitting sewage from these operations to be used for irrigation is conveyed by distribution canals to farming centers.

3.3.3 The Csongrád Barrage

The final addition to the chain of barrages in preparation will complete the canalization of the river in the central and lower Tisza region. From the El. 83.50 m A.O.D. retention level of the contemplated barrage, its main reservoir of 180 million cu. m volume (Alpar reservoir) will be filled by pumping water to El. 88.00 m A.O.D. Completion of the Csongrád System in 1985 will add 80,000 hectares to the area under cultivation. The irrigated area in the Tisza Valley will be about 500,000 hectares in Hungary and about 400,000 hectares in Yugoslavia. To meet the demands of the enlarged systems, importation of water from the Danube River will be inevitable.

3.3.4 Coordination of investments with development projects of agricultural establishments in the Kisköre Irrigation System

Hungarian agriculture remained almost unaltered for many hundreds of years until the last century. A feudal system until the end of World War II, a semi-feudal society until about 1960, it was transformed in a relatively short 10 year period--for the beginning of a highly industrialized socialist farming era. With the rapid industrialization of the country, the proportion of the population engaged in agriculture decreased from half to one quarter. The decrease is continuing. Fifteen to 20 percent of agricultural production is accomplished on large state-owned estates, 50 to 55 percent on cooperative farms, and 30 to 35 percent on private household farm plots of farmers in the cooperative. The farms have, on the average, moderate mechanization little specialized. Large scale utilization of chemical products in farming is becoming more common. As can be perceived from this brief outline, there is still opportunity to develop dry farming.

An attempt has been made, using the Kisköre Irrigation System as an example, how the diverse functions in water resources management may be directed in coordination not only with each other, but also with the general development of a region's agriculture, and to show how great infrastructural investments must be made 10 to 15 years in advance to ensure the growth of an economy.[1]

In order to exploit the benefits of irrigation, agricultural establishments in the area affected must be prepared to use irrigation water. For this task, the Tisza Region Agricultural Development Bureau was organized in 1966. Its primary responsibility is to make the necessary preparations in about 400 large-scale agricultural establishments in the area of nine counties affected by the barrage. A soil survey map at a scale of 1:100,000 for the entire valley and an irrigation sections map at a scale of 1:25,000 were prepared with the help of the Soil and Agrochemical Research Institute.

Planning is carried out in the following stages: the economic situation of the farming establishments is surveyed, eligible establishments are selected, medium-range plant development programs are prepared for these, consulting service is offered in agricultural-engineering problems, and farming activities of co-operatives participationg in the irrigation development program are continuously examined. By 1969, survey studies had been completed for 358 farming co-operatives with a total area of around 65,000 hectares of cultivated fields. The consulting service is in great demand.

Irrigation development from 1970 to 1975 will be financed from state funds at 24 model farming cooperatives in the project area. From the examination of agricultural establishments, it was concluded that in development stage I irrigation development preferably should be subsidized at farms with more advanced techniques.

The United Nations Food and Agricultural Organization (FAO) joined the agricultural development program and contributed 750,000 U.S. dollars in equipment for five model farms. The results attained by the FAO experts by linear programming are of great interest in that they optimize farming operations with the help of computers.

4. International aspects of prospective development schemes

4.1 The Council of Mutual Economic Aid (C.M.E.A.) cooperation plans in the Tisza Valley

The Carpathian Ukraine territory of the Soviet Union has a population of one million. The Czechoslovakian part of the Tisza River's catchment area has 1.3 million inhabitants. In both countries, important industrial plants and traffic centers are situated in the basin. In Rumania, industrial plants and large agricultural districts lie in the basin.

Mountain storage and high-head water power facilities are economically feasible on many small right-hand tributaries of the river's upper stretches in the North-East Carpathian Mountains in the Soviet Union and on a left-hand tributary in Roumania, where the closure of the Visó gorge offers an opportunity to design projects of 900 million cu. m storage. Investigations have begun for the utilization of 20 exploitable storage sites with a total capacity of 2,000 million cu. m. Industrial water utilization in the Upper Tisza region is important, as well. To meet water resources development demands in the Tisza Valley, the concerted common effort of the five interested countries is necessary. Since 1972, a C.M.E.A. subcommittee has been engaged in the elaboration of a common development program for the five countries. Outlines for the coordination of surveys, future projects, improvements in the exchange and forecasting of hydrometeorological data, organization of a common water pollution control service, and common flood control measures have been worked out.[1]

Implementation of multi-purpose developments began in Yugoslavia, in 1958 on the fertile plains of Bacska and Banat regions.[8] The project consists of the Danube-Tisza-Danube Canal and the Novi-Bechey Barrage scheduled to begin operation in 1974. The canal's intake is at Bezdan on the Danube River. Its 320 km long Bacska stretch makes use of long sections of the nineteenth century Franz Joseph and Emperor Franz canals. Connected to the headwater of Novi-Bechey Barrage, the canal crosses the Tisza River, and utilizing the old Bega Canal, reaches the Danube River via the 280 km Banat section.

Navigation on the canal is made possible by 13 navigation locks capable of handling 1,000-ton barges. Several sluices and diversions ensure its water supply at 60 cu. m/sec. The canal, partly in operation, will allow the irrigation of 360,000 hectares and the drainage of 760,000 hectares in the Bacska and Banat regions.[8]

4.2 The Danube-Tisza Canal project in Hungary

This multipurpose canal, having been proposed in many variations since 1715, is scheduled for completion after 1985. Recent preliminary plans[6] call for the importation of about 100 cu. m/sec water from the Danube River, which has a discharge varying between 2,000 and 1,400 cu. m/sec from May to September at Budapest. A 130-km canal will lead through the hilly region between the two parallel, southward-bound rivers (Figures 11 and 19). The canal branches from a dammed secondary channel of the Danube River at El. 96.90 m A.O.D. just downstream from Budapest. After a 30 km section with 2 percent slope, the canal reaches the hills. A 100 cu. m/sec pumping station and navigation lock No. 1 (12 x 85 m), both with 13-14 m lift, serve for the conveyance of water and as waterway for 1,350 t barges, respectively. After a 90 km meandering section near the ridge of the hills, the canal will descend about 20 m by means of three navigation locks and a power canal with a peak power plant through the Alpár reservoir to the reservoir and retention level of the Csongrád Barrage. The contemplated canal will shorten the water route from the Tisza Valley to Vienna and Rotterdam by 600 km.

4.3 The long-range development of international navigation

The Tiszalök, Kisköre, Novi-Bechey Barrages improve the conditions of navigation on their headwater reaches over a 270-km reach where formerly traffic was obstructed in summer by a number of shallows of 60-80 cm depths. After construction of the Csonrád Barrage, a 620 km section, almost the entire length of the Tisza River, will be navigable by 1,350 t vessels as a waterway conforming to the European III. and IV. class prescriptions except during a 60- to 90-day period in winter. After completion of the Rhine-Main-Danube Canal in 1981, not only Hungarian and Yugoslav, but also Soviet and Rumanian, ships can sail from the Upper Tisza Valley to Rotterdam, or through the Iron Gate to the Black Sea.

5. Conclusions

The authors hope they have succeeded in showing that multi-purpose water resources management of large areas is not a mere technical problem, although successful development schemes must be based on well-founded technical solutions. These infrastructural schemes calling for large investments and transformation of regional economies requires several generations. During this long process, new methods, new aims and improved technologies will be incorporated in the projects.

As already mentioned, natural conditions will change in response to human interference. Runoff coefficients and peak stages of regulated rivers will, for example, increase. Careful forethought will be required to meet and fight pollution problems. With the increasing world concern for environmental factors, expenditures will increase year by year.

The bulk of investment in the Tisza Basin serves agricultural purposes. Irrigation sections will be developed in successive stages, corresponding to increases in demand. For the authorities fixing the distribution of investment capital, it remains to find optimum dates for construction of barrages and main canals. They have to assess the interrelations between different users participating in the complex utilization of watercourses, in that some use the water, while others consume it, and they must find the best method for allocating costs among the interested parties.[9,10]

Figure 19 shows the barrages on the Tisza River, the connected distribution systems and August discharges in dry years. Figure a on the left shows conditions before the completion of the Kiskore Barrage. Figure b on the right, shows the distribution after the prospective development of the existing Kisköre and the Csongrád Barrages. The figures clearly illustrate how the balance of the region's water resources distribution, once disrupted by simple regulation and reclamation works, is being restored and made governable.

The schematic Figure 20 shows the extent of completed and contemplated works in the Tisza Basin between the years 1840 and 2000. A relationship is seen between the increase of population density plotted and the area of controlled flood plain, length of drainage canals, area equipped for irrigation, net storage volume and development of water demand.

The authors are fully aware, of course, that in the world there are many bigger projects completed, with exceptional means and efforts, in shorter time. But they think that this detailed description of experiences gained in the Tisza Valley, will be of assistance to those coping with similar tasks.

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Table 1

Precipitation and runoff in the Tisza Valley

Average annual precipitation

In the mountains:	1500 -1800 mm
On the plains:	600, on some places 500 mm

Precipitation in the growing season

Average year:	300-350 mm
Dry year:	150-200 mm
Wet year:	400-500 mm

Annual runoff volume of the Tisza River at Szeged

Dry year:	10.10^9 cu.m.
Average year:	25.10^9 cu.m.
Wet year:	40.10^9 cu.m.

Runoff volume of the Tisza River in summer

Dry year /1950/:	$2,9.10^9$ cu.m.
Wet year /1941/:	$22,0.10^9$ cu.m.

Extreme discharge ranges at Szeged

Minimum discharge:	95 cu.m/s
Maximum flood discharge:	4770 cu.m/s

Table 2

Characteristic data of the river regimes in the Tisza Valley

River	Gauging section	Stage fluctuation /cm/	Flood discharge cu.m/s	Mean discharge cu.m/s	Minimum discharge cu.m/s	Ratio of extreme discharges	Average runoff volume 10^9 cu.m/year
1.	2.	3.	4.	5.	6.	7.	8.
Tisza	Tokaj	1056	4000	464	53	76	14,6
Tisza	Szolnok	1141	3820	546	65	59	17,2
Tisza	Szeged	1173	4700	810	95	49	25,1
Szamos	Csenger	839	1350	620	15	90	3,8
Bodrog	Felső-berecki	651	1300	120	4	325	3,7
Sajó	Felsőzsolca	486	520	32	2	217	1,0
Hármas-Körös	confluence	1181	1330	67	4	333	2,1
Maros	Makó	658	1800	160	22	82	5,0

Table 3

Major floods on the Tisza River. Development of flood control in the Tisza Valley

Year	Area protected by levees along the Tisza River	Flooded area	Protected area	The protected area vs.protected flood plain	The protected area vs.all, formerly flooded areas	Stages on the gauge at Szeged
	in hectares			%		/cm/
1855	264 000	107 750	156 450	59,3	13,0	687
1860	669 400	475 000	475 000	71,0	39,5	670
1867	731 500	94 800	626 700	86,9	52,2	722
1876	812 000	52 150	759 850	93,6	63,5	786
1879	812 000	31 000	781 000	96,2	65,9	806
1881	875 000	56 900	818 100	93,5	68,2	845
1888	1 050 000	120 000	930 000	88,3	77,5	847
1895	1 050 000	29 000	1 021 000	98,0	87,0	884
1919	1 050 000	-	1 050 000	100,0	87,5	916
1932	1 050 000	-	1 050 000	100,0	87,5	923
1970	1 080 000	-	1 080 000	100,0	90,0	961
along the tributaries in 1970	720 000	60 000	660 000	91,6	-	-

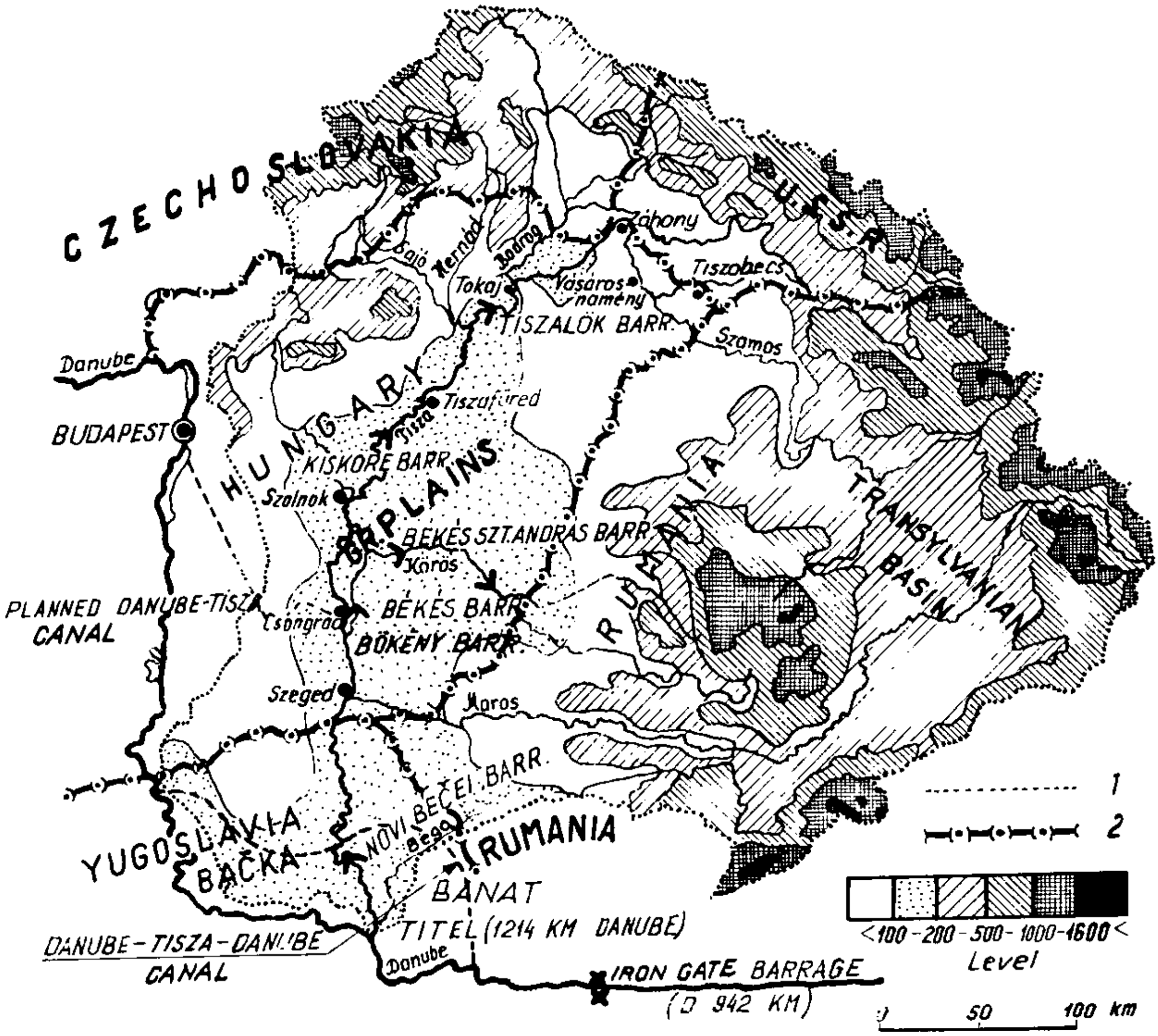


Figure 1. The catchment area of the Tisza River in the eastern part of the Carpathian Basin.

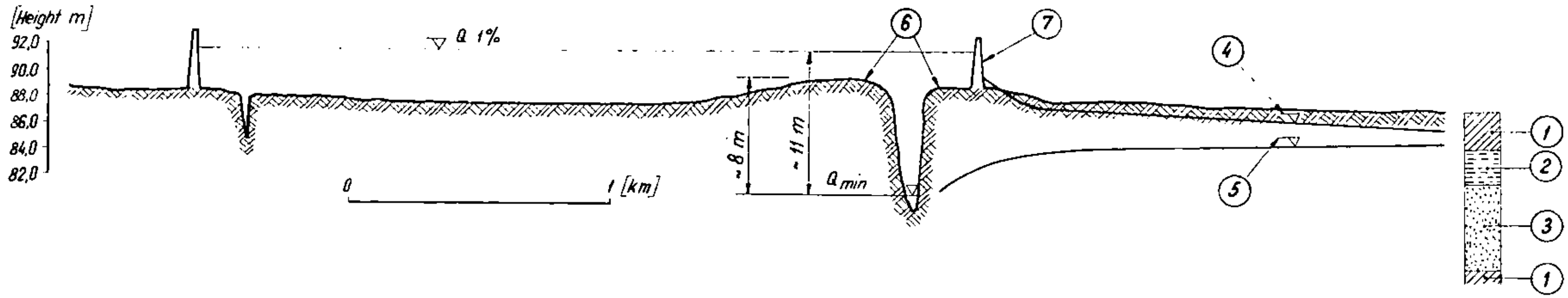


Figure 2. Typical cross-section of the Tisza River's channel and flood plain, Piezometric head of ground water vs. river stage:

1. Clay (impervious)
2. Transition layer (silty fine sand, K = 0,2 m/day)
3. Sand (k = 4 m/day)
4.
5. } Pressure gradients in different soil layers
6. Alluvial cone
7. Flood levee

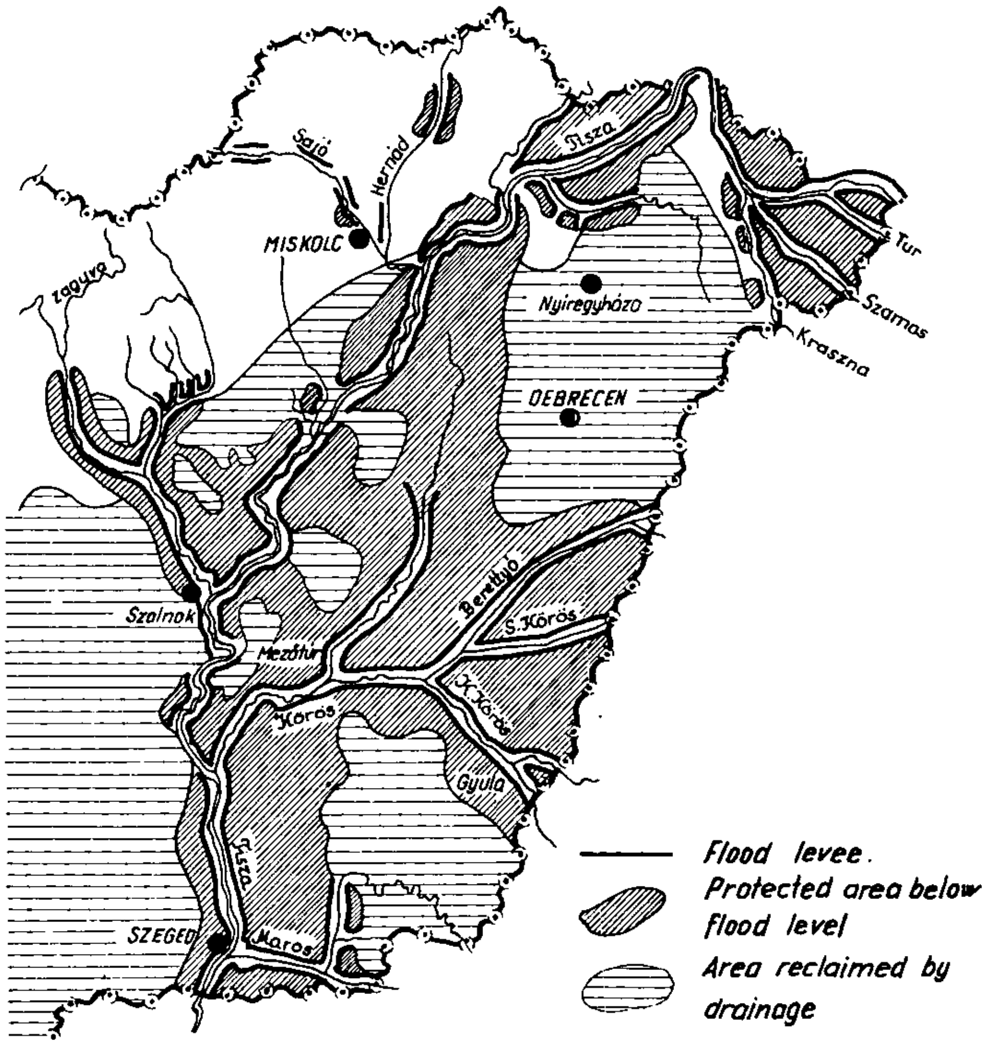


Figure 3. Flood control, levee system and drainage in the Tisza Valley.

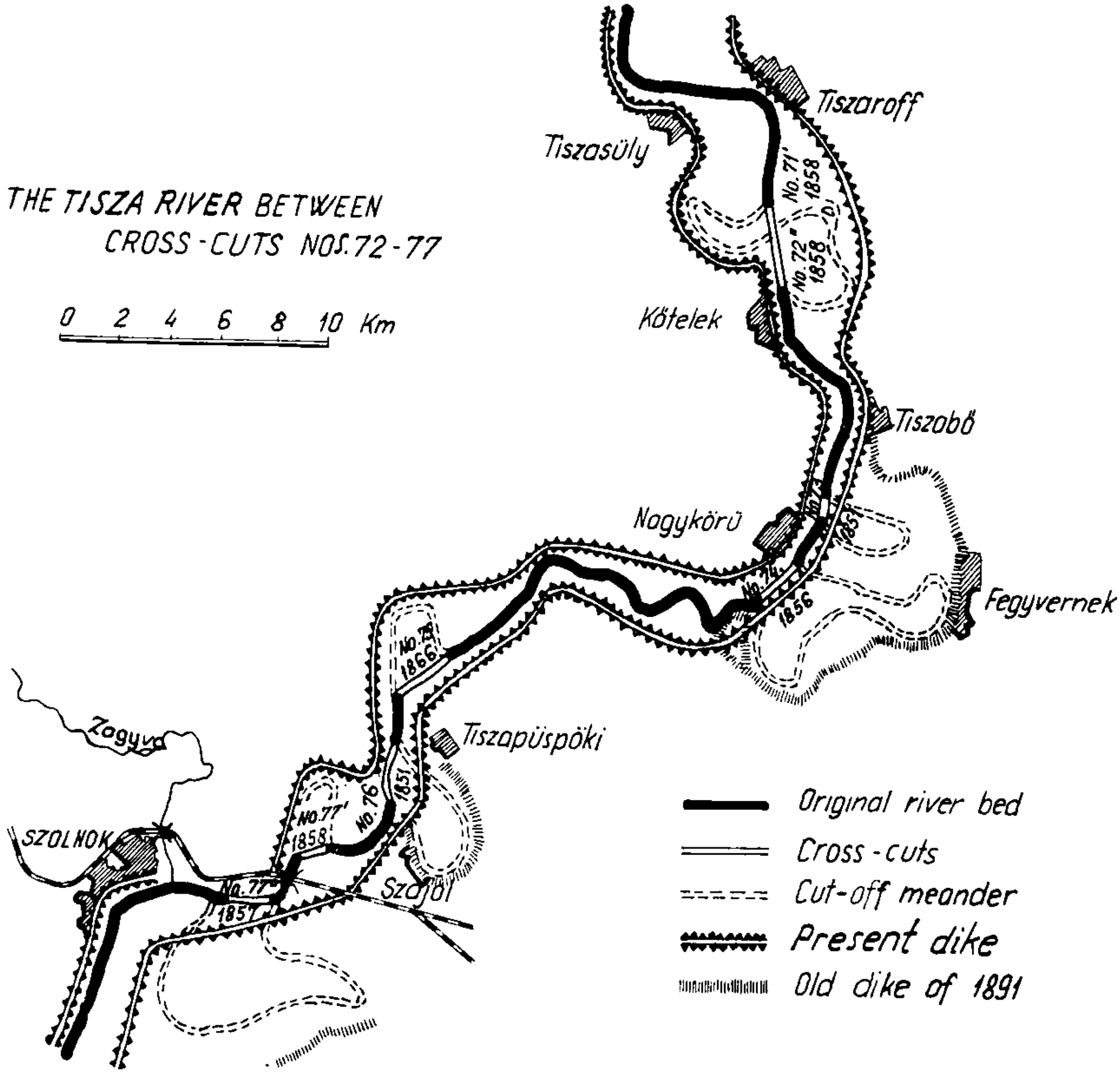


Figure 4. Cutoff meanders in the Tiszaroff-Szolnok section of the river (between cross-cut numbers 72-77).

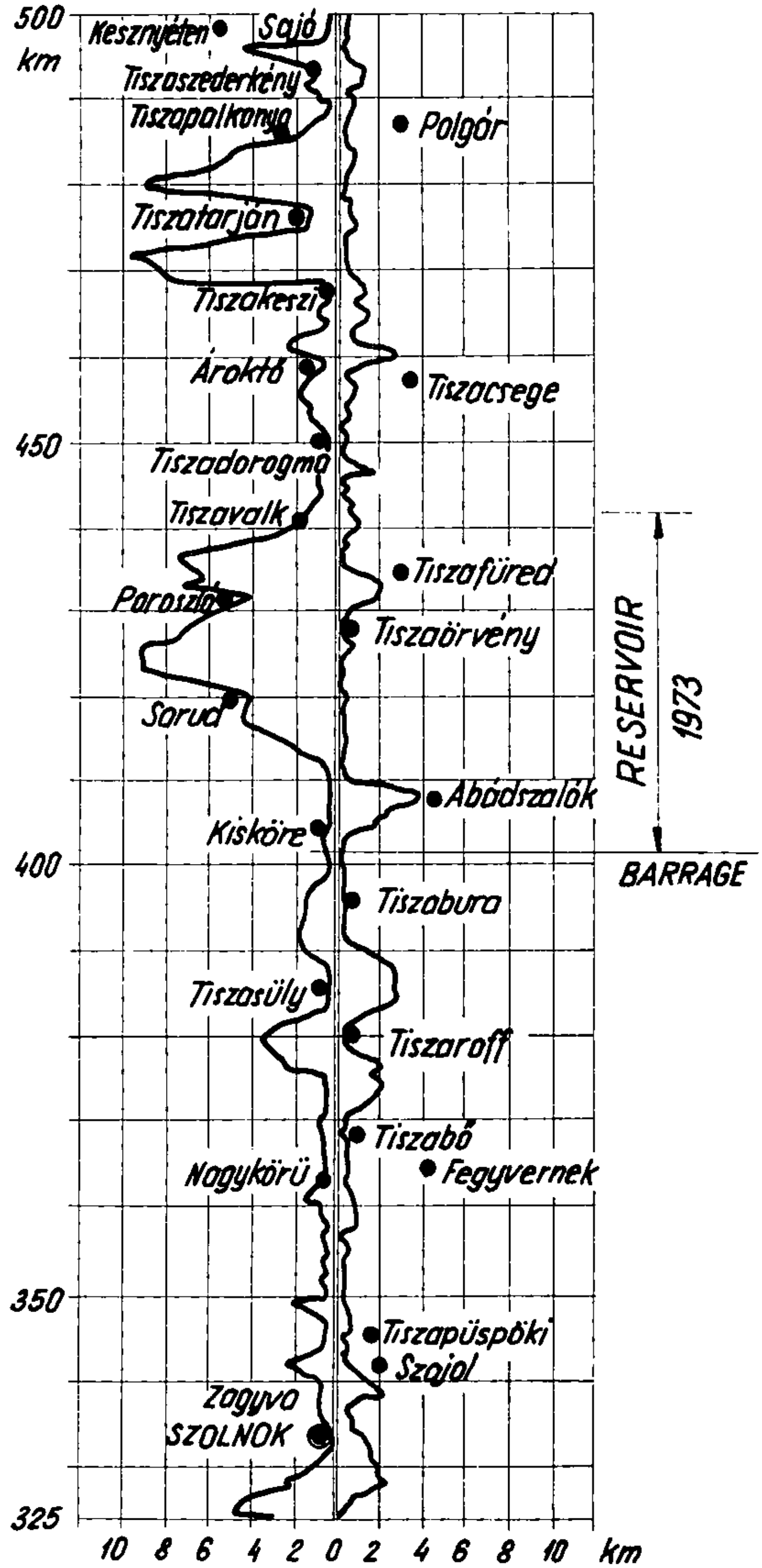


Figure 5. Schematic plan of the foreshore of the Tisza River between Rsts. 325-500 km of the Tisza River.

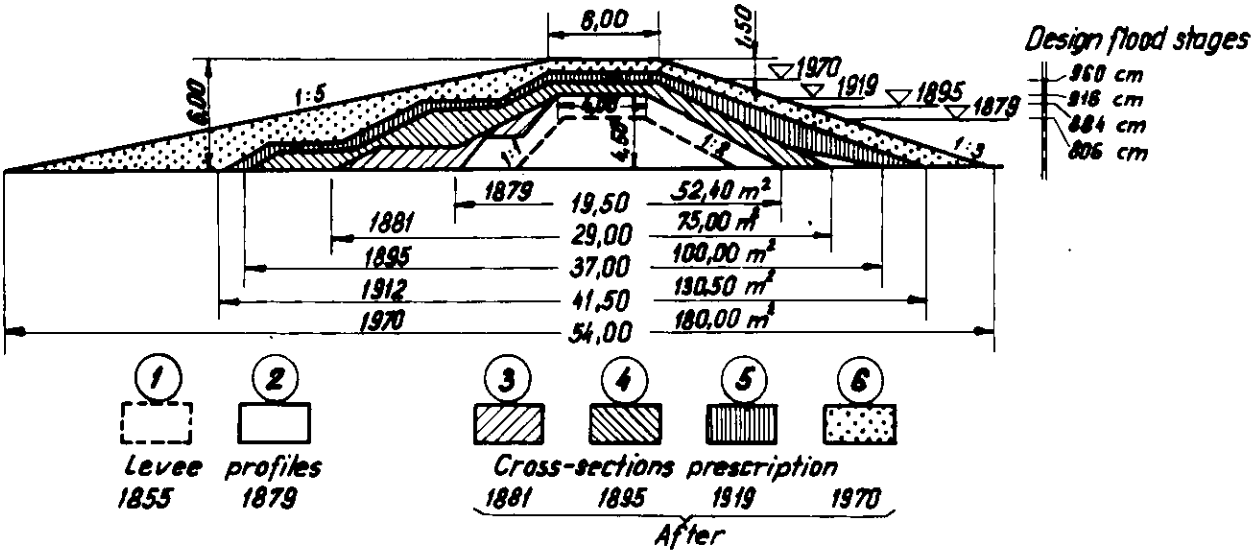


Figure 6. Development of levee cross-sections on the lower stretches of the Tisza River.

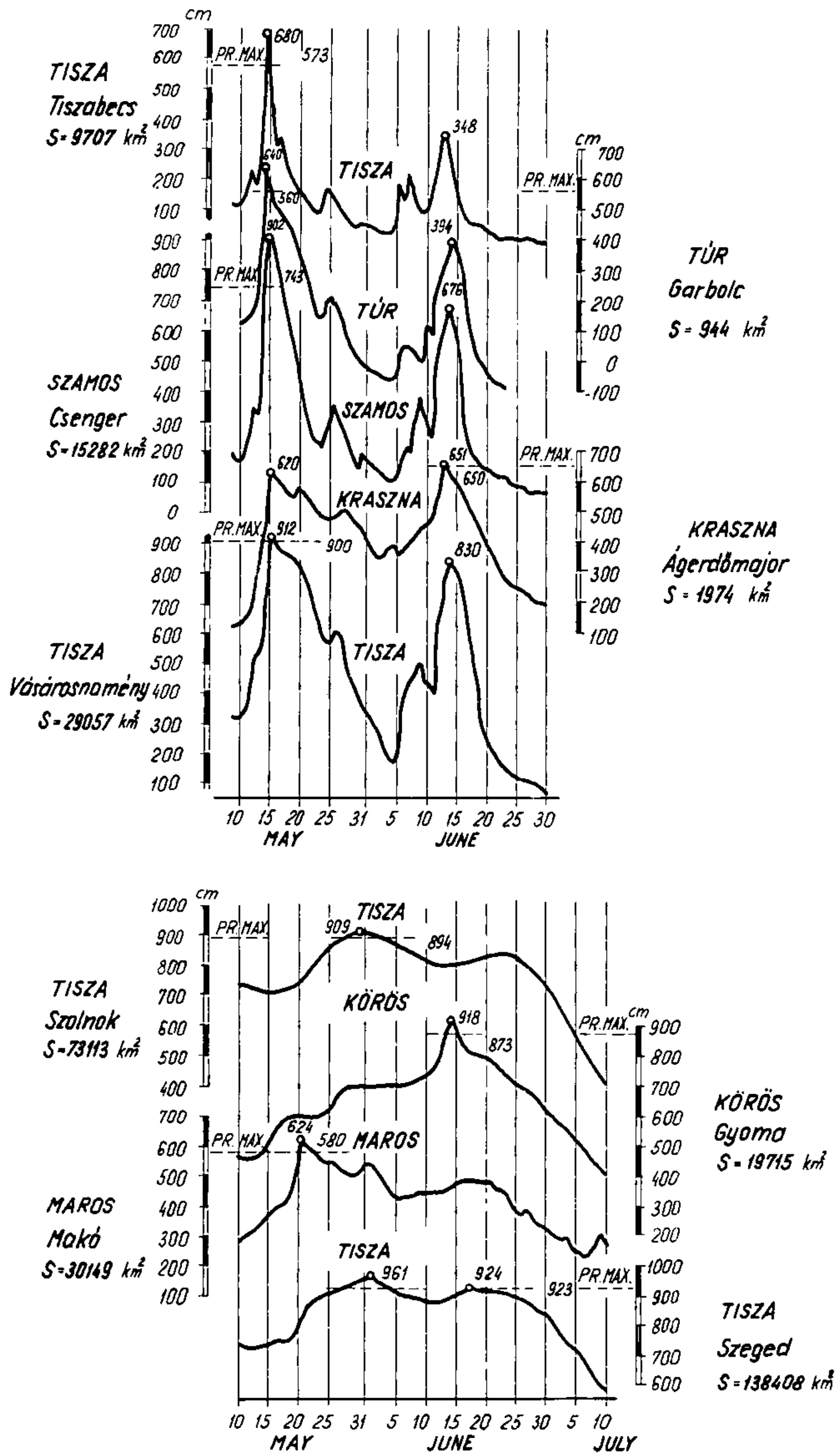


Figure 7. The travel of the 1970 flood waves on the Tisza River and her tributaries. Variations of river stages at different gauging stations.

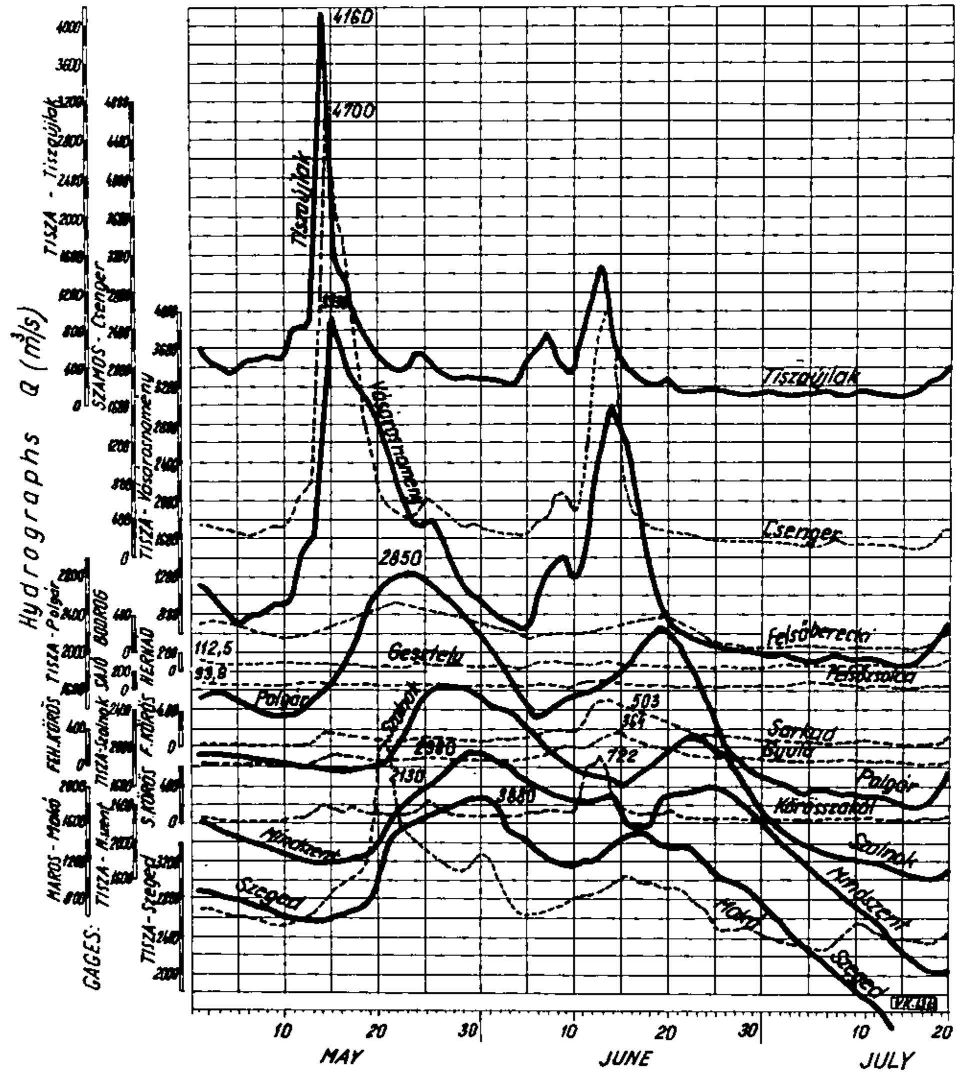


Figure 8. Hydrographs of the 1970 flood indicating the discharges conveyed by the Tisza River and her tributaries--at representative gauges between Tiszaujlak (Tiszabecs) and Szeged.

Figure 9. Aerial view of Tiszalök Barrage power station, three 37 m weir openings--17 x 85 m navigation lock. Closed old riverbed in the background.

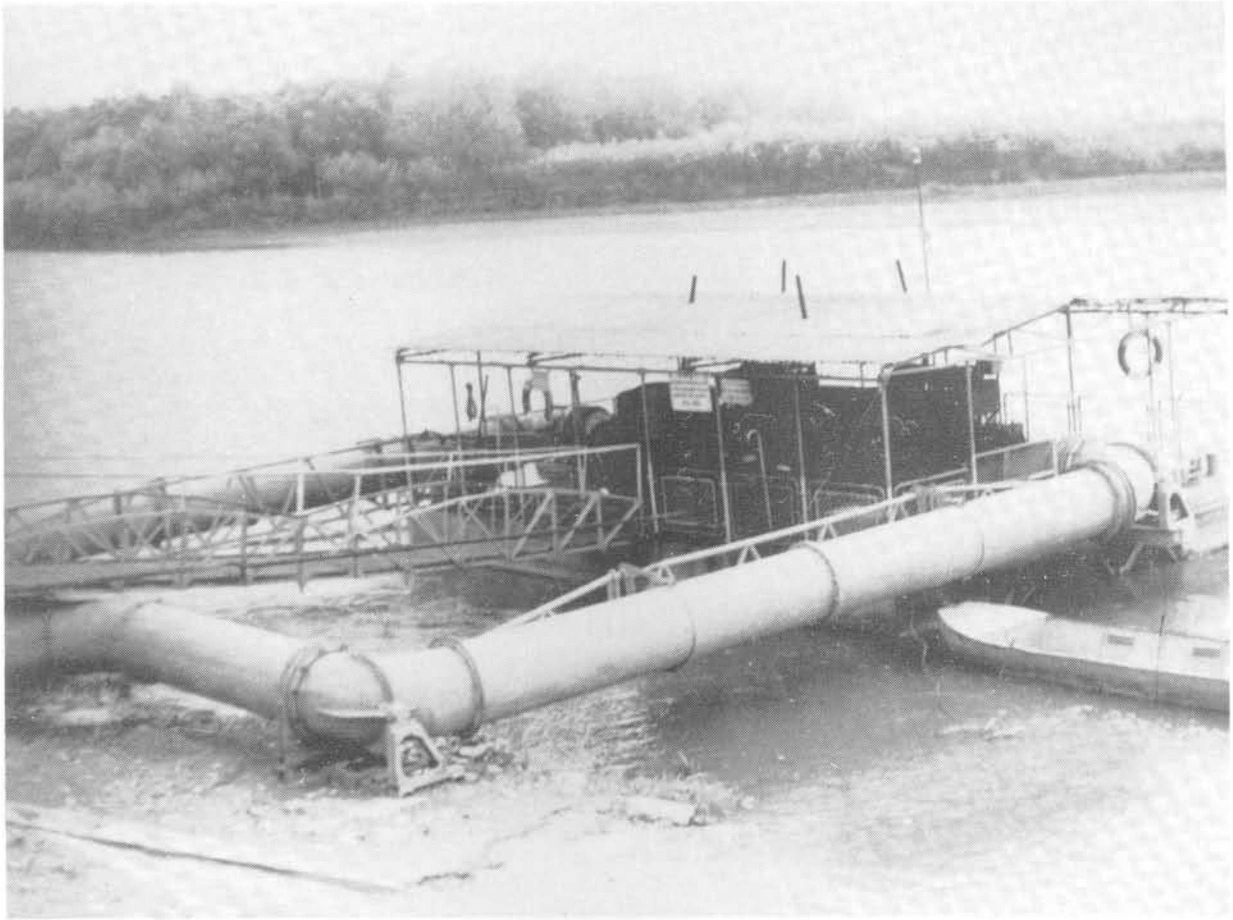


Figure 10. Floating pumping station with several 0.5 cu. m/sec pump units.

Figure 11. Plan of irrigation systems and main canals in the Tisza Valley
Irrigation systems:

1. Tiszalök Irr. Syst.
2. Kisköre I.S.
3. Körösladany I.S.
4. Csongrád I.S.
5. Bekes I.S.
6. Csanad I.S.

Main canals in the Tiszalök Irrigation System:

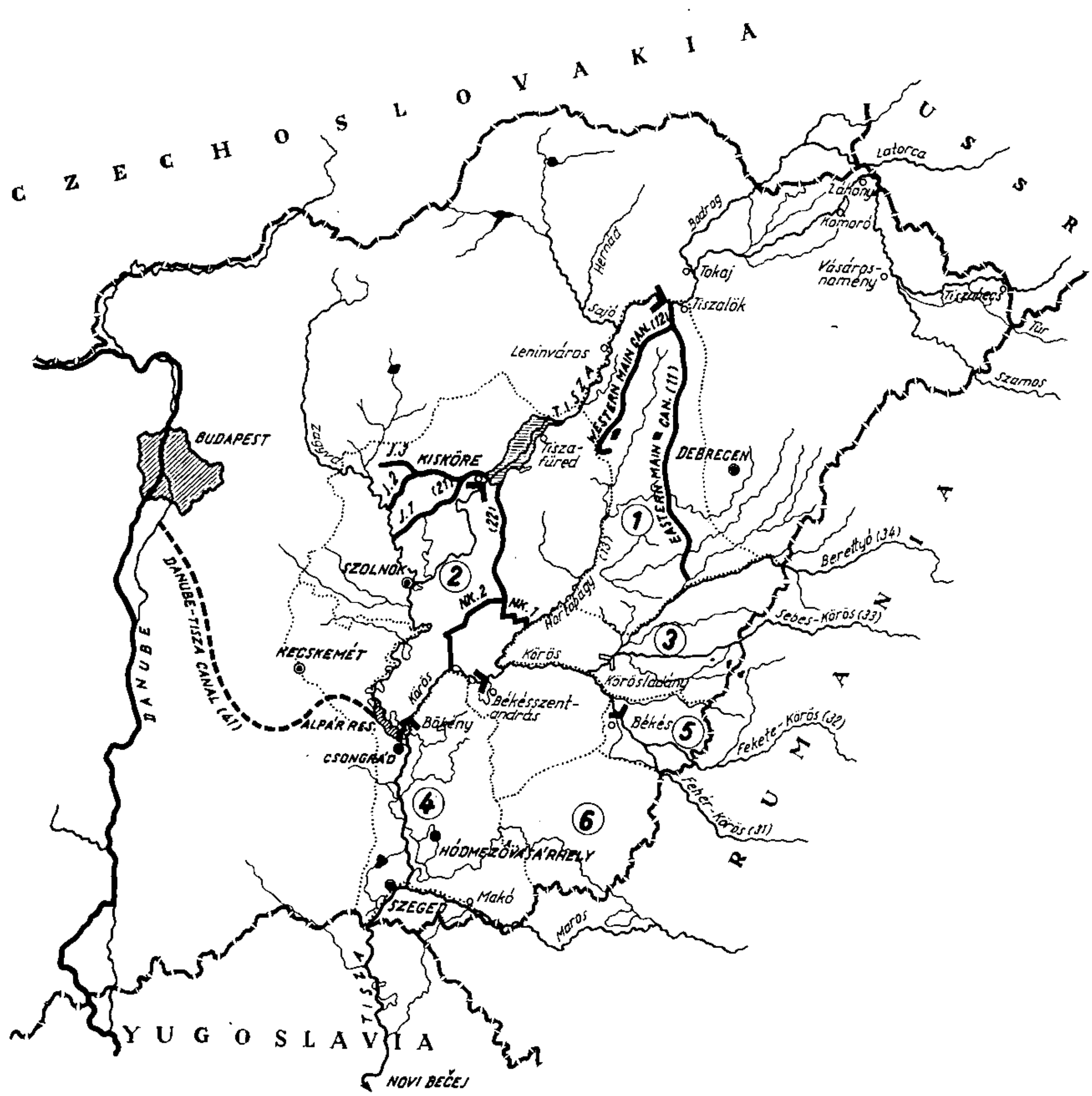
11. Eastern Main Canal (60 cu. m/sec)
12. Western Main Canal (12 cu. m/sec)
13. Canalized Hortobagy River (partly for drainage)

Main canals in the Kisköre System:

21. Jászság M. C., with branches J.1-J.2 (48 cu. m/sec)
22. Nagykunság M. C., with branches NK 1, NK 2 (80 cu. m/sec)

Tributaries of the Tisza River:

31. White Körös
32. Black Körös
33. Sebes Körös
34. Berettyó
41. Danube-Tisza Canal interconnected to the planned Csongrád Barrage reservoir
42. Alpar reservoir interconnected in the Kiskore Barrage



CZECHOSLOVAKIA
USSR
RUMANIA
YUGOSLAVIA
BUDAPEST
DANUBE
DANUBE-TISZA CANAL (41)
KISKÖRE
Tisza-füred
Leninváros
TISZA
WESTERN MAIN CAN. (12)
EASTERN MAIN CAN. (11)
DEBRECEN
Tokaj
Tiszalök
Sajó
Hernád
Bodrog
Latorca
Záhony
Komoró
Vásáros-namény
Tiszabecs
Túr
Szamos
Zagyva
SZOLNOK
KECSKEMÉT
ALPÁR RES.
CSONGRÁD
Bökény
Körös
Hortobágy
Berettyó (34)
Sebes-Körös (33)
Fekete-Körös (32)
Fehér-Körös (31)
Körösladány
Békésszent-andrás
Békés
HÓDMEZŐVÁSÁRHELY
SZEGED
Makó
Maros
NOVI BEČEJ

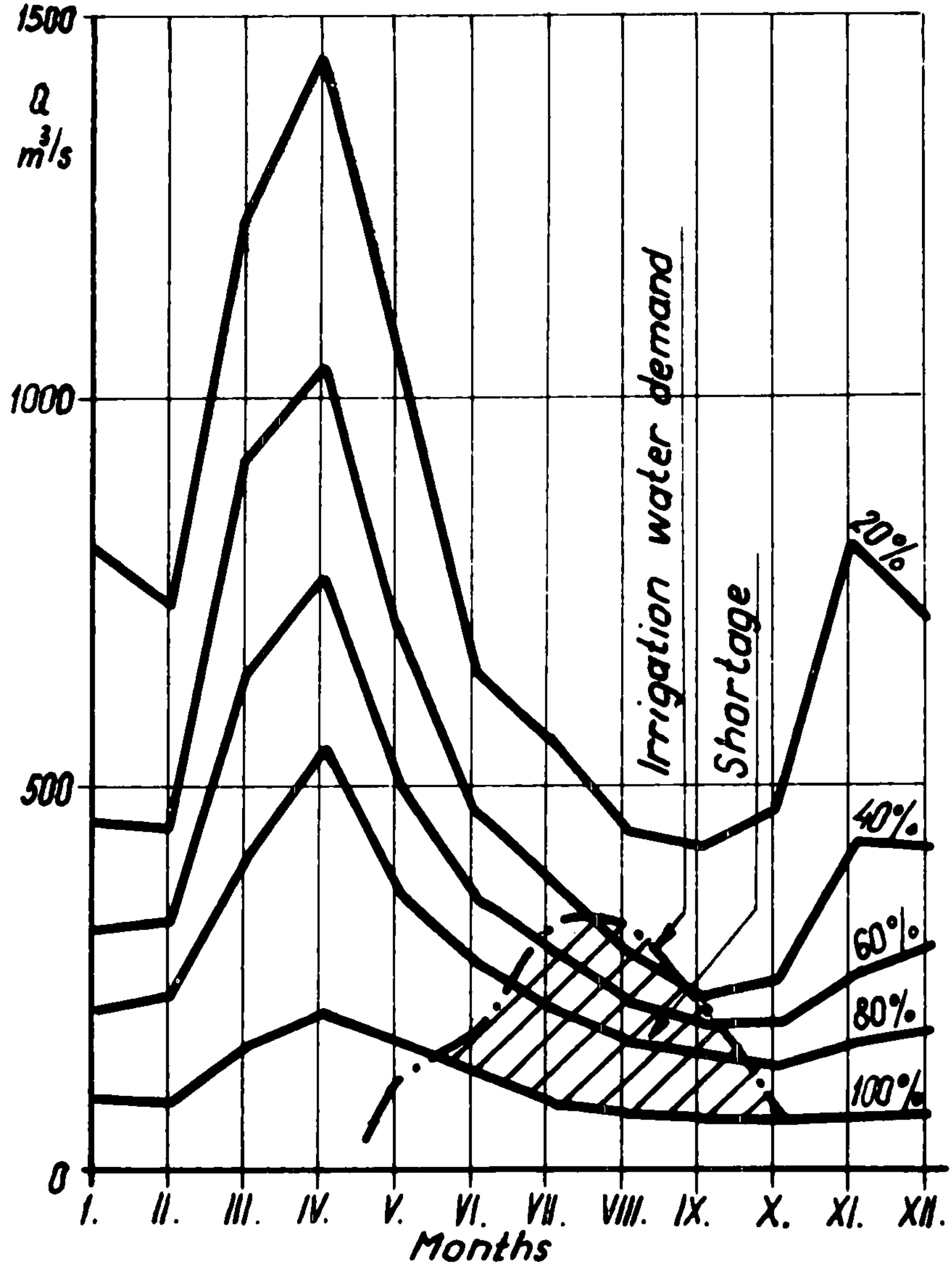


Figure 12. Monthly duration of discharges in the Kisköre Barrage section (1901-1960):
I. Water demand for irrigation

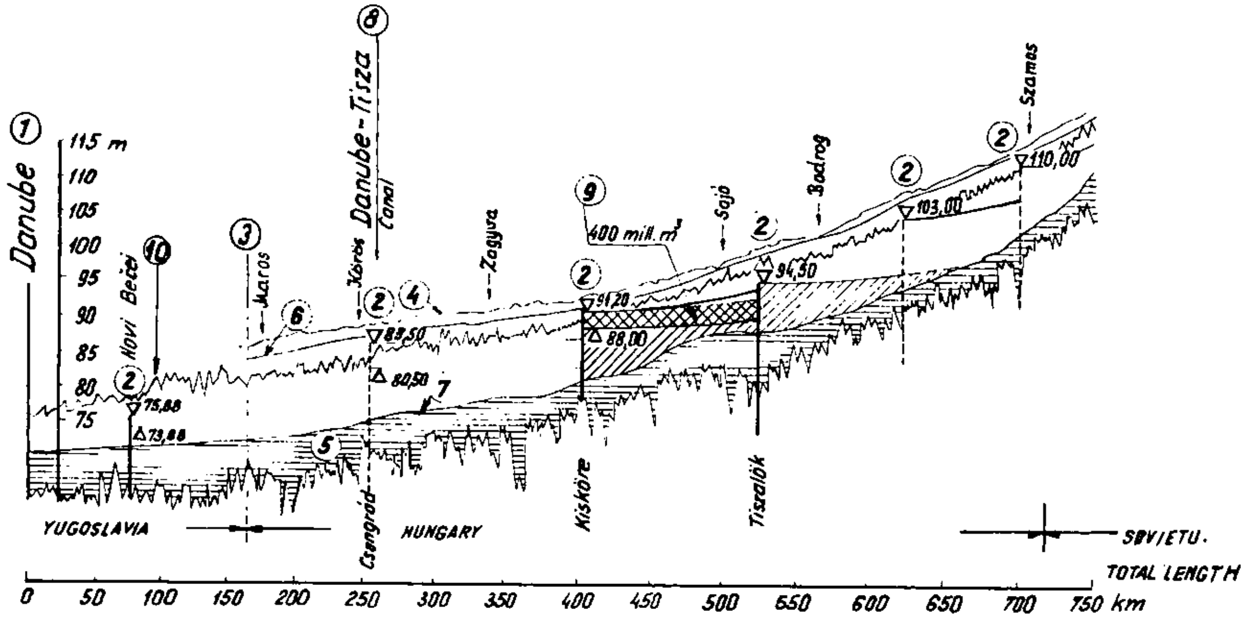


Figure 13. Longitudinal profile of the Tisza River and her barrages:

1. Confluence with the Danube
2. Barrages completed, contemplated, or under construction
3. Border between Hungary and Yugoslavia
4. Levee crest
5. Bottom
6. Highest flood level
7. Lowest water level
8. Danube-Tisza Canal
9. Net storage volume, 400 million cu. m
10. Danube-Tisza-Danube Canal in Yugoslavia

Figure 14. Aerial view of Kisköre Barrage (photo by O. Katanyi).

At left power station with four tubular turbine sets, five weir openings with 24 m width, a navigation lock of 12 x 85 m. For maintenance operation, two 40-ton cranes for service bridge. Closed riverbed in the background.

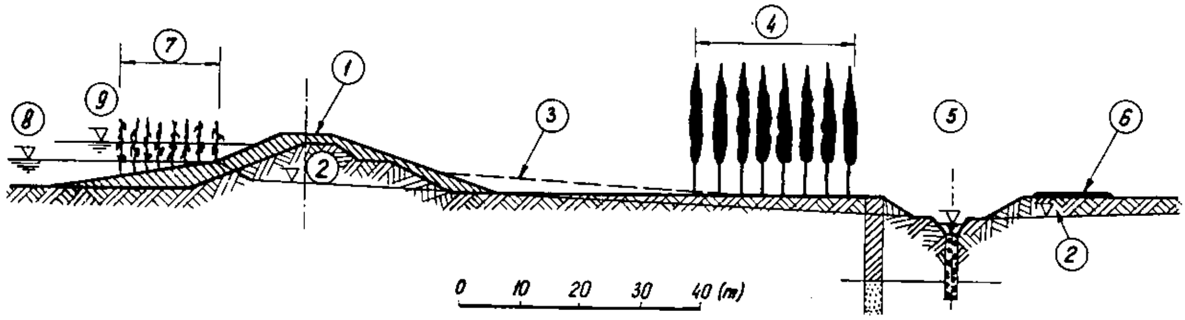


Figure 15. Section trough reinforced levee of the Kisköre reservoir:

1. Reinforced levee
2. Depression curve pertaining to raised water level
3. Dredge deposit
4. Forest belt
5. Intercepting canal with relief wells
6. Dirt road
7. Reed belt
8. Raised water level
9. Flood level

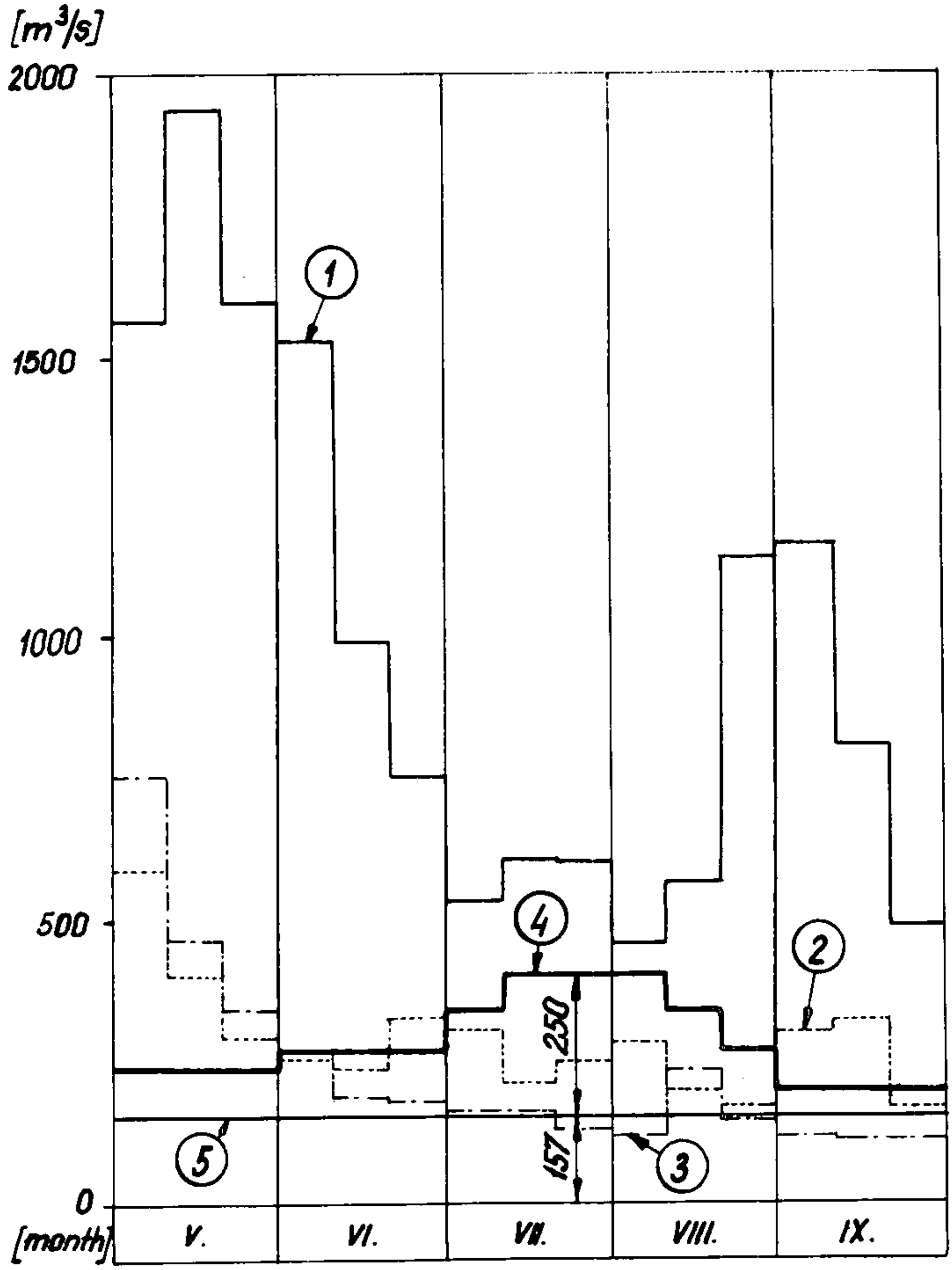


Figure 16. Water demands that can be met:

1.
2. Natural discharge hydrograph during the irrigation season (10-day averages) in different wet, average, and dry years
3.
4. Assumed distribution (in time) of consumption by agriculture (Q_{max} = 250 cu. m/sec)
5. Assumed distribution of other consumption (industry, communal, communal, minimum acceptable flow in the riverbed, discharge for abroad (Q_{max} = 157 cu. m/sec).

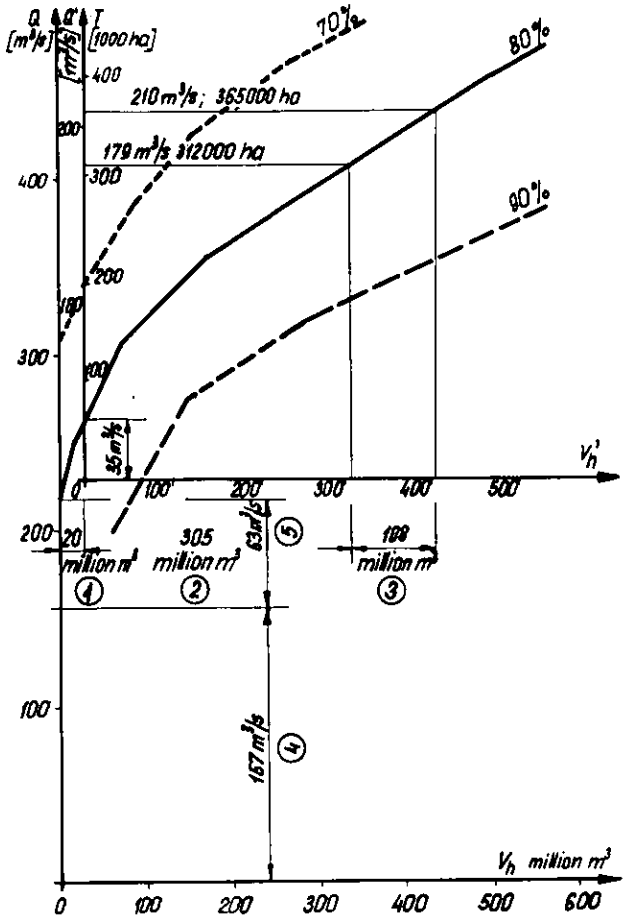


Figure 17. Discharge capacity curve for the Szeged section of the Tisza River (1901 to 1960):

V_h	=	Effective storage capacity in the Tisza River catchment, in million cu. m/sec
Q	=	Delivery capacity for agricultural uses of different duration, depending on storage and in terms of the design (greatest) consumption, in cu. m/sec
V'_h	=	Effective storage volume in the Kisköre reservoir, in million cu. m/sec
Q'	=	Delivery capacity of the Kisköre Project at the design duration of 80 percent, in cu. m/sec units
T	=	Agricultural consumption covered at Q', in 1,000 hectares
%	=	Duration of the agricultural consumption potentially covered, in percentages
1	=	Existing effective storage capacities in the Tisza Valley
2	=	Effective storage volume at Kisköre
3	=	Perspective increase of effective storage at Kisköre
4	=	Delivery capacity of 100 percent duration for other uses (industrial, communal, minimum acceptable flow, other countries)
5	=	Delivery capacity of 80 percent duration for agriculture, without storage

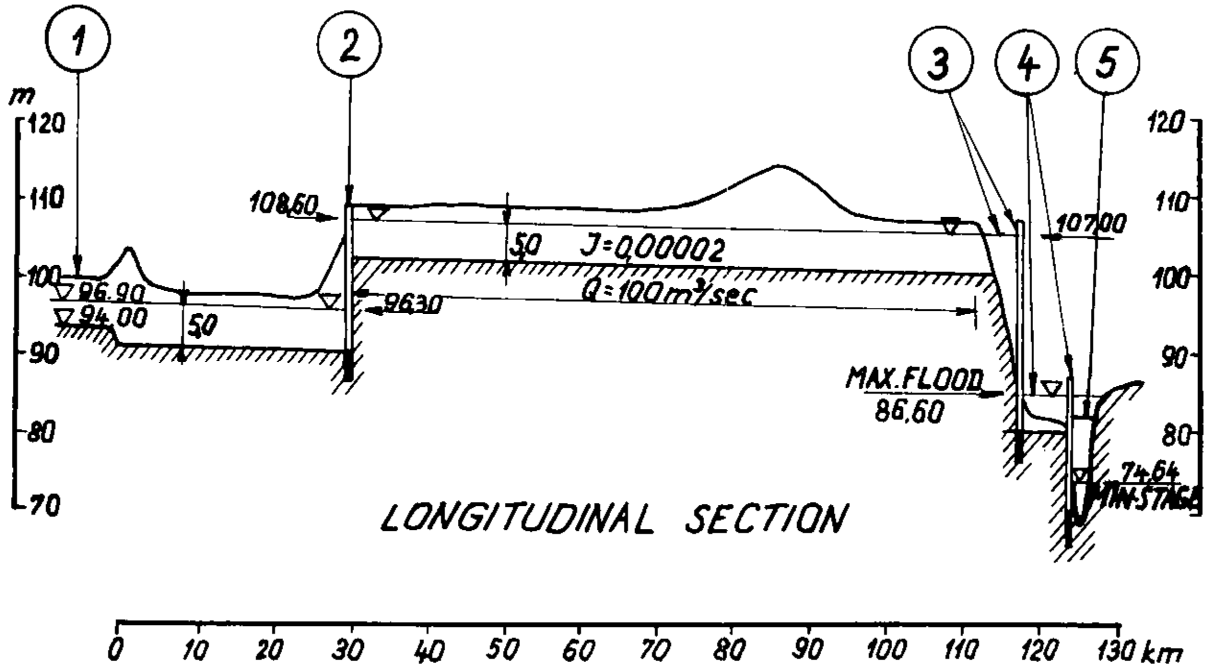


Figure 18. Longitudinal profile of the contemplated multi-purpose Danube-Tisza Canal:

1. Fixed water level of the Soroksr branch of the Danube River, 96,90 m A.O.D.
2. I. navigation lock and pumping station (100 cu. m/sec)
3. Upper reservoir (1300 ha), II-III navigation lock and 1. power station (400 cu. m/sec)
4. Alpár (Tisza) reservoir and IV. lock
5. Retention level of the Csongrád barrage on the Tisza River, 83.50 m A.O.D.

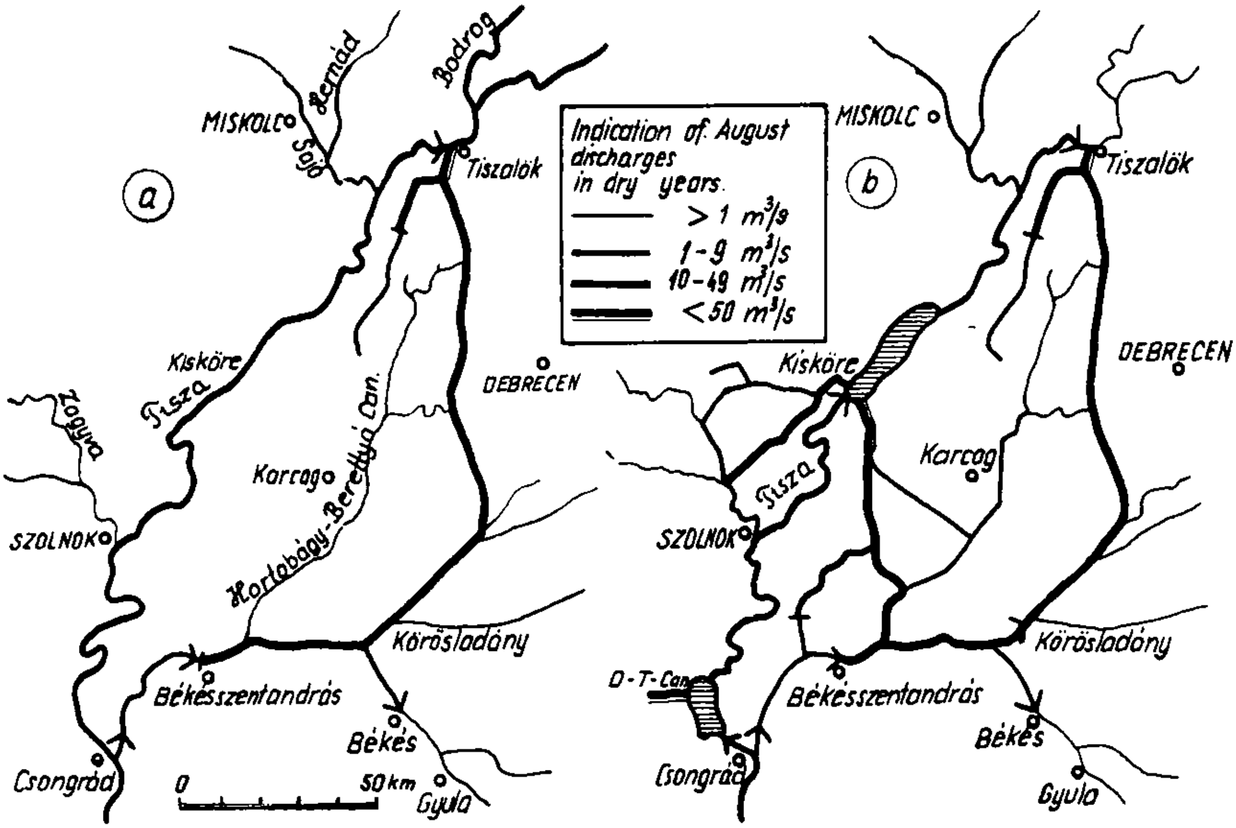


Figure 19. The barrages on the Tisza River and the related distribution systems:

a. Conditions after the construction of the Tiszalök Barrage and the Eastern Main Canal
b. Conditions when the Kisköre and Csongrád Irrigation Systems are fully completed, at about 1985

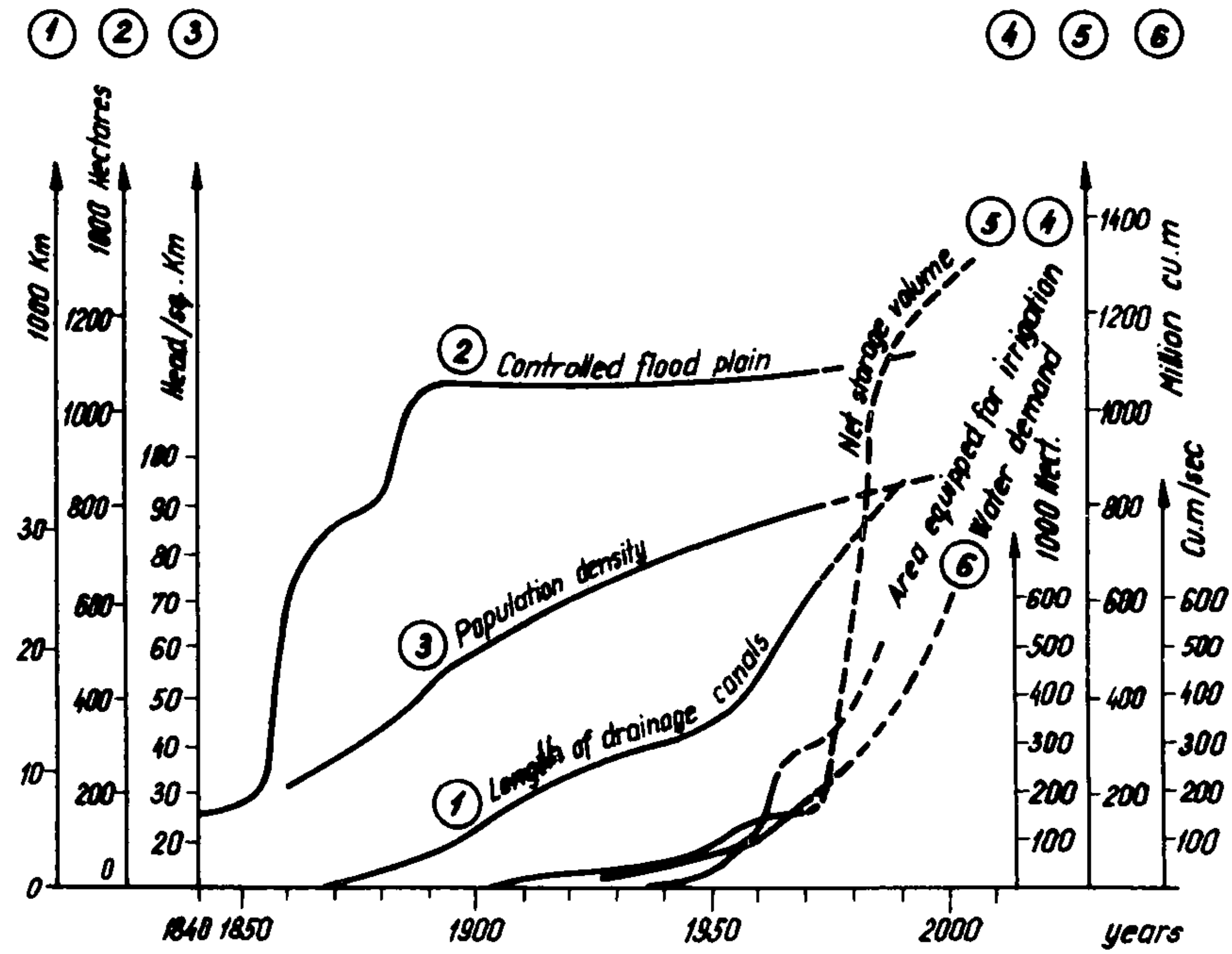


Figure 20. Water management development in the Tisza Valley, 1840-1985 (2000).

AIR POLLUTION - TWENTIETH CENTURY PLAGUE

by

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Introduction

Pollution in all its forms and the recognition of its harmful effects are not unique to the twentieth century. History recounts many isolated examples of pollution caused by vast concentrations of population where natural recycling of wastes, the decomposition and utilization of wastes as nutrients by other than human life forms was not possible on an efficient scale.

In the first century B.C., the Romans became aware of the contamination of their drinking water caused by the wastes produced by approximately one million inhabitants, and they built one of the first municipal sewage systems in history, the Cloaca Maxima. The Venetians, in ridding themselves of their wastes, took advantage of their unique, natural disposal systems by utilizing the canals that emptied into the sea with the twice-daily outgoing tide. But history also provides monumental examples of the incomprehension of some societies as to the necessity of maintaining an adequately pollution free water system. The result in these instances was dysentery and periodic epidemics of such diseases as cholera and typhoid.

Smog is, similarly, not the invention of the twentieth century--although not until the development of industrial technology and the advent of the combustion engine and automobile did it reach such staggering proportions. The most obvious examples of air pollution may be illustrated by the quality of the air over our major industrial areas and cities.

A return to the history books tells of the sixteenth century Spanish explorers' description of layers of smoke from Indian fires hanging over what is now Los Angeles. They witnessed what was later to be defined, scientifically, as an inversion layer. (Controlling Pollution, 1967). Sixteenth century England tried, with little success, to control through legislation the increasing air pollution resulting from coal and charcoal fires. This very same air pollution was encouraged, ironically, during World War II, to obscure bombing targets from the Germans. (Controlling Pollution, 1967).

In the past, the recording of history has enabled none but the more astute to recognize impending crises as the balance of nature is continuously offset and because of the impossibility of recycling, naturally, many of the new materials being used because they do not lend themselves to biodegradable decomposition. It is also becoming increasingly difficult or close to impossible, utilizing present technology, to cope with the waste produced by a highly affluent and exploding population. In ever increasing areas of the United States, the air quality problem has become so acute that in recent years the subject has not escaped public awareness and debate, with increasing pressure being placed on legislatures to pass legislation that will either improve air quality or maintain it at its present level.

The factors that contribute to the air pollution problem are both natural and manmade. The natural factors are primarily meteorological, sometimes geographical, and are generally beyond our control. A classic example of the way topography influences air quality is the condition that exists in Los Angeles. In this area, the prevailing winds are not strong, and those that come from the ocean are unable to carry the smoke over the hills surrounding the city. The smallest emission of gas or smoke affects air quality in this area where 80% of the air pollution is attributable to automobile use. (Controlling Pollution, 1967).

The man-made factors involve the emission of air contaminants from some human activity in quantities sufficient to produce deleterious effects; upon these factors control may be exercised.

Substances considered air contaminants fall into three classes based on their chemical composition and physical state: organic gases, inorganic gases, and particulates. (These will be discussed in greater detail in subsequent sections.) Organic gases consist entirely of compounds of carbon and hydrogen and their derivatives. Inorganic gases include oxides of nitrogen, oxides of sulfur, and much smaller quantities of ammonia, hydrogen sulfide, and chlorine. Particulates, also referred to as aerosols, may be organic or inorganic in composition and may exist in the liquid or solid physical state. Among the most common are carbon or soot particles, metallic oxides and salts, oily or tarry droplets, silicates, and other inorganic dusts, and metallic fumes. (Danielson, 1967).

Any solutions to the problem of protecting our air environment must take into account the flow, dispersion, degradation, or conversion of air contaminants; the means of avoiding or controlling air pollutants; the effects of air pollutants on plant and animal life and on objects and materials; and the means of detecting and measuring air pollutants.

The remaining sections of this paper deal with the gaseous environment called the atmosphere and the pollutants being pumped into the atmosphere that will in time not support life as we know it.

The Atmosphere

The atmospheric environment is a dynamic system that is both unique and finite. The density of the atmosphere decreases with altitude, approximately half the atmosphere by weight lying below the 18,000-foot level. (Air Pollution, 1970). The atmosphere can be considered a gigantic system in which countless, largely unidentified, chemical reactions occur and properties change with changes in temperature and pressure. The atmosphere absorbs a range of solids, liquids, and gases from both natural and man-made sources. These substances may travel through the air, disperse, and react among themselves and with other substances, chemically or physically.

Clean dry air contains 78.09% nitrogen by volume and 20.94% oxygen. The remaining 0.97% of air includes small amounts of carbon dioxide, helium, argon, krypton, and xenon, as well as small amounts of other inorganic and organic gases. Water vapor is normally present in air in concentrations of 1% to 3%. The air also contains aerosols, the fine suspended solid or liquid particles. (Cleaning Our Environment, 1969).

One of the most important phenomena that occur in the atmosphere is the oxygen-carbon dioxide cycle, a cycle fundamental to animal-plant relationships. Atmospheric oxygen is produced by photosynthesis, a process by which plants exploit solar energy and trap carbon dioxide to synthesize organic matter; a by-product of this process is oxygen. Maintenance of the atmospheric content of oxygen is dependent upon photosynthesis. Animals extract energy from plants and other animals by the burning of materials in the presence of oxygen. This results in oxides of fuel materials, such as carbon dioxide. When man burns matter in manipulation of his environment, other oxidized elements, such as oxides of sulfur and nitrogen, are generated.

Atmospheric pollution usually begins with the production of pollutants formed as undesirable or incidental waste products from various industrial processes. The emitted pollutants are transferred through the atmosphere, starting an airborne cycle that is completed by the contact of the pollutants with man, animals, vegetation, and other objects. This contact may result in the elimination of some pollutants from the atmosphere, or it may be followed by the repetition of a similar cycle. In its final stage, atmospheric pollution may cause damage to health and property.

Air pollution is created wherever energy is converted under human direction. Large urban areas, with their greater concentration of population and their major industrial centers, are the most frequent sources of air pollution. There are, however, numerous small communities and isolated sources that contribute to the overall pollution of the atmosphere, whether by the burning of hearth fires or from factory wastes.

The pollutants are even greater in number than the sources. A few groups of substances comprise the vast bulk of emissions to the atmosphere, and these will be reviewed in the following sections.

Sulfur and its Compounds

Sulfur occurs in trace quantities as an element in the atmosphere and appears mostly in oxidized form. Sulfur dioxide (SO^2) is one of several forms in which sulfur exists in the air; others include hydrogen sulfide, sulfuric acid, and sulfate salts. Approximately 80% of the molecules of sulfur dioxide in the air at any given time are emitted as hydrogen sulfide, which is later converted to sulfur dioxide, and approximately 20% are emitted as sulfur dioxide. It is estimated that about 80% of the sulfur dioxide emitted as sulfur dioxide is produced by the combustion of sulfur-containing fuels, such as coal and gas. The smelting of nonferrous metals and petroleum refining account for most of the remainder. The only apparent natural source of sulfur dioxide is volcanic gases. (Cleaning Our Environment, 1969).

The initial change for sulfur dioxide in the atmosphere is oxidation to sulfur trioxide. The sulfur trioxide dissolves rapidly in water droplets to form sulfuric acid, resulting in an irritant mist that may further react to form sulfate salts, such as ammonia sulfate. The primary oxidation process may take several different routes in polluted atmospheres: in air that contains nitrogen dioxide and certain hydrocarbons, sulfur dioxide is oxidized in a photochemical (light-stimulated) reaction process that produces aerosols containing sulfuric acid. Sulfur dioxide can also be oxidized in water droplets that contain ammonia, resulting in an ammonium sulfate aerosol. (Cleaning Our Environment, 1969).

The large percentage of sulfur in the atmosphere that is emitted as hydrogen sulfide is produced naturally by decaying organic matter on land and in the oceans, by some industrial operations, and by volcanic activity. Like sulfur dioxide, hydrogen sulfide is oxidized in the air and eventually converted to sulfur, sulfur dioxide, sulfuric acid, and sulfate salts.

Both the sulfuric acid and the sulfate salts formed by these reactions exist in the atmosphere as aerosols. They are removed by precipitation and to some extent by gravitational settling. A given volume of sulfur dioxide that enters the air will be removed by these mechanisms as acid or salt in a time estimated at from five days to two weeks. (Cleaning Our Environment, 1969).

The Effects of Sulfur and its Compounds

Sulfur dioxide can injure man and plant life, damage materials, and interfere with visibility. At high concentrations, sulfur dioxide irritates the upper respiratory tract due to its high solubility in body fluids. At low concentrations, it can make breathing more difficult by causing the finer air tubes of the lungs to constrict. The sulfur oxide criteria document published by the National Air Pollution Control Administration on February 11, 1969, reviews and summarizes the results of over three hundred studies, and states, in part:

> Under the conditions prevailing in areas where the studies were conducted, adverse health effects were noticed when 24-hour average levels of sulfur dioxide exceeded 300 micrograms per cubic meter (0.11 ppm) for three to four days. Adverse health effects also were noted when the annual mean level of sulfur dioxide exceeded 115 micrograms per cubic meter (0.04 ppm). Visibility was reduced to about five miles at sulfur dioxide levels of 285 micrograms per cubic meter (0.10 ppm); adverse effects on materials were observed at an annual mean of 345 micrograms per cubic meter (0.12 ppm); and adverse effects on vegetation were observed at an annual mean of 85 micrograms per cubic meter (0.03 ppm).
> (Progress in the Prevention and Control of Air Pollution, 1970).

When gases like oxides of sulfur leave a smokestack, they are quickly diffused. Oxidation to sulfur trioxide proceeds at a slow rate but is accelerated by the catalytic action of metal oxides in water droplets. This accelerated oxidation can be critical in conditions of high humidity, as during London fogs. In a two-day period during the London smog air pollution disaster of 1952, the average concentration of sulfur dioxide was 1.34 ppm. Concentrations of up to 3.2 ppm have been recorded in such commercial and industrial cities as Chicago and Pittsburgh. (Air Conservation, 1965). In New York City, the highest hourly average over a ten year period from 1957 to 1966 was 2.3 ppm in January, 1965. The highest hourly observation during the Thanksgiving episode in 1966 was 1.02 ppm between 6 and 7 a.m. on November 25. (Daily Air Pollution Index, 1968).

Sulfur dioxide when absorbed by plants in sufficient quantity cause both chronic and acute injury to the leaves. Sulfur dioxide and trioxide are highly corrosive to building materials, including limestone, marble, and mortar, all of which contain carbonates that are converted to relatively soluble sulfates that can be leached away by rainwater. Sulfur dioxide also causes the deterioration of a number of natural and synthethic fibers used in textiles, particularly cotton and wool. Women's nylon hose appear to be damaged either by extremely small atmospheric particles that contain absorbed sulfur dioxide or by tiny droplets of sulfuric acid that have formed around particles. (Cleaning Our Environment, 1969).

Hydrogen sulfide is objectionable because of its unpleasant odor even at concentrations ten to one hundred times smaller than the lowest concentration of sulfur dioxide detectable by smell. Silver and copper tarnish rapidly in the presence of hydrogen sulfide. House paint that contains lead compounds darkens rapidly in the presence of low concentrations of hydrogen sulfide by forming black lead sulfide.

Particulates

Probably the most widespread of all substances that are considered pollutants are the liquid and solid particulates. Both organic and inorganic particles emanate from a number of sources. Particles larger than 10 microns in diameter come mainly from mechanical processes such as erosion, grinding and spraying. Those between 1 and 10 microns in size are more numerous in the atmosphere; these stem from mechanical processes and include industrial dust and ash. Particulates in the size range between 0.1 and 1 micron tend to contain more of the products of condensation than do the larger particles. Ammonium sulfate and the products of combustion begin to predominate, along with aerosols formed by photochemical reactions in the air. Approximately 70% of the fine particles ranging in size from 0.02 to 0.06 micron are emitted from automobile exhaust systems. (Air Conservation, 1965).

Particles of all kinds and sizes share a number of physical properties. They grow by condensation, absorb vapors and gases, coagulate or disperse, and absorb and scatter light. Gravitational settling is the main mechanism by which particles are ultimately removed from the air, but there are intervening mechanisms that vary with the size of the particle. Particles that are less than 0.1 micron in diameter move randomly in the air, collide often with other particles, and grow by coagulation. Particles from 0.1 to 1.0 micron grow more slowly than smaller particles because they are less numerous, move less rapidly in the air, and thus collide less often with other particles. At diameters larger than 1 micron, particles begin to develop appreciable settling velocities, and those above 10 microns settle rapidly. The lifetime of a particle in the air is a function of the height at which it is introduced and of the turbulence of the air. (Cleaning Our Environment, 1969).

In 1966-1967, the rate of emission of suspended particulates in New York City was 87,100 tons/year with 31% from space heating, 36% from municipal and on-site incineration, and 14% from power generation. (Rickles, 1970). But in 1969 this figure was reduced approximately 23% to 69,000 tons/year as a result of the shift to low-sulfur fuels with half the ash content of formerly used fuels and by shutting down of three Sanitation Department incinerators. (Kihss, 1970).

The Effects of Particulates

The smaller particles measuring less than 5 microns in diameter have a greater effect than coarse particles in reducing visibility, in soiling, and in impairing human health. Particulate matter also damages materials by soiling and is a critical factor in the corrosion of metals, especially in the presence of pollutant gases of an acidic nature.

The effect of particulates on human health is determined not only by their chemical composition but also by the size of the particles. Particles of less than 5 microns in diameter may reach the lower respiratory passages and lodge in the tiny air sacs that terminate them. Sulfur dioxide may be absorbed on those particles and be slowly released into the air sacs. Other harmful components such as DDT and radio-isotopes in polluted air can also be carried into the lungs on these tiny particles. (Cleaning Our Environment, 1969).

Carbon Monoxide

Carbon monoxide is almost exclusively a man-made pollutant resulting from the incomplete combustion of carbonacious material. The automobile is estimated to cause more than 80% of carbon monoxide emissions, with smaller amounts resulting from other combustion processes. (Cleaning Our Environment, 1969). The behavior of carbon monoxide is governed by the fact that it is chemically inert and apparently reacts with no other constituent of the atmosphere to a significant degree.

During the period from 1965 to 1967, the average carbon monoxide reading at the New York City elevated sampling station in Manhattan was 3 ppm, with minimum and maximum daily averages of from 1 to 11 ppm. (Daily Pollution Index Background Information, 1968). However, since most of the carbon monoxide is emitted at street level, much higher readings are obtained within high traffic areas at this level. A study conducted in New York City in 1967 noted that at one in-street site the reading exceeded 15 ppm during periods between 9 a.m. and 7 p.m., but it then dropped to 1 to 2 ppm. The high readings can be correlated with the greater volume of traffic during daytime hours. (Johnson, Dworetzky, and Heller, 1968).

The Effects of Carbon Monoxide

Carbon monoxide is an odorless and colorless gas and it is also a poisonous inhalant that mixes with hemoglobin in the blood to form a carboxygen, a stable compound that does not permit absorption of oxygen into the bloodstream.

At concentrations of slightly more than 1,000 ppm, carbon monoxide kills quickly. The upper limit of safety for health is considered to be 100 ppm for an eight-hour period. At high levels of concentration, carbon monoxide has been identified as a participant in synergistic reactions. That is, the combined effect of carbon monoxide in the presence of hydrogen sulfide or nitrogen dioxide is more severe than the sum of the effects of each of the gases. (Air Conservation, 1965).

Carbon Dioxide

Carbon dioxide is not normally considered an air pollutant although man generates an enormous amount of it in the combustion process using fossil fuels such as coal, oil and natural gas. It is a normal constituent of the air and is essential to plant and animal life. However, worldwide atmospheric concentrations of carbon dioxide have been rising since about the middle of the nineteenth century. At the beginning of the twentieth century, the burning of coal produced 3 billion tons of carbon dioxide per year. Since 1950 the amount has averaged about 9 billion tons per year. (Air Conservation, 1965).

The Effects of Carbon Dioxide

Atmospheric carbon dioxide is responsible for the deterioration of building stones and in part for the atmospheric corrosion of magnesium and perhaps of other structural metals. Scientists are more concerned, however, by its possible future effect on the heat balance and hence on the climate of the earth. Carbon dioxide's effect on global temperature, its "greenhouse effect," is due to the ability of the gas to absorb the relatively long-wavelength infrared radiation emitted by the earth and radiate it back to the earth. It is generally agreed that global temperature rose about 0.4 degrees C between 1880 and 1940 and then reversed itself, falling nearly 0.2 degrees C by 1967. The concentration of carbon dioxide in the atmosphere meanwhile is believed to have risen from 296 ppm in 1900 to 318 ppm in 1969. Increasing carbon dioxide concentration could explain the higher temperatures up to 1940, and increasing turbidity could explain the lower temperatures since then. (Cleaning Our Environment, 1969).

Nitrogen Oxide

Oxides of nitrogen are one of the most important groups of atmospheric contaminants. The major source of nitrogen oxides is the combustion process in automobiles and electric power plants. Combustion converts nitrogen from the air chiefly to nitric oxide, which in urban air is oxidized slowly by oxygen and rapidly by ozone to nitrogen dioxide. Lightning and biological processes that take place in the soil are other contributors of oxides of nitrogen to the atmosphere.

Nitrogen dioxide is a strong absorber of ultraviolet light from the sun and is the trigger for the photochemical reactions that produce smog in polluted air. The gas can also combine with water vapor to form nitric acid, which can in turn react with ammonia or particles in the air to form nitrate salts such as ammonium nitrate. (Cleaning Our Environment, 1969).

The Effects of Nitrogen Oxides

The hazards associated with nitrogen oxides are a direct noxious effect on the health and well-being of humans and the photochemical oxidation of organic material. The threshold limit, established by the American Conference of Governmental Industrial Hygienists, has been tentatively set at 5 ppm for an eight-hour working day. (Air Conservation, 1965).

Treatment of Air Pollution

The preceding sections have attempted to review some of the major sources of air pollution and by no means do they represent the complete spectrum of pollutants. Nor does this section on treatment suggest an all-encompassing solution to the problem. In fact, treatment of air pollution is still rather primitive, but if even these methods were employed by the major polluters and adherence to present air quality standards enforced by local, state and federal agencies, the quality of air may be improved, or at least prevented from degenerating to even a worse level.

1. The best solution is to ensure that there is good combustion. This requires, that the combustion chamber be well supplied with oxygen and a good draft so that the temperature of the fire is as hot as possible.

2. The deployment of mechanical devices to trap pollutants before they reach the exhaust systems would also be useful. This may require the use of screens or filtering devices, electrostatic precipitators or merely relying on gravitational forces by having the exhaust pass over several settling chambers before being allowed to reach the chimney.

3. The use of oil or water scrubbers has proved to be an effective method of removing ash and even gases from exhaust. This method presents several problems to the user: (a) cost and (b) disposal of the collected waste.

4. Chemically treating exhaust neutralizing the pollutant or forming a by-product that may be of some economic value would also help.

Concluding Remarks

The well-being of present and future generations depends upon the steps taken now by local, state, and national governments and on the willingness of the general public to recognize the price they must pay for pollution abatement. One of the difficulties with pollution control is the time-honored concept that air and water is the property of all and is readily available and free to all that use them. However, when the use and subsequent abuse of our air and water environment presents a health hazard, then a system for establishing standards with financial values for their use will have to be established. The price for air and water use will, therefore, be reflected in the price of consumer products and services. Those consumer goods or services that pollute too highly for the good derived from their use will ultimately be priced out of consumer willingness to pay with the concomitant demand for those goods and services diminished.

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SOME APPLICATIONS OF SYSTEMS ANALYSIS TO WATER RESOURCES PLANNING

by

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Over the past decade a large number of mathematical optimization and simulation techniques for analyzing water resources systems have been proposed and discussed in the literature. By comparison, only a few papers have been written that discuss the application of these mathematical management models to actual water resources planning problems. Only in their application can one begin to fully appreciate both the advantages as well as the limitations of these analysis techniques and the need for further model and/or algorithm improvement.

This paper summarizes the methodology used in, and results obtained from, the analysis of four river basin planning and management problems. The first study focuses on the definition and evaluation of various water quality management policies for the St. John River in Canada. The next two studies are concerned with the evaluation of various investment alternatives for the future development and mamagement of both water quality and quantity in the Vistula River Basin, Poland, and in the Delaware River Basin, USA. The last study discussed in this paper involves an irrigation planning problem in Taiwan.

Each study made use of both optimization and simulation techniques adapted to the problem, the available data and computational facilities. Optimization was used not necessarily to find the best solution but to reduce the number of possible policy alternatives to a relatively small number for further analysis using simulation techniques. The need for preliminary screening is evident if one considers the complexity and number of variables involved in most water resources planning problems. If for example there were only 30 variables each of which could assume only two values, this results in 2^{30} or over 10^{9} possible alternatives. Assuming engineering judgment could eliminate 99 percent of these and each of the remaining one percent could be evaluated using a simulation model requiring only a minute of computer time, it would take over 19 years of computation. Of course the variables of most river basin planning problems number several orders of magnitude more than 30, and many of these may assume literally an infinite number of possible values.

While the complementary techniques of optimization, for preliminary screening, and simulation, for more precise evaluation, provide a potentially powerful method of analysis, the joint and coordinated use of these two techniques has not seen many applications. Nor do the results of all optimization models always correspond to the results of simulation of the same systems. Close correspondence depends in part on the form of the data, the model, and the skill of the analysts. Four case studies that have attempted to use both optimization and simulation, with varying success, are presented below.

Water Quality Management of the Saint John River[1]

The application of water quality management methodology to the Saint John River Basin consisted of three distinct aspects, (a) preliminary screening with an optimization model; (b) more refined analysis with the combined use of the optimization and simulation models; (c) considerations of various non-quantifiable aspects.

The primary purpose of the study was to demonstrate how the results obtained from water quality management models may be effectively used by water quality planners in Canada as well as to define and evaluate some specific policies for the Saint John River Basin.

The Saint John River Basin

In this section, the more salient aspects of the Saint John River Basin, illustrated in Figure 1, are highlighted. The magnitude and complexity of managing existing water quality problems are demonstrated, and thereby a focus is developed displaying the potential role of systems methodology in the management of water quality in a river basin.

The Saint John River Basin is located in the Provinces of New Brunswick and Quebec in Canada, and the State of Maine in the United States. Its total drainage area is 21,260 square miles, with 64 percent of the area in Canada. Measured along its stream bed, the river is approximately 435 miles in length and has a total fall of about 1,578 feet between its source in Little Saint John Lake and tidewater at Fredericton.

The hydrology of the basin is characterized by periods of high flow which usually occur twice a year, in the spring and again in November, although flooding can occur at any time of the year. The critical periods for low flow are in the winter, late summer and early fall. The critical period for water quality is in late summer to early fall, when low flows are concurrent with high temperatures, resulting in very low values of dissolved oxygen in the river. Over the section of the river between Edmundston and Fredericton, the river is well developed for hydroelectric power purposes, with a resulting artificial daily fluctuation in flows. Below the Mactaquac reservoir, the 60-mile stretch of river to Saint John is tidal, with the lower section containing saline water.

With 85 percent of the land area in the basin in forest cover, the use of this resource for pulp and paper production forms the leading industry in the basin. The second major industry in the basin is food processing, particularly potato processing. The potato processing industry is composed of two distinct components: potato foods, i.e., potato chips, french fries and mashed potatoes, and starch production.

Tourism is not a major industry in the basin. However, the various lakes in the basin do play a major role in providing water-based recreation, primarily for the inhabitants of the basin. Annual activity days for outdoor recreation for the entire basin are of the order of 600,000, of which 50 percent is estimated to be water based.

In the social and economic development of the basin, water, as a resource, has always been plentiful and has not been an important constraint upon the growth of any portion of the basin. Besides providing navigational and logging facilities for the early settlers, it has been more recently a major source of hydroelectric power, water supply and waste effluent transport. It has also served as a base for recreation and, of course, a spawning ground for various fish, particularly Atlantic salmon. Most communities continue to utilize ground water sources for the town water supply. Industries tend to draw their water from the town supply, or to use their own wells. In some cases, the water supply is obtained directly from the river. In any event, there is still an abundant supply of water, and this situation will remain unchanged for a considerable period of time.

However, the quality of the water is frequently poor. Fifty-nine significant effluent sources were identified in this study, resulting in a total daily effluent load of about 1,200,000 pounds of carbonaceous BOD, and a further 238,000 pounds of nitrogenous BOD. This very high effluent load results in major stretches of the river having extremely poor quality water, with major fish kills occurring quite regularly, particularly in the main stem between Edmundston and Mactaquac, and on the Aroostook and Presquile tributaries in Maine. The results of this poor quality water are readily apparent. Water-based recreational facilities on the main stem are limited. The preservation of the Saint John portion of the salmon industry is maintained at increased cost, land values along the river are reduced, and the social aesthetic and psychic gains obtainable from a clean river or stream are largely erased.

In examining water quality aspects, water resources planners became involved largely with satisfying competing demands. This conflict between water quality and other aspects was focused on two issues. The first issue existed within the water resources sector, and reflected competing demands for better water quality for improved water supply sources, water-based recreation, fishing and aesthetics, against demands for poorer quality resulting from a greater utilization of the assimilative capacity by effluent dischargers. The second issue, basically intersectoral in nature, existed between the water resources sector and aspects such as economic development and industrial growth. Clearly, costs imposed on industries to clean up pollution, in general, lead to reduced rates of economic development.

Certain international issues and potential conflicts were observed. In particular, the most severely polluted stretch of river (Grand Falls head pond) is on the international boundary, with both pollution sources and the effects occurring on both sides of the Canadian-U.S. border. However, on both the Aroostook and Presquile rivers, disbenefits to Canada, in the form of low water quality, accrue as a result of pollution in the State of Maine.

At the heart of the issue, then, was the manner of deciding the extent of effluent cutbacks, and the effective and efficient administering of this abatement. To make the best possible decisions, there was an initial need to establish the amount and type of effluent, its behavior and effects within the water body, and the costs and benefits resulting from removing varying amounts of the effluent.

While many different types of effluents are usually deposited in a water body (the Saint John River is certainly typical in this respect), there is a need to characterize wastes in accordance with their most deleterious effects on the water body. The most significant parameters have historically been the reduction of dissolved oxygen (DO) in the water body, due to oxygen consumption by wastes (BOD), toxic materials such as pesticides, acids, heavy metals, and nutrient additions, principally phosphorous. In the Saint John River, the most significant water quality problems result from low DO levels arising from very high BOD loadings. Thus, the focus in this study was on quantifying the magnitude and nature of the BOD load, and determining the costs of removing varying amounts of the load from the various effluent discharge locations. It is not to be implied, however, that low DO levels are the only water quality problems in the basin.

In examining the treatment plants available for treating municipal effluent, five different types were considered. These included primary, activated sludge, trickling filter, waste stabilization pond and tertiary treatment in the form of lime clarification and ammonia stripping. However, in many cases, the selection was reduced because some high BOD removal systems were cheaper than arrangements with a lower BOD removal. The annual costs included capital costs, and operation and maintenance costs. Capital costs were based on initial investment amortized over a 25-year period at 9 percent in Canada, and 7 percent in Maine. For industrial wastes, effluent reduction is normally obtained by a combination of process change, i.e., internal recovery of materials previously discharged, and normal treatment of the effluent. However, data for the cost of waste treatment in the various industrial sectors were very limited. Thus the approach adopted in this study was to consider that all industrial effluents would be reduced by installing treatment plants. The derived costs based on that assumption generally gave an upper limit to the cost of treatment.

Systems Methodology

In focusing on the water quality problems of the Saint John River, it was recognized that the most significant deleterious effects are caused by the abnormally low levels of DO. The heart of the problem, therefore, is to reduce the effluent discharge into the river system in a manner that ensures an optimum balance between increased costs in effluent treatment and improved quality of the water, with appropriate consideration of regional development, i.e., competing intersectoral objectives. In addressing this problem, two mathematical models were developed and utilized. These two models consisted of an optimization model which was used for screening and as a decision tool, and a simulation model for more detailed analysis.

The screening model was relatively simple, both insofar as construct and data requirements were concerned, as well as readily adaptable to different river basin configurations; and, by focusing on the entire river system, it quantified the significant linkages that exist between treatment costs and water quality. The simulation model, on the other hand, was considerably more detailed, and attempted to capture the physical system more precisely than did the screening model. In particular, it examined in greater detail the hydrology and effluent discharge, recognizing its time-varying stochastic nature.

During the screening phase, the results obtained from various analyses were particularly helpful in identifying the implications of alternative policy decisions. The ability to change the imposed constraints, while always maintaining the same objective (such as minimizing the basin-wide cost of treatment), permitted comparison on a consistent basis. Thus, the significance of alternative water quality standards could be readily and meaningfully quantified. For performing the screening function, a linear programming optimization model was used. In all cases, the mathematically defined objective was to minimize the total cost of treatment within the basin under different constraint conditions.

The basic form of the model used in this study can be described very simply. Consider a number of waste discharge sites i, each of which can reduce its total BOD discharge, BOD_i, by a fraction X_i costing $C_i\ (X_i)$. Also consider a number of quality sites j, at which standards specifying minimum allowable DO concentrations, DO_j^{min}, are defined. Denoting d_{ij} as the dissolved oxygen deficit concentration at site j resulting from a unit of BOD discharged into the stream at site i, DO_j^{sat} as the saturation dissolved oxygen concentration at site j, and D as the natural uncontrollable deficit at site j, the model can be written

$$\text{minimize} \sum_i C_i\ (X_i) = \text{total treatment cost}$$

subject to the stream quality standards at each site j

$$DO_j^{sat} - \sum_i d_{ij}\ BOD_i (1-X_i) - D_j \geq DO_j^{min}$$

together with appropriate limits on each X_i.

The Saint John River was characterized by 88 quality sites and 59 significant pollution source sites. Both carbonaceous and nitrogenous BOD were explicitly considered by defining both carbonaceous and nitrogenous BOD Loads, BOD_i, and transfer coefficients, d_{ij}, and by specifying nitrogenous BOD removal as a function of the carbonaceous BOD removal, X_i. Additional linear constraints were required to adequately define the non-linear treatment cost functions $C_i(X_i)$. During the preliminary screening, five basic cost minimization runs were performed, namely:

(a) Reference run--based on complete freedom of choice of treatment level with no existing treatment;

(b) Primary treatment--based on assumed minimum treatment of all effluents to be primary treatment;

(c) Existing situation--all existing treatment assumed in place and operating;

(d) Effect of changed flow--using the same constraints as in (c), but with the flow increased by 20 percent;

(e) Sensitivity analysis for reaeration--the value of the reaeration coefficient, k_2, in the Grand Falls head pond was increased by 50 percent.

The results of these screening studies included parametric variations on the cost-removal efficiency curves for various pulp and paper mills.

To obtain estimates of how any wastewater treatment or flow augmentation policy would perform under actual conditions of varying streamflows, a simulation model was developed that calculated the hydraulic and quality conditions every two days at each quality site for 25 years. Thus, while the screening model was used for preliminary sizing of facilities, and for obtaining costs under a variety of options, the simulation model catered to a more precise analysis of the routing of water and pollutants throughout the basin. Each simulation run required about 24 minutes on an IBM 360-65 as compared to about 30 seconds for a screening model solution. From the more expensive simulation model several parameters used in the optimization model were better defined and fed back to the screening model for performing further optimization runs. It is for these reasons that the optimization model was referred to as a screening and decision model, while the simulation model was considered as a performance-testing model. Due to the relatively long running times for the simulation model, it was important to be selective in determining the coverage of the model. Most of the running time was involved with (a) calculating the flow and velocity at the two ends of each reach at the end of every two-day period; and (b) calculating concentrations of every pollutant at the ends of each reach at the end of every two-day period.

In examining water quality, it was decided to determine the effects on basin-wide treatment costs of a variety of DO standards. The DO standards examined included 5 ppm, 4 ppm and 0 ppm. The first two standards reflected the potential threshold levels for salmon. The third standard of 0 ppm was adopted to reflect the lower extreme which might be considered acceptable, i.e., the threshold at which the river becomes anaerobic. An exception to these standards was the Grand Falls head pond, where a standard of 0 ppm was used. It was shown that to achieve a standard of 4 ppm or 5 ppm in the head pond would require an extremely high (99 percent) removal efficiency at the Edmundston-Madawaska pulp and paper mills, a level which has never been achieved, in practice for a pulp and paper mill without a process change. Also, the Atlantic salmon do not proceed upstream from the Grand Falls dam (previously a waterfall formed a natural barrier to upstream migration), and only a few landlocked salmon exist in the basin above Grand Falls. The head pond was therefore considered somewhat in isolation since poor DO conditions there do not necessarily affect the fish life above or below that area.

Summary of Results

Without going into specific details and implications, a sample of the results of the screening and simulation models are illustrated in Figures 2 and 3.

To obtain some idea of the expenditures required to meet various quality standards, it is interesting to observe that if no money whatsoever had been spent to date in the basin on waste treatment facilities, a general DO standard of 5 ppm, with 0 ppm in the Grand Falls head pond, could be achieved with an annual outlay of just over 3 million dollars. However, actual expenditure to date has been approximately 2 million, and yet a further expenditure of 3 million would be required to achieve the above standard. This results from the fact that the treatment required to raise the Grand Falls head pond quality will render the existing downstream municipal and industrial treatment redundant from a DO point of view, although such treatment, of course, gives other benefits. While limited reliance should be placed on this calculation because of the very crude input data, it amply demonstrated that the expenditure of public and private funds to date has not been focused effectively on those areas where the water quality problems are most severe.

A second general point which emerged is that in achieving basin-wide efficiency in effluent removal, the burden tends to fall on the larger polluters. This general pattern, which has also been observed elsewhere, displays the importance of economies of scale in waste treatment. Because economies of scale played such a prominent role in achieving efficiency in waste treatment, a policy of "primary treatment everywhere, at least" tended to introduce considerable additional expenditure. Primarily, this was due to the high costs (expressed in $/lb BOD removed) incurred by the smaller polluters.

It is clear from a comparison of the optimization and simulation results shown in Figure 3 that the optimization model failed to effectively capture the linkages between any two points i and j, if they were separated by significant travel times. This has not been a particular problem in most other river basins which have been studied with the use of water quality optimization models. The reasons for this problem in this study included the following:

(a) The Saint John River is a river with low-flow travel times exceeding one month, thereby placing greater emphasis on the effects of transfer effects associated with these increased travel times.

(b) The river is characterized by low values for k_2 because of the presence of various reservoirs. This has the effect of advancing the low point of the DO sag curve in terms of travel times, and thereby placing greater emphasis on points located significant distances downstream, in terms of travel times.

(c) The inclusion of the nitrogenous component of the effluent results in greater emphasis on the effects of the effluent at significant distances downstream, due to its lower decay rate.

(d) The river is dominated by a small number of very large effluent sources. Under the more usual condition of having roughly equal amounts of effluent discharge from a series of polluters distributed along a stream, the effects of certain dischargers, for quality points located at large distances downstream, become relatively insignificant in relation to other dischargers located nearer these quality points.

Later adjustments in the flow rates and transfer coefficients resulted in a closer correspondence between the results of these two types of models. Although some of the results of the simulation model tended to confirm answers obtained from the optimization model, the extent to which the optimization model did not initially capture certain linkages was not expected. Thus, the results of the simulation model, particularly its ability to capture the state of the system under a range of non-steady flow conditions, have demonstrated the importance of this type of model as a tool in planning. Certainly, by more precisely modeling the real behavior of the system, some of the ramifications associated with studying Canadian rivers were more than amply demonstrated. The additional insights, provided by the simulation model had a significant impact on the outcome of the study.

One of the facets in using models in the multidisciplinary approach adopted in this study is that it allowed the various disciplines to focus on the same issues from various viewpoints. Results from mathematical models were purposely presented in a form which permitted more ready understanding of their implications by each of the various disciplines. It turned out that the simplicity in conceptual foundation, and widespread understanding of the models used in a study was of extreme importance. In this particular case, the use of the screening model, in particular, remained a vital focal point for several interaction meetings involving professionals of various disciplines and citizen groups, and provided a highly effective language bridge between the participating disciplines and the interested public.

Planning the Future Development of the Vistula River Basin[2,7]

The Vistula River Basin Planning Project was initiated in October 1968, and involved the participation of the United Nations Development Program (UNDP), the United Nations (UN), and the People's Republic of Poland. The project objective was the development of a comprehensive plan for the orderly and optimal utilization of the water resources of the Vistula Basin, Poland's largest river basin. It was realized at the outset that to achieve this objective many alternative plans would have to be evaluated and that traditional planning techniques, where only a few select alternatives could be assessed, would not be adequate. The means had to be found for satisfying many competing demands for water - municipal, industrial, agricultural, power, navigation, water quality control, recreation, etc. over a future planning horizon extending at least to the year 2000. Many planning variants, numbering in the hundreds, were envisioned and enormous quantities of data had to be assembled, organized, and analyzed to describe the behavior of all the viable possibilities.

To cope with this problem in an orderly fashion and to achieve the objective of a comprehensive plan within the project period of approximately two years, it was proposed to employ a set of mathematical models whereby alternative strategies could be defined and evaluated. The modeling work was conducted in two phases. An initial phase of about three months' duration involved training of selected Polish specialists and initial model development. The second phase, lasting over six months, was devoted to the complete development of models and to the adaption of programs developed in Phase I to the available computer hardware in Poland.

The Vistula River Basin

The Vistula River System, as shown in Figure 4, drains an area of approximately 194,000 km^2 of which 168,000 km^2, or roughly 87 percent, lies within the People's Republic of Poland. It embraces more than half of the land area of the country, extending from the Carpathian Mountains in the south to the Bay of Gdansk in the Baltic Sea.

The river may be divided into three main reaches: (a) the Upper Vistula from the confluence of the main stem with the River Sola to the confluence with the River San; (b) the middle Vistula from the confluence of the San to the confluence with the Bug, and (c) the lower Vistula from the Bug to the point of discharge into the Bay of Gdansk. These reaches are, respectively, 280, 270, and 391 kilometers in length which, taken with the 127 km above the River Sola known as the Small Vistula, gives a main course distance of 1068 km (665 miles). Other major tributaries of the Vistula which were explicitly represented in the mathematical models are shown in Figure 4.

Topographically, the basin is comprised of fairly steep, narrow valleys in the region of the Small Vistula and the principal Carpathian tributaries, with broad, rolling foothills in the Upper Vistula, and flat, almost featureless terrain in the Middle and Lower Vistula regions.

Of particular importance from the water resources development point of view is the scarcity of good reservoir sites in the Carpathian Mountains. While the area is not without good dam sites, the terrain does not yield reservoirs of significant volume in relation to the annual yield of tributary streams. It is estimated that the maximum storage potential of developable sites in the Upper Vistula is probably less than 5 percent of the mean annual runoff.

The seasonal pattern of temperature and precipitation combines with the topographic characteristics of the basin to produce a fairly consistent hydrologic pattern over the year, varying somewhat with elevation. For the main stem of the Lower Vistula, monthly mean flows are usually maximal in early spring as a result of combined runoff from spring rains and melting snow. Minimal flows tend to occur in the months of December and January when precipitation is in the form of snow and the lower two-thirds of the river system is frozen over. In the Upper Vistula, the histogram of monthly flows tends to be bimodal, with a small peak in April with the first snow melt and a second sharp peak in July, occasioned by flashy storms in the mountain areas. These storms have been a major cause of flood damage in upper reaches of the river system. Mean monthly July runoff for the Upper Vistula (above Krakow) may be as much as 16 percent of the annual runoff as against 13.5 percent for April and a minimum of about 4 percent for the months of October and November.

The present (1970) population of Poland is roughly 32,000,000 persons, about 45 percent of whom live in the urban centers. Demographic projections indicate a probable urban growth in the next two decades of about 8 million persons, roughly 55 percent of the present metropolitan population. This growth in potential users of municipal water, and concomitant increases in per capita usage which can be expected, indicate a rapidly rising need for additional quantities of good quality water. It is estimated that the annual requirement for municipal supplies for the entire Vistula Basin will increase from a level of 452×10^6 cm in 1960 to 1570×10^6 cm by 1985. On a national scale, the total water requirement is expected to triple by 1985 (ref. to 1960 level) and the per capita requirement should rise about 70 percent. In 1960, per capita requirements for all metropolitan uses, exclusive of heavy industry, was about 175 liters per day; by 1985 it is expected to reach 300 liters per day.

While substantial expansion of industry is expected in the near future, there exists an awareness of limited availability of water for industry and a need to effect water economies. Considering these factors together it is estimated that industrial water needs will rise from a level of about 4.1×10^9 cm in 1960 to roughly 9.5×10^9 cm by 1985. Of the difference between these two levels of industrial water need, more than one half, say 2.7×10^9 cm would be allocated to the Vistula Basin. In the Upper Vistula alone the 1985 water requirements for industry is expected to reach 2.6×10^9 cm which is approximately equal to the mean annual flow of the Vistula at Krakow.

Agriculture's greatest problem in the Vistula Basin is protection from flooding. At the present time, annual flooding affects approximately 1.5 million hectares. Water for irrigated agriculture can be used initially for improving production of pasture lands, then for truck crops, and finally for bringing unproductive arable lands into use. It is expected that the area of irrigated pasture lands will increase from 0.37 million hectares in 1960 to about 2.7 million in 1985.

In 1960 the total wastewater return to the surface waters of Poland was about 2.3 billion cm, with three-quarters being received by the Vistula System. It is expected that by 1985 the total will reach about 8.3 billion cm, with 23 percent, some 1.9 billion cm, being returned to the Upper Vistula System where it is associated with industrial growth in the Katowice region. From the standpoint of overall water resource allocation, wastewater return flows are important; however, the problem of quality degradation is significant. Unless the pollution load in the streams of the Vistula system can be drastically reduced as the level of water use increases, a point may be reached where quality control could limit further development.

The hydroelectric power potential of Poland's rivers is not great when compared to the country's needs. A combination of hydrological circumstances and economic considerations has resulted in development of facilities capable of producing only about 2 percent of 43 billion kwh (0.9 billion kwh) developed from both hydro and thermal facilities in 1965. Hydroelectric power resources of the Vistula River System amount to about 40 percent of the country's total hydroelectric capacity. These resources are of additional importance relative to the total power capacity and the distribution of demand because the generation sites are located where coal, i.e., thermal power, is not particularly competitive. For example, about 60 percent of the Vistula's undeveloped hydropotential is located below Warsaw, far from the Silesian coal fields. Lower Vistula run-of-river projects could produce as much as 3.5 billion kwh of peaking energy. At the present time, these projects are probably the most attractive from the economic point of view.

Flood control is of major importance in the Vistula System; losses of over 5 billion zlotys, about 70 percent of the country's total loss, were incurred over the period 1958-1965. Such losses can be reduced by construction of protective levees or by providing additonal flood control storage capacity. At present the total flood reserve in this region is about 250 million cm. Additional capacity of 300 million cm could be added by the construction of all the reservoirs shown on Figure 5.

The basic problems of navigation are maintenance of adequate depths for the barge trains, providing for transport through barrages by lockage, and maintaining open river conditions during the freezing periods. While there seems to be considerable potential for expanding water transport on the main stream of the Vistula, it appears at the present time that development of rail transport may be more economic. Nevertheless, a transport of the order of 2 million tons and 0.2 billion ton-kilometers in 1965 might be increased by 1985 to 42 million tons and 14.4 billion ton-kilometers, according to recent transportation studies. The actual magnitude of future development will depend on the relative emphasis given to development of other modes of transport and whether adequate means can be found to control the river regime to improve navigation.

Systems Methodology

Given the background information, the basic problem in planning for the future development of the basin is how a limited and variable water resource should be allocated to various changing demands and competing water uses over a period of years into the uncertain future, in order that the net social and economic benefit associated with the allocation is maximum.

In designing the planning methodology for the Vistula River Project, certain practical considerations were recognized: the hydrologic-meteorologic data base was limited; that is, gaps existed, particularly over the war years, and a consistent record for a large number of gaging stations was not available for a period much longer than about 15 years. In addition, data were not available to permit a reliable assessment of quantitative benefits. With these considerations in mind, the following decisions and assumptions were made:

1. The historic hydrologic-meteorologic record period of 15 years, 1951 to 1965 was selected for the analysis of water allocation alternatives.

2. Because the planning project period was relatively short and because the computer facilities were limited, statistical characterization of the data was not attempted; in other words, a deterministic historic sequence was used for all studies.

3. Because of the complexity of the system and the large number of variants, a combination of mathematical optimization and simulation was determined as essential.

4. Because the system was so complex and because computer capacity and access in Poland was limited, the basic system was divided for analysis purposes into twelve subsystems. These are illustrated in Figure 5.

5. Water demands were considered for two stages of future development, 1985 and 2000.

6. The decision criteria for evaluation of each variant in each subsystem (and total system) were based primarily on costs and losses; benefits were considered but primarily as modifiers in the selection process.

A variety of models were proposed for the analysis of the Vistula System. The general scheme of model use is shown in Figure 6. Referring to Figure 6, the methodology developed for defining and evaluating various lternatives in each subsystem consisted of first defining a variant (1) described as to reservoir capacities, canal capacities, water use demands and locations. In parallel the basic data, hydrologic, water demand and user costs, were developed for all control locations (2). The design alternatives were reduced to a limited set d_{ij} by subsystems, i, and variants, j (3). A scheme for data management was devised (4) preparatory to analysis with a specially designed set of mathematical models.

Two alternative methods were developed for the water resource allocation problem. Each entailed subjecting a specific proposed variant to a simulation-optimization procedure which sought to determine the "least costly" way by which the available water resource could be allocated to meet a prescribed set of demands, say at the probable level of 1985.

Primary results of this analysis gave estimates of the "cost" associated with the alternative, the deficits incurred, if any, the storage histories and operating rules for reservoirs in the system, and flows at control profiles.

The so-called Three Step Method (Steps 5, 6 and 7 in Figure 6) entailed the use of three separate programs or models. The first program (5) determined a preliminary set of target outflows for individual reservoirs in the system. The second program (6) developed the operating rules for the reservoirs given the inflow hydrology and the target outflows. It also determined the flows at control profiles downstream of the reservoirs. Finally, an allocation program (7) determined the "best" program for supplying water to demand centers within the subsystem.

The Single Step Method was based on a modeling technique that could be solved by the Out-of-Kilter algorithm (OKA). In essence the method performed each of the operations of the Three Step Method in a single operation. Ostensibly, both methods were capable of achieving the desired results. It was initially intended that a detailed comparison of the two methods be made prior to the "production" runs. Difficulties in implementing the Three Step Method caused this to be abandoned in favor of the Single Step Method, using the OKA algorithm prior to any comprehensive comparison of results.

More or less in parallel with the allocation analysis, an evaluation was made of the problem of flood control. Two basic computer programs were developed and employed in this analysis, one for determining the desired operating rules to achieve a predetermined degree of flood control (9) and another for routing of floods through the system (10). Actually, in this latter case, two models were developed, one which was basically a volume-storage routing technique, and the other which was based on the dynamic equations of motion. For reasons of convenience, the former was used in the analysis of alternatives. The final evaluation of the flood problem (11) provided essential information for cost analysis of alternatives (16).

Special programs and models were developed to deal with the problems of hydroelectric power (13) and water quality (14). These also provided certain explicit information which assisted in the assessment of costs and benefits derived (16). In a similar way, although in a less quantitative fashion, the benefits or detriments to recreation (12) and navigation (15) were introduced into the evaluation procedure.

In principle, the analytical procedure outlined briefly above was carried out for all variants within a particular system or subsystem, at least to a degree where it was possible to select the most promising alternatives. In accordance with an early decision to decompose the Vistula System into subsystems, the procedure was applied in a like manner for each successive subsystem, with a view to defining the best possible schemes for the entire Vistula Basin (17). Finally, as a result of these analyses there emerged a recommended plan for the water resource development of the Vistula Basin (18) and a program for future investigation (19).

For the purposes of analytical treatment each subsystem of the Vistula Basin was reduced to a conceptual form in which all elements of the real and proposed future configuration for water resource development were represented. Basically, the conceptual model consisted of a network of river reaches, canals, and pipe lines connecting storage reservoirs, diversion points, wastewater return locations and demand centers. This idealization is illustrated by the basic node-link network for Subsystem I shown in Figure 7.

In Figure 7, existing reservoirs are shown as solid triangles (storage nodes), proposed reservoirs as open triangles, transfer or wastewater return locations as circles (transfer nodes) and demand centers (sinks) with the symbol "S". Stream reaches are shown as solid lines, and interbasin transfers as dotted lines. Gaging station nodes are indicated by solid circles.

The out-of-kilter algorithm, a special purpose linear programming algorithm, was used to solve each network allocation problems. The input to the algorithm required a network consisting of nodes and arcs, in which for each arc the source node number, the sink node number, the lower and upper bounds for flow in the arc, and the cost of unit flow in the arc had to be specified. Computer programs were used to generate the arc information for each subsystem. The algorithm proceeded to solve the network allocation problem for each month in each of the 15 years. In order to make the solution more realistic, the data on demand and hydrology for each month were those that might be expected at the beginning of each month. Each monthly solution consisted of the allocation of resources by the out-of-kilter algorithm to the subsystem network.

Summary of Results

The major portion of the mathematical modeling activity for the Vistula Basin was concluded at the end of November, 1971. Specific accomplishments of the mathematical modeling effort included (a) the organization of the Vistula River System into 12 distinct subsystems and the identification of planning variants for further analyses and evaluation; (b) the preparation of hydrologic and water use data and the data describing the physical characteristics and costs of subsystems and variants; and (c) the development, programming and testing of mathematical optimization and simulation models for optimally allocating water supplies, controlling floods, generating hydroelectric power, and maintaining adequate water quality.

Even with these achievements it is fair to acknowledge that there were some difficulties encountered along the way. Most of these may be charged to some over-optimism on the part of the modeling team concerning the practical problems of organizing, directing, and implementing model use on Polish computational equipment. Limited access to computers and peripheral equipment was a serious restriction on the productivity of the team, especially during the early stages of Phase II.

While the mathematical models developed and brought to an operational state in Phase II are currently being applied directly to the evaluation of investment alternatives, they are in a certain sense still in the development stage. For example, it is almost axiomatic that the aggregation of the best solutions for each of the subsystems which comprise the whole of the Vistula River System will not be the best solution for the entire basin. Moreover, it is clear that the solutions for many subsystems are interdependent, i.e., solution for one may partially determine the solution for one or more others and vice versa. Consequently, it is necessary to devise some procedure for analyzing all subsystems simultaneously in order to improve the performance of the Vistula System as a whole. These and other improvements will be needed, both in existing models and in the planning methodology generally. Certain improvements or modifications can be anticipated on the basis of the experience gained in Phase II. The production activity will no doubt reveal others. What is important is that those who have direct responsibility for the development of the Vistula Basin were trained in these modeling skills and now can use these skills for future planning.

An Economic Analysis of the Delaware River Basin[5]

In the mid-60's, Professors Hufschmidt and Fiering published their now widely known text on the development and application of digital computer simulation techniques for evaluating alternative water resource structures and operating policies.[3] To illustrate their methodology, they modeled the Lehigh River, a tributary of the Delaware River in northeastern United States. Their book describes in some detail the physical and hydrologic characteristics of the Lehigh River Basin, the economic benefit-loss and cost functions derived for evaluating alternative management policies, and the development and use of the simulation model for estimating the net benefits associated with any predetermined set of structures and operating policies. The final chapter discusses the extension of the Lehigh simulation model to one that encompasses the entire Delaware River Basin. This study involved the use of that simulation model, together with some optimization models, to define and evaluate alternative structures and operating policies for regulating the water resources of the Delaware Basin.

Unlike the other studies discussed in this paper, no one directly involved in this study was in the position of having to make policy recommendations or decisions. Of our primary interest was the further development and evaluation of techniques that could be used to assist those who had these responsibilities. Thus, the emphasis in this study was on the integrated use of some systems analysis techniques for analyzing relatively large and complex river basins.

Since the primary goal was to develop improved techniques for analyzing large river basin systems such as the Delaware, and not on providing project recommendations per se, the data collected from the U.S. Army Corps of Engineers, U.S. Geological Survey, Federal Water Pollution Control Administration, and other reports by the Harvard Water Program in the early 1960's were used. No major effort was made to update this basic data in order to produce results that would necessarily apply in 1968 when the models of the Delaware were completed and applied. Again, the aim was not to recommend specific project plans and operating policies for the future development and management of the Delaware, but to develop improved tools for those who are responsible for making such recommendations in basins as complex as the Delaware.

In this study, only economic criteria were used to evaluate various management alternatives. Economic optimization and simulation analyses such as those used in this study merely contribute to the decision-making process; they do not provide a substitute for the process. A consideration of the non-quantifiable effects is also necessary in order to define more socially beneficial management policies.

The Delaware River Basin

Approximately 22 million people and one major segment of industry in the United States depend on the water resources of the Delaware River system. The approximately 12,765 square miles drained by the river contain about a tenth of the nation's population. In addition, some of the basin's water is diverted into 22 counties of metropolitan New York and New Jersey, supplying an additional 15 million people in an area about equal to that of the Delaware River Basin itself. "In the number of people served by its waters, and the economic importance of their activities, the Delaware ranks first among American rivers."

Figure 8 shows the location of the Delaware River and its major tributaries. A schematic representation of the basin and the existing and potential reservoirs, hydropower plants and major water supply areas are presented in Figure 9. As shown in this figure, the possible projects considered in this study included 35 reservoirs, 12 variable-head and 9 run-of-river hydroelectric plants, 5 water supply diversions to New York and New Jersey, and 5 major water supply areas in the basin itself. These water supply areas included the Allentown-Bethlehem-Easton area, the Morrisville-Trenton area, the Reading-Pottstown area, and the Norristown-Philadelphia-Camden area. Low flow augmentation for stream quality control, together with biological (secondary) wastewater treatment, were considered at each of the water supply areas.

The benefit-loss and cost data that were incorporated into both types of models used to define and evaluate various alternatives in the basin were expressed as continuous functions of inputs and outputs. These continuous functions were usually estimated from discrete data found in numerous reports on the Delaware basin. Benefits and losses were determined, when applicable, by the alternative cost principle. Loss functions were used to evaluate the consequences of departures from planned target outputs. Many of the water supply, recreation lake level and firm power targets were considered unknown variables as were all the reservoir and hydroelectric plant capacities that did not exist at the time the data were collected. The functional relationships that were typical of the data used in this study are illustrated in Figure 10. Details on the derivation of these functions can be found in Hufschmidt and Fiering.[3]

Two assumptions regarding recreation and power were made in this study. It was assumed that the power load growth in the market area far exceeded the hydropower potential. Thus, any economically feasible peaking power, from either conventional or pumped storage, would presumably find a ready market. It was further assumed that the development of recreational facilities at any site in the basin would not affect the demand for recreation at other sites. Outdoor recreation demands of the state park type in the basin area were estimated to increase from the approximately 80 million visitor days that existed in 1965 to about 100 million in 1980 and 230 million in 2010. This is far in excess of the development potential of all reasonably feasible storage projects in the Delaware basin.

Systems Methodology

Both historical and synthetic monthly streamflows were used in this study. The unregulated monthly streamflows at any site within the Delaware basin were considered to be proportional to the unregulated flow at one of 25 gaging stations in the basin. The distribution of unregulated flows at each gage site in any particular month was expected to remain the same each year. In both models, the unregulated flows were considered to be random variables. The unregulated flows and their probabilities were derived from historical records. Synthetic streamflows supplemented the historical flows and were used extensively by simulation model. The time of flow from the uppermost site to the mouth of the Delaware was assumed to be well within the monthly periods defined by both types of models.

Within the simulation model, short duration flood flows were randomly generated and routed through the river system. The analytical screening models did not explicitly incorporate flood flows themselves, but rather considered flood control storage to be a variable whose value was dependent on the functional relationships between reservoir storage capacity costs and downstream expected flood control benefits.

Because a thorough description of the Delaware Simulation Model is given in Hufschmidt and Fiering's text, only a summary of this model will be presented here. The remainder of the discussion on simulation will involve its use together with the screening models for analyzing project development, timing and sequencing alternatives.

The input into the simulation model was divided into three components, the permanent data, the design data, and the simulated unregulated streamflows at each of 25 gaging stations. The permanent data specified the parameters that were not decision variables. This information included the economic and physical functional relationships that existed between the various components of the river basin system. This data essentially defined the functions illustrated in Figure 10.

The design data specified the values of the decision variables that were to be simulated. These variables included the reservoir and hydropower capacities, and the targets for recreation, hydroelectric energy production and water supply. In addition, and very importantly, the design data defined the operating policy, the rules for determining the reservoir releases, and allocations of water to the various uses. The operating policy, of course, was a function of all the other design and economic parameters of the system and is described in detail in Hufschmidt and Fiering[3] and in Maass, et al.[6]

The streamflow data, consisting of a 50-year monthly sequence of unregulated synthetic flows for each of 25 gaging stations in the basin, were stored on magnetic tape and used for each simulation run. These synthetic flows were generated from a multivariate model developed by Thomas and Fiering.[6]

The simulation model of the Delaware River system was used for both static and dynamic economic analyses. The first analysis was limited to simulating static or steady state economic conditions. Estimates of the annual benefits, losses and costs as functions of the input and output parameters were made for the years 1980 and 2010. Using these future projections in the preliminary screening models, several alternative design configurations were selected, based on their potential for achieving maximum net benefits in those years. These preferred alternatives were then simulated for a period of 50 years, using the same annual benefit, loss and cost functions. By routing 50 years of monthly synthetic streamflows through the basin, estimates were obtained of the annual net benefits that could be expected in the years 1980 and 2010 (together with their variance) for each alternative being simulated.

Static analyses provided snapshots of the performance of various basin management plans that could be expected in the years 1980 and 2010. Once these steady state analyses were completed, the questions of construction project timing and sequencing were examined far more efficiently and cheaply, using both the screening and simulation models, than would have been possible if such dynamic economic analyses were attempted at the beginning.

Completing a static analysis prior to a more comprehensive dynamic screening and simulation analysis permitted a significant reduction in the number of decision variables that had to be considered. It was assumed that those alternative designs eliminated in the static analyses would not need to be reconsidered when examining questions of project scheduling and sequencing. Thus, having used static analyses to obtain snapshots of the most economically beneficial river basin system in the years 1980 and 2010, the dynamic analyses which followed could concentrate solely on how best to achieve those conditions and not on how to define what those conditions should be.

A considerable portion of the time on this Delaware River Basin screening-simulation study was spent in the development of effective screening models. The objective was to construct a model that would be generally applicable for similar screening analyses of other river basins and one that could be fairly easily and quickly solved even given a relatively large number of decision variables and constraints. The decision variables of the model had to include a consideration of the long-term or over-year storage, together with short term or within-year storage requirements; reservoir and hydroelectric power plant capacities; recreation storage, firm power and water supply targets; and flood storage capacities. The model also had to contain the same economic functional relationships for evaluating various management alternatives that were included in the simulation model, e.g., functions similar to those illustrated in Figure 10. Finally, the model had to be adaptable to a dynamic economic analysis as well as to static or steady state analyses.

Optimization techniques do not yet exist that can ideally satisfy all of the above listed conditions. After some experiments with several non-linear models and algorithms, linear programming was chosen for the task of preliminary screening. Non-linear functions were made piece-wise linear. Concave cost functions required several trial solutions using linear approximations before a reasonably accurate solution was obtained. (Separable programming algorithms would have eliminated the need of some trial solutions had it been available at the time of this study.) Examples of such functions included project costs that exhibit fixed components and increasing returns to scale (concavity) and energy functions containing the product of two decision variables, storage head and flow. The task of approximation and iteration was much easier and less costly than first expected.

It quickly became apparent that deterministic linear programming models, using mean monthly flows, were not capable of capturing the significant variations in yearly and monthly flows which contributed substantially to the need for long term as well as short term water supply storage capacity in the basin. This led to the development of some stochastic linear programming models. These models incorporated some of the hydrologic risk inherent in the Delaware system by defining more than one possible streamflow and more than one possible reservoir storage level in each period within a year. Thus, a variety of discrete flows and reservoir volumes, and their corresponding probabilities of occurrence, were considered in the determination of optimal reservoir capacities; recreation, hydropower and water supply targets; and operating policies. Optimality was assumed to be achieved when the expected annual net benefits were maximized.

Summary of Results

As previously stated, the primary purpose of this study was to examine the problems involved in integrating optimization and stimulation methods for both defining and evaluating alternative design and operating policies for large, complex river basin systems. The Delaware River provided realistic and sufficient data for model development and comparison. Throughout this study, no attempt was made to find the optimal economic solution. It is unlikely that such a single solution can be defined, if for no other reason than the uncertainty of future economic conditions and technology. What was sought in this analysis was a relatively small set of near optimal management plans, each somewhat flexible as to possible changes in future conditions. Both types of models, but especially the simulation model, were employed to examine the effects of changes in both economic and engineering parameters on the design and operation of the entire system.

The river basin was analyzed as a whole only after several subbasins were examined separately. Subbasin analyses were made to compare the different solutions and to estimate the interdependence of one subbasin on another. Also of interest was the reduction in total expected net benefits if constraints were added to provide for more equitable geographic distribution or balance of benefits or projects among subbasins or between the four states (Delaware, New Jersey, New York and Pennsylvania) included in the basin.

Water quality control through streamflow regulation and augmentation was also examined. To examine the reduction in system net benefits if this method of stream quality control were used in addition to secondary treatment, low flow augmentation requirements were imposed at various levels of certainty. No attempt was made to evaluate the benefits or losses resulting from changes in stream quality. Dissolved oxygen, biochemical oxygen demand, acidity and salinity were the quality parameters considered in this study. The future streamflows that would be required, after secondary or biological wastewater treatment to maintain various quality standards, were provided by what is currently the Water Quality Office within the Environmental Protection Agency. Using these minimum required flows, the stochastic static screening models provided information on the reduction in the quantifiable expected annual net benefits associated with various probabilities of maintaining any fraction of these minimum flow requirements. Such procedures permitted the simultaneous evaluation of water quality and other non-monetary objectives, along with the other beneficial uses, and provided a means of estimating the economic system cost associated with any such non-monetary objective or constraint.

One of the difficulties in evaluating any study of a system in which a global optimum is not known is that it is never clear which particular set of project capacities and operating policies should be used as a basis for comparison. Nevertheless, there is some reason to hope that the solutions finally achieved in this analysis came fairly close to the global optimum. For one thing, the change in expected net benefits resulting from varying, even up to 10 percent, many of the engineering and economic parameters, taken one or at most several at a time, was insignificant. This implies a relatively flat expected net benefit response surface in the region of the best solution.

Possibly because of the flatness of the net benefit response surface, the expected annual net benefits derived from the screening models were always within 8 percent of the expected annual net benefits derived from the simulation models. This relatively close correspondence between the solutions of the two types of models can be considered nothing short of amazing, given the assumptions that were required during the development of the screening model. Of course, these results are based on only a relatively small number of comparisons. Given more time and money, some larger differences might have been found. Nevertheless, it appeared that at least for the models and data used in this study, the net benefits derived from the screening analyses of each subbasin as well as for the system as a whole were essentially similar to those derived from the simulation analyses.

More important than the correspondence between the values of the objective functions of each model was the correspondence between the values of the decision variables after attempts at system improvement using simulation were completed. Here, too, the results appeared to demonstrate the effectiveness of both screening and simulation models for the analysis of large river basin systems. For one tributary of the Delaware, the Schuylkill River, a better solution than that indicated by the screening model could not be found using simulation, even after an exhaustive search. This was the exception, however, since in general some improvement was obtained through sensitivity analysis using the simulation model. No "new" projects, i.e. those at zero capacity in the screening solution, were ever found that would increase the net benefits derived from the simulation model. Usually, some adjustments in the proposed project capacities or targets were beneficial. Again, given more time, it is possible that projects not defined by the screening solution could have been found that would increase the total net benefits. No doubt this fact has much to do with the assumptions made during the screening analysis where the intent was to eliminate the worst solutions.

It may be interesting to note that the use only of mean monthly flows in a deterministic screening model of the entire Delaware system resulted in null solutions, i.e. any change in the existing system would only decrease the annual net benefits. This clearly inaccurate result motivated the development of the stochastic screening models. However, by reducing each mean monthly flow in the Schuylkill River by 80%, the deterministic model of the Schuylkill yield approximately the same results as did the stochastic model.

No explanation of this coincidence is offered here, but it does appear that the smaller deterministic screening models might be used to define a series of solutions, each corresponding to a different fraction of the mean streamflow. These solutions could then be further analyzed, using simulation in order to select the best. However, such a series of possible solutions may not always include the best one for the system itself, as apparently happened using the Schuylkill model.

The dynamic screening-simulation analyses were equally as successful as those of the static analyses. The results showed that the demands for recreation, water supply, power and flood control, were sufficiently great in the Delaware basin to make it desirable to construct most of the projects proposed for 2010 in the first 20 years of the 50-year planning period. Perhaps if budget constraints were imposed, the scheduling of projects would have been more evenly spread out over the total 50 years. Of primary interest in any such dynamic analysis, of course, is the first period's decision since later analyses with improved data may suggest changes in the decisions of future periods. Using the dynamic-simulation model to explore the sensitivity of reasonable changes in future economic and technological conditions, no significant improvement for the first scheduling decisions was found.

Once the best solution was obtained from both the static and dynamic analyses, it was compared to solutions this writer and his associates had chosen after reading numerous technical reports prior to any model building solutions. The best guess was only 73% of the maximum total present value of the expected annual net benefits obtainable. Perhaps this is a clear indication of underdeveloped economic-engineering judgment. Those with more knowledge of the Delaware would have certainly exceeded our performance, but we suspect that most would have had difficulty proving it. If nothing else, analytical tools for analyses such as these provide them that opportunity.

Complete records were not kept on the man-hours and money spent in data preparation and model development and testing. Clearly, these activities contributed most to the total cost of this analysis. Whatever the cost, it was of several orders of magnitude less than the additional net benefits derived from the 37% improvement over our initial best estimate of the optimal solution. For the Schuylkill River alone, this represented more than one million dollars in expected annual net benefits.

Operation of the Tsengwen Reservoir Irrigation Project[4]

Taiwan is composed of a group of islands, situated in the Pacific Ocean about 100 kilometers off the coast of mainland China. It covers an area of 35,961 square kilometers, which is about one third that of New York State and a little larger than Holland. Taiwan's population in 1970 was approximately 14 million. About two-thirds of Taiwan consists of rugged foothills, ranges and mountains. The climate is subtropical, with a warm and humid summer. Crops can be produced throughout the year: The average rainfall is over 2,500 mm a year, ranging from 1,700 mm in the west coast to 3,000 mm in some parts of the mountains. Rainfall distribution is very uneven both with respect to season and location, thus, the storage and regulation of water for utilization can often be beneficial.

Irrigation in Taiwan can be traced back as far as the Yuan Dynasty of China (1291-1379). However, extensive irrigation development actually began around 1895. By 1933 the irrigated area had expanded to 400,000 hectares (988,000 acres). Since 1945, systematic research on .water application methods for paddy rice has been conducted. Data from laboratory experiments and field experience indicated that in water short years substantial savings in water could result from relatively precise management utilizing rotated water deliveries. Following the drought year, 1954-55, this practice has rapidly displaced the traditional continuous irrigation, and has played a very important role in the improvement of irrigation efficiency. The irrigated area also has been increased to 452,000 hectares during this period. Approximately 57% of the total cultivated land in Taiwan in 1970 was irrigated.

The competitive utilization of the limited land, labor and water resources between agricultural and industrial uses in Taiwan is becoming acute. As a result, the more efficient use of water and land has become of even greater importance. This study attempted to define improved cropping patterns and operational procedures for a reservoir irrigation system in Taiwan.

Tsengwen Reservoir and Irrigation Project

The Tsengwen Reservoir Project is a multipurpose development designed to meet the urgent demand for additional water resources in the Chianan Plain in southwestern Taiwan. The Tsengwen River is the major source of water for the project. It is one of the largest rivers in southern Taiwan, and has a length of about 138 kilometers (86 miles) along the main stem and a drainage area of about 1,177 square kilometers (458 square miles). The flow depends primarily upon the amount of storm rainfall in the typhoon season, which lasts from June to September.

Flow records for the Tsengwen River are available for a continuous period from 1919 to the present. The records are derived from staff gages read twice daily during periods of low flows and at hourly intervals during flood time. Current meter ratings of the site provide the gage-flow correlations.

The Chianan Irrigation Association's existing Wushantou Reservoir and related works are currently serving some 80,000 hectares of land in the project area. However, because of limited water supplies and the increasing demands of both growing population and industry, the existing reservoir has not been able to meet the demand for water services.

Currently being constructed is the reservoir on the upper reach of the Tsengwen River. This new reservoir is to provide a supplemental water supply for irrigation and domestic use. As illustrated in the schematic of Figure 11 it includes a large storage dam and reservoir on the Tsengwen River to regulate the flow of the river, and an afterbay weir to regulate and divert water through the existing Tungko Tunnel into the Wushantou Reservoir.

The Tsengwen Reservoir has been under construction since 1969 and is scheduled for completion in 1974. The Wushantou Reservoir was constructed in 1930. The Wushantou Reservoir is not able to supply all the irrigation water required by the area it serves. As a result, the land in this irrigation area cannot be utilized to its full potential. By the implementation of the Tsengwen project, the water storage capacity in this area will be increased six times.

The area of cultivated and irrigated farm land in the project area is listed in Table 1.

Table 1. Available Farm Area (Hectares)

I Three Year Crop Rotation Area			II Combined Irrigation Area	III Taiwan Sugar Corp. Farm Area
Loam soil	Clay soil	Saline soil		
51,100	9,025	3,470	9,675	8,158

Within the project area the crops consist of paddy rice, sugar cane, and a variety of miscellaneous crops. Rice is the staple food in Taiwan. Most of the investments for irrigation in Taiwan have been made for the purpose of increasing paddy rice production, the most important crop in this project. Sugar, a primary crop in this part of Taiwan, is one of the important exports. The most important of the miscellaneous crops grown in this area are sweet potato, jute, peanut, maize and soybean. They are normally cultivated in the loam soil area of the region.

Over a period of years a cropping pattern and an associated irrigation schedule have evolved in the Chianan Area. Primarily, on the basis of operational experience, water requirements have been established for individual crops grown on the major soil area for each of the crop seasons.

System Methodology

To examine the possibilities for increased irrigation benefits, two tentative models for optimizing the cropping and water allocation policies were formulated using deterministic and stochastic linear programming. Much of the data used in the models were based on "A Report on the Engineering and Economic Feasibility of a Multipurpose Project", issued by the Taiwan Provincial Water Conservancy Bureau in April 1965. It was assumed during the development of the models that the market price and production cost of the crops cultivated in the project area would remain rather stable. The design parameters and available farm area were fixed, i.e., the reservoir capacity and the areas for different soil types and kinds of districts, if any, were predetermined. The flows into the reservoir were defined by discrete first-order Markov chains in the drought season and by independent probability distributions in the wet season.

The model used to estimate optimal cropping allocations was a deterministic planning model. It was used to determine the cropping allocation at the beginning of each planting season. A sequential operating policy model, based on the result of planning model, was developed to derive the optimal allocations of water to each irrigation field, once the planting was complete. This sequential operating policy defined the water allocation to each crop and the reservoir releases as a function of current reservoir storage volume and inflow.

By analyzing some records of the existing irrigation area and combining these data with three years' experimental data of the Taiwan Irrigation Experimental stations, preliminary water production functions were formulated for each crop and soil type. The production cost and market price of crops were estimated from 1969 data in the project area. The availability of these production functions and economic information made it possible to compute water-benefit functions as shown in Figure 12.

These water-benefit functions were based on the following assumptions:

(a) The physical facilities were fixed and the water user has planned his operation expecting to receive a given target quantity of water.

(b) The irrigation activities always expect to receive their target allocations. Yet because of the uncertainty of streamflows, their actual allocation may be greater or less than their target. Deficits allocations might occur in case of a shortage of natural inflow. Surplus allocations would not occur, since appropriate field drainage and reservoir regulation facilities are available.

(c) Market prices of crops in Taiwan would not be affected by the production of various quantity of crops in the project area. (This assumption may not be true for extreme deviations in production.)

The objective of the overall planning model was to maximize the annual net benefits B (QX) from the allocation of water Q (volume/ha) to an area X (ha) of a particular farm area, crop type, and soil type.

$$\text{Maximize} \sum_{\substack{a \\ \text{farm} \\ \text{area}}} \sum_{\substack{c \\ \text{crop} \\ \text{type}}} \sum_{\substack{s \\ \text{soil} \\ \text{type}}} \sum_{\substack{t \\ \text{season}}} B_{acst} \quad (Q_{acst} \cdot X_{acst})$$

The solution of function will determine the optimal areal distribution of the various crops and optimal water allocations that will result in the maximum net benefit from the irrigation project.

Just how this objective might be achieved was analyzed using two linear programming models. The first estimated the optimal values of the X variables given estimates of available water supplies for the coming year, and the second model estimated the optimal values of the Q variables given actual water supply and crop conditions at the beginning of each decision period.

The constraints of the first model, used to estimate the optimal value of the X variables, i.e., what, where, when and how much to plant, can be summarized as follows:

(1) Reservoir storage volumes could not exceed the capacity of the reservoir.

(2) Water allocations to the irrigation project had to equal some multiple of the target allocation, depending on water availability and the amount allocated to each domestic and industrial use.

(3) The flow through Tungko tunnel could not exceed its capacity.

(4) Continuity of flow and storage volumes throughout the system had to be maintained.

(5) Constraints on available soil type acreages in each farm area.

(6) Special constraints on crops that are planted and grow for more than one season, e.g., sugar cane, or crops that must be planted in certain areas, etc.

The second model was a sequential linear programming model, used sequentially, to obtain as close to an optimal allocation of water to each farm area as is possible without perfect foresight. The form of the model is somewhat similar to those models developed for long-run planning of the Delaware Basin discussed in a previous section of this paper.

Summary of Results

The results of the first model are summarized in Table 2. Note the increased benefits that are obtainable except in the driest years by changing traditional cropping patterns.

When the optimal cropping allocation policy in the hydrological mean year is compared with the original cropping allocation planning, it can be seen that the annual benefit of the original planning is 13% less. This additional benefit in the optimal planning model is gained by replacing about 22,000 hectares of upland crops with paddy rice. However, losses due to a deficit of water in paddy rice may be much more serious than in the upland crops; so the risk of the replacement of upland crop by paddy rice must be carefully examined. In the drier year, the cropping allocation will result in relatively large areas of particular crops. Obviously, this kind of cropping pattern may not result in a higher benefit, because a surplus of these crops may result, with consequent lower prices, and a reduced benefit function.

The entire question of how to get an optimal crop area allocation adopted by farmers is a serious one that was not considered in this study. Possibilities exist for government control through water allocations or pricing policies.

Perhaps the most immediate benefit these models had was in the analysis of the effect of various capacities of the Tsengwen Reservoir on the net irrigation benefits. Reducing the capacity of the Tsengwen Reservoir from the design 76,740 hectare meters to 67,000, 50,000 and 35,000 hectare meters, the annual net benefits of the optimal crop patterns were determined, and are shown in Table 3.

From Table 3, it is clear that there is substantial reservoir capacity that may be used for purposes other than storage of water for irrigation. In decreasing the reservoir capacity by 54%, the annual benefit of irrigation is reduced only about 8%. From the view of short-run irrigational benefit, the capacity of the reservoir seems unnecessarily large.

Subsequent simulation studies have verified these findings. As a direct result, a decision was made early in 1972, to reduce the design capacity of the Tsengwen Reservoir. This will permit an earlier completion date than was originally planned.

Conclusions

This paper has summarized the experience obtained from the application of optimization and simulation models for defining and evaluating alternative investment and management policies for four river basin systems. In each study, optimization models were used to reduce the range of possible alternative design capacities, targets and operating policies for further more detailed evaluation by simulation techniques. Optimization was used for preliminary screening, simulation for detailed evaluation and sensitivity analysis.

In each case, without the information provided by the screening models, it would have been both impractical and expensive to simulate a sufficient number of design and policy alternatives to be able to conclude with reasonable confidence that an optimal or near optimal set of alternatives had been found. Yet without the ability to simulate the results derived from the solutions of the screening models, there would be little opportunity to test the effect of the many limiting assumptions that often must be made when structuring a mathematically tractable optimization model of a complex river system. Thus, both mathematical programming and simulation techniques were used in an effort to overcome some of the limitations that would exist if either had been used by itself.

Based on the limited results of these basin studies, the combined screening-simulation method of analysis appears to be both a practical and an efficient means of defining and evaluating alternative investment and management policies for large relatively complex river basin systems.

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Table 2. Optimal Cropping Allocation Policy in Different Hydrological Situations

Crop Category					Target Benefit (10,000 N.T.$/ha)	Cultivated Acreage (ha.)						
						Original Plan (By Experience)	By Optimizing Analysis					
X	a	c	s	t			Mean (1931-1964)	Wettest Year (1935-1936)	Driest Year (1954-1955)	25% more than Mean (1932-1933)	40% less than Mean (1936-1937)	20% less than Mean (1951-1952)
X	1	1	1	1	1.2944	17,037	28,617	28,535	0	28,326	7,912	28,883
X	1	1	1	2	1.05510	17,037	38,049	42,590	0	42,590	0	2,236
X	1	2	1	1	0.5667	26,455	9,600	9,600	9,600	9,600	9,600	9,600
X	1	3	1	2	0.8125	2,555	4,541	0	5,110	0	5,110	5,110
X	1	4	1	1	0.68880	10,140	2,500	2,500	31,621	2,500	21,706	0
X	1	4	1	2	0.68880	0	0	0	37,480	0	37,480	35,244
X	1	5	1	1	0.52520	5,146	0	0	0	0	0	0
X	1	5	1	2	0.52520	0	2,838	2,961	2,073	3,278	5,110	6,223
X	1	1	2	1	1.29780	9,025	9,025	9,025	0	9,025	9,025	9,025
X	1	1	2	2	1.2080	9,025	9,025	9,025	5,419	9,025	9,025	9,025
X	1	6	2	1	0.55310	3,249	3,249	3,249	12,274	3,249	3,249	3,249
X	1	7	1	1	0.7724	11,926	8,520	8,520	8,520	8,520	8,520	8,520
X	1	1	3	1	1.0511	3,470	3,470	3,470	0	3,470	3,470	3,470
X	1	1	3	2	0.6535	3,470	3,470	3,470	0	3,470	3,470	3,470
X	1	6	3	1	0.3721	1,249	1,249	1,249	0	1,249	1,249	1,249
X	2	1	1	1	1.26230	9,675	9,675	9,675	0	9,675	9,675	9,575
X	2	1	1	2	0.77090	9,675	9,675	9,675	0	9,675	9,675	9,675
X	2	6	1	1	0.50280	3,483	3,483	3,483	13,158	3,483	3,483	3,483
X	3	7	1	1	0.7724	6,454	8,158	8,159	8,158	8,159	8,158	8,158
Total Benefit (10,000N.T.$)						(87%) 133,865.3209	(100%) 154,625.8321	(100.68%) 155,686.6188	(59%) 91,111.6010	(100.6%) 155,582.5584	(83%) 128,380.240	(92%) 141,977.73[illegible]
Natural flow in T.W. Damsite (HA.-m.)						-	(100%) 119,181	(171%) 203,328	(25%) 30,602	(125%) 149,379	(61%) 73,171	(82%) 97,785
Utilization ratio of Dam Site natural flow						-	100%	70%	100%	81%	100%	100%

Table 3

Sensitivity of Annual
Irrigation Benefits of Tsengwen Reservoir Capacity

Unit: 10,000 NT$

Reservoir Capacity (ha.-m.) \ Hydrological Year	Mean (1931-1964)	Wettest Year (1935-1936)	25% more than mean (1932-1933)	40% less than mean (1936-1937)	20% less than mean (1951-1952)
(100%)	(100%)	(100%)	(100%)	(100%)	(100%)
76,740	154,625	155,686	155,582	128,380	141,977
(87%)	(100%)	(100%)	(100%)	(100%)	(100%)
67,000	154,625	155,686	155,582	128,380	141,977
(65%)	(98%)	(97%)	(96%)	(100%)	(98%)
50,000	152,559	151,167	149,507	128,380	139,577
(46%)	(92%)	(91%)	(90%)	(90%)	(96%)
35,000	142,344	141,986	140,322	127,494	139,967

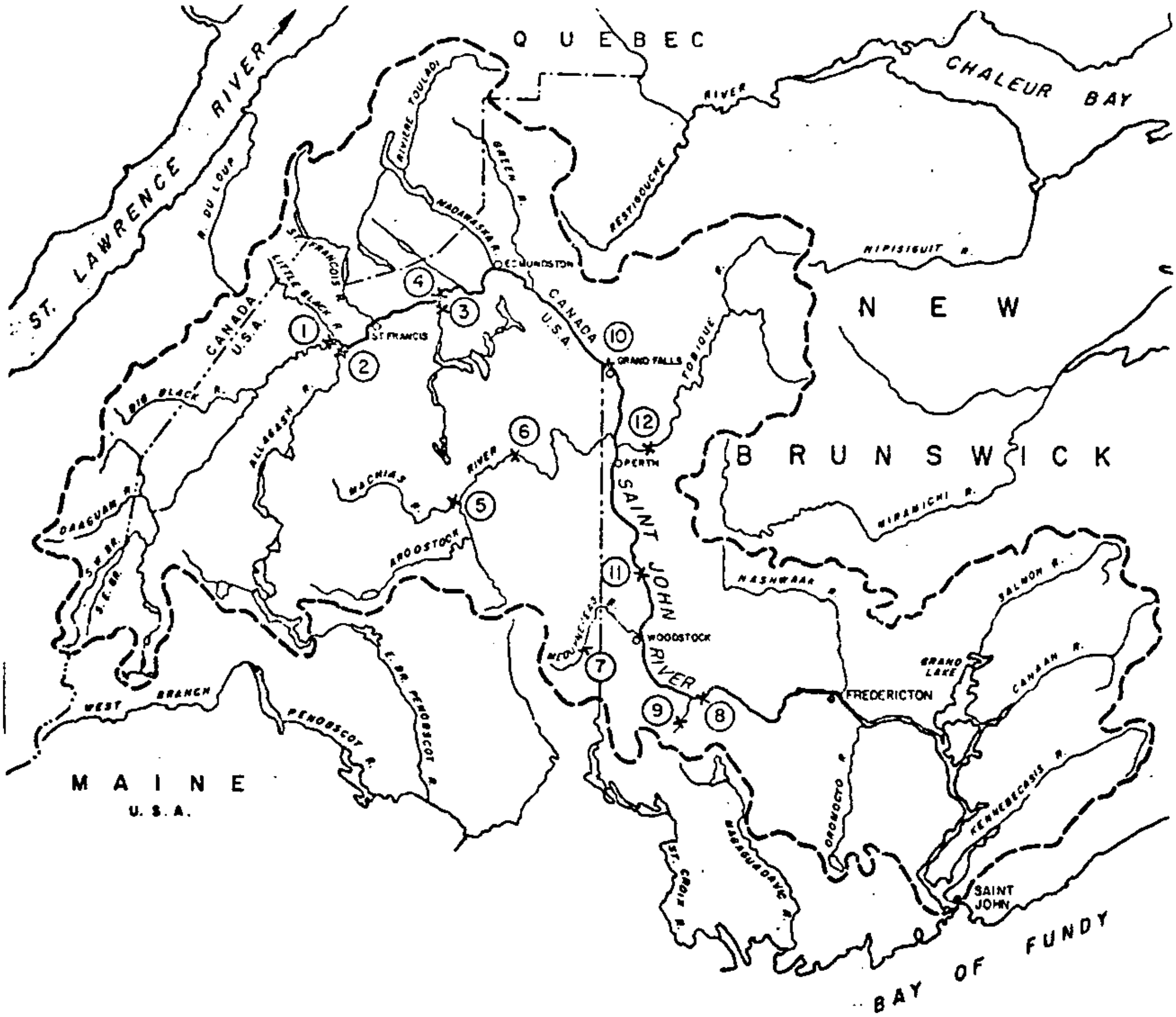
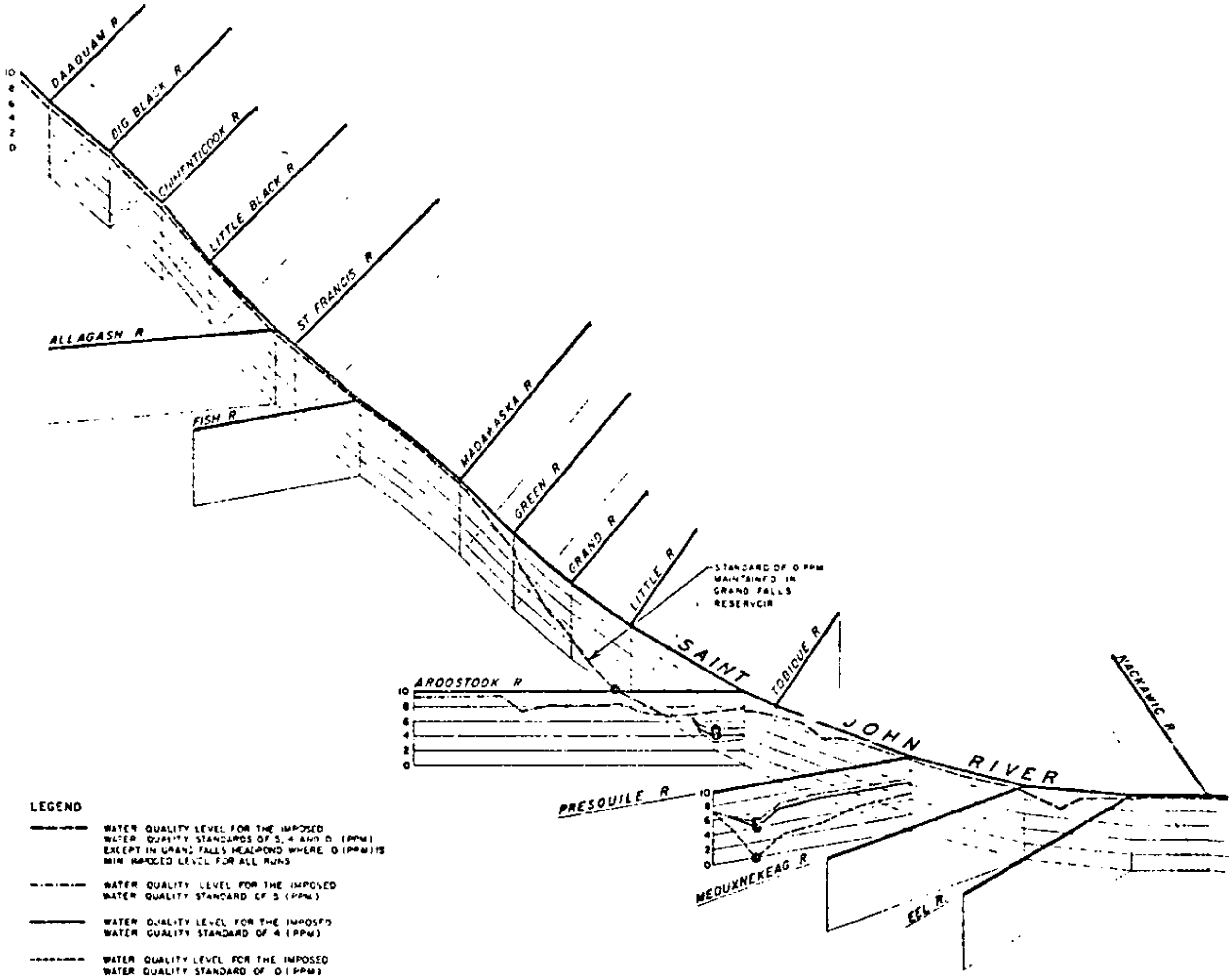


Figure 1.--Saint John River Basin



LEGEND

WATER QUALITY LEVEL FOR THE IMPOSED WATER QUALITY STANDARDS OF 5, 4 AND 0 (PPM) EXCEPT IN GRAND FALLS HEADPOND WHERE 0 (PPM) IS MIN IMPOSED LEVEL FOR ALL RUNS

WATER QUALITY LEVEL FOR THE IMPOSED WATER QUALITY STANDARD OF 5 (PPM)

WATER QUALITY LEVEL FOR THE IMPOSED WATER QUALITY STANDARD OF 4 (PPM)

WATER QUALITY LEVEL FOR THE IMPOSED WATER QUALITY STANDARD OF 0 (PPM)

WATER QUALITY LEVEL EQUALS IMPOSED WATER QUALITY STANDARD

NOTE

ALL TREATMENT PLANTS CONSTRAINED TO BE AT LEAST AT PRIMARY TREATMENT LEVEL

STANDARD DISSOLVED OXYGEN LEVEL (PPM) EXCEPT FOR GRAND FALLS HEADPOND	0	4	5
TOTAL ANNUAL COST OF TREATMENT	$ 4,702,000	$ 4,986,000	$5,247,000

Figure 2.--Sample Solutions From Screening Model

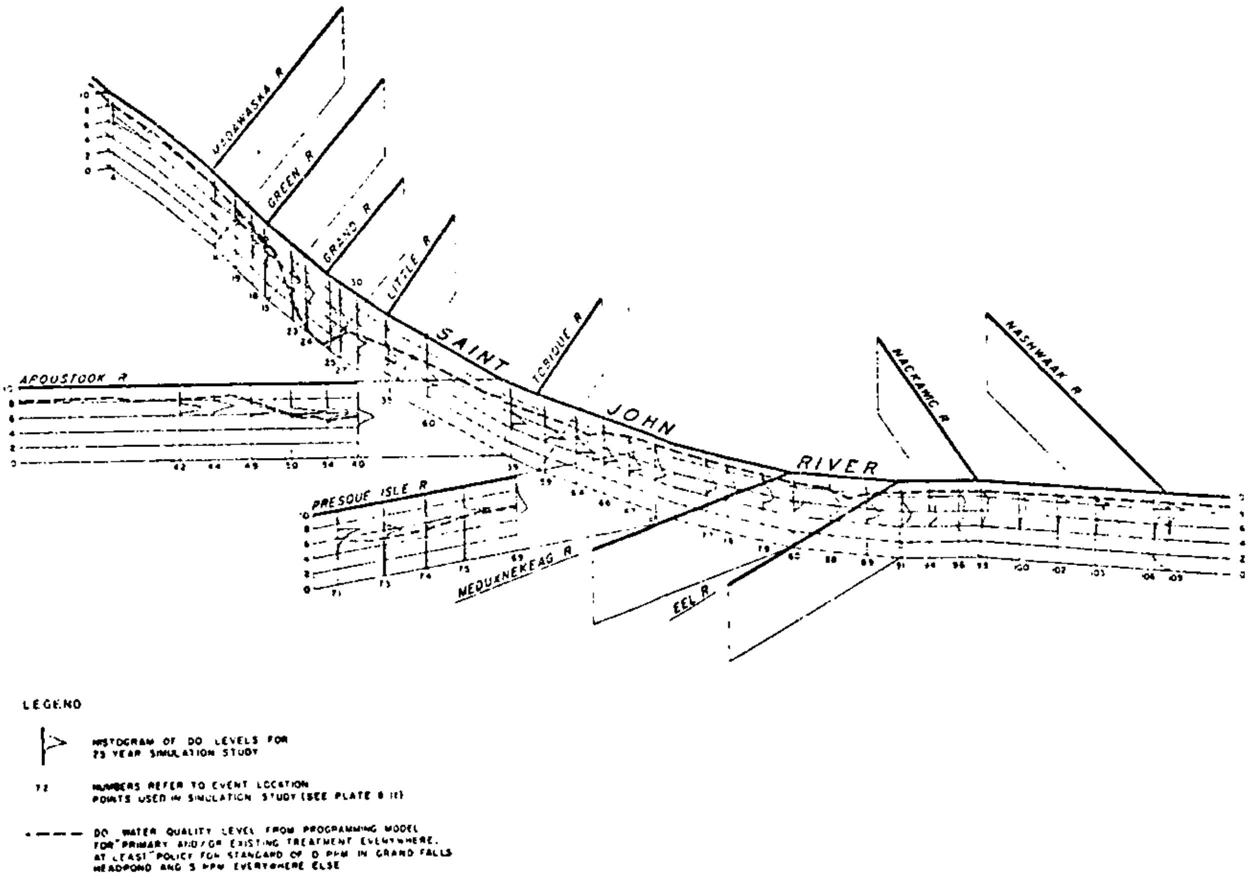


Figure 3.--Sample Solutions from Screening and Simulation Models

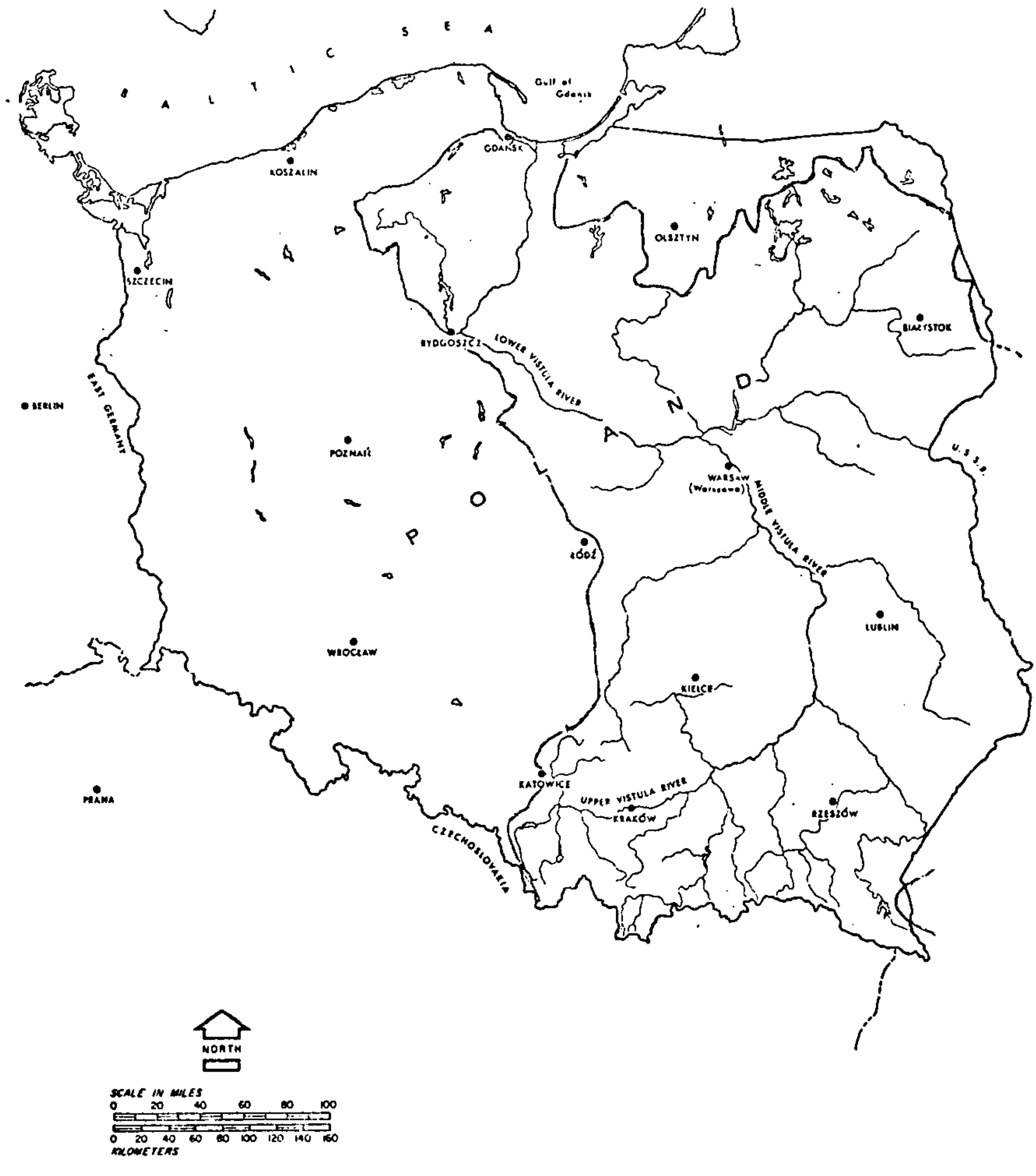


Figure 4.--Vistula River Basin

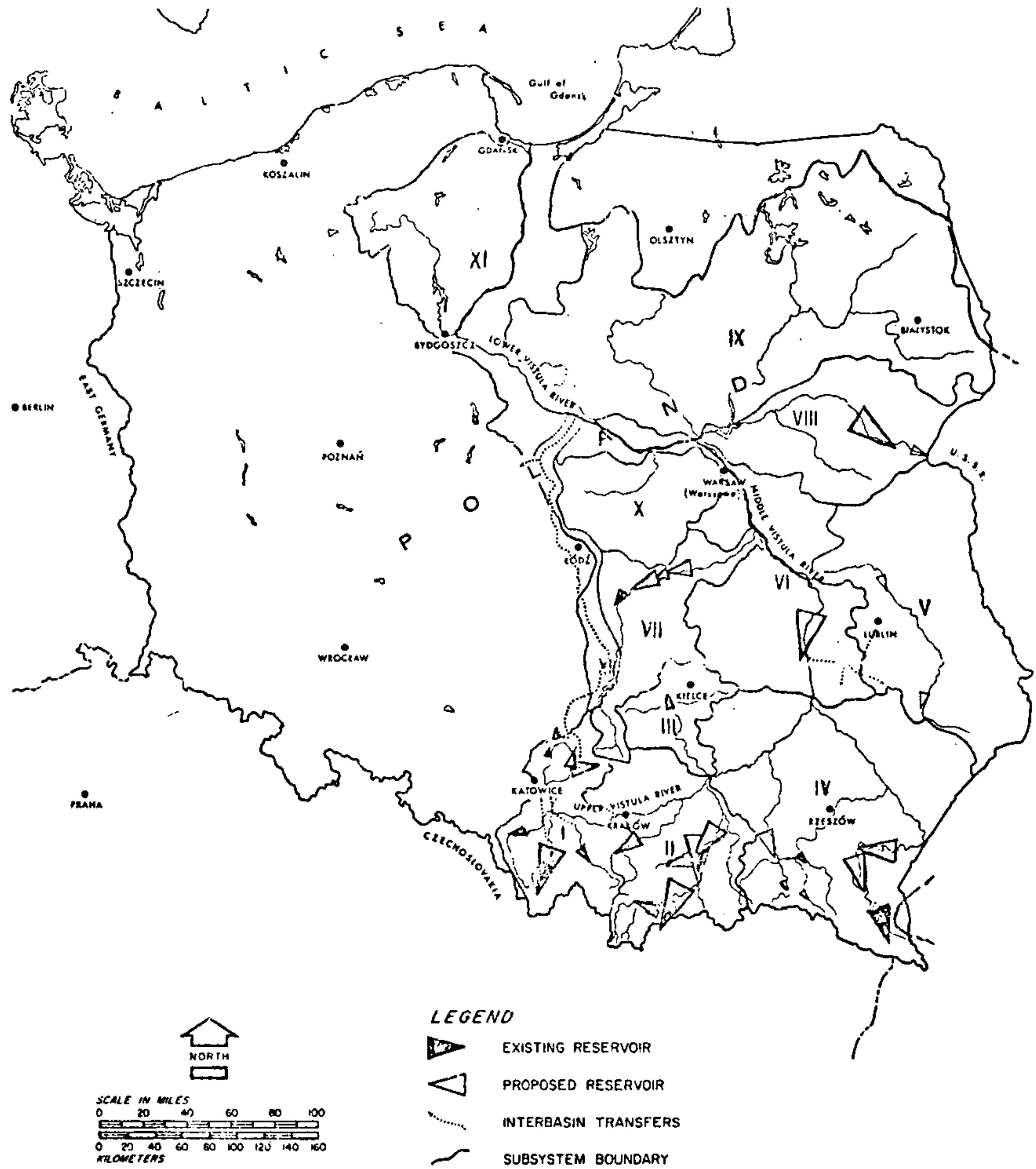


Figure 5.--Vistula Subbasins and Potential Reservoirs

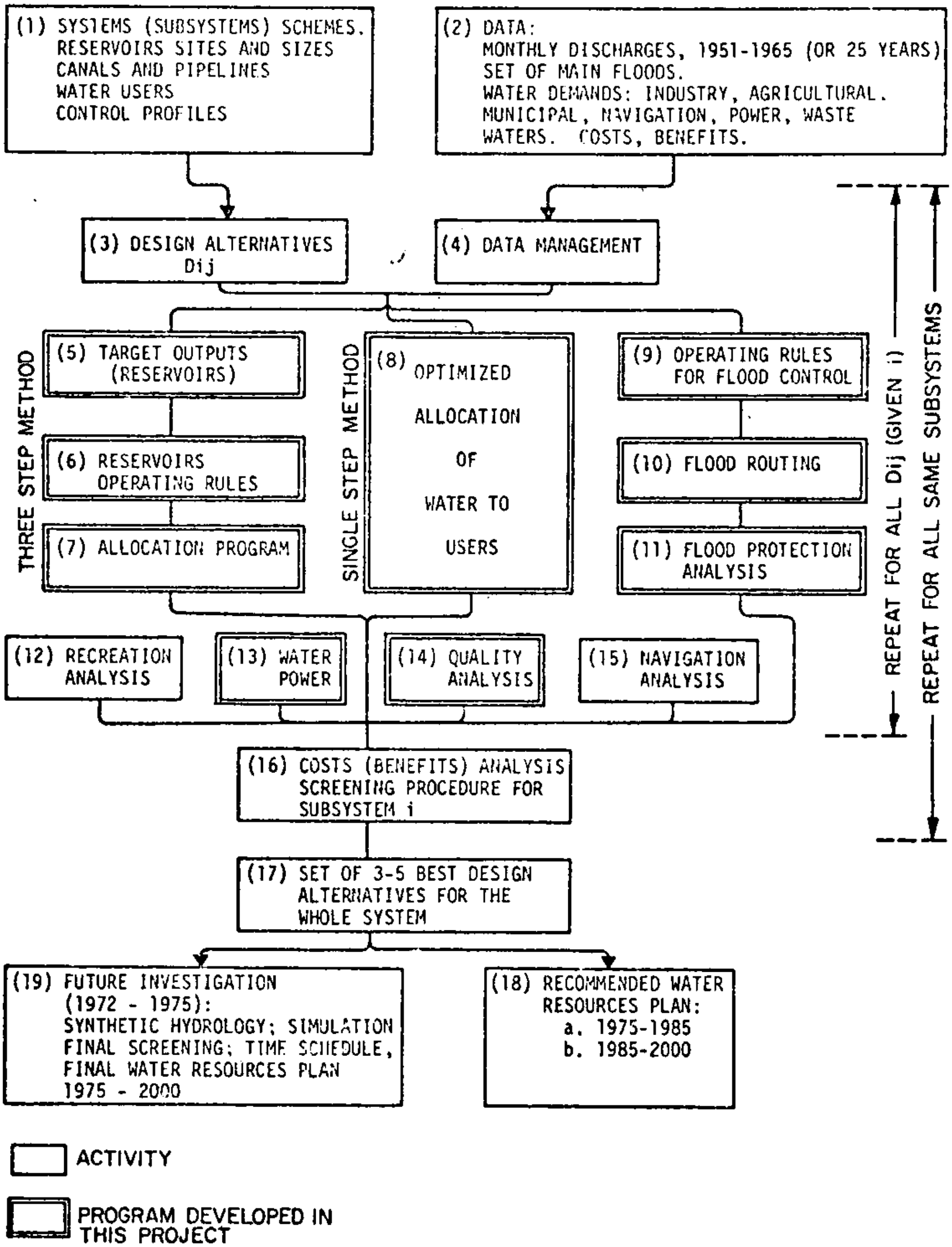


Figure 6.--Modeling Procedure for Vistula System

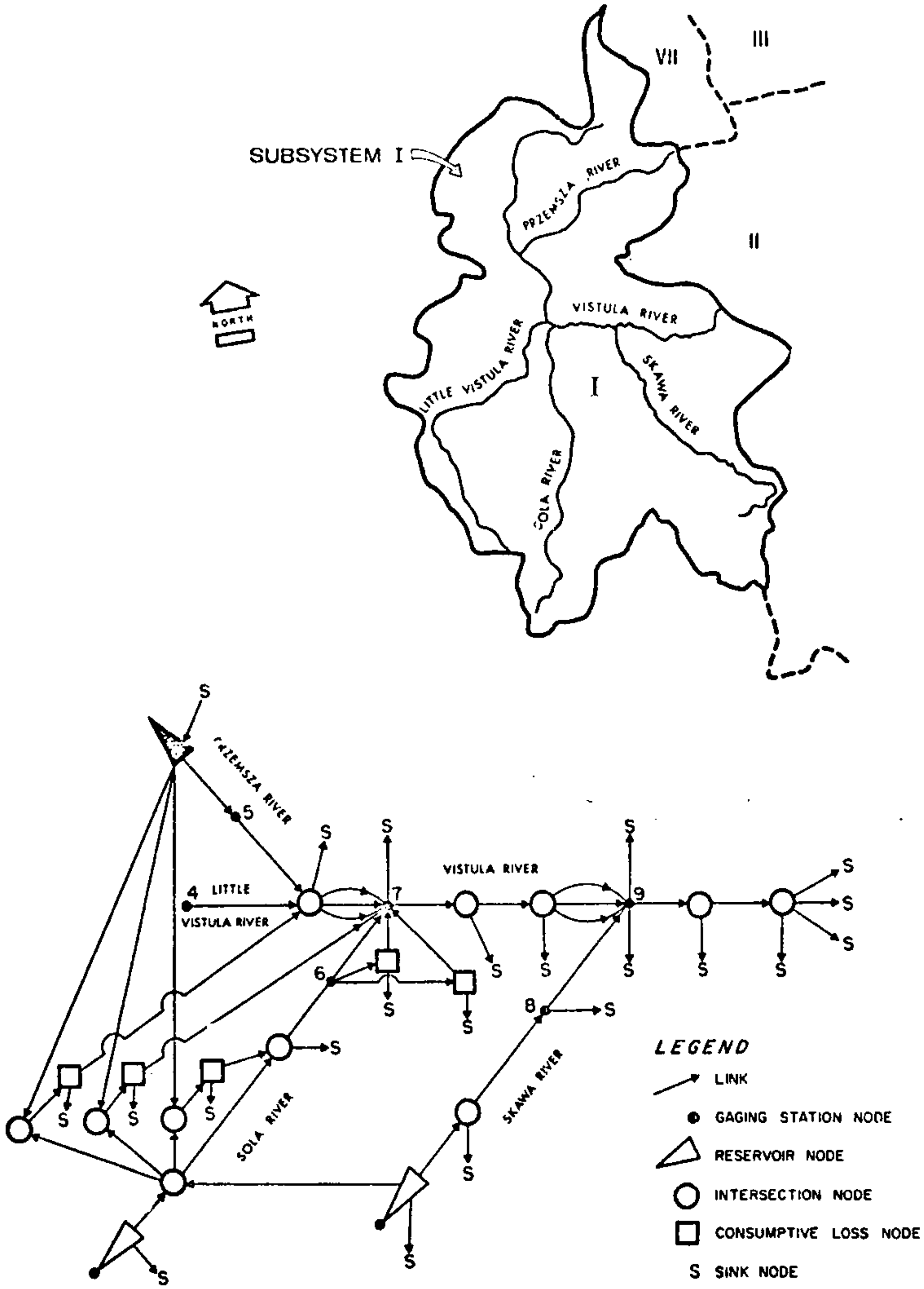


Figure 7.--Network Schematic for Subbasin I

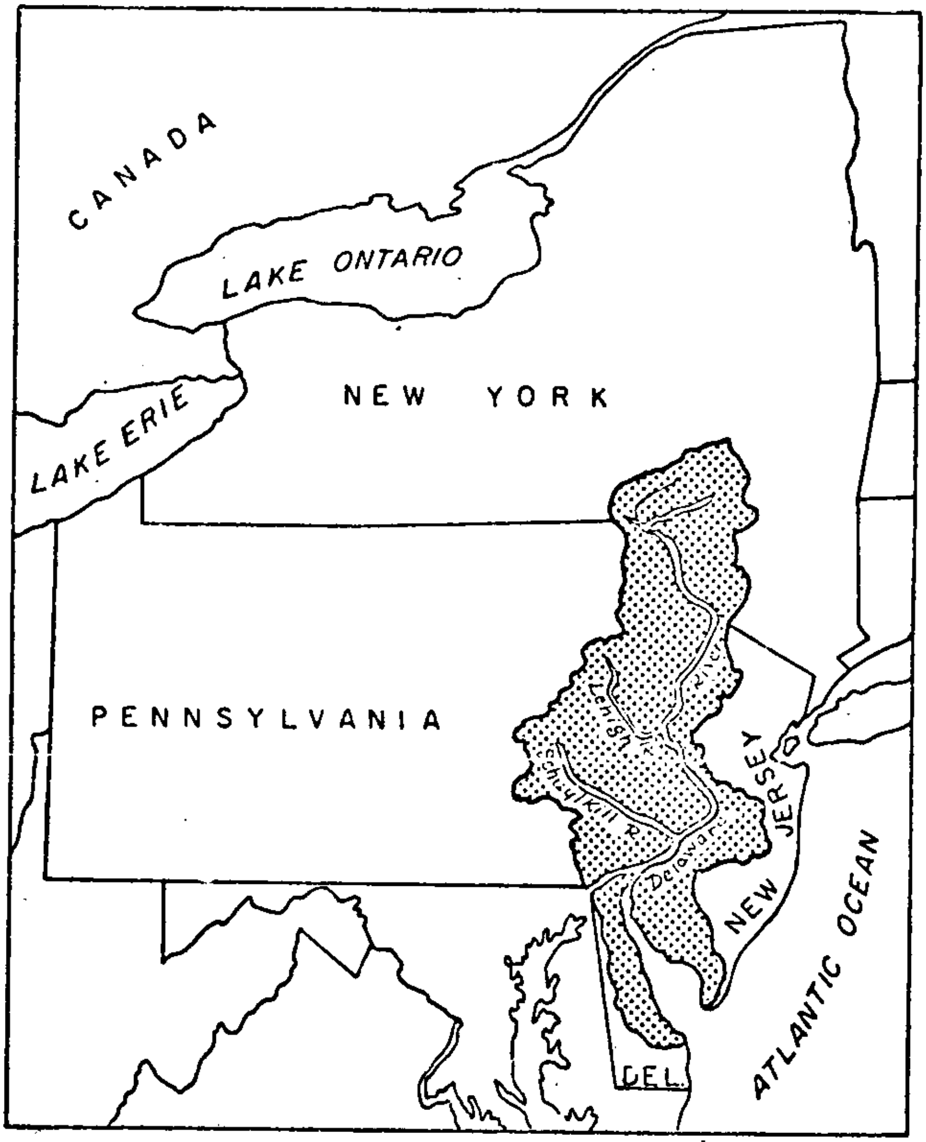
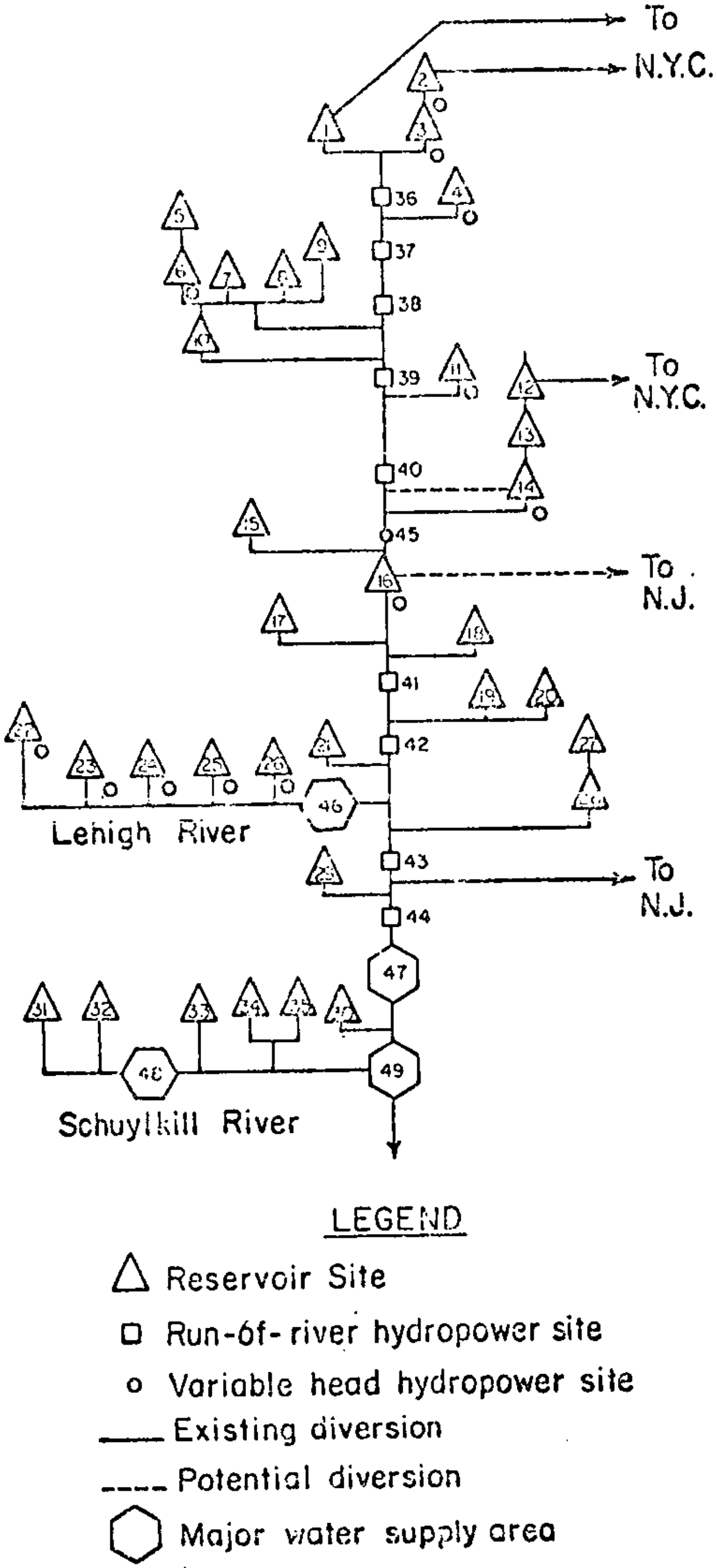


Figure 8.--Delaware River Basin



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*existing reservoirs have site numbers circled.

Figure 9.--Schematic of Delaware System Design Alternatives

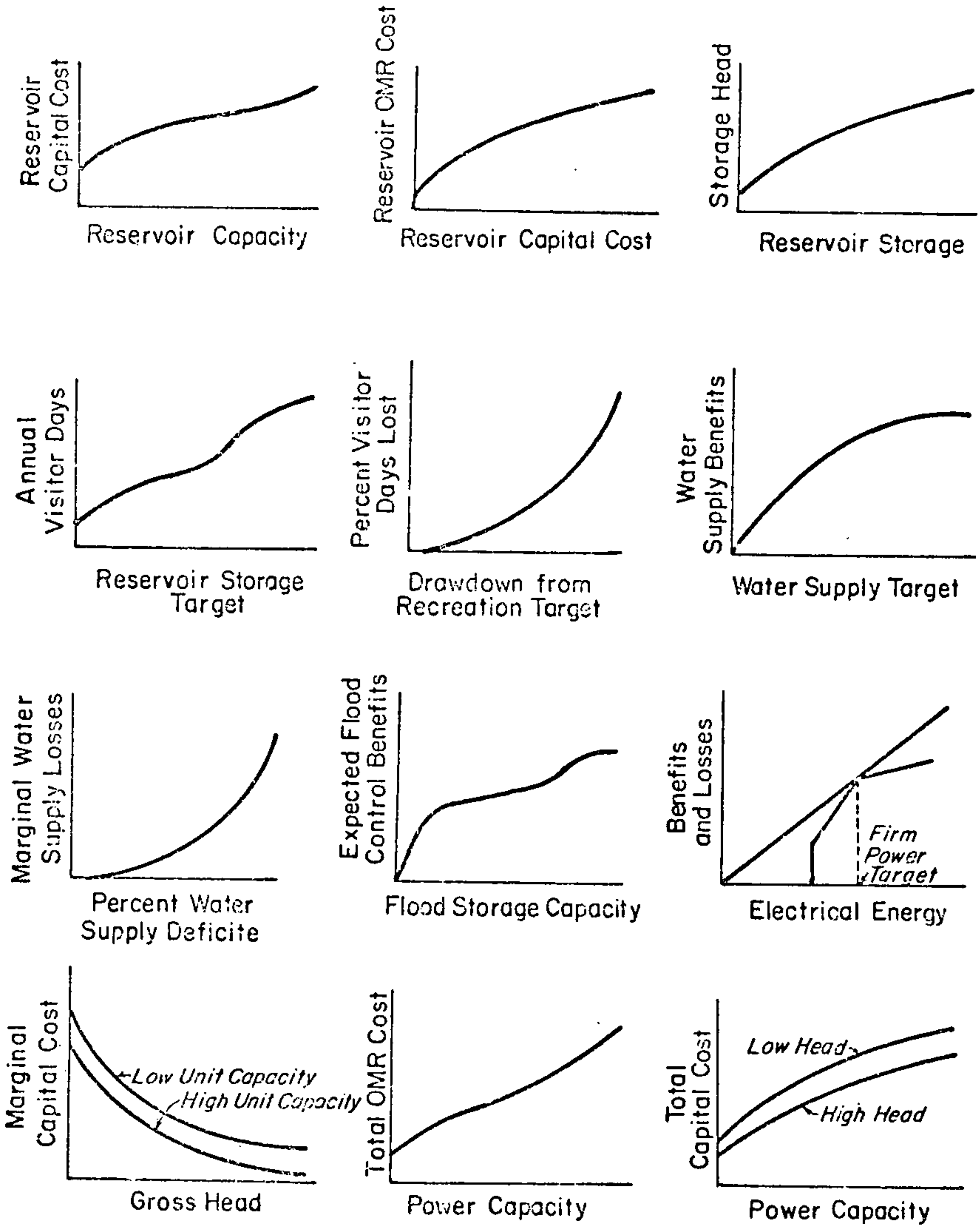
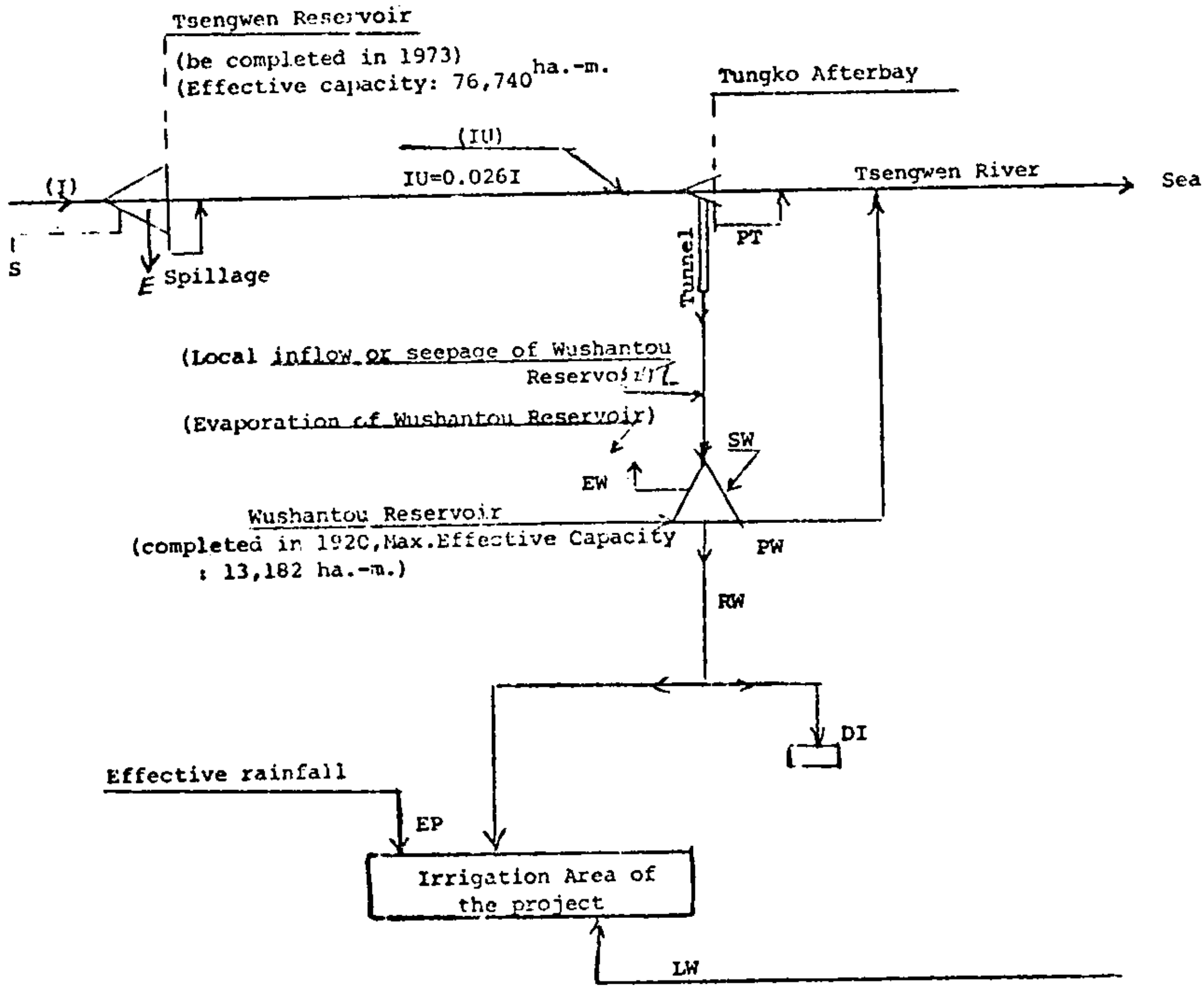


Figure 10.--Typical Functional Relationships for Delaware Basin



I = inflow
R = release
S = storage of reservoir
IU = Uncontrolled local inflow
E = evaporation of reservoir
PT = Spillage at Tungko Afterbay
IW = inflow of Wushantou Reservoir, also the discharge diverted from afterbay Tunnel max. capacity: C=14.424 ha.-m./month)
SW = storage of Wushantou Reservoir
PW = Spillage at Wushantou Reservoir
RW = Release of Wushantou Reservoir
DI = Domestic and Industrial Use
LW = Local water resources in irrigation area

Figure 11.--Schematic of Tsengwen Reservoir - Irrigation Project

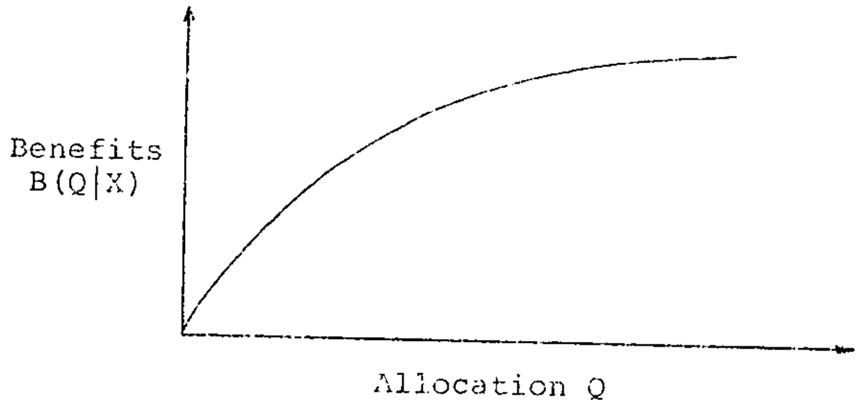


Figure 12. Water - Benefit Function Curve

THE EFFECTS OF HYDROSTATIC PRESSURE ON THE GROWTH OF *CANDIDA ALBICANS* IN A SIMULATED ENVIRONMENT

by

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The survival of microorganisms associated with the marine environment is generally dependent upon the physical and chemical factors which surround the organisms at any given time. Numerous studies have been undertaken concerning the nutritional requirements of terrestrial and marine microorganisms. Temperature and salinity tolerance experiments are also in abundance in the literature, particularly concerning marine bacteria (Wood, 1965). Investigations on the effects of pressure on ocean-borne microorganisms, however, have been relatively few, also limited almost entirely to marine bacteria in spite of the fact that marine yeasts were shown to be a ubiquitous member of the oceanic flora (Fischer, 1894).

Early investigators found that bacteria could tolerate pressures from 600 to 3,000 atmospheres (1 atm = 15 psi) and concluded that hydrostatic pressure did not influence the vertical distribution of these microbes (Certes, 1884; Cattel, 1936). Later studies indicated that many terrestrial and surface water organisms were killed or inhibited by pressure from 200 to 600 atmospheres and, in general, their reproductive rates decreased inversely as a function of pressure. Some bacteria, however, seemed to prefer greater pressure and were described as "obligate barophiles" by the authors (Zobell and Johnson, 1949; Zobell and Oppenheimer, 1950; Morita and Zobell, 1956; Haywood, et al., 1956; Morita, 1957; Morita, et al., 1957; Zobell, 1952).

The significance of the effect of hydrostatic pressure on microorganisms remains an unanswered problem (Sieburth, 1964). Oppenheimer, in a more recent personal communication to Sieburth, indicated that since water compressibility is slight at greater pressures, we can disregard pressure in working with marine bacterial populations. Sieburth (1964) also stated that ". . . from a practical standpoint, important _in situ_ recycling of organic matter must occur in the upper 1,000 meters while the reportedly inhibitory pressures of 200 atm are not reached until depths of 2,000 meters." Wood (1965), on the other hand states, "In the oceans, with depths up to 10,000 meters and consequent pressures of up to 1,000 atmospheres or 15,000 psi, hydrostatic pressure could be of great importance in ecology," and "the microbial flora so far observed in the ocean deeps suggests that the microbial processes with the possible exception of photosynthesis can and do occur at the highest oceanic pressures and the greatest depths."

Of the yeasts and yeast-like fungi which have been found to exist in the estuarine and oceanic environments, _Candida albicans_ is particularly worthy of mention. This yeast-like fungus belonging to the class Deuteromycetes (fungi imperfecti), order Pseudosaccharomycetales, was associated with human disease as early as 1839 (Langenbeck, 1839). Since this time, _C. albicans_ has become one of the most infamous of human pathogens, capable of inciting disease in almost any area of the body in various degrees of severity. This fact led to the publication of an extensive review by Winner and Hurley (1964) which, although admittedly incomplete, contained no less than 2,350 references.

When _C. albicans_ was isolated with alarming frequency from offshore areas, it was assumed that the presence of this microbe was unequivocal evidence of human contamination (Fell, _et al._, 1963b). Further studies, however, indicated that _C. albicans_ could be isolated readily in sub-tropical ocean waters from the surface to depths of over 1,000 meters (Fell, _et al._, 1963a). On this basis, the authors postulated that _C. albicans_ may be a free-living inhabitant of true marine waters, although, classically, there had been no conclusive evidence for its continued existence independent of a host.

Subsequent studies were undertaken in order to ascertain the factors which may play a role in the growth of C. albicans and other fungi in the marine milieu (Dzawachiszwili, et al., 1964; Madri, 1968). Investigations on the effect of sea water on the pathogenicity of these fungi demonstrated that C. albicans as well as Cryptococcus neoformans, another pathogenic yeast-like fungus, retained its infectivity and virulence for laboratory animals after prolonged incubation in sea water (Madri, et al., 1966).

Recent distribution studies on marine yeasts have revealed the presence of C. albicans in great numbers among beds of algae and aquatic weeds, notably Sargassum hemyphyllum (van Uden and Fell, 1968). A later study indicated that in certain estuarine areas where tide and wave action were minimal and the pollution load great, C. albicans could be readily isolated in numbers of 500 to over 20,000 cells per liter (Whalen, et al., 1970).

Although an investigation on the effects of hydrostatic pressure on various marine and terrestrial yeasts (including Candida van rigi) had been undertaken recently (von Baumgarten, 1967), no pressure studies had been attempted on C. albicans. This study, therefore, was designed to reveal the effects of hydrostatic pressure on the growth, morphology and viability of C. albicans in a simulated nutrient marine environment.

Materials and Methods

In general, the methods of von Baumgarten (1967) were used with some modifications.* At the time of this project, the identical pressure device used by the above author and described below was available for experimentation.

A. Organisms

Ten stock strains of C. albicans were used, all of which had been tested for their ability to reproduce in both filter sterilized sea water and in "salinized" Sabouraud's dextrose broth (Sabouraud fluid medium - Oxoid). These ten isolates had been grown and transferred for the previous two years on Sabouraud's dextrose agar slants to which enough sodium chloride had been added to simulate a salinity of approximately 30 ppt. All ten isolates were also tested for their ability to form mycelium, germ tubes and chlamydospores by the plasma method (Kaminsky and Quinlan, 1963).

B. Media

Sabouraud's dextrose broth salinized to 30 ppt was used for the pressure chamber experiments and for the atmosphere controls.

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*In this study Sabouraud's dextrose broth was used rather than GYP medium. Culture fluid to be introduced into the pressure chamber was agitated vigorously to provide for as much aeration as possible. Incubation temperature for this experiment was 25°C rather than 20°C. Yeast cell counts were performed with a mold counting chamber rather than a hemocytometer.

C. Pressure Chamber

The pressure apparatus consisted of a cylinder made from 8-inch diameter bronze marine propeller shaft stock, with a wall thickness of 1.5 inches. Oil from a reservoir bottle was fed through teflon tubing to a shutoff valve which was closed during pumping and open during charging. Pressure was obtained by a manual screw pump (American Instruments Co.). Oil was pumped through a check valve to the chamber and a 15,000 psi gauge. Pressure was lowered by loosening cap bolts until oil could escape past the Bridgman seal between the cylinder and cap. Light mineral oil served as the pressure fluid. Since temperature was an important variable, affecting the organisms through induced pressure changes due to expansion or contraction of the cylinder as well as being a stimulatory or inhibitory influence, all experiments were carried out at 25°C. Pressure tubes consisted of 16 x 100 mm borosilicate glass with a capacity of 14 ml.

In the experiments, the pressure tube with 9.9 ml salinized fluid medium was inoculated with a 0.1 ml suspension of C. albicans corresponding to 1×10^4 cells. A sterile glass bead was added for mixing. Additonal Sabouraud's broth was then added asceptically with a pipette until the tube was full. This quantity of medium was recorded and added to the original dilution of 10.0 ml. A thin sterile polyethylene film was secured over the top by a thin wire in order to prevent contamination.

Atmospheric control tubes were prepared in the same manner as the pressure tubes. Since the design of the pressure chamber necessitated the absence of air, or at least of its further introduction, the culture fluid was agitated previous to inoculation and incubation in order to insure the presence of dissolved oxygen in the system. The control tubes were incubated for 3 days at 25°C. The sets of pressure tubes were incubated as above but also under pressures which ranged from 1,000 to 8,000 psi at each 1,000 psi interval.

After incubation, the pressure was slowly decreased by loosening the cap bolts and the tubes were subsequently cleaned to remove the oil present. Counting was performed immediately thereafter with a modified Howard mold counting chamber (Howard, 1945).

Two separate but similar experiments as described above were undertaken in order to provide more meaningful data.

Results

The results of experiments I and II are summarized in Table 1. The fourth column in each experiment gives the result of the yeasts grown under pressure as percent growth of the atmospheric controls. All of the strains of C. albicans tested demonstrated a linear decrease in cell numbers as the pressure increased. From 1,000 to 2,000 psi, a decrease of approximately 25% was observed. Between 3,000 and 8,000 psi the organisms subjected to these pressures grew in numbers averaging approximately 3.6% that of the controls. Figure 1 illustrates the rapid decrease of C. albicans cell numbers which occurred up to 4,000 psi and the leveling off of this rate of decrease between 4,000 and 8,000 psi. Only minute fluctuations in growth were noted between 4,000 and 8,000 psi although extrapolation indicates that complete inhibition of cell division would probably take place at about 10,000 psi.

Discussion

As had been expected with C. albicans an inverse relationship between cell division (or growth) and pressure was found. The isolates tested demonstrated an approximate 20% decrease in cell numbers for every 1,000 psi increase until the drastic change in the decrease rate occurred in the 4,000 to 5,000 psi range. Above 5,000 psi the decrease rate lowered significantly to approximately 2.5% for every 1,000 psi increase. The reason for this significant change in the decreasing growth rate is unknown.

Relating these experimental results to a simplified scale of growth vs pressure as a function of depth it is evident that C. albicans and probably other Candida species would survive and even reproduce in depths of between 2 to 3 miles, with the greatest inhibitory effect on cell division taking place in the first mile of depth. As was previously mentioned, C. albicans has, in fact, been repeatedly isolated from depths exceeding 1,000 meters (Fell, et al., 1963a).

Under pressure, morphological changes were evident in only a few of the ten strains tested. C. albicans atmospheric controls in some cases produced mycelium, chlamydospores and pseudohyphae. Pressures of 1,000 to 3,000 psi, however, inhibited the formation of such structures, with the exception of the occasional elongation of some yeast cells indicating a tendency toward pseudohyphae or germ tube formation.

In general, yeast cells subjected to higher pressures tended to be rounded rather than ellipsoidal. Size of the cells was similar in both pressure and atmospheric cultures. Visually, budding in the pressure cells seemed to occur in much the same way as it did in the control cells.

The inhibition of the ability of C. albicans to produce mycelium which seems to be due to the increasing hydrostatic pressure may, in this case, explain the linear decrease in growth potential with increasing depth.

Mycelial development may represent an adaptation on the part of the organism to cope more efficiently with its surroundings. Various authors have suggested that filamentation facilitates adsorption of dilute essential nutrients and that hyphal development requires the presence of certain assimilable growth factors (Magni, 1948; McClary, 1952; Skinner, 1960).

If, in fact, this mechanism does occur, i.e., increased pressure, decreased ability to produce mycelium, its significance in the marine environment is of some importance. Since sea water is a source of nutrients in trace amounts and the increased surface area afforded by hyphae may be a deciding factor in the survival of certain fungi, the limiting effect of increased hydrostatic pressure on these organisms may be starvation rather than any other phenomenon. It is interesting to speculate on the effect of pollution enrichment in the depths of the oceans where, if only a more organically concentrated "medium" is needed for growth, and pressure, as such, is not a limiting factor, C. albicans may flourish as a budding yeast where once it could not.

Conclusion

The rate of reproduction of Candida albicans varies inversely with increasing hydrostatic pressure. A rapid linear decrease in cell division occurred between pressures of 1,000 and 4,000 psi. The inability of the organism to produce ancillary structures at higher pressures seems to be related to its decreased survival rate and may be due to a decrease in surface area needed for essential nutrient utilization.

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TABLE 1. DECREASE IN CELL NUMBERS OF C. ALBICANS AT INCREASING PRESSURES

	Experiment I			Experiment II			
	Number of Yeast Cells Counted			Number of Yeast Cells Counted			
PSI	Pressure*	Control*	%	Pressure*	Control*	%	Average
8,000	8.3	450	1.85	5.5	416	1.32	1.57
7,000	2.3	159	2.01	15.4	184	8.4	7.1
6,000	38.0	265	1.43	6.6	475	7.2	4.3
5,000	30.0	574	5.2	16.3	630	2.6	3.9
4,000	15.5	550	2.82	18.5	630	2.94	3.0
3,000	7.9	73	10.1	41.2	141	29.5	1.98
2,000	27.6	350	39.4	19.3	178	57.0	48.0
1,000	172.0	185	92.7	89.5	154	58.0	75.0

*Multiply number by factor of 10^4

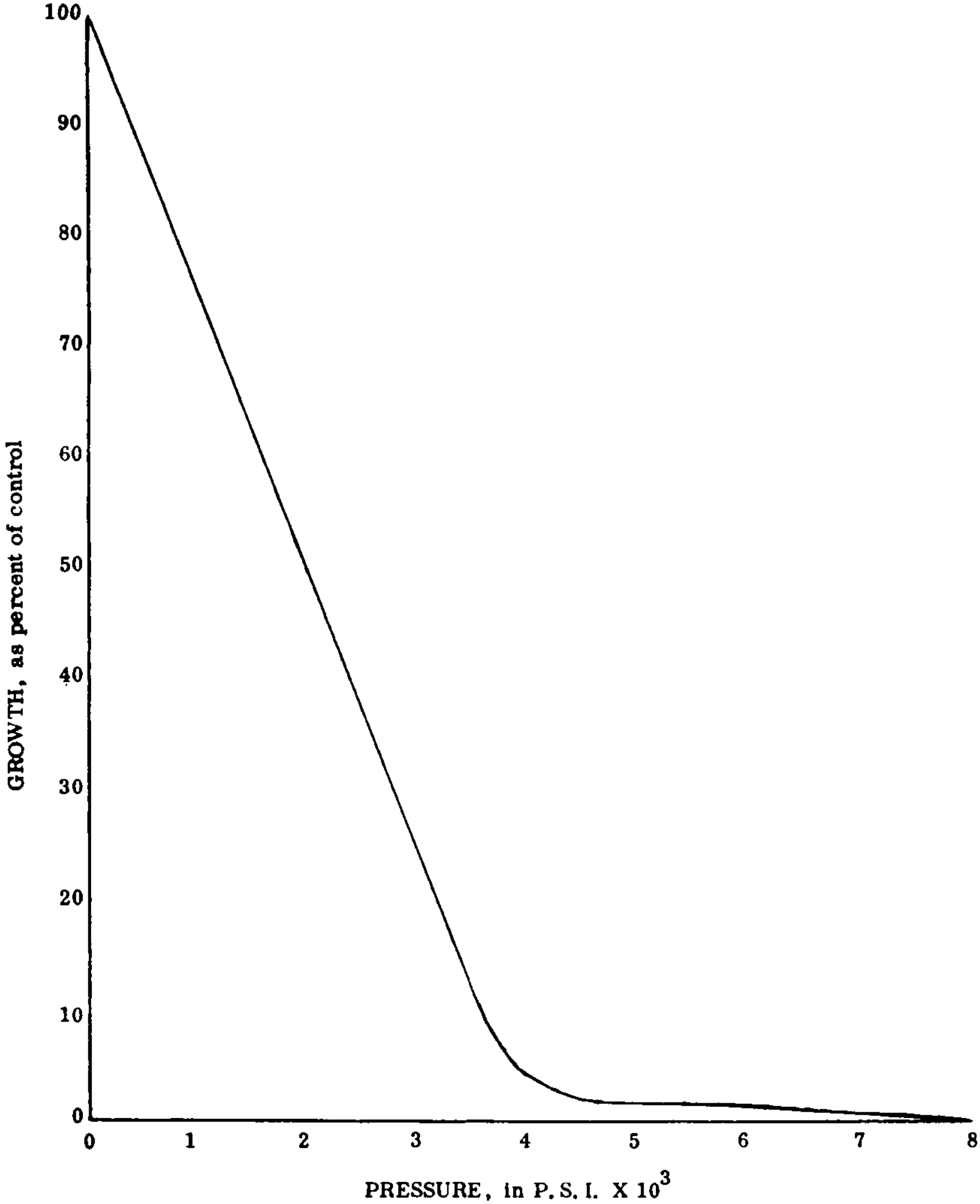

Figure 1. Effects of Pressure on Cell Numbers of C. albicans

HYDROGEOLOGICAL ASPECTS OF POLLUTION AND WATER RESOURCES IN URBANIZED AND INDUSTRIAL AREAS

by

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The securing of necessary water resources for our growing need and the controlling of water pollution are among the most serious problems facing our society and are subjects of national concern all over the world. The escalation of water demand and of water pollution resulting from expanding population, urbanization and industrialization has resulted in an enormous waste of our clean water supply. Development in the coming decades, combined with continued misuse of our water resources, will cause irreparable damage to our environment unless the securing of drinking and industrial water is regulated on a larger scale. We must realize the desperate urgent need to institute and expand environmental control, not as a matter of aesthetics or romantic nostalgia for the past, but as a condition for civilized survival.

Appalling mismanagement and lack of adequate research activities and regulation in securing water resources and regulation of water pollution have been causing health hazards, not only in great metropolitan areas like London, New York, Paris, or Tokyo, but also in highly-developed industrial regions, such as the Rhine valley with the Ruhr district, the Ligurian shore in Italy, the Great Lakes in the United States, or the Lake Baykal area around Irkutsk in the Soviet Union.

The primary task is to secure the available water resources, which means not only taking an inventory of facilities currently being used and of their modernization, but also reviewing possible ways to increase the quantity of clean water by further exploration for such resources as are near to the place of consumption. Water on Earth is abundantly present and is estimated to be 1,357,506,000 km^3, of which 97.2% is salt water, 2.15% is frozen and 0.65% is fresh water.[1] If we could place this fresh water on the nonfrozen land area of the Earth, it would reach a height of 73.16 m (8,506,000 km^3). Of this, the water of lakes, rivers and streams would account for only 1.08 m in height (126,250 km^3). On the other hand, 97.54% (8,300,000 km^3) of the fresh water is ground water (71.40 m in height) and half of this amount is available at a depth of less than 1 km under the surface. Of the annual water cycle of 422,000 km^3 of water, about 100,000 km^3 falls on land and 30% of this feeds plants by infiltration, supplies ground water, or runs off directly to the seas; the other 50% falls on frozen land or evaporates, and only 20% of this precipitation is controllable. Unfortunately, at present, no more than 1.3% (1,326 km^3) is controlled and used for consumption.

It is apparent that if we wish to secure the desperately needed pure water, the most economic way is to intensify the control and use of ground water. In this field, more research and more surveying--conducted by the hydrologist--is needed. The correlation between immense reservoirs (oceans), precipitation, runoff, evaporation, transpiration and infiltration together with percolation has been a subject of interest to many hydrologists over the past centuries. Until recently, the relatively new science of hydrology did not enjoy sufficiently large growth rate to cope with the problems created by a "rational" exploitation of the world's water resources. The Decade of International Hydrology sponsored by UNESCO is the first such impetus on an international scale. In the past, the computing of runoff quantities had little concern about the geological aspects, and the porosity of underlain rock formations was almost disregarded; and the different empirical runoff formulas, such as those of Iszkowski, Lauterburg, Specht, Hofbauer, Herbst, Pascher, Kressnik, Cramer, Huber, Tolkmitt, Dub, Kreps, Wundt, Dickens, Chamier, Craig, Murphy, Sherman, and Poljakov, were based on rainfall intensity and quantity, watershed area and form, vegetation and slope of terrain, elevation above sea level and yearly average precipitation, rather than on geological conditions and the permeability of underlain rock formations.[2]

Researches conducted in the past decades concerned maximum flow in watersheds with areas 300 km^2 or less, where the geological conditions, topographic characteristics and rock formations permit an evaluation of peak rates of runoff from smaller watersheds. The aim of the research conducted by the author in 1963-1966 was to suggest such equations for maximum runoff as could be used generally, not only for the chosen area, but anywhere the climactic and physiographic characteristics are the same.

Analogical aims were followed by the United States Department of Commerce, Bureau of Public Roads, in establishing runoff formulas for smaller watersheds.[3] This study divides the entire United States into only four zones, corresponding to the four main geological regions of North America. Similar investigations were conducted by Ogievskij[4] and Sokolovskij[5] in working out equations to compute peak floods for drainage areas in accordance with the latitude and regions of the Soviet Union, but generally did not take into account the geological conditions. Finally, Dub in Czechoslovakia attempted to work out formulas based on regional conditions.[6] In all cases, the formulas are rather more arbitrary than adapted to the physiographic regions and embrace too large territories without analyzing the local characteristics. In classifying the watersheds on such a big scale, the peak rate runoff of small areas cannot be expressed properly by these formulas because they become too generalized; they are not governed by the local hydrological conditions which correspond to the size of the small watershed.

It seems that research concerning the peak flow of smaller watersheds has very little in common with securing pure water for drinking or industrial purposes. On the contrary, these researches throw considerable light on the correlation of surface runoff, geological subsurface and ground water storage capacity. Besides this correlation, the most valuable water resources are those of smaller watersheds where treatment of water has very few problems since they are in general in protected, unpolluted areas, especially in sparsely populated areas. We should bear in mind that these areas can be used efficiently almost only for water supply systems of smaller communities and where group or regional water supply systems have limited possibilities, mainly for economic reasons. A further advantage gained from studying the geological influence on peak rate flow is the knowledge which can be, and is, gained concerning permeability of underlain rock formations and a better understanding of groundwater phenomena and their characteristics in percolation and storage. Detailed surveys and a better knowledge of these aspects are essential because the availability and quantity of ground water will govern any further development in urbanization or industrialization. On the other hand, we must realize that costly saline water conversions will be applied only in areas where not enough water is secured from the ground water available. Naturally, treatment of waste and of drinking water is necessary to avoid pollution of our water resources.

The correlation between the peak rate flow and the geological subsurface in smaller watersheds, based on research mentioned before, can be put in the following equation:[7]

$$q = \frac{c}{A^{0.4}} \quad \text{where}$$

q = peak rate of runoff ($m^3/sec/km^2$),
A = area of watershed (km^2),
c = coefficient depending on geological subsurface.

In accordance with these findings, the value of coefficient "c" showed a very close relation to the character of the geological formation of the watershed. The highest values were found at the Paleozoic formations, where the permeability is comparatively low because the rock formations are more lithified. The Mesozoic formations are better aquifers and, therefore their coefficient has a smaller value. The Tertiary formations are distinguished by an even smaller coefficient, and the Quaternary formations have the smallest one. Exceptions from this rule are early Tertiary formations which have an extreme aquiclude characteristic, and the highest runoff values occur in this region.

The values of "c" coefficient are as follows:

17.5-18.5	Tertiary: Paleogene formations such as Flysch.
14.0	Schist and shale, or Tertiary Paleogene formations mixed with Mesozoic zones.
8.0-12.00	Paleozoic, Proterozoic formations: Cambrian, Precambrian zones, crystalline complexes (granite, gneiss, micaceous schist, melaphyry, diabase, porphyry, prophyrite, trachyte, basalt, andesite).
6.0	Mesozoic (Cretaceous) - Limestone formations, tuffstone group (rhyolite, trachyte, dacite) and wrappings of crystalline complexes: Limestones and dolomites.
2.5	Tertiary: Neogene zones and glacial formations. Zone of the hilly country adjacent to Quaternary formations.
1.9	Quaternary formations - mainly of diluvial origin.
1.0	Quaternary formations - alluvial zones, fresh-water deposits or flood plains.

The formulas and coefficients of the peak rates of runoff from smaller watersheds were compiled from 110 cases collected and evaluated in Czechoslovakia. The Austrian hydrologist, Paplham, from Linz, proved the applicability of the formulas and coefficients for Austria also in 1968. By 1970, further data were collected from Austria, Federal Republic of Germany, and Switzerland which enlarge the applicability of and refine the described coefficients and equations. Besides the rainfall intensity and duration, the geological conditions are the only factors which substantially influence the values of peak rate of runoffs from smaller watershed areas. The vegetative cover and the shape of the watershed have only a $\pm 5\%$ effect on maximum flood. The values of peak runoff based on the developed formulas are apparent from the attached chart (Fig. 1).

It is obvious that the rate of surface runoff must be related in an inverse way to the permeability of the geological subsurface of the watershed, and the quality and quantity of ground water storage are directly dependent on these conditions. Therefore, it is interesting to compare the value of the hydrogeological coefficients in the developed peak runoff formulas with the permeability values of the corresponding geological formations. The comparison can be based only on average values of permeability measured at different conditions of the geological formations which are commensurate to formations used in establishing runoff formula coefficients. It must be pointed out that the various formations are in general already mixed or interwoven also in smaller drainage area. The existing cracks of the terrain, the uneven surface weathering, the disintegrated underlain rock formations at various depths and the possible present faults are of additional hardship to establish a practical average value of permeability even for a smaller watershed. Therefore, additional research is essential to obtain more reliable data.

Regulation of water pollution must be conducted by the civil engineer from this viewpoint too, especially, when toxic water pollutants are present, because geological formations with a higher value of permeability conduct the polluted water farther, not only preventing proper pollution control but also contaminating to a great extent the valuable ground water resources.

Near industrial areas, or at sea level as is the case for example in the New York City area, the demand for drinking water is satisfied by pumping ground water in such quantities that the ground water level is declining and salt water is intruding in the same area, and thus contaminates the ground water.[10] If ground water discharge exceeds recharge, it causes a net decrease in the amount of fresh ground water in storage. The decrease in storage was evidenced by declining ground water levels as far back as 1903 when the ground water table was recorded for the first time in the New York City area. The conditions were aggravated not only by intrusion of salt water from the sea (yearly 100 m since 1952 in a depth of 200 m the so-called "deep salty-water wedge" in an area of Jamaica Bay), but also by infiltration of contaminated waste water resulting from inadequate or deteriorated domestic or public sewage treatments, especially in the "garden city" type dwellings, region (e.g. on Long Island, with over ten million inhabitants). Dumping the sewage water into the sea without treatment does not solve the problem because, besides the contamination of estuaries by unnecessary discharge, an artificial vacuum has been created in the groundwater storage which has been and still is filled gradually by salt-water. The only solution for big metropolitan seashore areas, which are securing their water from groundwater storage is to keep a balance by taking less water than supplied by the recharge (precipitation and other resources) and to have a very efficient waste treatment system in order to secure recharge of the water resource system with treated waste water, too.

Our affluent society and the sharp increase in world population demand more water resources and more efficient water pollution control.

Water, the very symbol of life itself, has been quite often closely associated with suffering and death. Not only too much water, like floods, or too little, like droughts, have killed many millions; but the quality of drinking water or the contaminated inefficiently treated waste water have been having devastating effects on mankind. Waterborne diseases, such as cholera, dysentery, hepatitis, and typhoid fever, have played a significant role in population control. To enrich our knowledge concerning our most valuable water resources, the ground water storage and its hydrogeological potential in quality as in contamination, in discharge as in recharge, in survey as in its exploitation, are of utmost importance, and are a duty in the fight for civilized survival. As the French physiologist, Claude Bernard called it, homo sapiens keeps, in his "milieu interieur," the signs of his great ancestor, the water of the sea. In our beginning there was water. We should not mismanage it, because without it we are finished.

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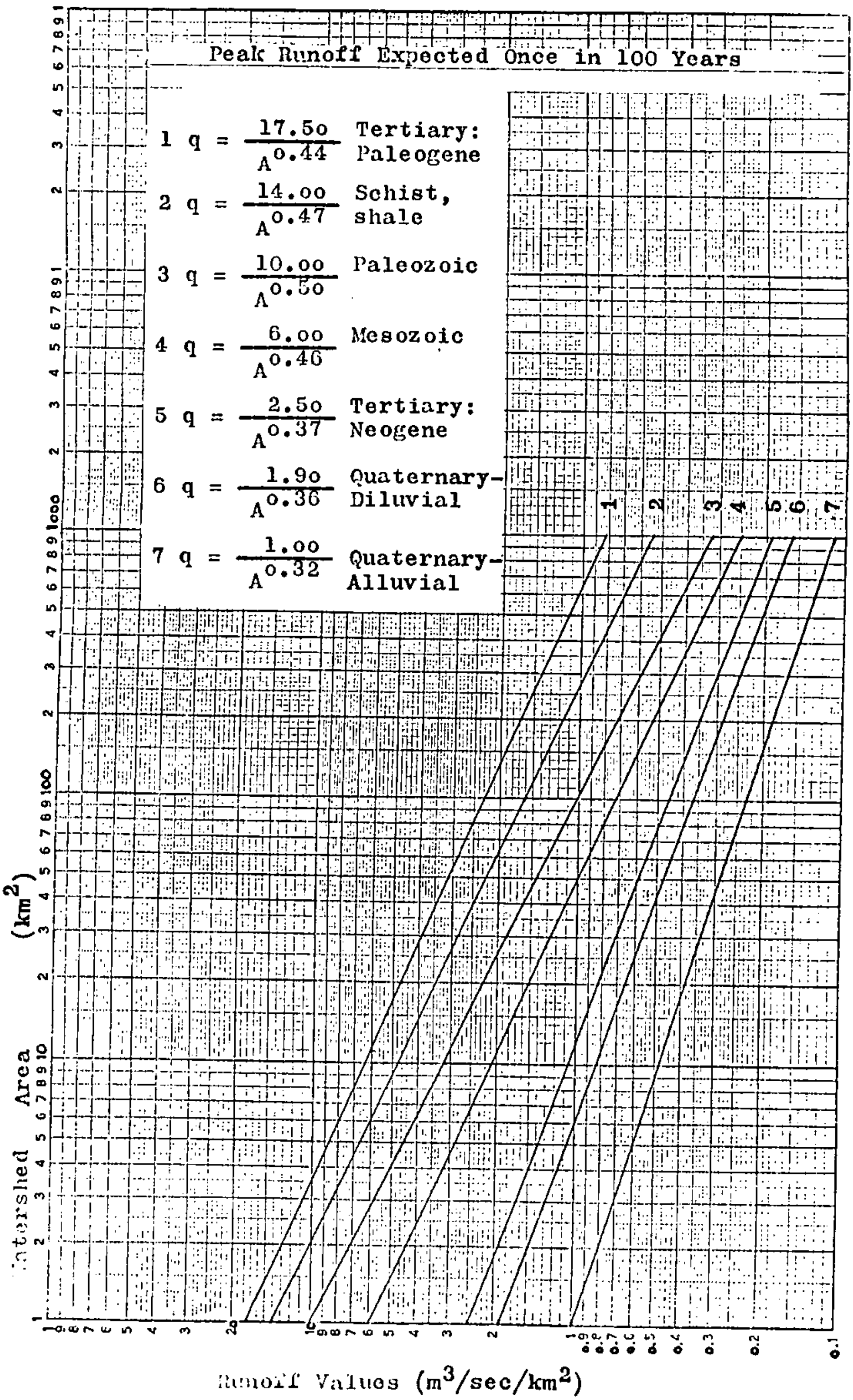

Peak Runoff Expected Once in 100 Years
1 $q = \frac{17.50}{A^{0.44}}$ Tertiary: Paleogene
2 $q = \frac{14.00}{A^{0.47}}$ Schist, shale
3 $q = \frac{10.00}{A^{0.50}}$ Paleozoic
4 $q = \frac{6.00}{A^{0.46}}$ Mesozoic
5 $q = \frac{2.50}{A^{0.37}}$ Tertiary: Neogene
6 $q = \frac{1.90}{A^{0.36}}$ Quaternary-Diluvial
7 $q = \frac{1.00}{A^{0.32}}$ Quaternary-Alluvial
Watershed Area (km^2)
Runoff Values ($m^3/sec/km^2$)

Figure 1

Geological formations	Runoff coefficient	Average millidarcys	Permeability[8-9] in millimeinzers
Tertiary: Paleogene (Flysch)	18.2	1.0	1.0 x 18.2
Schist, shale	14.0	1.9^2	1.9^2 x 18.2
Paleozoic (Precambrian, crystalline)	10.0	2.5^2	2.5^2 x 18.2
Mesozoic	6.0	6.0^2	6.0^2 x 18.2
Tertiary: Neogene	2.5	10.0^2	10.0^2 x 18.2
Quaternary - diluvial	1.9	14.0^2	14.0^2 x 18.2
Quaternary - alluvial	1.0	18.2^2	18.2^2 x 18.2

Figure 2.--Comparison of runoff coefficients with the permeability of various geological formations

APPENDIX

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