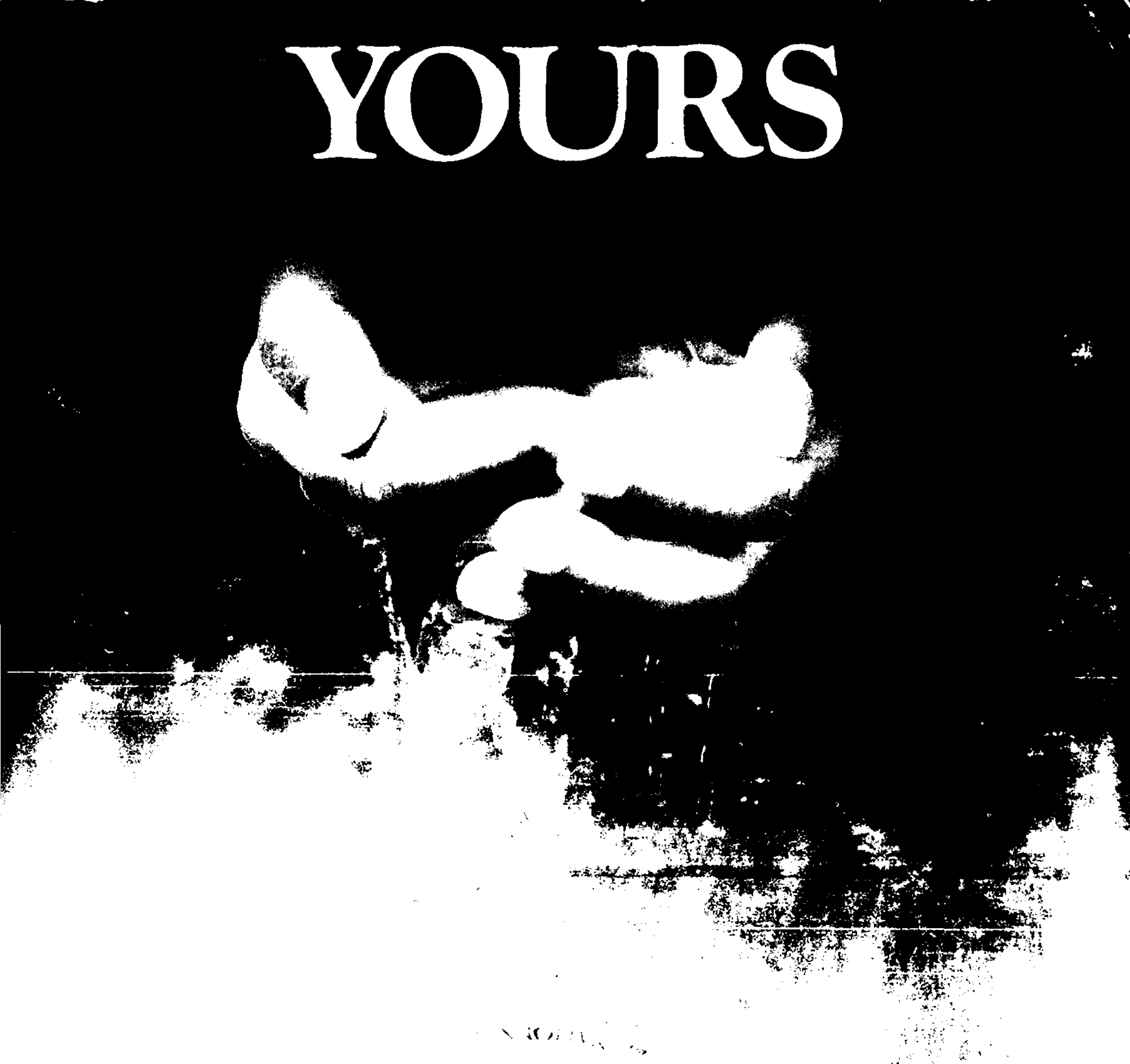

PROCEEDINGS

Vth WORLD CONGRESS ON WATER RESOURCES BRUSSELS 1985

VOLUME 1

WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

PROCEEDINGS

VOLUME 1

of the

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9-15 JUNE 1985 Brussels Belgium

organized by the

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FOREWORD

The Conference Committee has great pleasure in presenting herewith Volumes 1 and 2 of the PROCEEDINGS of the Vth World Congress on Water Resources. These Volumes 1 and 2 form the Pre-Congress part of the PROCEEDINGS.

There was a tremendous response of some 280 proposed papers to our first call. To start with, Abstracts were screened and classified by the Scientific Committee and the International and Belgian Rapporteurs. The next step was inviting selected authors to submit the full papers for final reviewing. On the basis of the accepted papers the initial Congress Programme was established in December 1984.

Initially, to ensure the smooth and organized running of the Sessions, the Conference Committee had the intention of printing prior to the Congress the full text of all the papers. In the Formal Sessions the papers were to be taken as read, by having been circulated before the Congress to all delegates who ordened a copy, and were to be highlighted by the authors in a short presentation. This is usually the best way to have well prepared and interesting discussions. The Conference Committee was however forced to proceed differently, as explained hereafter.

More than 200 papers have been accepted for the Congress, 83% of them for Formal Sessions. At an average of 10 pages per paper this would have given three Pre-Congress Volumes containing in all more than 1800 pages. However, as 6 weeks before the Congress only half of the authors could or would confirm their effective participation by paying their registration fee, the Conference Committee had to resort to circulating free of charge to all registrated delegates a Volume containing the ABSTRACTS of all accepted papers, and print for the opening of the Congress the full text of the only papers confirmed by the authors. All the other communications and contributions will be published in the Post-Congress Volumes of the PROCEEDINGS, provided they were effectively presented at the Congress by the authors.

However, this slight technical hitch will in no way impair the exceptional interest and value of our Congress, where scientists and technicians, educators and administrators, economists and managers, social, health and legal experts, politicians and philosophers from some 70 countries gather in an interdisciplinary effort for the advancement of the proper development of one of our precious resources: **WATER** wich is **YOURS**.

Victor de Kosinsky

The Scientific Organiser

PRÉFACE

Le Comité de Conférence a le grand plaisir de publier, en préparation du Congrès, les Volumes 1 et 2 des Comptes Rendus du V[e] Congrès Mondial des Ressources en Eau. Les Comptes Rendus porteront le titre anglais «PROCEEDINGS».

Notre premier appel avait suscité un immense intérêt et nous avons reçu quelque 280 offres de communications pour le Congrès. Pour commencer, ces communications, proposées sous forme de résumés, ont été triées et classées par le Comité Scientifique et par les Rapporteurs Internationaux et Belges. Ensuite, les auteurs sélectionnés ont été invités à soumettre leur manuscrits complets pour une appréciation ultime. Sur la base des communications retenues, un Programme a été établi en Décembre 1984.

Dans l'organisation initiale il avait été prévu d'imprimer le texte complet de toutes les communications avant le Congrès. Ainsi les participants auraient pu acheter ces volumes et prendre connaissance des communications à l'avance, pendant les séances les auteurs se bornant à mettre en exergue l'intérêt essentiel et culminant de leurs travaux. Cette procédure mène en général à des discussions bien préparées et fructueuses.

Quelque 200 communications ont été retenues pour le Congrès, dont 83% pour les Sessions Formelles. Comme la longueur des manuscrits a été volontairement limitée à 10 pages, il a été envisagé d'imprimer trois volumes des «PROCEEDINGS», contenant plus de 1800 pages en tout. Toutefois, avant d'imprimer les articles, nous avons demandé aux auteurs de confirmer leur participation effective au Congrès. Etant donné qu'à 6 semaines avant l'ouverture du Congrès seulement la moitié des auteurs ont répondu et réglé leur inscription, le Comité de Conférence a décidé d'envoyer gratuitement à tout participant régulièrement inscrit un Volume contenant les résumés, en anglais «ABSTRACTS», de toutes les communications acceptées, et d'imprimer pour l'ouverture du Congrès, seulement les articles complets dont les auteurs ont confirmé leur participation au Congrès. Toutes les autres communications et contributions présentées au Congrès seront publiées dans les Volumes des Comptes Rendus, imprimés après le Congrès.

Toutefois, ce petit ennui technique ne va nullement influencer la valeur et l'intérêt exceptionnels de notre Congrès, où réuniront scientifiques et techniciens, éducateurs et administrateurs, économistes et gestionnaires, experts en sociologie, en santé publique et en législation, hommes politiques et philosophes, venant de quelque 70 pays pour mener un effort interdisciplinaire dont le but est d'avancer le développement approprié d'une des plus précieuses de nos ressources: l'**EAU** qui est à **VOUS** tous.

Victor de Kiriaky

L'Organisateur Scientifique

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9 - 15 JUNE 1985 BRUSSELS BELGIUM
WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 11 Aspect number 8

MAXIMIZING USE OF SURFACE AND UNDERGROUND WATER RESOURCES IN RURAL AREAS - LEGAL CASE HISTORIES

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ABSTRACT

Inadequate water supply, because of poor quality or uncertain quantity, is but one consideration which causes migration from rural to urban areas in underdeveloped countries. The provision of an adequate water supply of good quality not only helps maintain rural communities, but affords potential for economic stability and development.

This paper will recount the experience of three western states of the United States in encouraging the development of adequate water supplies in order to discourage migration to metropolitan areas, while also creating a favorable climate for economic development in rural areas.

All three of these states, Colorado, New Mexico and Wyoming, follow the "no injury" doctrine in the development of water supplies. This paper will outline the methodology for maximizing water supplies in arid areas by joint use of tributary and non-tributary water and will explain the "multiplier" potential of such joint use by the use of specific examples.

Finally, there will be a discussion of the role of central governments in developing the guidelines for insuring adequate water supplies in rural areas.

INTRODUCTION

The migration of people from rural areas to major cities in an effort to improve their standard of living creates serious problems in all parts of the world. Cairo, Lima and Mexico City are just three of many examples of the movement of the poor, the undernourished, the dissatisfied, to major population centers in what is generally a fruitless effort to improve their standard of living.

The conditions under which these migrants must exist after their move to the metropolitan centers must be worse than the places they left. However, once the move is made people generally have neither the finances nor the inclination to return.

One way to help stem this migration is to improve the standard of living in rural areas. A great problem in most rural areas is the lack of a dependable water supply of adequate quality.

This paper will address attempted solutions to this problem in three arid western states of the United States - Wyoming, Colorado and New Mexico.

The mean annual precipitation in each of these states is less than 14 inches. No substantial amount of agricultural economy can survive without irrigation except in the eastern portions of the three states where the precipitation is barely sufficient for dry land farming.

Stabilization of rural economies in an environment which is becoming more and more sophisticated and in which there is a larger and larger disparity between the haves and have-nots is an important deterrent to the in-migration of people from the rural poverty level to the larger population centers in the hope of improving their lot.

Frequently, natural resources are found in isolated areas but unless adequate water supplies, both for the production of the minerals and the needs of the workmen, can be obtained, the development of these resources is either delayed, minimized or abandoned.

The central government of the United States and the government of each of the three states alluded to in this paper have recognized the desirability of assisting in the establishment of water supplies of adequate quality and quantity to ensure the continued occupation of the rural areas and to prevent the compounding of problems caused by excessive migration to population centers.

In addition to pointing out an example in each of the three states where the use of underground water has led to the stabilization of a rural economy, this paper will also address the question of maximization of the life of non-tributary ground water by the combination of the use of tributary and non-tributary wells.

Finally, there will be a discussion of the role of central governments in developing the guidelines for ensuring water supplies in rural areas with examples of various forms of local institutional organizations available for use in cooperation with central governments.

WYOMING

For the Wyoming example, we have selected the City of Gillette, which is a community in northeastern Wyoming booming with the introduction of coal strip mining on a large scale.

With oil exploration, Gillette's population moved from 3,580 in 1960 to 7,194 in 1970. The development of a number of major strip coal mines in the area has increased the mid-70's population of about 11,500 persons to an estimated 17,700 in 1983. The City has also pursued and is presently pursuing an aggressive policy of annexation of surrounding populated, unincorporated areas, further increasing the population for which the City supplies its various services including water and sewer. The City is expected to have a population in excess of 22,000 by 1990.

The City has owned and operated its own water system for over 90 years. Water has always been in short supply in northeastern Wyoming but, until recently, a series of wells drilled within the City limits provided adequate potable water (after treatment) to supply the needs of the City and its inhabitants.

With the discovery, development and exploitation of major energy reserves of oil, gas and coal in the region, the City of Gillette has assumed new importance as the major mining, commercial and retail center in Campbell County. One result of this importance has been increased population, and the population increases are projected to continue over the next decade. As a result, in 1979, the City began acquisition and construction of a major water project to permit the City to obtain necessary water supplied from the so-called Madison Formation, a limestone basin containing a substantial aquifer northeast of the City. The project, completed in 1980, provides the City with a good supply of potable fresh water and has permitted the City to supply some of the water needs of the nearby communities of Moorcroft and Rozet as well as permitting the City to supply water to developed areas which either have recently been or will in the near future be annexed into the City. The Madison Water Project actually began supplying water to the City in May of 1981. Prior to this Project coming "on line", the City had imposed strict water restrictions, particularly during the summer. The addition of the Madison Water Project to the Water System resulted in and should continue to result in significantly increased water revenues because of the elimination of this strict water rationing.

The present Water System of the City consists of approximately 99.5 miles of water distribution lines, 32 miles of which consists of the main transmission lines to the Madison wells which are located some 32 miles to the north and east of the City. Seventy percent of the total distribution lines have been constructed since 1971. Since the City obtains its water from wells, including the Madison Formation as well as the Fox Hills Formation and Fort Union Formation wells located in or near the City, the only water treatment required for human consumption is chlorination. The combined capacity of all of the Madison wells is 3,000,000 gallons per day and the combined capacity of the City's other wells is 1,750,000 gallons per day, which provides sufficient water for a population of approximately 19,000 to 20,000 people. As additional requirements for water materialize, the City will drill additional wells in the Madison Formation under valid ground water permits which the City presently holds. This water will be carried in the existing Madison pipeline which has a capacity of 10,000,000 gallons per day which would provide enough water for a population of 35,000 people.

Funding for the Madison Water Project was as follows:

$ 500,000.00	Mineral Royalty Grant (State)
4,500,000.00	Farmers Home Administration Loan (Federal)
15,000,000.00	State Legislature Loan
3,000,000.00	Farmers Home Administration Grant (Federal)
252,477.00	Town of Moorcroft
104,954.15	Interest on Investments

This is a classic example of funding from multiple sources to insure an adequate water supply for an energy impacted area.

COLORADO

Starkville, Colorado is a coal mining town in the extreme southeastern portion of the state where the coal mines have become exhausted and there is little work remaining. In 1976, many of the residents of the town were people on government assistance programs. There was a steady diminution in population caused by people leaving to find work elsewhere. There was no central water system and people depended on open ditches supplying water of poor quality. The residents of Starkville were concerned that within a short time the town would be totally abandoned and the population would move to the nearby community of Trinidad where they would become not only public charges as far as their subsistence was concerned, but would add to already crowded conditions.

The residents of Starkville cultivated small gardens and many of them commuted to work in Trinidad and returned to their homes in the evening.

The Town applied to the Colorado Water Conservation Board for a low-cost loan to construct a central water system for the Town of Starkville.

The Water Board, acting under the provisions of a 1971 law aimed at making water available to citizens of the state, approved the application and granted a loan of $300,000.00 at 1% interest to construct such a system.

The system was built and as a result, the population and the economy of Starkville has been stabilized and the migration of residents to nearby communities has ceased.

Not only has the population been stabilized, but the superior quality of the water obtained through the central system has improved the health of the population of Starkville.

NEW MEXICO

The State of New Mexico has a number of statutes which established programs to assist in the construction of rural or publically-owned water systems, including energy-impacted areas, and providing funds for such construction.

The first one of these was what is commonly known as "the Little Water Bill" adopted in 1947 with an initial appropriation of only $40,000.

As Caldwell expresses it:

> The present popular conception of New Mexico has to a great extent been influenced by the A-bomb development and the resulting changes in the mode of living. Another influence, however, that has been changing the pattern of living in rural New Mexico as much as the famed AEC development, although on a much quieter and smaller scale, is a program carried on by the Department of Public Health to provide safe water supplies in rural unincorporated communities.
>
> There are many of these small communities, especially in the mountain areas, inhabited for the most part by people of a culture and civilization which is older than our own. Noted for their courteous hospitality, their arts and their handicrafts, they are a quiet people who have depended primarily on agriculture and animal husbandry for their livelihood. Mining developments, however, are helping many of these people to work for wages on a stable, year-around basis.
>
> In many of these communities, there are periods during dry years when water is almost non-existent, and even a pot hole in a stream bed must be used as a source of domestic supply. Often the supply is an irrigation or drainage ditch, the water of which have been used in the fields where, as it sluggishly spread, it became contaminated and unfit for human consumption, and then returning to the ditch system it meandered on to the next user. Each drop was precious, to be used sparingly. A child carried what he could home to his waiting mother, in a small lard bucket or other pail, while his dog lapped at the open water as the pail was being filled, its

> contents muddy, contaminated, often odorous. Often the mother and children would have to walk great distances to bring home any appreciable quantity of water, or the busy husband would harness his team to the wagon, load the wagon with empty barrels, laboriously fill the barrels and drive back home. Small springs or seepage areas yielded limited quantities in some areas, enough to permit existence but rarely providing a surplus.
>
> Depressions were dug to hold the water, only too often muddy, unclean and wallowed in by the cows, horses and dogs of the community; nevertheless, it was all that was available. It was used, as in all homes, for cooking, cleaning, washing, bathing purposes, and for drinking. The household tasks were always difficult because of the never-ending lack of a plentiful water supply readily available. Seldom was there time to boil the water as they had been taught to do, and why bother anyway -- the same source of supply had been used by their parents, and their parents' parents. Those who had passed on while still young -- well, that was the way of things. Some lived, some died, and who was to say why?
>
> Not realizing the true part played by water supplies in the spread of disease, they continued to use contaminated water, thus increasing their chances of joining those who had already departed. Babies died in their first year, older children and adults might resist several times only to succumb or possibly develop immunities. There was no other way, no other supply -- or at least that was the thinking twelve years ago.

This same writer tells the story of the hamlet of Los Apodacas:

> In the villages to be affected, the people did not know where to begin, what steps to take first. Where was the Corporation Commission? Would their lands be taxed, or would they owe money if they took advantage of this tempting gift from the State. These were some of the questions people needed to have answered before embracing this new idea. For a long time the money lay idle with no one asking for a project.
>
> At last the Reverend Leopoldo Martinez of Los Apodacas decided that if he could just help his community get started, they could have that which they had never known existed for them -- safe water.

So the first Mutual Domestic Water Consumers Association was formed, duly incorporated under the laws of the State. It was, of course, necessary for these people to organize the members of the village into a working group with working officers. They then came to Santa Fe, the Capitol, to talk about their situation with the Health Department, the people who wanted to help them. There they signed papers agreeing to do their share of the work and asking for a grant-in-aid from the State; thence to the Corporation Commission to draw up the documents which would fully establish their association on a legal basis; from there they went to the County Clerk's office to record their corporation papers, deeds and easements. It was all very worrisome and serious, much money was involved, and maybe there were things they did not understand. Human nature prevailed, however, and mutual confidence and respect sided in solving what could have been major misunderstanding.

After completion of the paperwork, engineers visited the community and selected a site for a drilled well, a storage tank, and the location of the supply line. Plans were drawn, reviewed, and checked, and then sent to the State Purchasing Agent that he might call for bids for actual construction. Excitement reached a high pitch in the community when the well rig moved in and drilling started. Little children, old men, young men, ladies with rebozos over their heads and those without, all watched this big machine as the cable rolled up on the drum pulling up the drill, only to drop it with a thud and loosen up more soil, gradually deepening the hole. Never did a driller have a more attentive audience, and never probably was there more discussion about a well, as the drill pounded deeper and deeper into the crust of the earth.

With the well drilled and cased, and an ample supply of water assured, the villagers assumed their part of the work building a pump house, constructing a foundation for a storage tank and digging a trench for the line to connect the well to the tank. All labor and local materials needed must be furnished by the association.

The State will only assist in developing a basic project: the well and its equipment, a covered metal storage tank, the line connecting the two, and a supply line to the center or centers of population equipped with community hydrants as needed and as the legal and financial limitations of the law permit. Development of the distribution system and appurtenances such as crosses, tees, fire hydrants, yard taps, extra valves and house connections are encouraged, but at the expense of the association and its members. Once the mysterious pump begins its function of pulling the water up and then pushing it into the storage tank, there is another burst of enthusiasm as individual members hurriedly dig trenches to their homes for a sink or yard hydrant. The contractor does a land office business in small fittings and pipe, and his advice, tools, and help are in great demand.

This first project, serving twenty-three families, cost $4,600. The association supplied all of the adobes, vigas, sand, gravel, and other miscellaneous material which could be obtained locally, and excavated and backfilled the trenches for the supply line. They also mixed and poured concrete as well as performing the carpenter work needed, and secured the necessary easements and deeds for the pipeline and for acquiring the land for the well and pumphouse.

* * *

All of the families now have water in their homes or in their back yards. When we consider that twenty-three families were benefited by the priceless commodity, an ample supply of water, the actual cost is small especially if we compare this cost with the amount an individual home owner would have to pay for his own well development. We can also realize that over the years the purchase of washing machines, sinks, lavatories, hot water heaters, bath tubs and commodes, more pipe, fire hydrants, and all of the other items relating to water supply under pressure will continue acting not only to further the interests of public health, but is already and will continue to act as a business stimulus. A new way of life is being formed in our villages -- a way started by the water supply.

PLANS FOR AUGMENTATION

In the happy situation where there is available both tributary and non-tributary ground water and where existing users have a superior right to use tributary ground water or the flowing stream supported by tributary ground water, a technique exists which permits the utilization of tributary ground water augmented by diversions from non-tributary ground water to provide a totally new source of supply for an over-burdened system without injury to those who are relying on the presence of tributary ground water.

This is called a "plan for augmentation" and is defined by Colorado statute as follows:

> "Plan for augmentation" means a detailed program to increase the supply of water available for beneficial use in a division or portion thereof by the development of new or alternate means of points of diversion, by a pooling of water resources, by water exchange projects, by providing substitute supplies of water, by the development of new sources of water, or by any other appropriate means. "Plan for augmentation" does not include the salvage of tributary waters by the eradication of phreatophytes, nor does it include the use of tributary water collected from land surfaces which have been impermeable, thereby increasing the runoff but not adding to the existing supply of tributary water. (Sec. 37-92-103(9), C.R.S. 1973).

Although many of the citations in this paper are Colorado cases, the law of the arid western states -- those states and parts of states west of the 100th Meridian -- are substantially uniform and constitute what you all know to be the "Appropriation Doctrine" states. Sometimes the Appropriation Doctrine is called the "Colorado Doctrine" because that is the state which first announced it. Even those states in the West which formerly utilized the Riparian Doctrine have now grafted the Appropriation Doctrine onto it and all new appropriations are administered under the Appropriation Doctrine.

In the case of Cache La Poudre Water Users Association v. Glacier View Meadows, 191 Colo. 53, 550 P.2d 288 (1976), the Court reiterated what is commonly known as the "no-injury" rule. On page 293, the Court said:

> We hold here, and in the companion opinion announced contemporaneously with this one, Kelly Ranch v. Southeastern Colorado Water Conservancy District, 550 P.2d 297 (1976), that under the plans for augmentation involved, water is available for appropriation when the diversion thereof does not injure holders of vested rights . . .

and further:

> Here, where senior users can show no injury by the diversion of water, they cannot preclude the beneficial use of water by another . . .
>
> We rule that, in a matter such as this one, water is available for appropriation if the taking thereof does not cause injury. Therefore, the argument of the objectors, to the effect that water withdrawn from the wells must be replaced 100 percent, falls.

Very briefly, the plan for augmentation in Glacier View Meadows involved the purchase of reservoir rights which would be released to the stream in an amount equal to the consumptive use of water from alluvial wells plus an amount sufficient to compensate for transportation losses from the reservoir to the headgates of the objectors. Thus, by acquiring reservoir rights under which water was available year around, the applicant was able to maintain conditions on the stream as they were before the plan was put into effect even though there was provided, by the plan for augmentation, a water supply for 1,892 residential units.

The companion case, Kelly Ranch, instead of using reservoir rights, used a direct flow right. In the Kelly Ranch case, the historic consumptive use of the direct flow rights was computed under the Blaney-Criddle Method and the acre-feet historically consumed through evapotranspiration was made available to the stream through the drying-up of the previously irrigated land. The amount of consumptive use was then made available to the owners of wells who used the well water for domestic use up to the amount of the historic consumptive use of the direct flow right.

The determination of the historic consumptive use and the transfer of that quantity of water to new types of uses and to uses even outside the watershed has long been recognized in Colorado as another "no injury" situation. Once the historic consumptive use has been determined, the applicant for a change in place or type of use of water is permitted to do anything he wants with the quantity of water determined to have been historically consumptively used on the basis of the fact that the junior appropriators have never had available to them the water which had been historically consumptively used.

As engineers know, the determination of historic consumptive use can be made in a number of different ways and such determination is becoming more and more accurate. Once the engineers agree on the historic consumptive use, the rest of the proceeding is reasonably routine.

It is for this reason that a market has developed for irrigation water rights which may then be transferred to a higher and better use for municipal and industrial purposes. The historically irrigated lands are then removed from irrigation and the water saved thereby released to the stream.

In addition to reservoir water, water imported from another basin is frequently used for augmentation purposes, as any such water is an addition to the native water supply and obviously can be utilized to replace additional consumptive use charges against a stream system. It is important, of course, that the transbasin water be introduced above the headgates of any water users who might be injured by the increased consumptive use caused by the new development.

The return flow from water imported from another drainage may also be utilized for augmentation purposes provided the importer of the water continues to maintain control over the return flow and, provided further, that no appropriative rights have been acquired based on the return of the non-tributary water to the stream and commingling of that return flow with the native waters so that it can no longer be identified.

Only recently, with the development of non-tributary ground water, another tool has been found to aid in plans for augmentation.

It has been our experience that where no lawn irrigation is required and the distribution system is efficient, domestic consumptive use may amount to as little as 2½ to 5 percent of the amount of water diverted. See, Green v. Chaffee Ditch Company, 150 Colo. 191, 371 P.2d 775 (1962). See also, Colorado Springs v. Yust, 126 Colo. 289, 249 P.2d 151 155 (1952), quoting with approval the Supreme Court of Utah, in Tanner v. Humphries, 87 Utah 164, 48 P.2d 484 (1935), and Farmers Highline Canal and Reservoir Company v. City of Golden, 129 Colo. 575, 272 P.2d 629 (1954).

Using the 5 percent consumptive use figure for domestic use in a subdivision which allows no lawn irrigation (and this is frequently the case in mountain subdivisions), it is apparent that 1 acre-foot of non-tributary water may augment the diversion of 20 acre-feet of alluvial water utilized in the subdivision. This is a multiplier that is very attractive from an economic point of view. Even with the normal meld of lawn irrigation and household use and use by small businesses (omitting any industries which require a substantial amount of water), non-tributary water may have a two-for-one augmentation value and in the non-irrigating season, that value would again increase to the 20-times range.

The no-injury rule is not new and, as mentioned above, is not limited to Colorado. Hutchins, in his famous work, "Selected Problems in the Law of Water Rights in the West", states the doctrine this way:

> It has long been the general rule (with some exceptions hereinafter noted) that the appropriator may change the point of his diversion of water from the stream, or may change the place of use or even the purpose of his use of the water, so long as the rights of others are not thereby impaired. . . .
>
> The appropriator is entitled to have the stream conditions maintained substantially as they existed at the time he made his appropriation. This applies equally to senior and junior appropriators; the junior appropriator initiates his right in the belief that the water previously appropriated by others will continue to be used as it is then being used, and therefore has a vested right, as against the senior, to insist that such conditions be not changed to the detriment of his own right. This applies specifically to a change in place of use or diversion the effect of which will be to injure the holders of established rights. It is therefore a

> condition precedent to the right to make any change in diversion, place of use, or character of use, that the rights of existing water users will be properly safeguarded from injury resulting from the change. . . .

ROLE OF CENTRAL GOVERNMENTS

Because it is the impoverished areas of governmental subdivisions that desperately need a supply of good potable water or it is energy-impacted areas where local authorities are unable to keep up with the exploding demand, it is always necessary to have financial assistance from central governments to provide a stable water supply.

Certainly there is no higher purpose for which central governments may lend their support than the welfare and stability of their rural populations. Such efforts are to be commended and support for their expansion is desirable.

Only when safe, dependable water supplies are available to the rural areas can we look toward a diminution of immigration to already overcrowded population centers.

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A WATER PLAN FOR THE STATE OF NEW SOUTH WALES, AUSTRALIA

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ABSTRACT

The paper describes the water-related problems and issues that have prompted the preparation of Australia's first State Water Plan. It also outlines the planning concepts, methods and opportunities for public involvement leading to publication of the Plan. Water, so very variable in its occurrence in New South Wales, is controlled by several government authorities. The Water Resources Commission has overall responsibility for co-ordinating water management and development. Recent substantial development of the State's primary and secondary industries has resulted in a growing demand and competition for water. The water of all the State's regulated streams is now fully committed. Increasing competition for Government funds to be spent on other services and public agitation for environmental protection compound the problem. Money, attitudes and the degree of development already achieved restrict further dam building. Water planning in New South Wales has therefore reached a turning point - away from projects such as dams and towards the better management of existing resources. A comprehensive review of all water-related needs and expectations is necessary to ensure that economic, environmental and social goals are fully considered in future water resources management. This review is being done for the State Water Plan. It will identify the problems, determine priorities and assist in the resolution of conflicts. The paper concludes that more effective management and more efficient use of water could be the cheapest as well as the most environmentally benign way to augment as well as improve the quality of our water supplies.

Keywords: State Water Plan; planning concepts and issues; demand, competition and commitment of water; management of water; economic, environmental and social goals; water quality.

INTRODUCTION

No natural resource or physical commodity is more important in Australia than water. Most of the continent lies between 15° and 40° south of the equator, the latitudes of most of the world's deserts. New South Wales, with a third of the nation's 15 million people, lies largely between the 30th and the 35th parallels. And 80% of this third - about 4 million people - live in the Sydney-Wollongong-Newcastle coastal region where they neither see nor feel the landscape or climatic extremes for which Australia is famous.

If you have a picture of Australia it is probably that which its most popular 19th century poets, Henry Lawson and Banjo Paterson, put firmly in Australian folk lore and consciousness and which our landscape artists this century - Drysdale, Nolan, Williams - have continued to paint. Arid, dry, but erratically blessed with rains that can quickly become floods.

This paper first explains the physical facts that lie behind this picture, and then explains how Australians have responded to them. This includes discussion of the social ideals and choices that have shaped development. Special comment is made upon water quality problems. Finally, it looks at how the combination of physical setting with social, political and legal responses has created the needs for and directions in New South Wales water planning today.

PHYSICAL SETTING

By world standards Australia is a dry continent. Table 1 compares its average annual rainfall and runoff with that of other continents. Australia, about four times the size of Western Europe, has only two-thirds of its rainfall. Evaporation potential exceeds rainfall over most of Australia.

TABLE 1

Average annual rainfall and runoff of the continents

Continent	Rainfall (mm)	Runoff (mm)	Runoff % of rainfall
South America	1 350	480	36
Africa	710	180	25
North America	660	250	38
Asia	640	200	31
Europe	610	230	38
Australia	420	50	12

Source: Perrens (1982).

What is true of Australia is true of New South Wales. Forty per cent of the State is arid, receiving less than 300 mm of rain a year. Another 20% is semi-arid, receiving less than 500 mm. Despite this, Australia has more water per capita than other continents. The problem is where and how reliable this water is.

Averages disguise a most significant fact about rainfall: its huge year-to-year and season-to-season variability. This is reflected in river flows. The Gwydir River, one of the State's most variable rivers, has a minimum monthly flow of zero and a maximum monthly flow 26 times its average. The Murrumbidgee River, one of the State's least variable, has monthly flows ranging from 1/10th to 9 times its average. For comparison, the Columbia River ranges from 1/10th to 4-1/2 times, the Mississippi from half to double, the Ganges from 1/10th to 6 times and the Danube from 1/3rd to 2-1/3 times its average.

This variability suggests another component of the traditional picture of Australia: if not drought-stricken, it is flooded. In one week the Clarence River on the north coast discharged 79% of its average annual discharge and in another week the Murray River, the border between New South Wales and Victoria, 26%.

The single most important fact about the topography of New South Wales that explains its rainfall and its flooding patterns is a line of mountains, the Great Dividing Range. This Range, running the full length of south-eastern Australia, lies 50 to 200 kilometres from the coast and averages 900 to 1200 metres in altitude. This makes it a barrier to clouds passing west and determines the pattern of its rivers.

East of the Range 22 coastal rivers, relatively short, fast-flowing and independent of each other, drain only one-sixth of the State's surface area and supply only one-fifth of its water needs although they carry two-thirds of its surface water. These needs are those of the 90% of the population that has chosen coastal living and that of most industry. There is little warning when a coastal river floods but it is usually over in a few days.

West of the Range lies the State's great wheat and sheep country, and its irrigation. Twenty inland rivers traverse distances perhaps three to five times as long as the coastal rivers, drain five-sixths of the State, but carry only one-third of its water. Most flow into each other and eventually into the Darling River which joins the Murray at Wentworth in the south-western corner of the State before flowing on into South Australia. When the inland rivers flood, perhaps only once in every 10 or 20 years, they may inundate a vast area for months, as in 1971 when floodwaters from the Barwon, Macintyre, Gwydir and Namoi Rivers joined to form an inland sea covering 2 million hectares.

However, groundwater supplements these variable inland waters. Sixty per cent of Australia and most of New South Wales is underlain by groundwater basins. Groundwater supplies about 10% of the State's water needs, including the needs of 160 country towns, but sometimes saltiness limits its usefulness.

Yet physical facts - climate and topography - do not determine how people settle a continent or make a living from it. Human preferences for certain places in which to live, certain garden styles, foods and recreations have also shaped Australian responses to the physical environment. Preferences and needs can conflict, wasting public money and endangering public health and convenience, unless planned for by a responsible public authority.

INSTITUTIONAL RESPONSIBILITIES

In New South Wales many authorities share responsibility for the supply of water, and many others are interested in the management and development activities of the supply authorities because of the impact, say, the construction of a dam can have on the productivity of a region, on water quality, fish, and the recreational opportunities a river offers.

In 1976 the Water Resources Commission Act recognised the need for a single authority to co-ordinate the management and development of the surface and groundwater of the State. Through this Act the Water Resources Commission assumed this responsibility as well as the responsibilities of its predecessor for irrigation development. It also assumed new responsibilities for flood mitigation in non-tidal areas and for the monitoring of water quality.

WATER USE AND DEVELOPMENT

This reconstitution of the Commission implicitly recognised the changed and changing needs of society and the maturity of its water development, as evidenced by the irrigation of large areas in the south-west of the State, and the regulation of about 50% of the water of the inland rivers. It was time to reassess the context of water planning, and the constraints imposed upon planners by the history of water development and by changes in the economic, social and natural environments. Such an assessment is essential to clear perception of what our options are now and the direction in which water planning needs to go. This is the function of the State Water Plan.

Most human preferences and activities require a safe, reliable water supply. In Australia this has been a challenge because of the variability of our rains and rivers but also relates to living standards and economic development.

Looking at the urban scene first, Australia began as a settlement of a few hundred convicts with their soldier/warders at Sydney Cove in 1788. Two or three years later masons excavated storage tanks near the stream that remained Sydney's main source of water until the 1840's. This provided about 24 litres of water for each of the colony's 1300 people. Sydney today stores 932 cubic metres of water per capita, compared with London's storage of 18.2 m^3, and New York's of 250 m^3.

Why is Sydney's per capita storage so high? Largely because of the great variability in the stream flows upon which Sydney depends, but also because our consumption is high: 550 litres per head, expected to grow to 620 litres by 2000 AD. if no demand control measures are taken. This means that to supply Sydney's population, now three million, Sydney's water supply system must be capable of providing 1.6 million cubic metres of water a day on average. Peak consumer demand in summer requires twice this amount.

Looking at the rural scene next, we find that irrigation uses 75% of the State's regulated water. The history of its development is tied closely to the development of Australian social ideals as well as to a growing technical ability to gain some control over an environment unsympathetic to preferred land uses.

Australia was settled just as the industrial revolution was swinging into gear. Engineering solutions to old problems became possible. Thus early this century it was possible to establish large-scale irrigation in New South Wales. The million-megalitre capacity Burrinjuck Dam at the headwaters of the Murrumbidgee River was the first major irrigation dam built in New South Wales.

With irrigation the country could be more closely settled and shielded - slightly - from drought and flood. Irrigation was a way to fulfil people's dreams of self-sufficiency on the land. This dream was part of the idealisation of rural life that has been part of Western thinking for hundreds, even thousands, of years.

Today, 15 major rural dams supply water to 800 000 hectares of irrigated crops which earn about 15% of the State's export income. And, although only 2% of the country is cultivated, Australia has one of the largest cultivated and irrigated areas per head of population in the world. This is perhaps extraordinary given the extreme variablity of New South Wales' rainfall and streamflows, particularly west of the Great Dividing Range where 90% of the irrigation is. To irrigate, Australia stores ten times as much water as India per square kilometre of irrigation, five times as much as Egypt, and twice as much as the United States.

WATER QUALITY

A picture has been sketched of the factors that have shaped the development of New South Wales and of its capital, Sydney. Perhaps an outline of what this development has meant to water quality would be informative and interesting. As in Northern Europe, there is a strong land-use/water quality nexus, but problems differ in scale because of differences in population density and in the intensity of agriculture.

Since pollution control legislation was passed in the late 1960's there have been no gross pollution problems in New South Wales, but the problems which do exist will increase unless action is taken. The problems of the city differ from those of rural New South Wales.

In non-metropolitan New South Wales the main water quality problems are eutrophication, turbidity and salinity. Joint water and land-use management programmes and policies are needed to control each. For example, turbidity is common after heavy and prolonged rain where erosion and devegetation are excessive. This means much of western New South Wales. The cost of treatment to remove sediment from the water supplies of country towns would run into the multi-millions; nutrient removal and salinity control would also be extremely costly.

While all cities, with their bitumen and impervious concrete, alter the hydrology of the area and, as they grow, exceed the natural limits of the environment to assimilate their untreated wastes, Sydney is luckier than most European cities in three important respects:

on its eastern perimeter the Pacific Ocean offers Sydney its vast natural capacity to assimilate wastes, especially liquid wastes.

- it is not forced to use the same water bodies for water supply as for waste disposal. It draws little of its piped water from the rivers within its own metropolitan area and hence only small sections of these rivers need the protection of their waters to drinking standard.

- its short coastal rivers are not used for waste disposal by any other urban population. Hence the adequacy of any external authorities' pollution control does not affect Sydney.

Sydney's main problem, however, is one familiar to cities all over the world: a growing number of people and hence an increasing volume of human liquid wastes. For example, the north and west of the city are drained by the Nepean-Hawkesbury River system. If, as forecast, by 2000 AD these rivers must receive the waste of 700,000 people instead of today's 300,000 people, then the volume of this waste will double. While today's tertiary treatment methods are usually satisfactory, they will become inadequate as waste loads increase. Nutrient removal will be necessary. Modifications of treatment processes and changes in discharge controls will also be necessary. But such relatively easy solutions will be harder to find where non-point source pollution - the rural problems of eutrophication, turbidity and salinity - are involved.

NEEDS AND CONSTRAINTS IN N.S.W. WATER PLANNING

Between the 1890's and 1920 every Australian State nationalised surface water. Groundwater followed in the 1960's. Nationalisation was thought the best way to ensure the fair distribution of the resource. Thus, in each State a government authority owns the water and issues licenses for its use and control. Riparian and prior appropriation rights do not exist. This has spared Australia legal argument over water rights but it has not prevented all problems.

Why? Basically, because no matter who owns the water, more potential uses exist for it than can be supplied. Also, many people have difficulty accepting change. A government authority should exercise its responsibilities and manage activities and resources for the people's benefit. Change may be necessary and desirable, but sudden change can be both difficult and unfair.

Therefore, pure economic theory cannot be applied to the main water users - irrigators - overnight. Irrigation was established for social rather than economic reasons. It was the way to create a more just society by creating many family-run farms where once there had been only huge sheep properties. Often the irrigation farms were far too small and, partly to compensate for this, the Government offered cheap water. The irrigators were required to pay only the operation and maintenance costs of the irrigation systems, not the capital costs of the dams.

This has continued to the present day so that cheap water has been built into the capacity to compete for markets, into land values, regional economies and popular expectation. Now, although water demands are growing and conflicts between uses are emerging, much of the water remains with these irrigators and is attached, as a right, to their land.

How is the Water Resources Commission to solve these problems? Should it keep building dams? The Commission has recently completed construction of two major rural water supply dams and is now constructing two more. But development is approaching its cost-effective limits: the best sites have been dammed, so putting more money into further development will add less and less water per dollar spent to our regulated water supplies. And today there are fewer dollars to spend on public works such as dams. In 1900, 1930, and 1950 Australians agreed on the need for these works. They symbolised progress and helped settle the country and control its often harsh environment. But today people expect a much wider range and higher quality of services from their government than in their grandparents' day. This means more competition for public funding and different priorities that reflect different social aspirations and needs from those of 50 or 20 years ago.

Another constraint, partly reflecting the success of past development, is that many Australians today generally prefer the unique and distinctive features of their natural and cultural environments to be preserved. They question each fresh proposal for a new dam and want to know who will be paying for it and who benefiting.

Altogether, then, physical, economic, financial, environmental and social factors suggest that water planners and managers must look first to non-development options to meet growing needs for water. They must ensure that existing supplies are used as efficiently and effectively as possible before expecting the public to accept, and pay for, new dams.

THE STATE WATER PLAN: IMPETUS AND PROCEDURE

The Water Resources Commission began work on the State Water Plan in 1982. Until this time there had been little attempt to control growth in the demand for water and planning had been project or valley-based. This provided little basis for determining priorities between land salinisation, flood mitigation, water treatment plants for the country towns whose water is often turbid, making irrigation supplies to cotton growers more reliable, or developing an effective groundwater policy.

To get a proper picture of how other people saw these problems and needs, comments were invited from government authorities and environmental and water user organisations. An outline for the Plan was then developed and studies initiated with which interested parties helped.

Some studies containing information not previously available to the public, for example on groundwater and irrigation efficiency, will be published as supplementary papers. Others will be blended and summarised and published as position papers. One of the major merits of this planning procedure is that it generates informed public discussion.

THE STATE WATER PLAN: CONCEPTS AND OPTIONS

The options and blends of options, both management and development, through which problems and needs associated with water in New South Wales may be met or modified are now explored.

First, the State Water Plan must estimate the future water needs of each sector of the economy as well as domestic and the non-consumptive needs for water of natural ecosystems and of people for recreation.

To make these estimates while allowing for the uncertainties of the future, the Task Force preparing the Plan is using the alternative futures concept. In applying the concept, the Task Force hopes to relate significant water demands to economic variables such as Gross Domestic Product, Gross State Product, and average weekly earnings where a reasonable correlation can be demonstrated. In this way it will be possible to determine a range of future water demands covering various levels of expected economic activity.

Demand forecasts for the various uses of water for the three time horizons of the planning period, 1990, 2000 and 2010, will be made. Then a blend of management and development programmes to meet or modify the forecast demands of each alternative future will be presented. This will enable the community to weigh up the possible futures and their implications and to express a preference for a particular future.

Recent studies of water problems and management conducted by the Adelaide water authority in the neighbouring State of South Australia, and by the Hunter District Water Board 160 kilometres north of Sydney, indicate the likely directions of water planning in New South Wales.

The key concept in Adelaide's study is demand management: the identification and implementation of various measures to change demand (in scale, type or distribution) so that fewer capital works, and thus lower expenditure, are required in the planning period. The study identified three basic components of demand management:

- the pricing of water to encourage careful and effective use and to ensure equity between users

- community education and involvement in water conservation

- technical innovations to improve efficiency of existing appliances and of water conveyance, etc. and to encourage the recycling and re-use of water.

This emphasis on demand management is also the response of the Hunter District Water Board to the problems it faces.

Turning now to the rural sector, where irrigators use 75% of the State's regulated water, efficiency and effectiveness in the supply and use of the available water is the cheapest way to augment supplies. Therefore, studies of how much water is available, how it is used and the potential for doing more with the water already regulated are vital to the State Water Plan. Studies to date for the Plan have covered efficiency and effectiveness in conveyance from a storage to a user and in use on the farm. The studies sought to identify opportunities to minimise waste and reduce individual demands on existing supply systems.

For example, a recently completed en route re-regulating storage on the Murrumbidgee River cost $3 million. In its first year of operation it saved 70 000 ML, at about $6.00 per ML. Compare this with the most cost-effective possible new storage on the same river: it would cost $35 million and add 200 000 ML to supplies which works out at $30 per ML. This would be easily the cheapest water from any new dam west of the Range.

Both irrigation equipment and irrigation policies can also help achieve greater efficiency in irrigation, for example:

- Under-tree irrigation uses about 40% less water than fixed overhead sprinklers use to water citrus orchards.

- The Commission has implemented volumetric allocation schemes in river valleys controlled by major dams. These schemes allow irrigators to use a specific volume of water rather than authorise irrigation of a specific area as was formerly the case. They have encouraged the more careful use of water and ensure a more equitable distribution which is particularly important as regulated water reaches full commitment.

New South Wales' relatively low water charges mean that, as competition for the available supplies increases, irrigators lack the incentive to use water wisely and maximise returns per megalitre.

A market system, or trade in water entitlements, would place an economic value on water and hence encourage its efficient use. If irrigators are able to purchase or rent some portion of another irrigator's allocation or sell or rent their own allocations, they will have a monetary incentive to use water in the most beneficial way so that they, and the community, will obtain the greatest return from each megalitre and from their investment in it.

A temporary market transfer scheme for the 1983/84 irrigation season only was implemented by the Commission as a means of obtaining the best use of water during drought.

The Commission monitored the scheme which was of interest to water authorities, irrigators and resource economists throughout Australia. A slightly modified scheme was implemented for the 1984/85 irrigation season at the request of irrigators. Permanent implementation of a market system for water must consider implications for other policies. For example, it could affect, or be affected by, the strictly controlled land lease and ownership arrangements that apply in the State's irrigation areas. Pressures to permit the transfer of water out of irrigation to mining companies or thermal power stations may also affect a transfer system.

Other management initiatives affecting the rural sector include:

- adjusting charges to reflect the reliability of supply

- conjunctive use of surface water and groundwater

- farm design and management advisory service for irrigators.

It can be seen that while the maturing of New South Wales society has placed constraints on water planners and re-defined goals, objectives and priorities, there are also many opportunities in both urban and rural systems for meeting or modifying various needs for water within these constraints.

CONCLUSION

The reviews and studies undertaken for New South Wales' first State Water Plan suggest that, in the foreseeable future, greater emphasis will be put upon management than upon development. This applies equally to urban and rural water supplies. Such emphasis is necessary because of the high level of development that the New South Wales' water industry has reached. More effective management and the more efficient use of water could be the cheapest as well as the most environmentally benign way to augment as well as to improve the quality of water supplies.

However, a "greater emphasis" on management does not exclude development. The Plan will explore a range of management and development options so that the decisions made may enhance community welfare, and preserve or enhance the utility and quality of the natural assets - land and water - upon which our prosperity ultimately depends.

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IMPLEMENTATION OF DATA BANKS AND NUMERICAL MODELS IN GROUND WATER INVESTIGATIONS AND MANAGEMENT

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ABSTRACT

Continuous collecting of ground water data and their sucessive analysis can in many cases replace expensive and long-term investigations. Process of gathering and interpretation gradually leades to creation of an information system that need not be especially sofisticated and though to be able to satisfy successfully all current needs for the knowledge and management of ground water reserves. For the application of such an information system it is necessary to take into account that the ground water reservoir is a subsystem of the environmental system, with all the positive and negative consequences and that the exploitational reserves, among other things, depend on the way of exploitation. Using data banks, it is possible to define hydrogeological systems and mathematical models and by their application to manage ground water reserves. As an illustration of this we are presenting authors' experience with survey of the obtained results in a part of the Sava river basin.

INTRODUCTION

By this work we want to indicate the simple and cheap way of ground water reserves management for individual and collective needs. According to our experiences it is proved that the demand for the additional and special investigations is minimized by a systematic approach to problem solving. Problems that we initiate surely require a broader discussion which we are not able to enter because of space limitation and short time. We are apologizing in advance, that our presentation here is going to cover only assertions without any explanations and existing dilemmas.

STARTING ASSUMPTIONS

Insisting on classification and standardization of investigation works (Peek, 1980) and ground water reserves, that we meet in literature, points to the complexity of problems and misunderstandings among authors. These misunderstandings arise still from the definition of ground water deposit and definition of ground water reserves.

G r o u n d   w a t e r   d e p o s i t belongs to the three natural macrosystems (fig. 1).

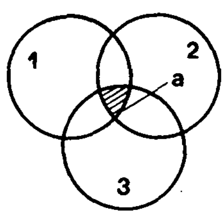

Fig. 1 Shematic illustration of macrosystems influencing the ground water subsystem

Circles 1,2 and 3 represent macrosystem one after the other: lithosphere, hydrosphere and biosphere. Surface (a) presents enviromental system to which ground water subsystem belongs. For a simplification of analysis, in ground water subsystem we set aside the systems (aquifers, ground water deposits), whose boundary conditions and constraints are determined by other environmental subsystems. Neglecting the influence of environ ment on the ground water deposit causes the growth of investigation works and emission of inconplete informations (Miletić & Heinrich-Miletić, 1984).

Without starting giving definitions and detailed discussion, one can easily made certain, that e x p l o i t a t i o n r e s e r v e s (the mined ones and safe yield) of the same system differ greately relating to the kind, number, position and performance of exploitation devices as well as to limitations arising from environmental system. Therefore we can determine and evaluate exploitation reserves only by the performance of real objects for exploitation or by model simulation (Miletić & Heinrich-Miletić, 1984).

ABOUT MATHEMATICAL MODELS

For the analysis of hydrogeological systems, flow models and transport models are most commonly applied. Considering the features of the observed processes we use deterministic or stochastic models (Mercer & Faust, 1980). Taking the investigation objective into account, deterministic numerical models can be devided into models of: predication, identification and management (Bachmat et al., 1980). In our case the predict models (detailed description: Prickett, 1975) are the advantageous ones. Because of the frequent application of the predict model it is necessary to point out its positive and negative sides discussed by Prickett (1979), Darr (1979), Basci (1979), Lehr (1979) and Anderson (1983, for transport problem). Model application on the estimation of reserves of hydrogeological systems can be devided into three groups (Heinrich-Miletić, 1980). We differ the model of demanded quantity of water, the combined hydrogeological and environmental model (Young & Bredehoeft, 1972; Schwarz, 1976) and the model involving optimalization processes (Aquado et al., 1974; Alley et al., 1976;

Evenson & Mosley, 1970; Schwarz, 1976). Our experience proved that the role of a dominant investigation method must not be attributed to the model, but it should be used in continuity of investigation works (Miletić, 1975; Boreli & Djordjević, 1978; Heinrich-Miletić, 1980). Model application is the most complete within the scope of water economy information system. Keeping the model constantly active in the system, enables an optimal management of water reserves and optimal use of environment (Heinrich-Miletić, 1982).

ABOUT THE ORGANIZATION OF INFORMATION SYSTEM WITH APPLICATION

A need for collecting, selecting and use of data, has been known for a long time. Accummulation of data in number and kinds, with possibilities given by computer technology, leads to the organization of information system. A good information system is based on two requirements. These are a real usability and a clear objective. Considering the first one- a good information system has to be simple, cheap and reliable. The second one is in our case an optimal management of ground water reserves. In our case, the organization of information system for the Sava river basin in Socialist Republic Croatia (approx. area 25000 sq.km.) consists of: data bases (series of the identical data), data bank (series of data bases with the possibility of simple data processings) and bank of the applied softwear. For each particular case of problem solving on the basis of data from the above organization, the operational data base is formed, which we use in processing.

On the field where we perform professionally, succession of the above mentioned took place in the following way.

The first systematic gathering of hydrogeological data started in 1962. During some years, thousands of individual data were gathered, whose complete interpretation resulted in hydrogeologic rayonization of the terrain (Miletić, 1969). Data are arranged in basic data bases about: tube wells, investigation boreholes, springs, thermal wells, exploratory oil wells, piezometers, morphological features, geological and geophisical explorations, and known ground water reserves (Miletić et al., 1972). For all investigations carried out since then, the Water Authority enables free use of data bank, free feasibility reports and plans of investigation, and even assured financial participation if only the user had respected the planed works and delivered the new data to information system. In this way the data bases were filled, whose analysis enabled separating of hydrogeological systems on which regional water supply can be based.

Systems are standardized and applicable programs of numerical processing are completed for them in the sense we were discussing earlier (Heinrich-Miletić, 1982). Individual described approaches are shematically presented in fig 2.

By successive verification and inprovement of the model during time, the model became more and more reliable investigation tool. With the example we have been presenting, we are now able even to assume the consequences that will be caused by the changes in environmental use. We are also capable of lacating and testing new pumping plant in individual hydrogeologic system by minimum investigation works.

CONCLUSION

Following the fundamental objectives of IRWA we considered our experience from ground water investigation works that we present here, to be of some interest. During location and execution of rural, local and regional water supply schemes the greatest difficulty arises due to the lack of financial means and avalable time necessary for the performance of the investigation works. Compromises being made in these cirumstances lead to many mistakes. We think that negative effects of the mentioned difficulties can be alleviated and in some cases prevented, following systematically all relevant events related to ground water in complex water economy systems. A large number of collected data and informations enable hydrogeological rayoniza-

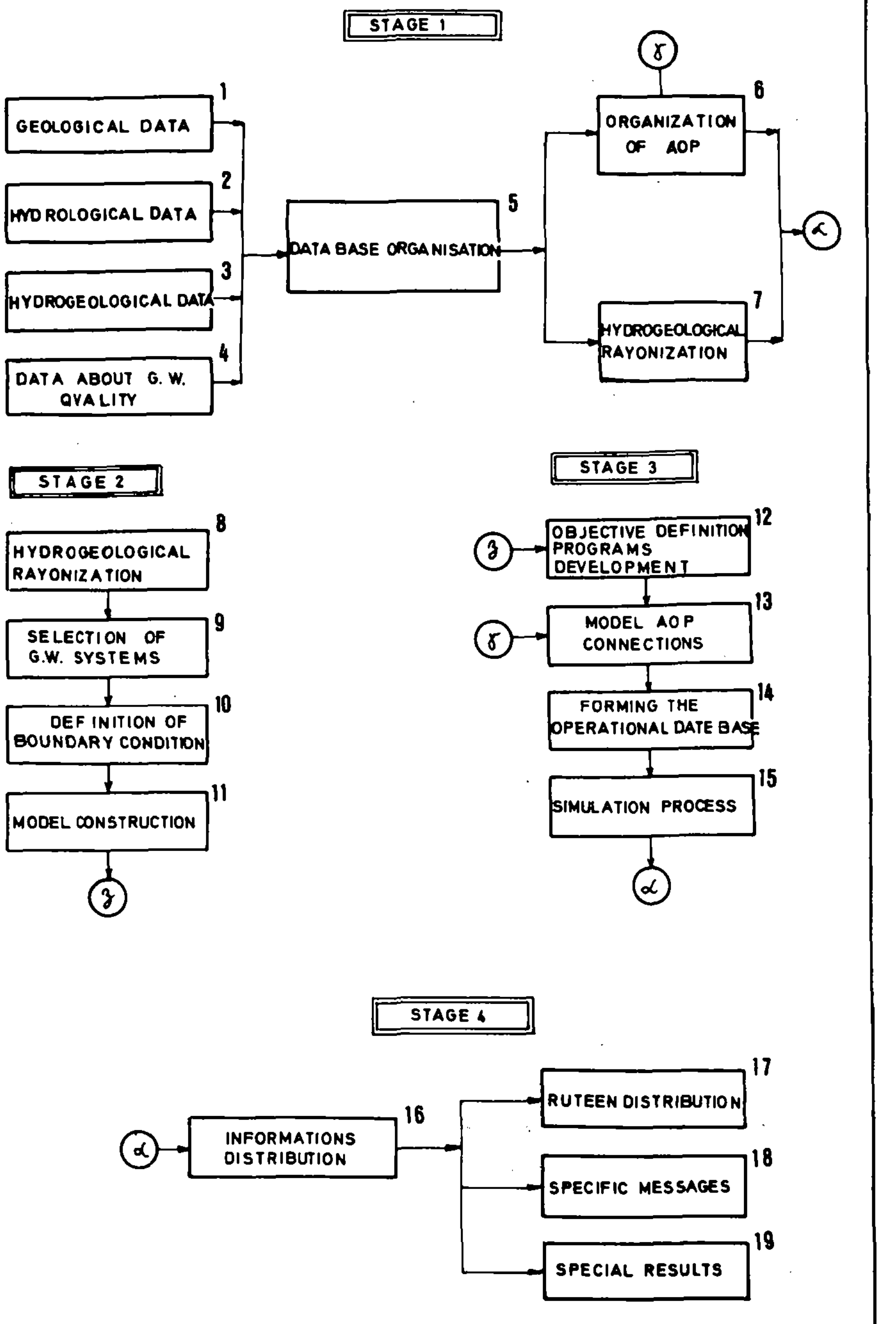

Fig. 2. Schematic presentation of individual procedures within the information system

tion and separation of hydrogeological systems. By maximum utilization of data and informations referring to particular hydrogeological system, by the organization of regional monitoring of ground water levels and quality, and by construction of mathematical models which are permanently active, we are capable to achive a high accuracy of conclusions with relatively modest local investigation works. Continuous activity at data collecting and their appropriate interpretations lead to the organization of an information system de facto, disregarding that it is not necessary, even not desirable, that information system should be very sofisticated. Shortly, the described activity has the consequence, that in large water economic systems each particular investigation work does not present a separate occasion but only an episode in the continuity of investigations which are connected through information system.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 17 Aspect number 6

URBANIZATION AND NONPOINT POLLUTION

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ABSTRACT

Sources of pollution of surface waters can be divided into two groups; effluent discharges (point) and diffuse sources (nonpoint). Pollution from nonpoint sources is due to use of land by man and due to his activities taking place on the land surface. The pollution causing activities include agriculture, urbanization, construction, fertilizer application, transportation, etc. This pollution is distinguished from the background water quality contributions caused by the contact of water with rocks, undisturbed soils and geological formations, natural erosion and other natural processes responsible for chemical enrichment of surface waters.

The sources of nonpoint pollution due to urbanization include: atmospheric deposition, litter deposition, traffic emission, erosion from open lands and by construction activities, road surface deterioration. On the other side of the spectrum, soil loss is the major source of pollution from agricultural watersheds.

In Wisconsin, several large research projects have been conducted in the last 10 years, the result of which enabled to characterize the loadings and the strength of pollutants from small watersheds with a uniform land use. A synthesis of the results and computer simulation yielded then an overall picture on the effects of changed land use - - urbanization - - on the water quality. Consequently, the major factors causing the change, such as increased imperviousness, increased traffic densities, were defined and by a sensitivity analysis their effect was determined. The pattern of the change depends also on the type of drainage system used for conveyance of increased volumes of stormwater.

Keywords: Nonpoint pollution, urbanization effects, unit loadings of pollutants, water quality, urban erosion, hydrologic modeling, hazardous land uses, land use effects, traffic effects.

INTRODUCTION

Many areas of the world are undergoing rapid urbanization, which is imposing severe stresses on water resources, increasing water demand and deteriorating water quality.

Traditionally, deteriorating of water quality due to urbanization has been associated with pollution from point sources that included mostly municipal and industrial, treated or untreated sewer outfalls. However, a significant portion of the overall pollution load from urban and urbanizing areas can be attributed to nonpoint sources such as construction, washoff of dust and dirt from impervious surfaces, or sewage inputs from unsewered suburban areas.

Watershed in transition

Water quality modifications in a watershed in transition from a pristine unaffected drainage area to a fully urbanized one progress in the following qualitative phases:

1) Deforestation increases the hydrological activity of the watershed and erosion potential. Patrick (1975) quoting works by Leopold (1978) and Borman and Likens (1969) pointed out that cutting down a forest - reducing the forest from 80 to 20 percent - increases the sediment load entering the riverine system eight times.

2) Conversion to intensive agriculture deprives the watershed of protective vegetation cover. This increases erosion potential by one to two orders of magnitude. Besides sediment, the pollutants from intensive agriculture include pesticides and nutrients (nitrogen and phosphorus).

3) Watershed during development. The soil loss from construction sites can reach magnitudes of over 100 tonnes/ha (Figure 1). In urbanizing watersheds, few percent of the watershed area under construction can contribute a major portion of the sediment load carried by the stream.

4) Unsewered suburban and urban development. Disposal of sewage in soils (septic tanks) eliminates only pollutants that can be filtered out or adsorbed on soils. Mobile pollutants such as nitrates can cause severe contamination of groundwater resources. When the adsorption capacity of the soil disposal systems is exhausted, contamination of surface waters by organics and fecal microorganisms will occur.

5) Fully developed sewered watersheds. As the imperviousness of the urbanizing watersheds increases, the watersheds become more hydrologically active. This also increases the frequency of the runoff events. Pollutants that accumulate on the impervious surfaces from traffic, litter deposition and other sources are washed by the surface runoff into the storm or combined sewer systems and, subsequently, into the receiving water bodies. The pollution potential of urban runoff is similar to that of treated sewage from the same area for BOD_5, it is between treated and untreated sewage for bacterial pollution, and exceeds that of untreated sewage for suspended solids (Field and Turkeltaub (1981)). Urban runoff contains also a variety of potentially toxic substances, including lead, asbestos and hydrocarbons from traffic, PCB's, cadmium and other components associated with urban air pollution deposition, and components from damaged infrastructure due to elevated acidity of urban precipitation (Novotny and Kincaid (1982)).

Figure 1. - Pollution from a construction site. (Photo, Univ.of Wisconsin)

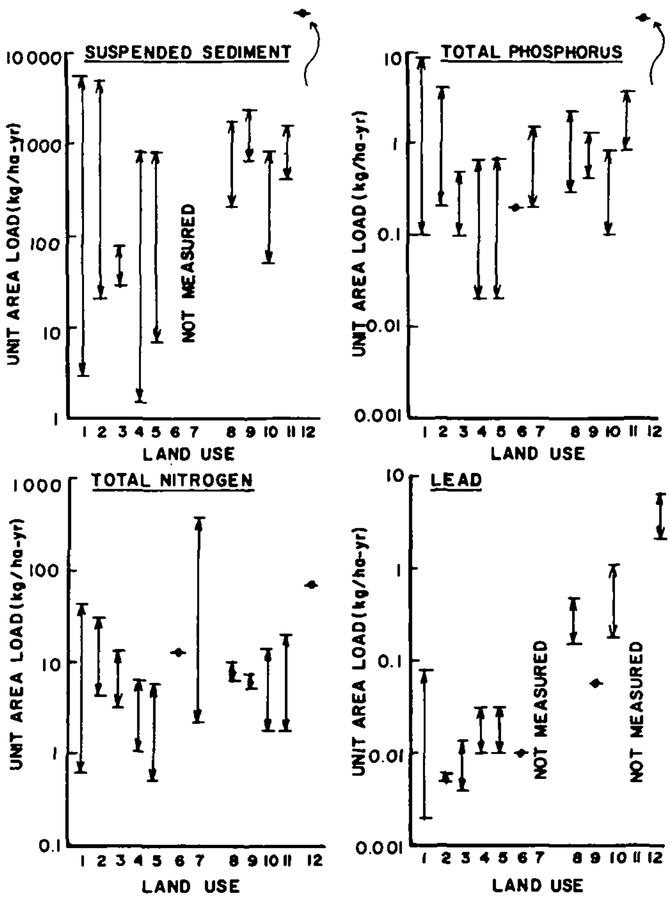

Figure 2. - Ranges of loadings of pollutants from pilot watersheds in the Great Lakes region. (Source - International Joint Commission.) Land uses: 1 - agriculture, 2 - cropland, 3 - pasture, 4 - forests, 5 - idle, 6 - sewage sludge, 7 - spray irrigation, 8 - general urban, 9 - residential - 10 - commercial, 11 - industrial, 12 - developing.

Table 1. Flow Weighted Mean Concentrations of Pollutants From Nonpoint Sources (Midwestern USA)

	Suspended solids (mg/ℓ)	BOD_5 (mg/ℓ)	Total nitrogen (mg/ℓ)	Total phosphorus (mg/ℓ)	lead (mg/ℓ)	Total coliforms no/100 mℓ
Precipitation[1]	4	10	0.8	0.01	0.01	NA
Agricultural Wisconsin priority watersheds[1]	780	NA	9	1.20	NA	NA
Animal feedlot Runoff[2]	30	5,000	1,400	300	NA	NA
Urban stormwater[1]	400	30	5.0	0.6	0.5	$10^3 - 10^8$
Combined sewer[3]	400	115	10.0	1.9	0.4	$10^5 - 10^8$
Snowmelt from freeways[4]	570	40	3.1	0.3	1.0	$10^2 - 10^3$
Construction site runoff[5]	20,000	NA	NA	NA	NA	NA
Treated municipal sewage[3]	20	20	30	5	NA	$10^2 - 10^4$

NA = Data not available or insufficient

Source: 1 - Wisc. DNR, 2 - Loehr (1972), 3 - Lager et al. (1977)
4 - Marquette University, 5 - Water Resources Center-University of Wisconsin

PROCESSES CONTRIBUTING TO NONPOINT POLLUTION

Land use effects on nonpoint pollution

Land use is a simple term describing the prevailing activity occurring in the area. As such it bears little relationship to pollution generated from the area. Nevertheless, some of the most important causative factors for nonpoint pollution generation are indeed a function of land use. These factors include population density, atmospheric deposition, vegetation cover, street litter accumulation rates, traffic density, pollution conveyance systems, and degree of imperviousness. Therefore, attempts to relate pollution loadings from diffuse sources to land use activities are justified. Factors not related to land use such as slope, soil texture and fertility, and drainage density are less important for urban areas which have impervious surfaces than for rural lands (Novotny and Chesters (1981)).

Figure 2 shows the ranges of unit loadings of pollutants from pilot watershed studies in the Great Lakes Region. Using these figures one could predict the changes in loadings as urbanization is progressing from forested or idle land to agriculture or urban land use. Table 1 shows the weighted average concentrations of pollutants in runoff from various land use watersheds.

Woodlands

Undisturbed forests or woodlands represent the best protection of lands from sediment and pollutant losses. Woodlands and forests have very low hydrological activity due to high surface water storage by interception of rainfall by vegetation and due to improved permeability of forest soils. Streams draining lowland forests and forested wetlands may have elevated organic and nutrient levels caused by leaching from soils by interflow. Most reported and simulated sediment loadings from forested lands are less than 100 kg/ha-yr.

Pasture and rangeland

As seen in Figure 2, unit loads of pollutants from pasture and rangelands are at least one order of magnitude less than loads from croplands. Timmons and Holt (1977) studied organic pollution and nutrient losses from native prairies in west-central Minnesota. Average annual total COD losses ranged from 2.2 to 40 kg/ha, annual total N losses ranged from 0.1 to 1.7 kg/ha, and annual total P losses ranged from 0.01 to 0.25 kg/ha. It should be noted that N and P loads contributed annually by precipitation are significantly higher than the nutrient losses from native prairies. Thus, native prairies are actually nutrient sinks.

Agricultural land use

Many factors affect pollutant emission from farm croplands. Pollutants arise from surface runoff (erosion of topsoil and irrigation return flow), interflow (mostly tile drainage and leachate of excess irrigation), and groundwater flow.

Erosion and soil loss by surface runoff is a predominant source of pollution from croplands. As noted by Alberts, Schuman, and Burwell (1978), over 90 percent of the nutrient losses are associated with the soil loss. Although the nutrient losses usually represent only a small portion of the applied fertilizer, their pollution impact almost always exceeds the standards accepted for preventing accelerated eutrophication of surface waters. Bacterial contamination of runoff from manure application and unconfined livestock is also considerable (Dudley and Karr (1979)). Agricultural runoff is also a source of pollution by organic chemicals.

The measured ranges of pollutants from agriculture in the Great Lakes Region are shown on Figure 2. Load differences amounting to several orders of magnitude are common for croplands. The variability is caused by variations in slope, soil erodibility and texture, drainage characteristics, vegetational cover, tillage and planting practices (up and down slope or contouring), and meteorological factors.

By simulation with a hydrological model, the author has found that agricultural cropland with slopes less than 3 percent and with a medium texture or better soils, produce minimal pollution loadings. Only cropland located on higher slopes and/or poor soils represent a pollution hazard (Novotny et al. (1979), Novotny and Chesters (1981)).

Feedlots and barnyards can be the land uses producing the highest pollution loads in rural areas. With the advent of improved feeding techniques, cattle are no longer put out to pasture but are held in relatively small areas. The majority of feedlot wastes reaching surface waters are transported by surface runoff. The high organic content of the surface crust protects against erosion and, consequently, sediment yields from feedlots are somewhat lower.

Nevertheless, as reported by Loehr (1972), barnyard and feedlot runoff has extremely high BOD_5 concentrations (1000 to 12,000 mg/liter), COD (2400 to 38,000 mg/liter), 6 to 800 mg/liter of organic N, and 4 to 15 mg/liter of P.

Construction sites

Construction sites produce the highest amounts of pollutants, ranging up to 500 tonnes/ha-yr of sediment particles with corresponding amounts of other pollutants. The principal cause of high pollution loads arise from stripping topsoils and exposing bare soils with no protection. Furthermore, compaction of soils by construction machinery reduces permeability and surface storage of soils and increases hydrological activity. Factors affecting sediment and other pollutant losses from construction sites are: slope, proximity of the site to a stream channel, existence of buffer zones of natural vegetation, erodibility of the soils, erosion control practices on the site, meteorological factors, use of heavy machinery, and length of time the soils are exposed and unprotected.

Developed urban lands

The nonpoint pollution generation in urban areas is quite different from that in rural or suburban lands. Several factors cause the difference.

1) Large portions of urban areas are impervious, resulting in their much higher hydrological activity. The runoff coefficient (defined as a ratio of runoff volume divided by the volume of rainfall) is proportional to the degree of imperviousness.

2) Except for construction sites, most of the pervious surfaces in residential or city areas are protected by lawns, and as a consequence, erosion is reduced.

3) Over a longer period of time (e.g. a season or year) almost all of the pollution deposited on impervious surfaces that has not been removed by street cleaning, wind, or decay, will eventually end up in surface runoff. On the other hand, soil represents an infinite pool of sediments and potential pollutants in non-urban and suburban areas and in construction zones. Their removal into runoff depends on surface protection and on the energy of rain and surface runoff that liberate the soil particles.

4) The frequency of pollution carrying runoff events is greatly increased in watersheds with higher imperviousness. Nonpoint pollution loads from pervious lands (croplands, pastures, woodlands or urban lawns and parks) occur only during very large storms or during spring frozen ground conditions. Polluted runoff from impervious urban surfaces is generated during all rainfalls that exceed a certain threshold value of depression storage (1 to 2mm).

Pollution or urban runoff is a resultant of several diversified input sources and processes. The nonpoint inputs can be categorized as follows:

1) Wet and dry atmospheric deposition

2) Street refuse deposition including litter, street dust and dirt, and organic residues from vegetation and urban animal population

3) Traffic emission and impact

4) Urban erosion

5) Road deicing.

Dry atmospheric deposition rates in many U.S. cities have dropped substantially as a result of switching from coal to natural gas as a primary source of energy for household heating and industrial energy production. The deposition rates measured in Milwaukee, Wisconsin were around 1.5 tonnes/km^2-month and represented about 5 percent of all pollution inputs into several experimental urban watersheds monitored in the period of 1979-1982. The wet atmospheric deposition (precipitation) in Midwestern U.S. is acidic with pH around 4. It contains appreciable amounts of nitrogen (above 1 mg/ℓ). As shown by Novotny and Kincaid (1982) the acidic rainfall is neutralized primarily by dissolution of cations (calcium and magnesium) from urban pavement and sewer materials.

Litter and vegetation residues constitute the major components of street refuse that accumulates on curb and median barriers or urban streets and roads. The reported deposition rates ranged from about 5g/curb-meter/day in relatively clean residential areas to over 100g/curb-meter/day in some urban industrial areas (Novotny and Chesters (1981)). Since the curb represents a trap for pollutants, its height has an effect on pollution loadings. Roads with no curb will have lower loadings. The accumulated deposits are washed by runoff into receiving water bodies. Pollution inputs increase dramatically during the Autumn leaf fallout in areas with trees.

Traffic corridors and their impervious surfaces (roads) or partially pervious surfaces (railroads and secondary roads) are another source of pollutants in urban and interurban land use.

Lead has been identified as the primary pollutant attributed to road traffic; however, other pollutants - hydrocarbons, phosphates, asbestos, and particulate matter can be closely correlated with traffic density, road conditions, and abrasion.

The deposition ranges of lead may vary according to traffic density. They can range from 2.3 mg/m^2-day on a busy highway to 0.04 to 0.18 mg/m^2-day for background deposition rates. The measured levels of lead associated with street dust and dirt ranged from 1.0 to 20.0 mg of Pb/g of solids. These values are much higher than the lead levels found in soils (Laxen and Harrison (1977)).

In northern parts of the U.S. application of salts for road deicing elevates the pollution levels of winter runoff. Application rates for road salts for highway deicing range from 110 to 350 kg per road km per application (Anon. (1971)). These application rates result in chloride concentration in the highway snowmelt runoff ranging from 3000 to 25,000 mg/ℓ). The salts that are used for highway deicing operations contain corrosion-inhibiting compounds, mostly phosphates, chromates, and complex cyanides. These compounds represent an additional pollution problem since little is known as to their fate and deposition. The highway deicing salts, in spite of the anti-corrosion additives, are also a major cause of vehicular corrosion. Most of the metal loss due to corrosion is also incorporated into the snowmelt runoff.

<u>Drainage system effects on nonpoint pollution</u>

Urbanization represents also a profound change in land drainage with a consequent effect on pollution loadings. The effect of overland and channel drainage systems on nonpoint pollution loadings is expressed by a delivery ratio factor, DR, or

$$Y = DR * EP$$

where Y = sediment or pollutant yield measured in the receiving water body

EP = erosion potential for the sediment or pollutant at the source.

The delivery ratio factor expresses the fact that pollutants are redeposited, biodegraded or infiltrated during transport from their source to the point of measurement in the stream. Most of the loss occurs during the overland flow phase.

In watersheds with natural drainage the delivery ratio is correlated to the drainage density or area size and can range from 50 to few percent. Besides area of the watershed, other factors determine the magnitude of the delivery ratio factor (Novotny (1980)).

In watersheds that are urbanized, the presence of impervious surfaces and of storm or combined sewer systems results in delivery ratios for most pollutants approaching one (100%), meaning that all pollutants generated at the source will reach the receiving water bodies. Thus urban areas, in spite of lower nonpoint pollution emissions from the sources, have pollution yields that may equal or surpass those from agricultural watersheds as shown in Table 2.

CONCLUSIONS

Watershed transition from agricultural to urban land use in Midwestern U.S. may not result in drastic changes in overall yields of pollutants as demonstrated in Table 2. Higher source emissions from agricultural lands are buffered to a degree by low frequency of polluted runoff events and by a lower delivery ratio. The most profound qualitative and quantitative step in worsening water quality occurs in transition from forested or native prairie watersheds to intensive agriculture or urban land use.

However, a significant change of frequency of polluted runoff events occurs as a result of urbanization. Most of the flow in streams of rural watersheds is due to groundwater (base) and interflow contributions with a relatively low suspended solids content. Watersheds with drainage tile systems, or which are receiving excess irrigation, do have elevated pollution levels from tile and irrigation return flows. Highly polluted surface runoff events are relatively infrequent (few times per year) but the suspended solids and pollutant concentrations are commonly very high, reaching over 1000 mg/ℓ of suspended solids. Due to increased imperviousness of urban watersheds, surface runoff events are more frequent, however, concentrations of pollutants are lower and pollutants are more diluted. Toxic pollutants from urban sources (mostly metalic compounds and hydrocarbons from traffic) are elevated over those from agriculture but this may be balanced or even surpassed by contamination of agricultural runoff by pesticides and herbicides.

Construction in urban areas is the most polluting activity and, frequently, few percent of the urban areas under construction are responsible for a larger portion of the suspended sediment yield in urban streams. On the other side of the spectrum, animal feedlots are the most polluting land use activity in the agricultural land use, yielding pollutant loadings that surpass any other

Table 2

Comparison of Water and Pollution Yields From Wisconsin Experimental Watersheds - 1980-81
(Source - Wisconsin Department of Natural Resources)

Watershed type	Watershed area ha	% Imperviousness	Runoff[1] coefficient	Pollution yield, kg/ha-year[2]		
				Suspended solids	Total P	Total Lead
Urban -						
Storm sewers						
Commercial I	11.7	77	0.58	718	1.48	1.53
Commercial II	18.2	81	0.69	1197	1.50	3.90
Residential I	14.6	57	0.40	487	1.12	0.90
Residential II	25.3	51	0.33	272	0.62	0.28
Residential III	13.3	50	0.33	161	0.54	0.21
Residential - 10% under construction	522	47	0.31	767	0.75	0.21
Suburban -						
Low density residential, partly sewered	4974	7	0.10	217	0.30	0.12
Agricultural I	3900	< 5%	0.06	752	1.11	id
II	1528	< 5%	0.14	743	0.74	id
III	2615	< 5%	0.09	470	id	id
IV	2144	< 5%	0.06	386	0.63	id

[1] Annual runoff volume/rainfall volume

[2] Excluding winter

[id] Insufficient data

nonpoint pollution sources.

Most of the agricultural and urban,highly impervious areas contributing runoff require management to reduce pollution loadings. Exceptions (lands with a low pollution potential) include lower density urban residential lands, parks and recreational areas, agricultural farming on low slopes and low cattle density pastures.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 19 Aspect number 3

ECONOMICAL ASPECTS OF GROUP RURAL WATER SUPPLY SYSTEMS

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ABSTRACT

The system which supplies water to at least two rural communities is termed a group water supply system. Its essential feature is the necessity of water transit from a water supply plant to the individual villages. Feasibility of a group water supply system in a given area should be determined with economic calculation including a comparison of economic effectiveness index calculated for a group water supply system with the mean effectiveness index calculated for all local water supply devices to be replaced by the group system. Analysis of economic effectiveness was carried out on adopted models of water supply systems /capacity within the range of 75÷6000 m^3/d/, using the formulae which permitted to evaluate feasibility of the group water supply system and its economic range. Values of the system economic range were calculated for three adopted schemes of transit pipelines: radial, ring, and in-series mains. The results have shown that the in-series system is the longest, and the radial system the shortest one. In practice, selection of pipelines should be based on economic calculation.

Keywords: Water supply, group water supply system, transit pipelines, economic distance of water transit.

Application of a group water supply system in a given area is economically expedient when the economic effectiveness index of the system is better than that of all the local water supply devices to be replaced by the group system. This condition may be expressed with the following formula:

$$E_G^S + E_G^H + \sum_{i=1}^{n} \frac{Q_{Mi}}{Q_G} E_i^T \leqslant \sum_{i=1}^{m} \frac{Q_{Mi}}{Q_G} \left(E_{Mi}^S + E_{Mi}^H + E_{Mi}^T\right) \qquad /1/$$

where: E_G^S - index of economic effectiveness of the group water intake and water supply devices /facilities for water intake, treatment, storage and pumping, in zl/m^3/,

E_M^S - index of economic effectiveness of the local intake and water supply plant, in zl/m^3,

E^T - economic effectiveness index for water transit devices from the group water supply plant to the rural community /transit pipelines, and if necessary, zone pump stations, in zl/m^3/,

E_M^T - index of economic effectiveness of water transit devices in a local water supply system, in zl/m^3,

Q_G - mean 24-hour capacity of the group water supply plant, in m^3/d,

Q_M - mean 24-hour capacity of the local water supply plant, in m^3/d,

E_G^H - economic effectiveness index of water pumpage associated with level differences between the area of the group system water intake and the area of water users, in zl/m^3 /this index does not take account of capital expenditure on pump stations, included in E_G^S/,

E_M^H - economic effectiveness index of water pumpage associated with level difference between the area of local water plant intake and the area of water users, in zl/m^3 /this index does not take account of capital expenditure on pump stations, included in E_M^S/,

n - number of communities to which water is supplied with transit pipelines from the group water supply plant /assuming that the group water supply plant is located in one of the communities serviced by the plant, n=m-1/,

i - serial number of the community.

The economic effectiveness index for water transit devices in the formula /1/ consists of two components:

$$E^T = E^{T'} + E^{T''} \qquad /2/$$

where: $E^{T'}$ - effectiveness index of transit pipelines, in zl/m^3,

$E^{T''}$ - effectiveness index of zone pump stations, in zl/m^3.

As the economic effectiveness index of transit pipelines, the index related to 1m of pipeline /single or double/ was adopted, expressed with the following formula:

$$e_T = \frac{E^{T'}}{L} \qquad /3/$$

where: e_T - unit economic effectiveness index for transit pipelines, in zl/m^3, m

$E^{T'}$ - as in the formula /2/

L - length of transit pipeline, m.

The economic effectiveness indices in the formulae /1/ and /2/ may be calculated using different methods, according to the adopted principles of evaluation of the projet effectiveness. The formula /1/ is adjusted to such an index which expresses the expenditure /benefits ratio/ in such case, the lower the effectiveness index, the higher the economic effectiveness of the project . In this paper, the following simplified formula, currently applied in Poland for evaluation of so-called "non--productive" projects, was adopted in calculation of the effectiveness index:

$$E = \frac{J\,/r+s/\, + K}{W} \qquad /4/$$

where: E - economic effectiveness index, in zl, per unit of usable effect /in this case per 1 m^3 water supplied, then in zl/m^3/,

J - capital expenditure on the group of facilities under consideration, frozen during construction,

r - discount rate in the year^{-1},

s - mean depreciation rate of the plants under consideration, in the year^{-1},

K - predicted annual operation cost, reduced by depreciation of resources, zl/year,

W - size of the usable effect expressed in natural units, in this case in the quantity of water supplied to the users, in m^3/year.

Significant for the feasibility of a group water supply in a given area may be also the distance and conditions of water transit, as well as the number of communities serviced, their size expressed with the quantity of water supplied. In view of this, analysis of economic effectiveness of group water supply systems covered different capacity of water supply systems /range: 75 m^3/d through 6000 m^3/d/, different number of communities serviced, and three different schemes of transit pipelines, i.e.:

- radial scheme, in which water is distributed to each community with a separate pipeline from the group water supply plant,
- ring scheme, in which water is distributed to communities with a closed system of pipelines,

- in-series scheme, in which water is distributed to all the communities sequentially with one pipeline.

Diagrams of the analyzed systems supplying water to four rural communities with Q_M water requirements are presented in Fig.1.

Three different models of technical solutions of water intakes and water supply plants were analyzed:

1. W1/G - water supply plant with ground water intake and treatment, with one-step pumping /subaqueous pumps in wells, iron-removing apparatus/ and tower tank,
2. H2/G - water supply plant with ground water intake and treatment, with two-step pumping /subaqueous pumps, iron--removing apparatus, double-stage pumps and hydrophores/,
3. H2/P - water supply plant with surface water intake and treatment, double-stage water pumping /1st step pumps, treatment facilities, plant distribution tank, 2nd step pumps and hydrophores/.

Economic characteristics, illustrating the relationship between the project economic effectiveness and its capacity, expressed with the mean daily quantity of water supplied, were prepared for the above models of water supply plants, together with water intakes, transit pipelines and zone pump stations. Economic characteristic of the adopted technical models of water intakes and water supply plants is presented in Fig.2, and that of the transit pipelines in Fig.3.

The term of economic range of the group water supply plant may be interpreted as the length of the transit pipeline distributing water from the plant to the farthest community. As such, the economic range of the group water supply system may be calculated using the formula /1/, taking account of the formula /3/. The estimated range values for the simplest case, when the group water supply system services only two communities, the water supply plant being located at the larger community, are presented in Table 1. The calculations have shown that in the system of water supply to two communities of different size, the economic range of water supply plant increases with the increased capacity of the group water supply system.

For the radial scheme of water transit devices, the general formula for the economic range of the group water supply system is as follows:

$$L \leqslant \frac{/E_M^S - E_G^S/ \frac{m}{n} - E^{T''}}{e_T} \qquad /5/$$

where: L - average economic distance of the individual rural communities the same water requirements from the group water supply plant, equal to the water supply range, in m,

- the other symbols as in the formulae /1/, /2/, and /3/.

For the ring and in-series schemes of transit pipelines, the values of the economic range were calculated assuming the identical distance between the rural communities and the identical water demand, Q_M. The values of the economic range of the group water supply system, without zone pumping stations, are presented in Table 2.

Necessity of applying the zone pumping station is determined by configuration of the terrain along the water transit route, and total head loss in transit pipeline during maximum flow. The calculations carried out for the Polish conditions and flat terrain have shown that with the zone pumping stations the final value of the water supply plant economic range may be reduced to a significant extent, even by 50 percent, as compared to the original value.

CONCLUSIONS

Calculations of economically justified size of the range of the group water supply plant, carried out on three technical models have shown that with the same plant capacity, the longest range is found in the in-series scheme, followed by the ring scheme, the radial scheme being of the shortest range. To secure water supply, the radial and in-series schemes should be designed using double transit pipelines. This brings about significant reduction /by 40 percent/ of their economic range. However, in practice the in-series schemes may occur frequently, as a result of the spacial distribution of rural communities, e.g. along the rivers of transportation routes. In such case the most economic solution is to locate the supply source possibly in the centre of the system.

Feasibility of the group water supply plant in a given area should be determined by economic analysis containing comparison of the effectiveness index for the group system with the mean effectiveness index for the local water supply plants. Choice of a transit pipeline scheme should be also determined by economic calculation. The scheme should be also most favourable as regards operation reliability.

Decision on construction of group water supply systems, apart from economic effectiveness, should also take account of other factors. Lack of possibilities of drawing water from the area close to the community, because of too small resources or their inadequate quality, may be also of significance. The prerequisite for application of rural group water supply systems should be improvement of the standard of living and development of agricultural production in the area.

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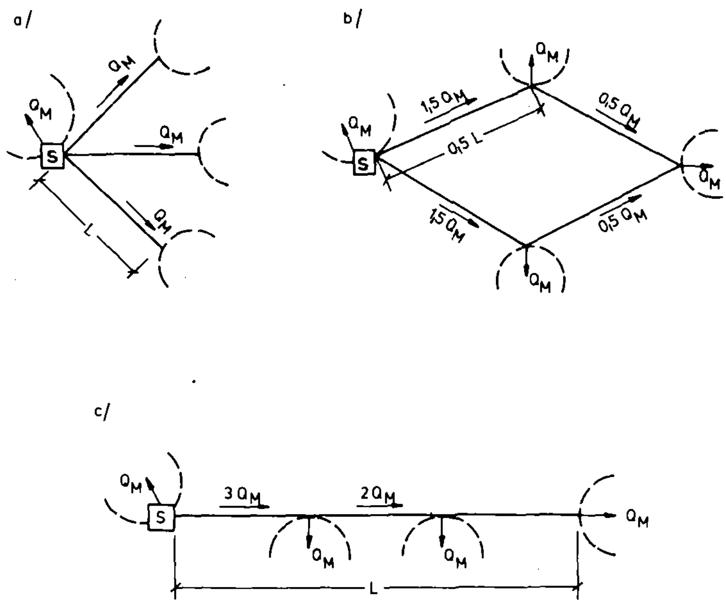

Fig. 1. Analytical diagrams of transit pipelines a/radial scheme, b/ ring scheme, c/ in-series scheme, S–water supply plant, L–scope of the group water supply system, Q_M–water demand of rural settlement.

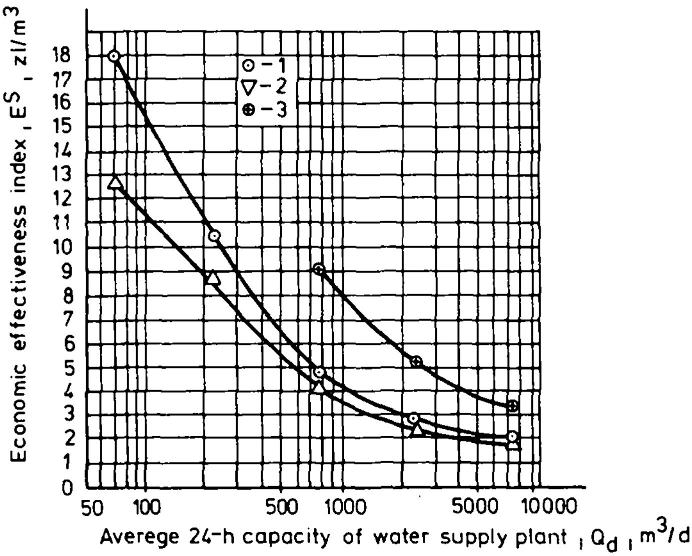

Fig. 2. Relationship between the economic effectiveness index of the model systems and their average 24-h capacity: 1 – indices calculated for H2/G/model, 2 – indices calculated for W1/G/model, 3 – indices calculated for H2/P/model.

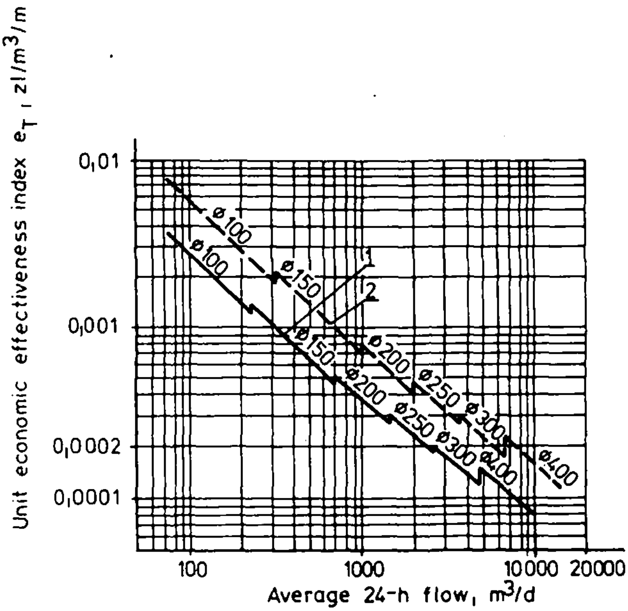

Fig. 3. Relationship between unit economic effectiveness index of transit pipelines and average 24-h flow: 1 – single pipelines, 2 – double pipelines.

TABLE 1
Economic range of the group system supplying water to two rural communities
/water transit to the smaller community/

System model	Average 24-h water demand of communities, m^3/d			Economic range of the group water supply system L, km	
	Q_{M1}	Q_{M2}	$Q_G=Q_{M1}+Q_{M2}$	Single transit pipeline	Double transit pipeline
H2/G/	75	225	300	3,7	1,7
	75	375	450	4,5	2,3
	230	690	920	6,2	2,8
	770	2310	3080	4,8/P/	3,5
	770	3850	4620	5,1	3,7
	1500	4500	6000	6,0/P/	3,8
H2/P/	770	2310	3080	10,0/P/	6,5/P/
	770	3850	4620	10,0/P/	6,6/P/
	1000	3000	4000	7,3/P/	7,1
	1000	5000	6000	7,3/P/	7,4
/P/- zone pump station					

TABLE 2

Economic range of the group water supply system calculated for systems supplying water to four rural communities

System model	Plant capacity, m^3/d		Economic range of the system L, km		
	Local, Q_M	Group, $Q_G=4xQ_M$	Radial scheme	Ring scheme	In-series scheme
W1/G/	75	300	1,9	3,1	5,8
	230	920	4,9	7,8	14,6
	770	3080	5,5	8,7	14,3
	1500	6000	5,1	8,4	13,8
H2/G/	75	300	3,1	5,4	9,9
	230	920	6,1	9,9	18,2
	770	3080	6,2	9,8	16,3
	1500	6000	6,3	10,3	15,8
H2/P/	770	3080	12,6	19,8	32,8
	1000	4000	14,2	20,7	35,5
	1500	6000	14,1	23,0	37,5

Note: Calculations carried out for single transit pipelines.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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REGIONAL PLANNING METHODOLOGY FOR MUNICIPAL/RURAL WATER SUPPLY

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ABSTRACT

In a comprehensive regional planning setting, it is important but difficult to make "best" decisions as to type of water development and transmission facilities, their capacity, and investment timing. Since there is never enough money to construct all needed projects, they must be prioritized (thereby changing the structure of groups which may optimally be connected to a single source) and priority sequences must be easily modified in response to changing political, institutional, and technical constraints as available information increases. Each of these changes requires the solution of a new possibly very large optimization problem. A planning methodology is presented which has the following capabilities: 1. The planning problem is efficiently converted to a mathematical model solvable by an appropriate solution algorithm via a truly generalized input data model generator. 2. Alternative solution algorithms include mixed integer programming, nonlinear and iterative linear programming. Recommendations as to which approach is appropriate vary with size of the problem and stage of planning (preliminary screening or more detailed). 3. The software is interactive and user friendly in order to allow very rapid modification in response to "what if" type questions from decision makers. Application of the procedure to an example problem is presented.

Keywords: Optimization model, water supply planning, rural water supply, model generators, integer programming, nonlinear optimization

INTRODUCTION

The basic components of any municipal or village water supply system can be categorized as supply sources, transmission facilities and demand zones. The supply sources can be further disaggregated as being springs, wells, surface diversions and the associated treatment facilities (if required) for each source. The development or expansion of these supply, treatment and transmission facilities which deliver water to a demand zone or to many demand zones represent a planning optimization problem which is solved daily by consulting firms or by other engineers employed by water utilities or government agencies. If the system involves very few potential alternatives, the optimization problem is handled well by traditional engineering economic calculations. If the system includes a very complex urban setting or a regional problem with many village systems (with possible interconnections) then traditional calculation approaches may result in a far from optimal solution since only a few intuitively selected alternatives can be compared.

A generalized model for this problem was first presented by Hughes (1973). A discrete decision model was proposed using a mixed integer programming (MIP) algorithm since many of the facilities required are produced in only standard sizes. An interactive user friendly software package call WASOPT was later developed for generating very efficiently the matrix required for any size of planning problem using the MIP structure (Hughes, Pugner and Clyde 1977). Transfer of this technology to many U.S. water supply engineers has been accomplished by short courses presented at Utah State University and the software has as a result been used by several consulting firms and water utilities in planning complex water systems.

The original WASOPT package had two principal constraints on its use: 1) It required use of a proprietary model generating and report writing language (Burroughs GAMMA). This restricted use to the Burroughs computer system. 2) The MIP solution approach works well for most problems but for extremely large problems such as regional planning of many village systems the number of integer variables can cause very large computer costs and can eliminate the possible use of personal computers even for moderately large problems.

The principal objective of this paper is to describe the current version of this methodology which will be referred to as WASOPT2. Both of the drawbacks of the original version have been overcome by the revised software package. The model generator requires only a Fortran 77 compiler and is therefore portable to most systems. WASOPT2 also includes several alternatives to the MIP model structure and solution algorithm--all of which are more efficient computationally than branch-bound and therefore allow solution of much larger problems. Another important capability of WASOPT2 is the concept of pre-optimization whereby the model generator selects facility capacities which explicitly balance pumping cost vs. capital cost (rather than requiring the user to intuitively select the capacities for consideration).

A complete description of the WASOPT2 software and an example application to a large proglem are given by Al-Eryani (1984). Only a summary of the principal concepts will be presented here. The paper will proceed in the following manner: (1) A mathematical description of the planning problem will be presented using very simplified notation to allow concentration upon the functional forms and concepts rather than rigorous detail. (2) The model generator and pre-optimization concept will be described. (3)

The various model structure/solution algorithms will be described briefly. These include: mixed integer programming, iterative linear programming (LP) by objective bounding, non-linear discrete programming, and non-linear continuous programming. (4) Application results and preliminary conclusions regarding a comparison of solution approaches will be presented.

PROBLEM DESCRIPTION

The water supply problem can be conceptualized as a network in which some nodes are sources, some are junctions, some are demand nodes and there is a possibility of transfers between demand nodes. A simplified version of this problem has been referred to in the literature as the fixed-charge transportation/transshipment problem (for example, see Gray 1971 and Steinberg 1970). The transshipment characteristic (initial destinations are not necessarily final) makes it much more difficult than the classical transportation problem. The water supply problem is also much more difficult than the usual transportation/transshipment problem because of demands which vary seasonally. This presents serious difficulties for most programming approaches because of the need to select alternative facility capacities prior to knowing seasonal flow rates (and therefore operating costs).

Consider the following discrete version of the problem:

Problem 1 formulation:

$$\text{Min } W = \sum_{m,k} (F_{mk} \cdot X_{mk} + F'_{mk} \cdot Y_{mk}) \qquad \text{(Fixed cost of proposed pipes)}$$

$$+ \sum_{n,k} F_{nk} \cdot X_{nk} \qquad \text{(Fixed cost of proposed source facilities)}$$

$$+ \sum_{m,k,t} (C^P_{mkt} \cdot Q^P_{mkt} + C^{P'}_{mkt} \cdot Z^P_{mkt}) \qquad \text{(Operating cost of proposed pipes)}$$

$$+ \sum_{n,k,t} C^P_{nkt} \cdot Q^P_{nkt} \qquad \text{(Operating cost of proposed source facilities)}$$

$$+ \sum_{m,t} C^E_{mt} \cdot Q^E_{mt} + \sum_{n,t} C^E_{nt} \cdot Q^E_{nt} \qquad \text{(Operating cost of existing pipes and source facilities)}$$

Subject to:

(1) Continuity at each source node:

$$Q^E_{nt} - Q^E_{mt} = 0 \qquad \text{(each t at each appropriate n-m combination)}$$

$$Q^P_{nkt} - Q^P_{mkt} = 0 \qquad \text{(each t and k at appropriate n-m combinations)}$$

(2) Continuity at each demand node:

$$\sum_k (\text{inflow-outflow}) \geq \text{demand (each demand node, each t)}$$

(3) Continuity at junction nodes:

A junction is modeled as a demand node with zero demand and the inequality in (2) replaced by an equality

(4) Capacity constraints

$$Q^E_{mt} \leq a_t \cdot b_m \qquad \text{(each existing pipe: each t)}$$

$Q^E_{nt} \leq a_t \cdot b_{nt}$ (each existing source; each t)

$Q^P_{mkt} \leq a_t \cdot b_{mk} X_{mk}$ (each proposed pipe; each k and t-direct flow)

$Z^P_{mkt} \leq a_t \cdot b_{mk} Y_{mk}$ (each proposed pipe; each k and t-reverse flow)

$Q^P_{nkt} \leq a_t \cdot b_{nkt} \cdot X_{nk}$ (each proposed source; each k and t)

(5) Finally, the integer constraints:

$x \ \varepsilon (0,1)$ all x's
$y \ \varepsilon (0,1)$ all y's

Solution of this problem by an MIP algorithm produces a global optimum. Solution by any other method except dynamic programming (which is not capable of handling large problems) guarantees only a local optimum because of concavity of the objective function. This is a generic difficulty of any non-linear problem where the objective is to minimize a function with economies of scale. The solution algorithms discussed subsequently therefore, although computationally much better than MIP, suffer from a common limitation--the possibility of getting trapped in a local optimum.

THE MODEL GENERATOR

Regardless of the solution method, manual preparation of the problem matrix (commonly called the MPS file), even for small problems, is cumbersome and vulnerable to human error both in the computations of the matrix coefficients and in the spelling and MPS format requirements. Furthermore, since the methodology proposed here involves solution of the same problem using a number of different formulations, a different form of the MPS file is needed for each problem version. For these reasons an efficient matrix generator is mandatory.

The WASOPT2 generator reads problem specifications from a relatively concise input data file from which it develops all of the data files necessary for each of four possible solution algorithms. In addition to four MPS files (linear portions of the four problem forms) and three files for non-linear data for various problem forms, the generator also produces a file which gives results of the pre-optimization calculations.

Consider for example, the very small problem shown in Figure 1. The three service zones (nodes 1,2,3) are currently served by existing wells and springs (nodes 4,6,7,9,10,11,13). Growth in demand has motivated consideration of developing possible new wells (nodes 5 and 8), a new treatment plant (node 12), and possible interconnections between the service zones (pipes 1 and 2). Including pipes from the new sources, two alternative capacities of the treatment plant and pipe 2 and possibility of reverse flow in pipes 1 and 2 there are 14 0,1 decisions to make (plus more than 100 continuous flow variables).

The input data file for this problem is shown in Table 1. The initial entries define the number and length of each season and the number of and type of nodes and pipes. Other sections such as DN, PP, EP, etc. define the seasonal demands, the proposed pipes and existing pipes, etc. For example the first row under PP indicates that pipe 1 is assumed to have direct flow from node 1 to 2, it is 3200 feet long, the downstream node reservoir is 94 feet lower than the upstream, only one alternative capacity--0.75 mgd is to be considered. Proposed and existing wells,

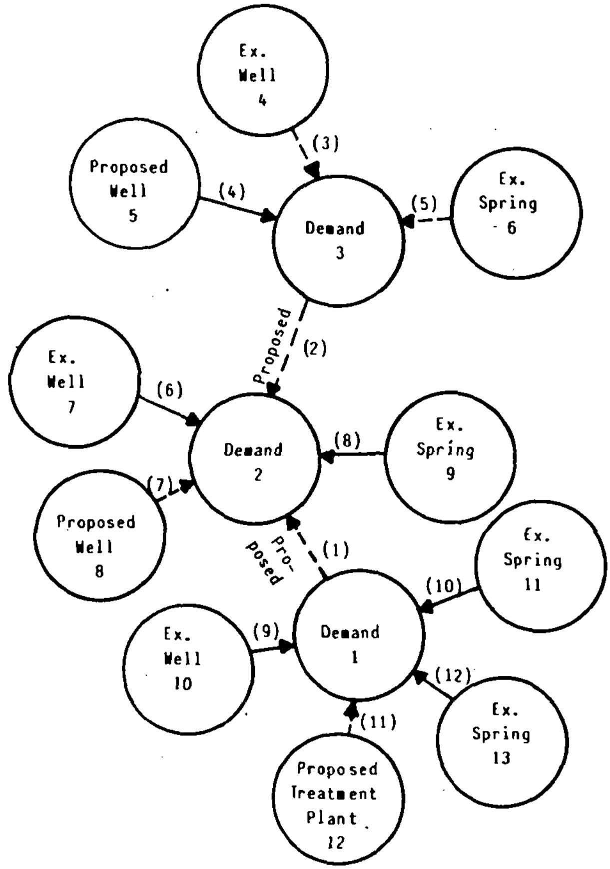

Figure 1. Example problem network.

springs, and treatment plants are similarly defined. Note that pipe diameters are not specified--only capacities. Given this definition of the network configuration, demands and capacities, the model generator selects pipe and pump size alternatives, calculates all investment and operating cost coefficients, and generates the MPS and non-linear data files for input to whatever solution algorithms are specified. Cost and friction loss coefficients are calculated by default functions included in the generator (all of which may be revised interactively at a users option during execution of the generator code).

SELECTION OF PIPE DIAMETERS

One of the very difficult sub-problems inherent in any water supply network problem with time varying demands is selection of pipe/pump capacities which will minimize fixed plus energy costs when seasonal flows are unknown a priori. What is desired but unavailable is a good estimate of average annual flow rates in each pipe for calculating total energy costs. Because of possible interzonal transfers which change seasonally, however, the

Table 1. Input data file.

```
 $genspec nseasons=5,days(1)=1,days(2)=62,days(3)=91,days(4)=121,
  days(5)=91,tnnodes=13,npp=5,npw=2,nep=7,new=3,nes=4
  ,nps=0,npt=1,net=0,ri=.06,yearpipe=40
  yearpump=15,yearwell=25,probname='tricity'$
 $powrspec gamma=65.,bhpinful(1)=1.34
  ,pumptype(1)=1,cstofful(1)=.05,mxprsure=150$
PKDADJST(1,1)=.42,PKDADJST(2,2)=.16,PKDADJST(4,1)=.035$
DN
1,2,1.7,1,.7,1.12
2,1.8,1.5,1.1,.76,1.12
3,3.8,3.2,2.2,1.5,2.
PP
1,1,2,3200,94,1,1,.75,1
2,3,2,4500,60,1,2,.6,1.,1
4,5,3,100,-467,0,1,1.1,1
7,8,2,100,-408,0,1,2.4,1
11,12,1,1000,200,0,2,.22,.72,1
EP
3,4,3,1.59,1.59,1.59,1.59,1.59,187,187,187,187,187
5,6,3,3.,3.,3.,3.,3.,6,6,6,6,6
6,7,2,1.,1.,1.,1.,1.,6,6,6,6,6
8,9,2,.4,.4,.4,.4,.4,6,6,6,6,6
9,10,1,4,4,4,4,4,250,250,250,250,250
10,11,1,.5,.5,.5,.5,.5,6,6,6,6,6
12,13,1,.9,1.,1.,1.,1.,300,300,300,300,300
PW
5,1,1,10,500,1.15,0
8,1,1,16,500,2.9,0
EW
4,1.59,1.59,1.59,1.59,1.59,6,6,6,6,6
7,.9,.9,.9,.9,.9,6,6,6,6,6
10,3.17,3.17,3.17,3.17,3.17,6,6,6,6,6
ES
6,1.3,1.5,1.29,.97,2.6,6,6,6,6,6
9,.3,.32,.32,.32,.32,6,6,6,6,6
11,.4,.43,.43,.43,.43,6,6,6,6,6
13,1.,1.,1.,1.,1.,6,6,6,6,6
PT
12,1.,.8,2.,2.,2.,2,.72,.22,3
GO
```

average use factor on a particular pump may be very unpredictable prior to solving the optimization problem. There is no rigorous optimal solution to this problem. Even with a continuous non-linear model where standard pipe diameters are ignored and pre-optimization is not necessary (no selection of pipe size alternatives before the complete optimization problem is solved) there are still difficulties involved because there is no unique relationship between pipe diameter and maximum flow rate when pumping is involved. In the continuous model setting, a logical approach would seem to be taking the derivative of the total cost function with respect to seasonal flows (Q_t), since flows rather than pipe diameters are the optimization problem decision variables. The total cost function can be written as:

$$TC = C_1 + C_2 + C_3$$

where the C_i are annual investment cost of the pipe, the pump and the annual operating cost respectively.

By proper handling of lengths, interest rates, static heads and other constants, these costs can be reduced to:

$C_1 = f_1\ (D)$ where D = diameter

$C_2 = f_2\ (Q_m, h_f)$ where Q_m = maximum flow rate and h_f = pipe friction

$C_3 = f_3\ (Q_t, h_f)$ $(t = 1, 2, \ldots T)$

In the case of gravity flow pipes a maximum velocity can be assumed, thereby implying a known relationship between maximum flow rate, pipe diameter, and cost C_1.

In the case where pumping is required however $[\partial(f_1(D))/\partial Q_t]$ (t = 1, 2, ... T) produces a very complex system of non-linear concave equations which must be solved simultaneously with other partial derivative equations. The principal difficulty relates to expressing D as a function of H_f and Q_t for back substitution the appropriate equations. A simplified - version of this system of equations, however, was developed for use in WASOPT2 by eliminating terms which have only a small impact on capacity. A comparison of diameters selected by the exact and the simplified functions showed very small errors.

Consider now, the discrete decision model where a few standard pipe size alternatives are to be selected (one for each Q_m alternative specified by the user) prior to solving the complete problem. WASOPT2 performs the pre-optimization process by assuming initially that the ratio of average annual flow rate to maximum flow rate will be equal to the ratio of average annual to maximum season demand for the entire problem area. This allows calculation of total costs (investment plus energy) for any pipe diameter. Those possible pipe diameters which would cause velocities exceeding 6 ft/sec are then eliminated. Total costs $(C_1 + C_2 + C_3)$ are then calculated for each of the remaining pipe diameters for each of the alternative Q_m values and the lowest cost is selected for inclusion in the optimization problem. The actual seasonal flows in a particular pipe may of course vary substantially from those implied by the demand ratio and may need correction later. This is done by calculating the actual ratio of average to peak flows for each pipe in the first solution and running the generator again. An example of this correction is the seventh line in the input data file (Table 1) which was added after the initial solution. There is still no guarantee that the ideal diameters will be selected (because actual flows may again change) but results indicate that no serious convergence problems are likely.

The pre-optimization results for the example problem are given in Table 2. Note that for pipe #2 the smaller alternative pipe will be considered as 8" diameter for direct flow and 6" for reverse flow. In reality, of course, only one pipe will be built but four outcomes in regard to this alternative are possible: (1) The smaller alternative may not be built; (2) If only direct flow is actually used--the 8" pipe will be built; (3) If only reverse flow is used--the 6" will be built; (4) If both reverse and direct flows (in different seasons) occur--the 8" will be built.

Table 2. Pre-optimization pipe size alternatives.

Mode of Flow	From-To	Pipe#	Length (ft)	Del H (ft)	Pumping Head (ft)	Npumps	Alt	Peak Flow (mgd)	Pipe Dia. Type	(in)	Velocity (fps)	Annual Fxd Cost	Annual Var. Cost
Div	1-2	1	3200	94	0	0	A	1.	1	8	4.4	5493	0
Rev	2-1	1	3200	-94	116	1	A	1.	1	8	4.4	9068	9339
Dir	3-2	2	4500	60	0	0	A	0.6	1	8	2.6	7725	0
Rev	2-3	2	4500	-60	109	1	A	0.6	1	6	4.7	7773	4247
Dir	5-3	2	4500	60	0	0	B	1.	1	8	4.4	7725	0
Rev	2-3	2	4500	-60	91	1	B	1.	1	8	4.4	10636	6498
Dir	5-3	4	100	-467	467	2	A	1.1	1	8	4.8	12942	47247
Dir	8-2	7	100	-408	409	2	A	2.4	1	10	6.8	20201	90094
Dir	12-1	11	1000	200	0	0	A	0.22	1	6	1.7	1184	0
Dir	12-1	11	1000	200	0	0	B	0.72	1	6	5.6	1184	0

SOLUTION TECHNIQUES

A basic objective of the research on which this paper is based was to determine if solution algorithms which are computationally more efficient than MIP could be developed. WASOPT2 now includes three such techniques, each of which have their own advantages and limitation, as well as MIP itself. The mathematical description of the basic optimization problem given previously (Problem 1) is in MIP form. Two of the techniques to be presented here are also discrete so that what is desired is a way of satisfying constraints type 5 without using branch-bound. The final algorithm is continuous so that discrete alternatives do not exist (the k subscript disappears).

MIP

The MIP algorithm solves Problem 1 without modification. A global solution is guaranteed if the search proceeds long enough and if the available computer system storage exceeds the possibly very large quantities required for memory of all unfathomed branches. MIP algorithms have a large literature including several textbooks. A brief description of the branch-bound concept in laymen's terms is given by Hughes et al. (1977).

Non-linear discrete technique

The central idea of this approach is to change the binary variables of the MIP problem version to non-linear variables, then attempt to force correct accumulation of the capital costs by setting an upper bound of 1 on all such variables and by a suitable transformation that will either:

(1) Force a (0,1) solution of the non-linear variables, or

(2) Cause the facility's capital cost to be fully incurred whenever its associated non-linear variable is in the range $1 \geq X > 0$.

The solution algorithm used in this method is MINOS, a widely used code which handles non-linear objective functions via any of several gradient search techniques (Murtagh and Saunders 1980).

Two transformations are included in WASOPT2, each of which is written by the generator into a separate file.

Transformation #1

One possible transformation is:

$$C_i = F_i X_i \ (101\text{-}100\ X_i) \qquad (1)$$

where

C_i = the contribution of facility (i) toward the objective function
F_i = the capital cost of facility (i)
X_i = the non-linear variable associated with facility (i)

The above transformation has the property that if $0 < X_i < 1$ then C_i is very large compared to its value if X_i is 1. Hence, one should expect the objective function to force all the X_i's (and Y_i's) toward the lower bound of 0 or the upper bound of 1. However, for this property to be effectively enforced, the problem has to be non-degenerate (see Ragha

Vachari 1969). Because of the introduction of the upper bounds on X and Y, and the use of alternative facility sizes for proposed facilities, the water supply problem is highly degenerate. That is, all the non-linear variables are basic either at zero activity or at an activity ≤ 1. Consequently, fractional values of X's (and Y's) are commonly found in the basis of the optimal solution and, hence, the non-linear (capital cost) part of the objective function is greatly exaggerated.

In spite of this limitation, a feasible solution can still be found by MINOS. In all likelihood, the solution will be a local optimum. The true objective function value is recomputed by requesting a final call to the non-linear function subroutine when and if an optimal solution has been found. During this final call, all X's that are in the range $0 < X_i \leq 1$ are set to 1 and their capital cost is correctly accumulated.

Transformation #2:

Another possible transformation is:

$$C_i = (F_i X_i)/(X_i + \varepsilon) \tag{2}$$

where

ε = a very small positive number (say 10^{-4}).

With this transformation it is obvious that C_i will be zero when X_i is zero and will be approximately equal to F_i whenever $0 < X_i \leq 1$. Hence, the fixed cost will be incurred without requiring X_i to be $(\bar{0},1)$. Again, as was the case in the previous transformation, once an optimal solution has been found, all the basic nondegenerate X_i's (and Y_i's) are set to 1 and the capital cost portion of the objective function is recomputed.

We can now define Problem 2 (the non-linear discrete problem) by deleting constraints type 5 and adding constraints 6 and 7 as follows:

6. All X and Y variables ≤ 1.

7. Either of the transformations described above (on both X and Y variables).

Objective Bounding Technique

Because the solution of the non-linear discrete and non-linear continuous problems are not guaranteed to be globally optimum, it will be useful to obtain a lower and an upper bound on the global optimum value of the objective function. Such bounds will give the analyst an idea of how good or bad the other may be as well as providing a starting point for an additional method of optimization. This method follows a concept suggested by Balinski (1961).

The lower bound (W_ℓ) is the absolute minimum value that the objective function can assume when the binary variables are allowed to take any value in the range $0 < X_i \leq 1$ and the problem is solved as an ordinary LP problem without regard to the fixed costs. That is, the fixed costs are allowed to be proportional to the values of the X_i's (and Y_i's).

The upper bound (W_u) is then obtained by forcing all the nondegenerate X_i (and Y_i) variables in the lower bound's solution basis to a value of 1 and solving the problem again as an ordinary LP problem. Because of degeneracy, however, the basis of W_u may contain new X_i (and Y_j) variables (at fractional values) that were not in the lower bound's basis. This is not common, but if it occurs, the new fractional valued variables are fixed at 1 and the new LP problem is solved. The result is a new candidate for a good solution which because of the fixing of some X and Y variables at their upper limit will rarely be a global optimum.

Problem 3 can therefore be defined as Problem 2 with the last type of constraint (type 7) removed. It is therefore an LP problem which is solved at least twice.

The solution described above can usually be improved by comparing the actual maximum flow rate to that specified for each alternate pipe and source facility constructed. If the difference is significant the specified capacities should be reduced, the model generator run again, and the new LP problem solved as before. This should be done for any of the discrete problem techniques including MIP. The correction in average to peak flow rates as discussed previously should be made at the same time. Also, if desired, the problem can be made much smaller by eliminating from the data file all alternative capacities except those selected during the first solution. This is especially recommended for the MIP problem.

A slightly modified version of this procedure is to obtain the initial LP W_ℓ and then adjust capacities of oversized facilities and obtain a new LP solution before fixing fractional valued X's and Y's to 1. This can sometimes achieve a better solution than the previous approach.

Continuous non-linear technique

The continuous problem has no discrete alternatives and therefore no pre-optimization. The continuous problem (Problem 4) has a totally different objective function than Problem 1. There are no X or Y variables. The capital costs as well as operating costs are defined as functions of flow rates (the Q's and Z's) as discussed previously (the $C_1 + C_2 + C_3$ functions). The constraints are different in that the type 5 are deleted and the type 4 constraints for proposed facilities have the X and Y variables deleted.

This non-linear continuous problem is solved by MINOS. The pipe sizes and costs are then adjusted by rounding up to the next largest standard diameter.

RESULTS

WASOPT2 has been applied to a few problems to date including one with 23 separate but connectable community systems. This fairly large problem (103 binary variables and several hundred continuous variables) required from 1 to 10 minutes of CPU time (on a VAX 11/780 system) for the various solution techniques described here except for MIP which took much longer. The MIP run was aborted after 28 munites of CPU without having verified that the current solution was optimal. It apparently was not optimal since the objective bounding algorithm found a slightly better solution after 9 minutes (including 3 iterations).

A surprising result was a large variability in solutions achieved by different algorithms--thereby indicating the potential significance of local optima in producing bad solutions. The continuous non-linear algorithm, was extremely efficient (less than 2 minutes of CPU) and seems mathematically more elegant; however, it produced the worst solution in terms of the objective function.

The very voluminous problem description and solutions for the large 23 city problem are presented by Al-Eryani (1984). The following results summarize the solutions to the much smaller 3 service zone problem presented in this paper (Figure 1). Because of space limitations, the seasonal flows (5 in each pipe) cannot be displayed here--rather only the capital investment decisions are included in Table 3. The various solutions generally look quite different in terms of configuration and capacity; however, they are quite similar in terms of total annual cost. Again, the continuous solution is worst but only 11 percent higher than the global optimum obtained by the MIP algorithm. The non-linear discrete and objective bounding solutions are only 1 percent and 4 percent, respectively higher than the MIP solution. Again the frequency of local optima by techniques other than MIP is demonstrated, however, in this case good solutions are obtained by all except the continuous algorithm (the objective function is apparently relatively flat in the region of reasonable solutions.

The CPU time required for the small example problem varied from a few seconds to about 1 minute.

CONCLUSIONS

The usual difficulties associated with local optima for any minimization problem with a concave objective function are present and can produce bad solutions for the general water supply planning problem described here. For the two problem applications discussed here the local optima difficulties were more serious for the larger problem for some but not all of the solution algorithms. The objective bounding approach produced the best and what appears (but is not proven) to be a very good solution to the large 23 city problem. The non-linear discrete approach was closest to the global (MIP) solution for the small 3 city problem. The continuous non-linear solution was consistently the worst in terms of objective function (but best in computational efficiency).

The MIP algorithm has the significant advantage of producing a global optimum if allowed to continue to the verified best solution (which can involve a very high cost and/or an unavailable storage requirement for large problems). The global optimum is not important per se and should not be pursued as if it were the Holy Grail. As demonstrated by the 3 city problem, more than one very good solution was obtained and the objective function appeared to be flat in the vicinity of the global optimum. Indeed, other good solutions may in fact be preferred due to non-economic reasons (such as reliability) which are difficult to model. However, a global solution is always reassuring if one is not sure if a particular solution is within that flat region.

The WASOPT2 software has the following very desirable capabilities:

1. It can produce an MIP global optimum when such a solution is obtainable at reasonable cost.

2. When a global optimum is not obtainable, local optima produced by several different techniques, all of which are computationally efficient can be compared. The probability that at least one of these will be a good solution appears to be very high.

Table 3. Example problem partial solutions.

Facility			Solution Algorithm			
Type	No.	Capacity (mgd)	MIP	Objective Bounding	Non-linear Discrete (Trans. No. 2)	Non-linear Continuous
New Pipes	1	0.75	6" (D&R)*	6" (D&R)	8" (R)	10" (R)
New Pipes	2A	0.6	8" (D)	-	-	8"
New Pipes	2B	1.0	- -	8" (D&R)	-	(D)
New Pipes	4	1.1	8" (D)	8" (D)	8" (D)	10" (D)
New Pipes	7	2.4	-	-	10" (D)	8" (D)
New Pipes	11A	0.22	-	-	-	-
New Pipes	11B	0.72	-	-	-	-
New Wells	5	1.15	yes	yes	yes	yes
	8	2.9	-	-	yes	yes
Treat. Plant	12A	0.72	-	-	-	-
	12B	0.22	-	-	-	-
Objective Function (1000$)			131.1	136.1	132.3	145.7

*D = Direct flow
R = Reverse flow

3. Input data for all of the solution techniques are produced by executing the model generator only once and using a single compact initial data file.

4. Any intuitive solution can be easily compared to solutions obtained by these algorithms by fixing the binary variables.

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LIST OF SYMBOLS

a. Indexes:

t = season index (season 1 is always peak day)
k = alternative capacity index
n = node number
m = pipe number

b. Decision variables:

X_{nk}, X_{mk} = 0,1 variables indicating construction of proposed source facility (well, spring or treatment plant) and pipe respectively

Y_{mk} = 0,1 variable indicating construction of pipe equipped for flow in reverse direction of that assumed for X_{mk}

Q^P_{mkt}, Q^E_{mt}, Q^P_{nkt}, Q^E_{nt} = flow per day in proposed or existing pipe m and from proposed or existing source n respectively during season t

Z^P_{mkt} = flow per day in proposed pipe m in reverse direction

Constants:

a_t = number of days in season t

b_m, b_{nt}, b_{mk}, b_{nkt} = daily capacities of existing pipe m or source n and proposed pipe mk or source nk during season t (sources may be constrained either hydraulically or hydrologically)

F_{mk}, F'_{mk}, F_{nk} = annual fixed cost of direct and reverse flow pipe mk and source nk respectively

C^P_{mkt}, $C^{P'}_{mkt}$, C^P_{nkt} = unit cost of operating pipe mk (in direct and reverse flow) and source nk during season t

C^E_{mt}, C^E_{nt} = unit cost of operating pipe m and node n during season t

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 22 Aspect number 11

WATER RESOURCE MANAGEMENT IN AREAS SUBJECT TO LAND SUBSIDENCE

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ABSTRACT

An extreme, often irrational exploitation of underground water resources in a great number of areas resulted into the emergence of alarming subsidence. Such occurrences are most frequently reported in the Po Valley and caused considerable damage, especially in the coast areas (Po Delta, Ravenna, Venice, etc).
After briefly reviewing the main factors responsible for this phenomenon, the results of some experimental studies carried out at the Mining Science Institute of the University of Bologna are reported. Special emphasis is placed upon the compressibility features of some soils in the Po Valley and how their compaction is affected by salinity variations.
A few instances of subsidence which occurred in the eastern erea of the Po Valley as a result of underground water withdrawal are reported and the most severe damages to the environment and, in particular, to agriculture, are described.
Eventually, some investigations conducted in the Emilia-Romagna Region with a view to working out a sensible scheme of action are referred to as well as suggestions advanced as to how a rational integration between underground and surface waters should be achieved.

Keywords : Subsidence in the Po Valley, subsidence due to gas production, subsidence due to ground-water production, salinity variations, economic impact of subsidence.

INTRODUCTION

Soil sinking is caused either by naturally-occurring phenomena or by man's intervention. Natural causes are the slowly-progressing compaction of sediments and the motion of the Earth's crust. This is a considerably slow process with low speed variation rates.

This paper is mainly concerned with the subsidence instances occurring in the Po Valley and, more specifically, in the Emilia-Romagna plain (Fig.1).

In these areas, over the period 1880/1950 the values of naturally-occurring subsidence ranged from 1 to 2.5 mm/year, as was pointed out by Bertoni (1980), Montanari (1983) and Pieri and Russo (1980), with peaks of 4 to 5 mm/year in the Po Delta. If the positive eustacy of the Mediterranean Sea, estimated around 1.5 mm/year over that very period, is considered, soil lowering due to natural phenomena can be rated at a few millimeters per year, as compared to the average sea level (with the only exception of the Po Delta). This is, therefore, but a remarkably slight subsidence, if compared to the ones which have been taking place in the last 30 years in the same area and which exceeded this by many orders of magnitude. Hence, subsidence due to man's meddling with the environment is the most commonly reported case in these areas: load variations on the ground, resulting from the mushrooming of built-up areas, underground water and hydrocarbon withdrawal, land reclamation works. All these subsidence instances brought about considerable and, in quite a few cases, irreversible damage to the environment.

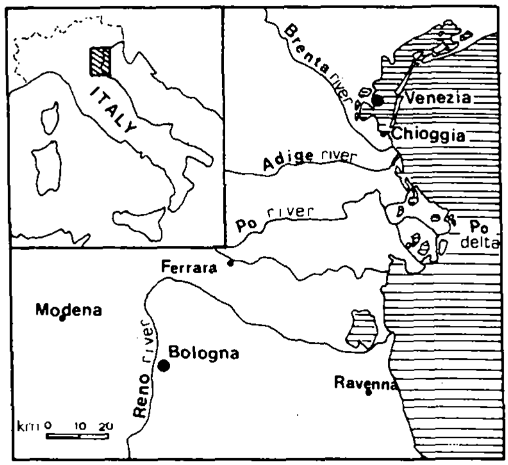


Fig. 1 - Eastern Po Valley

MAN-DEPENDENT CAUSES OF SUBSIDENCE

General survey

An increased loading on the ground is known to involve a certain degree of strain; this is, however, a localized phenomenon which, in the areas considered, has only played a minor role in bringing about subsidence on a large scale, even in coastal areas.

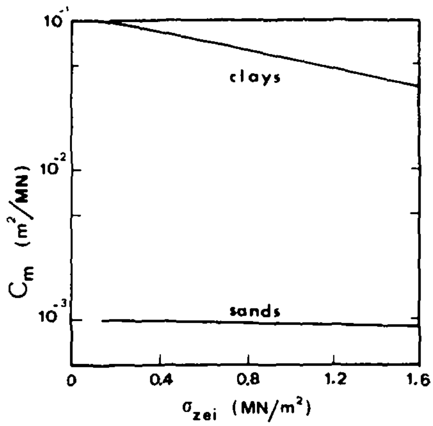
As to underground water withdrawal, it causes pore pressure to fall and, consequently, according to the well-known Terzaghi's theories, effective intergranular stress to increase: this, in turn, results in a volume decrease in the de-pressurized area with resulting strain in the surrounding medium, up to the surface. This drop in pore pressure spreads about depending on the medium permeability. Therefore, it occurs immediately in aquifers while it is extremely slow in the adjacent aquitards. A further effect of pore pressure drop should be mentioned here, i.e. the possibility of salt water intrusion in aquifers, thus causing significant variations in the features of the adjacent aquitards containing certain clay minerals. Sinking of the water table , as a consequence of land reclamation works, also increases the effective stress in the medium, the effects of which are virtually the same as those previously described. It should be noted, however, that, on account of their softness, these soils have an high compressibility. In land reclamation works, this phenomenon is then coupled with the oxidation of organic matter, which occurs when the latter comes into contact with the atmosphere. Based on the investigations carried out for over 20 years in the Bologna Mining Science Institute, the Author's attention will be focused in what follows on the effect of decreased pressure in deep water-bearing strata and of salinity variations in interstitial waters.

Effects of decreased pore pressure

In this paragraph the main factors which cause the occurrence of this phenomenon,-namely the geometrical dimensions as well as the mechanical and petrophysical features of both the layers undergoing the pressure drop and the adjacent ones- are highlighted, whereas, regarding the specific measurement and laboratory research techniques, reference is to be made to the two papers thereupon by Brighenti and Fabbri (1983,1984). As far as the size and depth of the low-pressure layer or, even better, their ratio is concerned, a distinction needs to be made between region-wide aquifers and hydrocarbon reservoirs (or particular, small-sized aquifers). The former are much wider than deep; here subsidence is studied, accurately enough, by means of monodimensional strain models; surface sinking at one given point virtually corresponds to the compactedness of the layers beneath. In a hydrocarbon reservoir, on the contrary, average radius and depth are, as a rule, of comparable dimensions; in this case, therefore, a three-dimensional approach, as clearly shown by Geertsma (1973), is required for subsidence to be studied. With respect to the soil mechanical features, it should be borne in mind that their rheological behaviour is predominantly elastic-plastic, with a marked prevalence of the plastic feature, which gives rise to mainly irreversible strains. In addition, the compressibility of layers basically changes depending not only on the mineralogical make-up of the same, but also on the sedimentation mode, consolidation state and depth. By way of example, the uniaxial compaction coefficient is shown in Fig.2

$$C_m = -\frac{1}{z}\left(\frac{\partial z}{\partial \sigma_{ez}}\right)_{\varepsilon_x = \varepsilon_y = 0}$$

(where z is the sample height, ε_x and ε_y the strains in the horizontal plane and σ_{ez} the effective stress), as a function of σ_{ez}, for some soils of the Po Valley saturated with fresh water. It is apparent that C_m of cohesive soils is significantly higher than the ones of sandy soils. Deep soils, which are, at the same time, saturated with salt water and much more consolidated, have a markedly smaller uniaxial compaction coefficient.



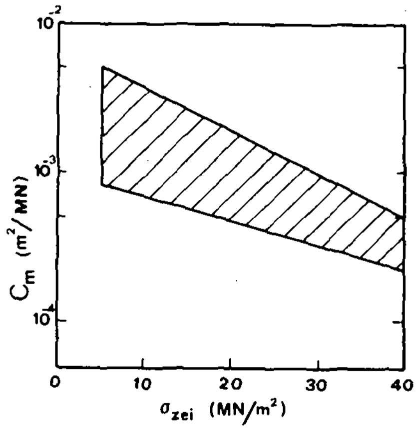


Fig. 2 - Uniaxial compaction coefficients for soils near Ravenna

Fig. 3 - Range in uniaxial compaction coefficients for deep soils in the Po-Valley

Moreover, as depth increases, C_m values of both sands and clays tend to be similar, as was enhanced in the works by Van der Knaap and Van der Vlies (1967) and Brighenti and Fabbri (1983). In Fig.3 the range in uniaxial compaction coefficient for deep Po Valley soils is given. Besides, it is worth pointing out that subsidence is affected by the mechanical features not only of the layer which underwent pressure drop, but also of the layers lying in the surrounding medium, which experienced the stress variation due to such depressurization. In this connection, the papers by Van Opstal (1874) and Gambolati, Gatto and Ricceri (1984) may be referred to.

Salinity variations in the interstitial water

The hazard of salt water intrusion from deep-lying aquifers or from the sea is encountered in the Po Valley. Since some researchers suggested that this might be responsible for a compaction of aquitards rich in active clay minerals, an investigation is being accomplished in this Institute with reference to local soils.

As to trial techniques, an introductory work by Brighenti and Fabbri (1982) should be consulted. Some results obtained in three different soils with sea water fluxing tests using an oedometer and having σ_{ez} = cost. are shown in Fig.4. The results obtained point out that salinity variations in interstitial water may, in some instances, lead to considerable layer compaction. It goes without saying that for the phenomenon to be accurately framed and outlined, the mineralogical and chemical properties of the material (such as clay and non-clay mineral composition, organic material, geologic history, specific surface of fines, ion exchange ability, pH, etc.) must be carefully defined. So far, no subsidence taking place in the Po Valley could be definitely ascribed to salinity variations.

SOME SUBSIDENCE INSTANCES IN THE PO VALLEY

Production of methane-bearing water in the Po Valley

This was the first noticeable region-wide subsidence which occurred in this country. For a critical assessment of facts, refer to the papers by Borgia, Brighenti and Vitali (1982). In the '50s and '60s a huge land expanse extending over 2,500 Km between the end tracts of the Po and Adige Rivers experienced this phenomenon. This was mainly due to the production of water with dissolved natural gas then under way (gas/water ratio: 1-1.4 Nm /m) from within 100 to 600 m deep layers. After a low-tone start, marked by poor outputs as early as 1929, water production rates soared in the '50s. In the years 1951-1957, this involved a piezometry fall ranging from 15 to 40 m depending on the sites and a max. 150 cm soil lowering over the same period, until a rate of about 30 cm per year was reached in the 1957/1959 period (the maximum sinking value recorded throughout the duration of this event exceeded 3 m) A Commitee set up by the Ministry of Public Works resolved to suspend production in the most affected area and, later on, once ascertained the close relationship between gas production and soil lowering (Fig.5), it laid down that production should be gradually abandoned in the years 1963 to 1965. After production stopped, piezometric levels rose and subsidence slowed down according to a decreasing exponential law. In 1982 a rate of about 2 cm per year was still recorded in some areas; this, however, may have been largely the result of works undertaken by the land reclamation syndicate in these years.

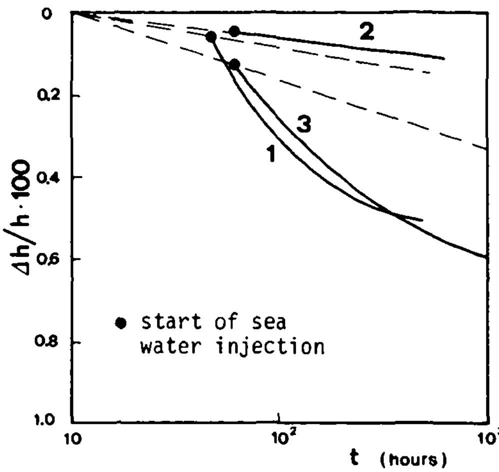


Fig. 4 - Sea water fluxing tests

1 - σ_{ez} = 1.5 MN/m^2, fine fraction ($<2\mu m$) = 21% (mainly Illite and Chlorite)

2 - σ_{ez} = 0.9 MN/m^2, fine fraction ($<2\mu m$) = 25% (40% Illite, 60% quartz and Calcite dust)

3 - σ_{ez} = 1.2 MN/m^2, fine fraction ($<2\mu m$) = 35% (mainly Illite and Chlorite)

Underground water withdrawal

In several areas of the Po Valley underground water withdrawal has by far exceeded the output capacity of local water-bearing strata. Many of the data on the areas described above are available for reference in the papers presented at the Conference on "Soil Subsidence and Resulting Problems" (Bologna, 1983) and at the "Third International Symposium on Land Subsidence (Venice, 1984). Here it is pointed out that over the 1952/1970 period, piezometries fell by 5 m in Venice towncentre, over the period 1950-1973 in the Ravenna district by 41 m and in the years 1943/1950 - 1970/1973 by 36 m

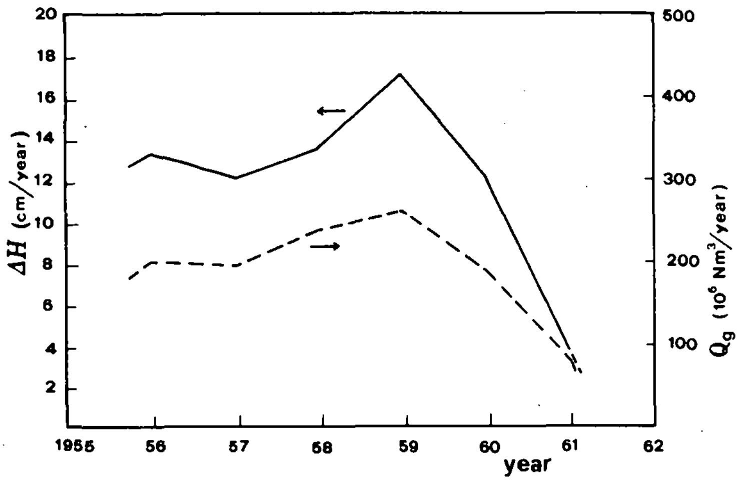


Fig. 5 - Gas production and average land subsidence in the Po Delta.

in the Bologna area with subsidence peaks of 9 cm, 84 cm and 140 cm respectively during the same periods. Severe soil lowering was also recorded in Modena (about 80 cm), the Ferrara Province and in the area surrounding Chioggia as well as in many other land patches of the area concerned. Needless to say, these phenomena, although they affected Venice but mildly, caused considerable damage specially to Venice and Ravenna, both sites being situated at sea level or, in some spots, even beneath it. The main reason for the occurrence of subsidence are easily found in pore pressure drop; it should be noted, however, that every single area exhibited its own particular features, depending on the subsoil type, the sediments' homogeneity and the possible existence of rigid substrates. In the Bologna area, for instance, the subsidence peaks were not matched by a particularly intensive drainage off strata lying in scarcely compressible gravelly layers. Moreover, sudden variations in lithology gave rise to significant differential drops in sites quite close to each other. In every single case different problems are to be coped with, which require special treatment lying beyond the scope of this dissertation. Suffice it to say that, whereas in Venice and partly in Ravenna, the steps taken successfully resulted in a re-pressurization of the strata and, consequently, in a sharp fall in the soil lowering rates, in the Bologna and Modena districts as well as in many other areas mentioned above, no considerable rate slackening was recorded.

Natural gas production off deep-lying reservoirs

The Po Valley is rich in natural gas reservoirs and some of them have been held responsible for extensive subsidence. Though it cannot be ruled out that they did account for some localized sinking of the soil surface, they are not thought to have caused considerable damage to the environment. The system of land and offshore reservoirs in the Ravenna area, however, should be treated separately. Detailed investigations on how and to what extent it affects local subsidence are under way, whose results, though, have not been published yet (as of May 1984) owing to the complex nature of the issue, above all related to the concurrence of several different causes.

SUBSIDENCE-RELATED DAMAGE TO AGRICULTURE

The damages caused by subsidence to the land are well known: on the coast, the moving back of the shoreline (according to Montanari (1983), for instance, in some coastal areas of Romagna a 20 cm lowering recorded in the decade 1957/1967 was followed by a 60 m regression of the shoreline), the reversal of slope and the resulting breaking apart of underground canal networks, injuries to buildings, etc. Limited to the area being dealt with, the following damages to agriculture are mentioned:

- rising of the watertable with respect to the plane of site resulting in considerable harm to the crops, such as the fruit tree blight recorded by Zambon (1968);
- need for the embankments of rivers and reclamation canals to be raised with the resulting danger of failure by piping and damage to the structures embedded in these embankments (e.g. bridges, supporting walls, wharfs, etc.);
- failure to let meteoric waters collected from entire districts naturally flow to the sea and, consequently, the need to build lifting plants;
- need for rebuilding canals which have acquired a reverse gradient;
- foundations' settlement of the existing structures and unavoidable cessation of use of the same;
- need for more powerful lifting equipment and, as a whole, increased expenditure for the running and maintenance of land reclamation plants;
- worsened water quality standards and consequent smaller yield of crops;
- severe injuries to irrigation works.

MEASURES REQUIRED

As the main reason for the development of subsidence lies in fluid and, in particular, water withdrawal from the subsoil, the most obvious step to be taken is to restore the underground water balance, in other words to refrain from exploiting strata more than they can yield, so as to stabilize piezometric surfaces or, even better, to cause them to rise. In this respect, it should be emphasized that these do not have to be brought back to their original levels at all; actually, subsidence being basically an irreversible phenomenon, this would hardly prove beneficial. Only seasonal or, occasionally, multi-seasonal ground water level fluctuations are allowed. One should never forget, in fact, that deep-lying strata are frequently supplied with water from far-away areas and, on account of the sluggish rate at which water seep through, replenishments may require quite a long time. Top-quality underground waters will only be meant for civil purposes as well as to supply isolated users. Further needs for industrial and agricultural uses (in addition to excess requirements, if any, for civil purposes, in which case, water must be suitably treated before use) will be met using surface waters coming from rivers or reservoirs obtained by means of retaining dams.

The recycling of sewage suitable for irrigation uses as well as the adoption

of close-circuit equipment in industrial plants will also be promoted. If this approach is to be pursued, an investigation of demand and supply in the areas being considered is needed. This is precisely what was done by the Emilia-Romagna Regional Authorities which, along with other local institutions, such as the Ravenna, Modena and Bologna Municipalities, carried out and made available a large number of investigations and suggestions. One example is given by the scheme worked out by IDROSER (1981) which, though incomplete and needing deeper investigations and reviews as far as recording and processing techniques are concerned, provides a great deal of information as a guidance towards a general rational planning.
This investigation is concerned with identifying regional aquifers and evaluating their capabilities, recording all water takings and present consumption, establishing their estimated trend up to 2001, assessing the number of infrastructures currently in operation and planned as well as their potentialities, searching for new primary and secondary sources of supply and possibly finding new ways of reclaiming waste waters.

CONCLUSIONS

In Italy, only recently has the Government's concern in the preservation of environment against subsidence been aroused, as was pointed out by Caia (1983). As far as hydrocarbons are concerned, the law favours the proper exploitation of reservoirs more than the preservation of environment. As to underground waters, subsidence is only given due consideration by the law-making organisms with regard to particular areas of the country, where conspicuous subsidence has taken place. Only the Ministerial Decree dated 21.1.1981, specifically ruling soil and rock investigations, refers to the opportunity to carry out prevention studies on the consistency of possible soil lowering with the stability and effectiveness of built structures in the event of underground water withdrawal; in this case too, the Legislator's "narrow-mindedness" stands out.
The main problem is the preservation of the environment and of the activities performed in it by man. On the assumption that in some special cases (such as the safeguard measures for the preservation of Venice and Ravenna towncentres) historical, artistic or environmental considerations demand the absolute preservation of environment, the Author argues that, in general, the rational integration between underground, surface and waste waters, especially in crop areas well above the sea level, must be accomplished on the basis of predominantly economical criteria, as was suggested by Jones and Warren (1976). This amounts to saying that, when computing the water overall cost, not only the cost of production and distribution, but also the cost of the measures required to control inevitable damage to the environment must be considered. This approach makes it imperative that the diverse demands are met by dividing supplies among all available sources, so that minimum expenditure is attained. Obviously, this cost must be imposed on the users, not so much based on the supply source, but on the water quality obtained. Following this guideline, a better and greater use should be made of surface and waste waters, as also pointed out by some investigations we carried out in the Romagna and Bologna plains. The results obtained can also be extended to areas having similar features, i.e. highly-populated and cultivated with intensive crop areas.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 25 Aspect number 4

MANAGEMENT AND TECHNIQUES FOR LOW-COST, LOW-DISCHARGE RURAL WATER SUPPLY

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ABSTRACT

The importance of providing safe water to the more than two billion people who lack this basic service in developing countries has been repeatedly stressed by national governments and international agencies. Among the activities of the International Drinking Water Supply and Sanitation Decade designed to address this problem are the United Nations Development Programme (UNDP) Global and Interregional Projects for laboratory testing, field trials and technological development of handpumps, executed by the World Bank.

Investment, operation and maintenance costs of mechanical rural water supply systems have made them prohibitive for most of the rural populations in the developing countries. Conventional handpumps and boreholes lead to early and frequent failures and rely on mobile maintenance teams which again create a prohibitive situation in many, if not most, regions. A new strategy had to be adopted to develop the technology, motivation, training, education, and incentives for the delegation of management responsibility to the villages. Adequate water supply for all the rural poor worldwide is a major objective; only low-cost options present a realistic solution. An enormous magnitude of costs can be saved through low-cost options.

Handpumps installed in wells where groundwater is easily available provide one of the simplest and least costly methods of supplying the rural population with water. Therefore, the program of the Decade has placed special emphasis upon handpump development and installation. Meeting the goals of the Decade would require the manufacture and installation of about five to seven million handpumps, each serving an average of 150 - 200 persons.

BACKGROUND

More than seventy percent of the over three billion people in the developing countries do not have access to adequate supplies of safe water and adequate sanitation facilities. The more than two billion who lack these basic services include 1.5 billion in rural areas.

The importance of providing safe water to these people has been repeatedly stressed by national governments and international agencies. Recognizing the urgent need for improved water and waste management, the United Nations has declared the 1980s the International Drinking Water Supply and Sanitation Decade (IWSSD). Among the activities of the Decade are the United Nations Development Programme (UNDP) Global and Interregional Projects for laboratory testing, field trials and technological development of handpumps, executed by the World Bank.

An ambitious goal has been established by the United Nations to provide drinking water to about 1500 million rural people who presently do not have access to adequate safe water. Handpumps installed in wells where groundwater is easily available provide one of the simplest and least costly methods of supplying the rural population with water. The program of the Decade has therefore placed special emphasis upon handpump development and installation. Meeting the goals of the Decade would require the manufacture and installation of about five to seven million handpumps.

Supplying pumped water to rural areas requires a significant capital investment in engineering, hydrogeology, borehole drilling, well digging, and pump installation. The effectiveness of this massive investment ultimately depends on the proper performance of the pumps installed.

Despite research and development of handpumps already undertaken by manufacturers, governments, bilaterials and international agencies, a number of serious technological problems remain. These problems are manifested in poor design, unsatisfactory performance, shortened working life, and often in pump failure. There is also a lack of reliable data on handpump performance and on the comparative performance of different handpumps designs. These data are required to facilitate selection from among the array of available handpumps.

The UNDP/World Bank program for laboratory, field testing and technological development of handpumps addresses these problems. The main objective of the program is to improve the dependability and reduce the cost of rural water supply systems that employ handpumps, so that the majority of people in developing countries can have access to safe drinking water. The program is providing the necessary technological basis for the development of new low-maintenance and cost-effective handpumps for installation in developing countries.

Extensive field trials are being conducted in approximately fifteen countries in various regions of the developing world with the testing of some 2,000 pumps. Each test site encompasses a defined area and includes 25 to 50 pumps each of three to four different types, for a total of about 100 to 200 pumps per site. Handpumps are monitored by the project staff and the local project participants. Detailed data are being collected, analyzed and disseminated.

One of the main objectives of the project research activity is the development of Village-Level Operation and Maintenance (VLOM) pumps, which can be manufactured in the developing countries and repaired by trained village operators. Unlike the conventional pumps, the light, simple pumps can be repaired without incurring the delay and expense of employing heavily equipped, highly skilled mobile maintenance units.

The pumps to be field-tested have been procured through funds provided for handpump programs supported by international, bilateral and national agencies, as the UNDP funds are intended mainly to cover the technical assistance and project management expenditures. The following list is a summary of the support in each field trial country:

Region/Country	External Donor/Funding
East Africa:	
Sudan	UNICEF
Kenya	SIDA
Malawi	UNICEF, DANIDA
Tanzania	FINNIDA
West Africa:	
Ghana (2 trials)	BMZ/KfW, CIDA
Ivory Coast (2 trials)	CIDA, Government of Ivory Coast (World Bank project)
Niger	BMZ/GTZ
Upper Volta	Netherlands, USAID
South Asia:	
Bangladesh (2 trials)	CIDA, UNICEF
India	Government of India, UNICEF
Sri Lanka (2 trials)	BMZ/GTZ, CIDA, UNICEF
East Asia and the Pacific:	
People's Republic of China (2 trials)	BMZ/GTZ, Government of PRC
Thailand	Government of Thailand
The Philippines,	Government of the Philippines, (World Bank project)
Papua New Guinea	University of Technology
Latin America and the Caribbean:	
Bolivia	UNDP

To provide maximum impact and to encourage continuing support for the project objectives, the World Bank, as executing agency, is cooperating with other international organizations such as UNDP, UNICEF, WHO, UNEP, the major bilateral agencies, and others. In particular, the Project will coordinate with WHO/UNDP Water Supply and Sanitation Programs, the UNDP/World Bank Project on Low-Cost Sanitation and the UNICEF Rural Water Supply Programs.

It is expected that the overall Project will make significant contributions to the programs for low-cost water supply in developing countries. The Project will develop a standard methodology for handpump testing leading to identification of effective pumps. The Project will provide local training and technical assistance to district-level handpumps operation, maintenance and monitoring teams. Manuals will be prepared to assist in the selection, installation and maintenance of pumps. The Project will also promote and assist in the development and local manufacture of appropriately designed handpumps.

It is anticipated that the Project will enable governments to obtain greater benefits from funds available for rural water supply. Moreover, by improving the effectiveness of handpumps, the Project is also expected to encourage increased investment in rural water supply during the Decade and thereafter.

FIELD TRIALS

Following are brief synopses of the ongoing field trials being carried out under the INT/81/026 Project.

South Asia Field Trials

The objective of the South Asia Region Projects is pump evaluation based on total life cycle costs versus productive work capacity. A total of 500 pumps of ten different types are being tested (deepwell, non-suction, low-lift, suction mode, primarily piston). Field trials are taking place in Bangladesh, India, and Sri Lanka. The Projects are working closely with UNICEF in the testing work. One important output being sought is the development of a reliable, low-cost non-suction pump which may be applied where the minimum static water level of the groundwater table exceeds eight meters. Potential sites for such a pump include the entire delta area of Bangladesh. Accelerated R&D work is being carried out in an effort to produce a VLOM-Mark II--a collaborative effort in India. Work is also being carried out on manual small-plot irrigation.

West Africa Field Trials

The participating countries in the West Africa region with field trials executed by the INT/81/026 project management are Ghana, Ivory Coast, Niger, and Upper Volta. Mali is participating with field trials executed by other organizations (secondary field trials). Field trials in the region are concentrating on performance evaluation and water quality and evaluation. Water quality problems have arisen primarily in Upper Volta and Ghana.

Some initial data from the region indicate that the performance of pumps is much influenced by the quality of well construction (sand/silt, yield, dry wells) and pump installation (well pad, loose rising mains and rods, etc.), structure and organization of pump maintenance (responsibilities, preventive maintenance), and attitude of users (training of pump attendants, education campaigns). Non-acceptance of water by the population may be caused by unpleasant or uncommon taste, color change in foodstuff cooked with such water, and laundry stains.

East Africa Field Trials

Special emphasis in East Africa is being placed on the study of borehole/pump systems. From preliminary findings, it seems as if the single largest cause of handpump failure is borehole design. In the past, it was generally thought that no attention need be given to the design of boreholes for handpump supplies (i.e. low yields). Very poor gravel pack and well screen design has been the rule rather than the exception in most cases. Even the best pumps will have a poor maintenance record in "bad" boreholes while poorer pumps may operate for a considerable time in well-designed boreholes. In this region, trials are taking place in Malawi, Kenya, Tanzania and the Sudan.

East Asia and the Pacific Field Trials

Handpump field trials and technology development are taking place in the People's Republic of China, Thailand, the Philippines and Papua New Guinea. The conditions in the region are characterized by a majority of shallow and intermediate water tables, a large potential demand for handpumps for small communities and extended families, relatively high in-country capability for manufacturing and technology development, and, in some cases, the prospect of a later conversion of handpumps to power driven pumps. Special emphasis in the region is on small-scale irrigation by handpumps (primarily in the People's Republic of China).

Latin America and the Caribbean Field Trials

No full-scale field trials are yet being carried out in this region, as handpumps in these countries are used primarily by individual families instead of entire communities. An agreement in principle was reached at the beginning of 1984 with the Government of Bolivia for a field trial and possible technical assistance for local manufacturers.

Outlook for INT/81/026 Project Field Trials

The field trials taking place as part of the INT/81/026 Handpumps Project are proceeding well; initial results should be available within 12-24 months. Field trials will continue in the latter half of the IDWSSD, although some may be moved to other sites and focus on the well/pump package. Developing country manufacturers will continue to be aided in overcoming production problems and governments will be assisted in building the capacity to maintain pump evaluation after the project teams are gone. The project will see that a mechanism to continue the data gathering and dissemination will continue throughout the IDWSSD and perhaps beyond. Management will also formulate a handpumps marketing strategy and the means of implementing a large-scale distribution program of those pumps found appropriate under the VLOM strategy.

LABORATORY TESTS

The Consumers' Association Testing and Research (CATR) of the United Kingdom is conducting full-scale comparative testing on selected handpumps. The original laboratory test project, sponsored by the British Overseas Development Administration (ODA), tested 12 different types of pumps under simulated field conditions. The long-term test (4,000 hours of operation) provided results on performance, durability, user comfort and other engineering data. The present UNDP/World Bank project, which began in 1981, continues to test handpumps in cooperation with CATR, and is broadening the scope of its testing.

The test program is extensive and takes about 20 months to complete. It includes detailed inspection of the pumps as received, including their packaging, engineering assessments with suggestions for design improvements, and user trials. Tests carried out include endurance, performance before and after endurance, impact and handle shock where applicable. Assessments of ease of installation, maintenance and repair are also made.

CATR is now involved in plastics development work for VLOM handpumps. The laboratory is primarily using new designs, trying out some other external designs following detailed discussions with consultants from the plastics and rubber industries.

Since one of the most important elements in establishing an economic analysis of rural water supplies is a knowledge of the amount of water extracted by each pump, it was necessary to develop a means of measuring this output. CATR was asked to develop a water monitor which has now satisfactorily completed twelve months of testing in the laboratory and nine months of field trials in Lesotho. CATR has developed two systems, both of which are activated by the flow of water of the pump. One system is based on the use of the counting mechanism in a digital watch which totals the time of water flow from the pumps. The second system is based on the ionization of a column of mercury. A small gap in the mercury column moves up a scale indicating the time water has flowed. The water monitors can also serve to measure water flows in gravity irrigation schemes.

COST ANALYSIS

It is probably correct to state that the primary issues in rural water supply are that of cost, investment and maintenance. This is especially true of recurrent costs. Project after project has had to be deemed a failure because the resources necessary for its continued operation and maintenance have not been available. Yet very little attention continues to be paid to recurrent costs during the preparation of rural water supply projects.

Until the last few years, capital costs have not been a major issue. The prevailing attitude was that poor villagers could not be expected to contribute toward them and as a result there has been little or no cost recovery. Capital costs have typically been subsidized by governments or funded fully by aid agencies. The IDWSSD has, however, forced the realization that the global needs are desperately high and cannot be met at current conventional per capita investment levels.

The search for lower capital and recurrent costs is, of course, the context for the entire handpumps project. If global needs are to be met, then the technology with the lowest capital cost in most regions is essentially the groundwater source equipped with a handpump. If existing conventional handpumps can be made more reliable, they will require less maintenance and so their recurrent costs will fall. If VLOM pumps suitable for villager maintenance are developed, the present level of maintenance organization will no longer be necessary and recurrent costs will be drastically reduced. If pumps can be locally manufactured in developing countries, capital costs will be reduced, especially their requirement for scarce foreign exchange resources, and standardization policy can be enforced.

This general context for the entire project also provides the specific context for its cost analysis work. This has four components, designed to give quantitative substance to the general goals:

(1) the establishment of a data bank of rural water supply costs (both capital and recurrent) for a variety of technologies and countries;

(2) the detailed examination of handpump recurrent costs to determine which are the critical elements;

(3) the comparison of different handpump maintenance regimes and their costs; and

(4) the comparison of the costs of specific rural water supply systems.

Data for the project are being collected from primary and secondary sources, monitoring forms, response-soliciting request papers and studies in individual countries. Three conclusions that can be drawn from initial data are as follows:

(1) recurrent costs are higher than earlier believed and have an impact on total costs per household or water unit;

(2) the key element is the recurrent costs of the transport of mobile maintenance units and fuel for diesel-driven systems; and

(3) high proportional costs are accounted for by overheads of expatriates during execution or O&M management.

CONCLUSIONS

Most of the existing handpump technologies which are presently available and are being installed in the least developed countries (LDCs) may be classified as inappropriate as a result of frequent breakdowns that necessitate the use of mobile rotating district or regional units to pull out and repair the pumps relatively frequently while covering large distances. As it is inevitable that any handpump will need at least a certain amount of maintenance occasionally, the question of whether LDCs can rely on mobile (trucks/vehicles) maintenance units has become a major issue in rural water supply management. The concept is prohibitively expensive in terms of cost and logistics. Based on the concensus of a majority of experts, the focus of the UNDP/World Bank INT/81/026 Rural Water Supply Handpumps Project is the promotion of technical assistance and development and testing of VLOM handpumps--pumps that can be pulled out and repaired by trained village operators. These pumps are light, simple to use and manufactured by regular machine-tools and extruding machines.

The technology of VLOM pumps must be suitable for local manufacturing in the developing countries and for standardization and distribution of spare parts across the countries through many outlets. When manufactured locally, it is fairly safe to assume that even bilaterals giving earmarked assistance will not import other pumps, thus disturbing the whole standardization policy.

The long-range rural water supply management concept is therefore designed as follows:

(1) Training courses for village operators (men and women--stress on women in the future, as it is their responsibility and their problem if a handpump ceases to function).

(2) VLOM-type pumps locally manufactured.

(3) Distribution of spare parts to a large number of outlets--within bicycle distance from the village.

(4) Villages paying the District Maintenance Units a lump sum of X.

When the village takes upon itself the responsibility for the handpump, approximately 2/3 of the sum is reduced. (The village will now pay the trained operator for his work and for spare parts.) The remaining 1/3 will be sufficient for infrequent, general supervisory visits by the district team (possibly every 18 to 24 months) and for the replacement of the pump in due time.

Part of the general supervision function is to verify the O&M procedure in the village and prevent early need for replacement. When a village is found to be incapable of keeping its pump in order, the original lump sum will be imposed, and the Mobile District resumed.

The entire management system of billings and payment is, of course, much easier in those countries and villages where agricultural products are dealt with by marketing boards (like Cocoa and Coffee, for example), or in non-subsistence farming regions.

(5) An intensive education and social campaign should precede and follow the entire exercise. Undoubtedly, transferring maintenance function to men and especially women would be a difficult and lengthy process in many African and Asian societies, but when the cycle starts, it is possible to assume

that it may spread from one country to another, subject, of course, to local politicians' willingness to cooperate and to the leverage that bi- and multi-laterals will have.

Experts in the countries considered agreed that the task of promoting local manufacturing of VLOM pumps is the most important function of UNDP and the World Bank on the subject of handpumps. In most countries visited, however, the large-scale installation of handpumps is the only practical solution to water supply in the rural areas, and consequently, a large number of today's technology pumps are being, and will be installed. The results and improvements derived from our laboratory and field testing are therefore of the most immediate and highest importance.

Rural water supply based on groundwater and improved handpumps could present a reliable and potable source for 1-1.5 billion persons at a reduced cost of 1-2 USDollars per household of six persons per year. With such recurrent costs, reliable rural water supplies become affordable to the greatest majority of the rural population in the developing countries.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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HUMAN RESOURCES AND WATER SECTOR MANAGEMENT STRATEGY

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ABSTRACT

It is a sad fact that the developing world contains many examples of water supply schemes which upon completion fail to reach even a small fraction of expected levels of performance. The reasons given for these failures are numerous, and are often attributed to "inappropriate technology". But it can be argued that the true causes of many failures so attributed lie more in the realms of manpower deficiencies rather than technological problems.

The traditional approach to human resource development has often been to set up short training schemes for water supply staff, timed to coincide with completion of physical works. This approach fails to take account of the fact that training is not a once-and-for-all event, but, especially for managerial staff, is a continuous process of development. Furthermore, the activities necessary to create a fully competent water supply organisation go far beyond the relatively narrow confines of job training.

In addition to knowledge and skills employees need other things before they can perform their work satisfactorily : (a) motivation and (b) tools, materials, equipment etc. To tackle these issues effectively involves consideration of the whole institutional structure of the water supply sector, as well as the organisation of individual enterprises. All contributors to water supply development, especially donor agencies, national governments, water supply managers and consultants, have a role to play in ensuring that the potential of human resources is fully developed and utilised.

This paper is based on the author's involvement in water supply both in UK and in developing countries, primarily Indonesia. It attempts to look objectively at some of the issues and constraints to be faced during human resources development exercises, and some of the lessons learnt.

1. INTRODUCTION

It is a sad fact that the developing world contains many examples of water supply schemes which upon completion fail to reach more than a small fraction of expected levels of performance. The reasons given for these failures are numerous, and are often attributed to "inappropriate technology". But it can be argued that the true causes of many failures relate more to deficiencies in human resources i.e. the way in which human resources of the water sector are developed and deployed.

This paper attempts to address this issue, firstly in the general context of water sector management strategy, secondly in the specific area of human resources development.

The theme is pursued under three main headings :

Institutional arrangements
Organisation development
Development of the individual

Each of these can be considered as links in a chain - a chain on which hangs the realisation of an effective, on-going water supply system. Deficiencies in any link can seriously weaken the chain and lead to less effective, or even disastrous, results.

2. INSTITUTIONAL ARRANGEMENTS FOR WATER SUPPLY

Institutional arrangements for water supply vary from country to country :

(a) In some countries the responsibility for water supplies is split between two or more authorities. For instance, in Indonesia the Ministry of Health is responsible for rural schemes whereas the Ministry of Home Affairs carries responsibility for urban and semi-urban supplies.

(b) In other countries responsibility for both rural and urban supplies is borne by the same authority, which may for instance be a State Water Board or a National Body.

Whatever type of water supply system is under consideration, be it a supply for a single village, or a more extensive and complex system for an urban community, there are five basic requirements for management and operation :

(i) MONEY is required for staff salaries, energy bills etc.

(ii) MANPOWER is necessary to operate and maintain the system

(iii) MATERIALS are needed for maintenance, repairs, water treatment etc.

(iv) MACHINES, equipment and tools are required for carrying out day-to-day operations.

(v) METHODS and systems are needed to regulate activities.

All these elements are inter-dependent, and all have a bearing on the ways in which human resources can be developed and deployed.
The institutional arrangements must allow for meeting these needs, both at the time of commissioning of the water supply, and on an on-going basis. It is useful to look at some of them in more detail :

Money is of course the most important issue. The characteristics of a healthy enterprise are that it must be capable not only of meeting day-to-day commitments efficiently and effectively, but must also be able to plan and prepare for the future. Budgets are required for immediate operational needs, also for more forward-looking activities such as plant overhauls, renewal of system components, and manpower development.

Ideally a water supply system should be able to recover enough money from water sales to meet its financial needs. But water must be made available at a price which can be afforded by potential consumers otherwise it will not be sold. Balancing these two conflicting elements can be a problem for many water undertakings.

Urban water systems are more fortunate than rural schemes in this respect. In urban areas there is usually a stratum of the population who are relatively affluent. They can afford to pay water charges which not only cover the costs of the water supplied to them, but which also, through use of an appropriate tariff system, can subsidise supplies to poorer sections of the community.

Rural schemes are less fortunate. The proportion of affluent consumers is usually very small, and in the author's experience all but very simple systems have difficulty raising sufficient revenue from water sales to adequately provide for operation and maintenance.

Ideally, then, the institutional arrangements should allow for some form of financial subsidy for less affluent sections of the population, especially those in rural areas. In this way adequate budgets can be made available to ensure a satisfactory environment for staff, with sufficient tools and materials for their work and opportunities for self-development.

This situation can best be achieved by an arrangement such as described in (b) above, where a system of direct cross-subsidy between urban and rural can be adopted internally by the water authority. In situation (a) above, subsidy for rural schemes has to come as a grant from government. It is thus outside the direct control of local management and is more exposed to the winds of the political climate.

<u>Manpower</u> needs vary according to the size and complexity of the water supply system. Often a wide range of skills is required, and even a medium-large water undertaking may meet difficulties in recruiting and retaining people with the right level of educational background, and in finding the resources for training new recruits.

With regard to professionally qualified staff e.g. engineers and accountants, the water sector is in competition with other sectors of the economy. To recruit and retain these groups the water undertaking should be able to offer competitive salaries, and, to good performers, opportunities for career progression.

Rural schemes, on the other hand, usually can neither afford nor justify full-time professional staff, although there will be a need for their services from time-to-time.

A major factor also to be considered is motivation of staff. The water undertaking needs, in addition to sufficient money, sufficient authority regarding conditions of employment, status and salaries of staff to be able to avoid situations which result in demotivation and demoralisation of employees.

All these factors point towards the desirability of having larger, rather than smaller, water undertakings, which embrace rural as well as urban supplies. They also highlight the need for the Regulations relating to the establishment of water undertakings to be drafted in such a way as to allow maximum autonomy and flexibility regarding employee conditions.

Materials and machines (equipment), especially those required for repairs and maintenance, are often prime targets for budget cuts whenever it is felt necessary to reduce expenditure. Even when money is available, it can sometimes be a laborious business to process all the documentation necessary for procurement of equipment from overseas, and in extreme cases may take years to achieve.

Yet manpower cannot function effectively if it is deprived of the resources it needs to do the job. A few examples of situations encountered by the author in recent years may help to illustrate this :

1. Staff from several new "package" treatment plants were given training in operation and maintenance, including use of a "comparator" for measuring Cl residual and pH. (The equipment had been supplied as part of the treatment plant contract). On return to their plants the staff were unable to use the comparators as the necessary chemical reagents were not made available to them.

2. Several water undertakings were aware that their water losses were very high, estimated at 50 - 60%. Each sent an employee from the distribution section to learn techniques of leak detection and control. On return to their undertakings many of these staff were frustrated, because :

   - their undertakings lacked bulk water meters to measure water production

   - more than 40% of domestic water meters were malfunctioning, so that water consumption could only be estimated

   - the undertakings did not possess any leak detection equipment.

3. Staff from a number of water undertakings were given training in the financial and administration procedures required by the central auditing authority. Following the training many of them experienced difficulties in implementing the systems, as their undertakings did not provide all the necessary forms and documentation.

Situations such as these can be counter-productive, in that they demotivate staff and make them cynical towards future development opportunities.

Lack of materials and equipment can be a particularly severe problem in rural areas, where a total system may come to a standstill because of breakdown of a major element - a breakdown which might have been avoided by regular workshop overhauls or ready availability of spare parts.

Institutional arrangements need to ensure that as few constraints as possible are placed in the way of water undertakings wishing to purchase materials and equipment (especially items which have to be imported).

Also, for rural schemes, the need for easy access to workshop and stores facilities (such as those available in may urban undertakings) must be recognised.

3. ORGANISATION DEVELOPMENT

All water supply schemes, other than very simple village supplies, have need of a team of people to keep the systems operational. This implies that :

(a) there will be specialisation of work i.e. staff will have different jobs to do

(b) there will be need of a framework for planning, coordinating and controlling the activities of team members i.e. an organisation structure/management system will be required.

As described in section 2, the team must be set up in such a way that the undertaking not only operates effectively on a day-to-day basis, but also is able to plan and prepare for the future. The decisions taken in relation to (a) and (b) above can have a critical effect on the ability of the team to meet these objectives.

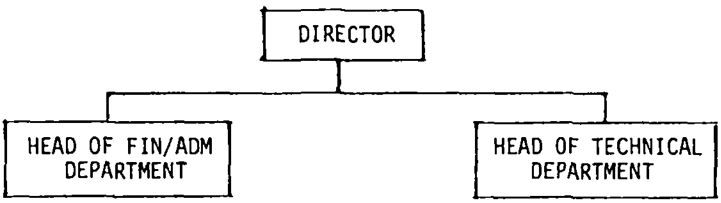
For a new water undertaking, the questions to be addressed are :

(i) how many staff are needed ?

(ii) what will their jobs be ?

(iii) what kind of organisation structure will be most appropriate ?

In general, the tasks to be carried out within a water undertaking fall into two main categories : financial/administrative tasks and technical/scientific tasks. It is common practice to consider these tasks as the responsibilities of separate departments, and heading the organisation we have



But what structure is required within each department ? Each must be considered separately :

Finance/Administration Department

The majority of activities within this department are of a routine nature, involving the application of standard procedures. In most developing countries, where water is supplied to consumers via domestic water meters, a large percentage of staff will be involved in meter reading, billing and bookkeeping. The total number of staff required is predominantly a function of the number of water bills to be processed each month. The actual relationship will depend on the complexity of the financial/administrative procedures to be followed.

In Indonesia, central government have issued recommended staffing levels for finance/administration departments. For undertakings having between

1000 and 50,000 connections these recommendations approximate to :

$$(\text{nos. of staff}) = 0.2 \times (\text{nos. of connections})^{0.63}$$

Technical Department

Technical staffing requirements tend to be more difficult to determine. Departmental activities can usually be grouped into five areas : production, laboratory, distribution, maintenance, plans/records. But actual staffing levels will depend on a number of factors, e.g. :

- the number and geographical distribution of the water sources
- the type of water treatment to be carried out
- the capacity of the treatment plant and the number of hours of operation each day
- the types of water analyses to be undertaken
- the amount of new pipelaying work to be carried out and the policy of the undertaking concerning use of private contractors for pipelaying
- the policy of the undertaking concerning leak detection and control
- the amount and complexity of electro-mechanical equipment to be maintained
- the scope of in-house planning activities

"Standard" staffing levels for technical departments are not appropriate, and each undertaking must be examined individually.

Determining appropriate staffing structures for both finance/ administration and technical departments involves the process of "organisation design". One cannot be dogmatic about this process, as a number of subjective elements enter into it, but one can identify a number of key steps, which in the author's experience have been found to work reasonably well. These are described below, as applied to the technical department.

Step 1 Task Identification

Consider the lower levels of the organisation structure. Identify all the basic (non-supervisory) tasks which must be carried out for operation, maintenance and forward planning. The amount of work to be done at these levels will determine the number of non-supervisory staff required, which will in turn show what middle-management structure is appropriate.

Step 2 Job Design

Group tasks together to form jobs for employees. This should be done carefully. All tasks in a job should as far as possible be inter-related and require similar levels of ability. Some tasks requiring unusually high levels of ability may have to be reserved for later inclusion in a supervisory/managerial job.
"Performance standards" are useful for this exercise, and some examples from Indonesia are given below :

House connections	- 1 pipelayer/plumber able to connect a minimum of 5 consumers per week.
Pipelaying	- 1 pipelaying team able to lay, test and commission 40m of medium diameter pipe per day under normal conditions.
Water meter repair	- 1 man able to overhaul 8 water meters per day.

Step 3 Determination of Supervisory/Middle Management Structure

At supervisory level (first level of management) a crucial consideration is the "span of control" i.e. the range of skills and numbers of employees that one man can supervise effectively. When subordinates are carrying out similar, routine jobs the span of control of the supervisor may be 10 persons or more. If subordinates are involved in different, non-routine jobs the span of control will be less than this.

A factor also to be considered is the need for the supervisor to act as "standby" should one of his key subordinates fall sick. If this contingency is to be catered for then the range of in-depth skills required by the supervisor must be borne in mind.
At the next level of management, where different groups are combined under a single head, a determining factor is the interdependence of one group with another e.g. meter repair staff with distribution staff, production staff with laboratory staff.

Step 4 Job Descriptions

Once an appropriate organisation structure has been determined, detailed Job Descriptions need to be prepared for all staff. This is especially important for a new undertaking, where staff may have little or no previous experience of water supply. The job descriptions help to ensure that :

- all tasks necessary to maintain a healthy enterprise have been accounted for and allocated to one job holder or another
- there is a basis from which to proceed with staff recruitment
- each job holder knows exactly what his duties are
- the supervisor/manager also knows what his subordinates' duties are, and can supervise them more effectively
- the person responsible for staff training has a basis from which to determine training needs and design training programmes

Step 5 Management Information System

Managers need reports and records from various parts of the organisation to enable them to plan and control activities. Firstly, it must be decided what kind of information should be recorded, how frequently, and to whom it should be given. Secondly, standard report forms may have to be prepared to facilitate the flow of this information through the organisation. For example, a treatment plant operator may need to record plant operating hours, water production, water quality, usage of fuel and chemicals, frequency of backwashing filters, etc.

Establishing a workable organisation structure can be a laborious exercise, but unless it is carried out in a systematic way, then there is a risk that the total effectiveness of the team will be much less than the sum of their individual abilities. Human resources development is important for groups as well as for individuals.

4. DEVELOPMENT OF THE INDIVIDUAL

The need to give attention to the development of individual members of an organisation is self evident. They represent an asset which can appreciate, under the right conditions, to the positive benefit of the enterprise.
If neglected or abused, they can also become a negative influence within the system.

Obviously employees need minimum levels of knowledge and skill in order to satisfactorily perform the jobs which they are paid to do. Furthermore, the many specialised aspects of water supply usually mean that new employees must acquire much of this knowledge and skill after they join the water undertaking. It is in the interestsof the undertaking to guide its employees through these learning situations, and this can be done through some form of organised training.

Training, when properly carried out, can not only improve knowledge and skill, but also influence the attitude of the employee towards his work. Many new recruits to a water undertaking have only ill-formed ideas of the total objectives of a water supply system, and the difficult operations involved. Training is an opportunity to show them the purpose and benefits of water supply, and the importance of their own role in helping the organisation meet its objectives. In other words, training can help generate a positive attitude towards the job.

This aspect is also important for those at the receiving end of the water supply system - the consumers. The image of a water undertaking within the community is an important consideration. The more knowledgable the consumers are concerning the benefits of clean water and the difficulties in providing it, the more supportive they will be of the undertaking's activities. Also, an enhanced image for the undertaking brings with it a psychological improvement in the morale of employees and their social status within the community.

Returning to the question of employee development, the techniques of systematic industrial training have been developed over many years and are well documented. Unfortunately their application within the water sector is not as widespread as it could or should be.

Too often those involved with water supply development appear pre-occupied with the technical aspects, and are disinclined to divert their efforts towards the human element. Perhaps this is a natural consequence of their own technically-orientated education, sometimes combined with a lack of awareness of principles and practice of human resources development.

Training is sometimes approached as if it were merely an extension of the classroom education we all received in our youth. But training and education are not the same, and although there are many similarities there are also important differences, not least of these being the vastly different timescales available to each.
Through necessity, training has to be more sharply focussed, more precisely executed, than education. A 50% correct performance may be sufficient to earn a Diploma in an examination, but it is hardly an acceptable level of job performance in a water undertaking !

The stages of systematic training can be summarised as :

- identifying the training need
- preparing the training programme
- implementing the training
- evaluating the results

To enlarge here on the activities involved in each step would merely be re-iterating standard text book material and would occupy a prohibitive amount of space. What can be said, however, is that whilst in theory the steps are straightforward, in practice the way is confused by constraints and comprise. Also, good results are not only difficult to achieve, they are frequently difficult to measure !

The resources to be used for the training may be many and varied, including perhaps staff of consultants, suppliers, contractors, as well as government bodies and other institutions. Co-ordinating and controlling the activities of these groups can present many problems. However, adherence to the basic principles of systematic training, if not wholly to the detail, will enhance the chances of a successful outcome.

A further point to be made about training is that it should be seen as an on-going exercise. The development of senior managers is a long-term process and cannot be achieved in a single training event. Also, organisations are constantly changing in response to changes in the internal or external environment. Water undertakings (especially in developing countries) are in a continual process of growth. Employees retire, resign, are promoted. Systems and technologies are modified and updated. Thus new training needs are constantly being generated.

For effective long-term development of human resources it becomes highly desirable to have a central training resource to support the individual water undertakings. This should be able to provide training courses, training materials, trainers and the like. Such a resource not only ensures the quality of training, but, if properly planned, ensures greater cost-effectiveness. It also creates a focal point for training other groups involved in water sector activities e.g. contractors.

5. SUMMARY AND CONCLUSION

The effectiveness of human resources in the water sector is affected by the activities of, and decisions taken for, three levels : Institutional, Organisational, Individual.

Institutional arrangements for both urban and rural water supplies must pay due regard to the following factors, which influence the quality and performance of staff :

- ability of the water undertaking to recruit and retain good people
- resources for staff training
- staff motivation (conditions of employment, career opportunities, etc)
- availability of tools, materials and equipment

In general these requirements are more easily met in larger, rather than smaller, water undertakings.

Organisational considerations involve issues such as :

- task identification
- job design
- organisation design
- management information system

These influence the work performance of employees, both as individuals and as members of a team.

Individual development has to be considered not only for employees of the undertaking, but also for members of the wider community e.g. consumers, contractors.

Systematic training is a well developed, professional technique for achieving this and should be more widely applied. Also, training is a long-term exercise and some form of central training resource is desirable.

Over recent years a lot of money and effort has been directed towards helping developing countries acquire the "hardware" components of water supply. Perhaps it is now time to pay more attention to the "software" aspects, such as are discussed in this paper.

All contributors to water supply development have a role to play, especially those involved in discussions with senior government decision-makers, such as donor agencies and consultants. The importance of human resources needs to be given greater emphasis, so that strategies for water supply take due regard of the factorsaffecting their deployment and development.

Experience is showing that the human element presents one of the greatest problems and challenges in water sector development, not only for the International Water Supply and Sanitation Decade, but for future decades to come.

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ASSESSMENT OF KARST WATER RESOURCES

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ABSTRACT

Karstified carbonate rock formations are predominant over a large number of regions throughout the world, being of primary importance for certain countries. These formations significantly affect the potential and the use of water resources, both quantitatively and qualitatively.

Certain new or modified hydrological techniques, based primarily on regression analysis and structural mathematical modelling, have been developed to assess the areal and timely distribution of karst water resources.

The application of these techniques to basins in Turkey, where karst formations cover almost on third of the country and affect more than one third of the water potential, proved to yield satisfactory results..

INTRODUCTION

The peculiarities of karst formations attracted, especially during the last two decades, intensive interest with regard to hydrology on an international basis (I.A.S.H. 1967; I.A.H. 1975; Yevjevich 1976; U.A. 1977; W.K.U. 1977; Yevjevich 1981a), as well as on the specific case of Turkey (Öziş 1979a; Günay 1980; Yevjevich 1981b).

Karstified formations of soluble, mostly corbonate rocks cover almost one third of Turkey, being predominant along the entire southern coastland and a large part of the eastern regions (Eroskay and Günay 1980). Basins with intensive contributions of karst spring effluents to river runoff are shown in Fig. 1 (Öziş and Keloğlu 1976).

Turkey has built hydraulic works like Keban/Euphrates and Oymapınar/Manavgat dams as well as numerous smaller dams, irrigation and power schemes, and is going to develope many other projects; hence, every effort towards better planning and efficient operation of such projects will be of great importance.

Surface waters of the upstream basins in karst regions drain through underground channels, resulting in the depletion of surface water resources to be developed; whereas in the downstream basins, they appear again at the surface as karst springs and contribute significantly to streamflow.

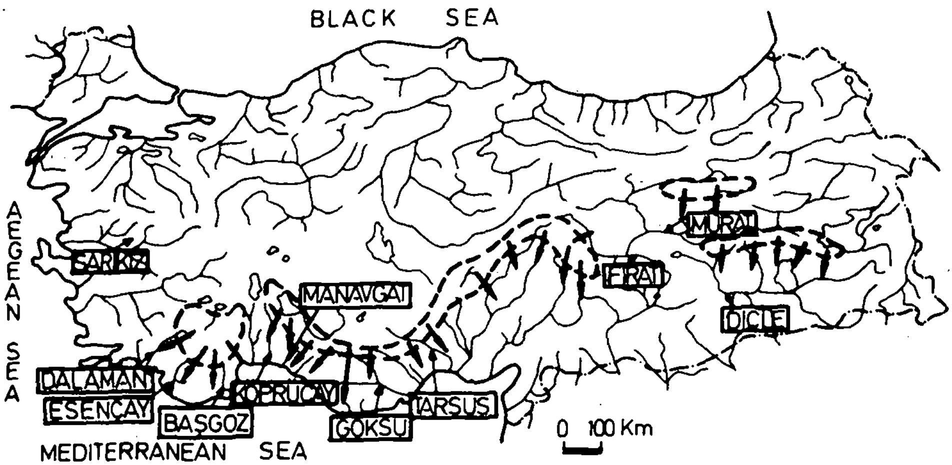

Figure 1- Basins with significant karst springflow contributions in Turkey.

Karst springflows exert a positive effect to decrease the required relative reservoir volumes for flow regulation; because the propagation through underground karst formations leads to seasonal flow variations occuring within narrower limits by karst runoff than by direct surface runoff, and possible cavities which may remain within a reservoir increase the storage capacity.

The development of hydrological techniques with regard to karst water resources will not only enable the direct use of the results in the exploitation of water resources, but also provide an information flow from hydrology to geology, which has usually been in the reverse direction (Yevjevich 1976). The paper presents such techniques, based primarily on regression analysis and structural mathematical modelling, to assess the areal and timely distribution of karst water resources and discuss the results of their application on Turkish karst basins.

COMPARISON OF MODULAR RUNOFF RATES

The comparison of modular runoff rates q ($m^3/s, km^2$) of basins with karst springs effluents to those upstream or neighboring basins, where such contributions do not exist, may yield a rough estimate of the mean karst runoff Q_{py} (m^3/s) as:

$$Q_{py} = (q_y - q_x) \cdot \Delta S_y + Q_{pu} \qquad (1)$$

where y and x denote the respective basins, ΔS_y (km^2) the differential drainage area, Q_{pu} (m^3/s) any eventual karst runoff from upstream basins (Öziş 1979b). The runoff rates might be modified to account for differences in precipitation rates.

INTERCEPT OF SINGLE LINEAR BIVARIATE REGRESSION

The intercept A_y (m^3/s) of the linear bivariate regression equation, of Y on X, where Y (m^3/s) denotes the discharge of the basin with karst springflows and X (m^3/s) without tham, describing :

$$\lim Y\ (X \to 0) = A_y \quad \text{and} \quad Q_{py} = A_y \qquad (2\ a,b)$$

can be regarded as the base flow of the karst runoff (Öziş and Keloğlu 1975).

However, it should be remembered that the existence of such karst flow in the X set may distort the results, even yielding A_y almost equal to zero, if the ratio of karst flows lies in the range of the value of the regression coefficient B_y.

PAIR OF LINEAR BIVARIATE REGRESSION EQUATIONS

Applying the same considerations to the intercept of the reciprocal regression equation, the estimate of the base flow of the karst runoff can better be approached by :

$$Q_{py} = (A_y - A_x/B_x)/2 + (B_y + 1/B_x).Q_{px}/2 \qquad (3)$$

where the second term enables to take account for an eventual karst runoff component in basin X (Öziş et al. 1977).

REVERSIBLE LINEAR BIVARIATE REGRESSION PARAMETERS

Confidence limits of regression parameters

Following the Student-t-distribution with N-2 degrees of freedom, the standardized variable of the distribution of the regression coefficient of a linear bivariate regression equation is (Yevjevich 1972a):

$$t_{by} = \left[(B_y - \beta_y)/B_y\right] \cdot \left[r(N-2)^{1/2}/(1-r^2)^{1/2}\right] \qquad (4)$$

where β denotes the regression coefficient of the population, B and r the regression and the correlation coefficients of the sample, N the sample size.

Hence, for a given probability level b with the corresponding t_b value, the upper and lower confidence limits for $\beta = B$ are (Yevjevich 1972a):

$$B_y\ (u.,l.,C.L.) = B_y \pm t_{by}(B_y/r)(1-r^2)^{1/2}/(N-2)^{1/2} \qquad (5)$$

Similarly, the standardized variable of the distribution of the intercept is :

$$t_{ay} = \left[(A_y-\alpha_y)/B_y\right]\left[r(N-2)^{1/2}/(1-r^2)^{1/2}\ (S_x^2 + \bar{X}^2)^{1/2}\right] \qquad (6)$$

where α denotes the intercept of the population, A that of the sample, S_x the standard deviation and $\bar{X}$ the arithmetic mean of the sample's independent variable.

Again, for a given probability level a with the corresponding t_a value, the upper and lower confidence limits for $\alpha = A$ are :

$$A_y(u.,l.,C.L.) = A_y \pm t_{ay}(B_y/r)(1-r^2)^{1/2}\ (S_x^2+\bar{X}^2)^{1/2}/(N-2)^{1/2} \qquad (7)$$

Limit of correlative association

The theoretical maximal limit of correlative association is a pure functional relation. Therefore the practical, approximate maximal limit of the correlative association between two hydrological processes, when the sample size N tends to infinity and the correlation cofficient converges to unity, will be an almost functional relation with regard to the correlation parameters. Consequently, the regression coefficient β will converge to a limit value β^*; which can be called the reversible regression coefficient and expressed as (Öziş et.al. 1977; Öziş and Benzeden 1978) :

$$\beta_y^* = 1/\beta_x^* = \lim \beta_y(r \rightarrow 1) = 1 / \lim \beta_x(r \rightarrow 1) \tag{8}$$

so that the sample value can be obtained as :

$$B_y^* = 1/B_x^* = B_y \text{ (upper C.L.)} = 1/B_x \text{ (upper C.L.)} = B_y/r = r/B_x \tag{9}$$

as shown in Fig.2.

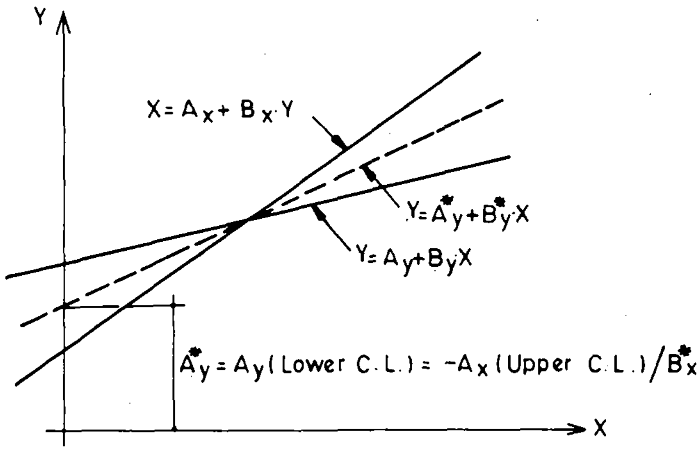

Figure 2- Reciprocal linear bivariate regression equations and the reversible regression equation.

Similarly, the intercept will converge to a limit value α^*, which can be called a reversible intercept and expressed as :

$$\alpha_y^* = -\alpha_x^* / \beta_x^* = \lim \alpha_y(r \rightarrow 1) = - \lim \alpha_x (r \rightarrow 1) / \beta_x^* \tag{10}$$

and computed form the sample as :

$$A_y^* = -A_x^*/B_x^* = A_y\text{(lower C.L.)} = - A_x \text{ (upper C.L.)} /B_x^*$$

$$= A_y - B_y^* (A_y - B_y^* .A_x) (S_x^2 + \bar{X}^2)^{1/2} / \left[(S_y^2 + \bar{Y}^2)^{1/2} + B_y^* (S_x^2 + \bar{X}^2)^{1/2} \right] \tag{11}$$

the final form obtained by substitution of equations 7 and 9 in appropriate terms.

It should be noted, that this approach is conceptually different than the orthogonal regression, although the numerical results may happen to be in the same order of magnitude (Benzeden and Özdağlar 1981).

Estimation of karst runoff

Assuming an almost functional relation as the approximate maximal limit of correlative association, the statistical testing can be interpreted as to whether the regression parameter pairs computed from the sample differ significantly form the reversible parameters. If, for confidence levels of equal probabilitiy the differences are insignificant, the reversible regression equation may be considered as the best estimate of the representative almost functional relation.

Hence, the baseflow of karst runoff can be computed as :

$$Q_{py} = A^{*}_{y} + B^{*}_{y}.Q_{px} \tag{12}$$

where the second term enables to take account for an eventual karst runoff component in basin X (Öziş et al. 1977).

CORRELATION OF STOCHASTIC COMPONENTS OF CONVENTIONAL MATHEMATICAL MODELS

The linear bivariate correlation is theoretically based upon the normal distribution of the variables. Since daily or monthly sequences of hydrological processes include a strong deterministic component in form of periodical seasonal variation, a more sophisticated approach will consist of computing the reversible regression parameters through correlation of the stochastic components after removal of the deterministic (Yevjevich 1972b) ones.

Consequently, the baseflow of karst runoff can be computed as :

$$Q_{py} = Y_{d} - B^{*}_{y,s}.X_{d} \tag{13}$$

where Y_d and X_d denote the deterministic components (m^3/s) of the streamflow sequences, $B^{*}_{y,s}$ the reservible regression coefficient of their stochastic components (Öziş and Benzeden 1979).

MATHEMATICAL MODELS WITH EXPONENTIAL FUNCTIONS

The recession hydrograph during dry periods, in the exponential form of :

$$Q_t = Q_o.e^{-at} \tag{14}$$

given first by Maillet in 1905 (Milanovic 1981), where t denotes the time (day) and a (1/day) the aquifer discharge coefficient, has been generally accepted as representative of dry period karst spring discharges.

Recently, as shown in Fig.3, reciprocal exponential functions, a decreasing one for the dry period of 6 to 7 months, an increasing one for the wet period of 5 to 6 months, has been proposed to model the karst-related part of the deterministic component of river runoff with significant karst spring effluents, whereas the surface runoff part of the deterministic component will be represented by periodical Fourier functions, covering only the wet period (Keloğlu 1984).

Hence, the average karst runoff (m^3/s) can be estimated by :

$$Q_{py} = \left[\int^{t_{dry}} Q_t . \, dt + \int^{t_{wet}} Q'_t . \, dt \right] / \, (t_{dry}+t_{wet}) \tag{15}$$

where the Q_t and Q'_t functions represent the descending and ascending limbs of the karst-related part of the deterministic component of the mathematical model.

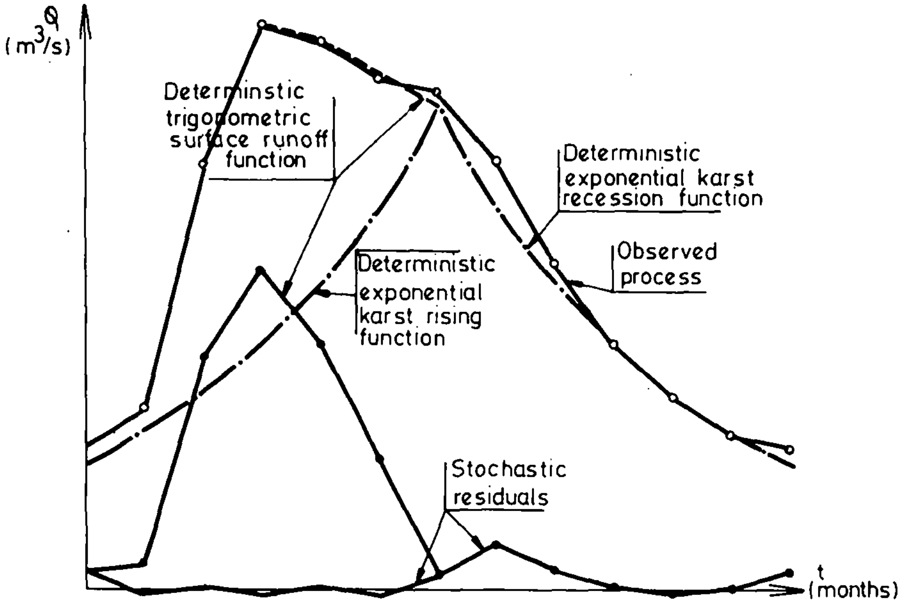

Figure 3- Mathematical model with exponential simulation of river runoff receiving significant karst spring effluents.

RESULTS OF APPLICATIONS

The applications of the above-mentioned methods to various river basins with significant karst spring effluents in Turkey yielded fairly consistent results.

Although every method has not been applied to every basin, the average baseflow contribution of karst spring effluents is in the order of 10 m^3/s or one half of total flow at Örenköy/Eşençay; 40 m^3/s or slightly less than one half of the total flow at Beşkonak/Köprüçay; 110 m^3/s or more than two third of total flow at Homa/Manavgat; 35 m^3/s or one third of the total flow at Karahacılı/Göksu; 12 m^3/s or one third of the total flow at Muhat/Tarsus; 200 m^3/s or more than one third of the total flow at Aşvan/-Murat; 110 m^3/s or roughly one third of the total flow at Rezuk/Dicle; to quote only the major basins (studies until 1979 summarized in Öziş 1979; later in Atış el al. 1980; Keloğlu 1984).

CONCLUSION

An important part of Turkey's water resources are affected by karst formations. Karst spring effluents are emerging at several locations near the river bed and are not readily measurable by direct hydrometric methods.

Mathematical hydrological techniques, based primarily on linear bivariate regression analysis and various structural simulation models, have been developed and successfully applied to the appraisal of karst water contributions in various basins. Most of these techniques are suitable for application to basins in other parts of the world with similar karst spring effluents.

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	WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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ON THE OPTIMAL LOCATION OF WASTEWATER TREATMENT PLANTS IN LOWLAND AREAS

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ABSTRACT

This paper reports on a particular application of a previously developed and well-tested model for the optimal location of wastewater treatment plants. The model allows one to select and design a set of treatment plants and the appropriate conveyance system for collecting and treating wastewaters coming from various discharge points in a given region, in such a way that the global discounted cost be minimized. In the primary version of the model, an average depth is assumed for each wastewater collector and no depth compensation allowing e.g. for gravity flowing on flat sections is possible. In previous uses the model was applied to moderately undulating areas, resulting in most cases in gravity flowing solutions for which the model is well suited. The paper is mainly devoted to a description of more recent experiments of the model, performed on an area with important parts where relief is practically absent. Accordingly, the model was modified in order to incorporate the possibility of optimizing, in addition to each collector's diameter, both its slope and excavation depth, thus enabling more frequent gravity flowing even on flat or slightly climbing grounds. The optimal costs are significantly affected by adding this new component to the model, while the required number of pumping stations is strongly decreased, resulting in more centralized optimal solutions where the number of treatment plants is much reduced. The model is proposed as an useful aid in regional planning of wastewater treatment.

INTRODUCTION

The regional management of water quality has been the object of various types of mathematical models in the literature. An important class of these is concerned with the optimal location of wastewater treatment plants, which has given rise to extensive research projects (Deininger, 1969, 1972; Joeres et al., 1974; Jarvis et al., 1978; Smeers and Tyteca, 1983). The purpose of such models is to seek the least-cost design of a wastewater collection and treatment network, by specifying the layout and size of collectors as well as the number, location and size of treatment plants. When the number of wastewater discharge points in the region under concern, as well as the number of possible plant locations and collector arrangements, are important, the solution of this problem is far from being evident, due to its rather complex combinatorial nature, in such a way that least-cost solutions may be easily overlooked by simply enumerating a few possible layouts. Mathematical programming techniques, as used in the aforementionned references, can be of considerable help in solving this type of problem.

In previous applications of the optimal treatment plant location model, the areas studied were more or less undulating, thus enabling one to take advantage of the possibilities for gravity flowing in most collector layouts. For that reason, the slope of each collector was not considered as a variable in these applications, but instead was supposed to be equal to the field slope, with a constant depth of excavation. The possibility of adjusting both the collector slope and depth was not taken into account and was assumed to have no significant influence on the total network cost. This is no longer valid in lowland areas where relief is completely absent or unsignificant : assuming constant collector depth would then result in installing pumping plants almost everywhere, which considerably increases the network cost. The first purpose of this paper is to show how the model can be modified and applied by incorporating the collectors' depth and slope as variables. A second part of the paper is devoted to a description of the case study and the results obtained with the model.

MODEL FORMULATION

Classical formulation

The classical model for optimal location of wastewater treatment plants can be formulated as follows (see e.g. Smeers and Tyteca, 1983; Tyteca, 1977) :

$$\min \sum_{i=1}^{N} \sum_{j=1}^{N} C_{ij}(x_{ij}) + \sum_{k=m+1}^{N} C_k(y_k) \tag{1}$$

such that :

$$\sum_{j=1}^{N} x_{ji} - \sum_{j=1}^{N} x_{ij} = -\pi_i \qquad i = 1, 2, \ldots, n \tag{2}$$

$$\sum_{j=1}^{N} x_{j\ell} - \sum_{j=1}^{N} x_{\ell j} = 0 \qquad \ell = n+1, n+2, \ldots, m \tag{3}$$

$$\sum_{j=1}^{N} x_{jk} - \sum_{j=1}^{N} x_{kj} - y_k = 0 \qquad k = m+1, m+2, \ldots, N \tag{4}$$

$$x_{ij} \geq 0 \ , \quad y_k \geq 0 \ , \qquad \forall \ i, j, k \qquad (5)$$

in which : N is the total number of points in the network, composed of points 1 to n : wastewater discharges, points n+1 to m : intermediate collect points, points m+1 to N : possible treatment plant locations; x_{ij} is the flow rate (in m^3/sec) from point i to point j; $C_{ij}(x_{ij})$ is the cost for conveying flow x_{ij} from point i to point j; y_k is the flow rate (in m^3/sec) treated at plant number k; $C_k(y_k)$ is the cost for treating flow y_k at plant k; π_i is the wastewater flow rate (in m^3/ sec) discharged at location i; equation (1) represents the total discounted (i. e., capital plus operation and maintenance over the lifespan) collection plus treatment cost to be minimized; equations (2) to (4) are flow conservation relationships formulated at each point of the network.

Cost functions

The objective function given in equation (1), to be minimized, includes two components, one for collection, the other for treatment, obtained as sums of cost functions, that is, expressions yielding the collection or treatment cost as a function of the flow rate collected or treated. The formulations previously used for these functions are as follows (Smeers and Tyteca, 1983):

Collection costs

$$C_{ij}(x_{ij}) = A_{ij} + B_{ij} \ x_{ij}^{0.375} + E_{ij} \ x_{ij}^{0.75} \qquad (6)$$

where A_{ij}, B_{ij}, E_{ij} represent "constants" or parameters obtained for collector ij. These parameters are computed for each possible collector of the network, through a preoptimization analysis : primarily the cost is given as a function of the collector length L_{ij} and diameter D_{ij}, assuming a fixed excavation depth :

$$C_{ij}/L_{ij} = a_{ij} + b_{ij} \ D_{ij} + c_{ij} \ D_{ij}^2 \qquad (7)$$

where the constants a_{ij}, b_{ij}, c_{ij} account for the field characteristics (concrete, asphalt, bare soil) and for various types of crossings (roads, railroads, rivers).

For gravity systems, the diameter appearing in equation (7) is substituted by expressing the condition for free gravity flowing, yielding directly equation (6). For pressure systems, pumping costs are to be added to the pipe costs; for a set of possible flow rate values, the least-cost combination of pumping power and diameter is then computed, yielding equation (6) through a regression analysis. The interested reader is referred to Tyteca (1976 or 1977) for more detail about this methodology.

Treatment costs

$$C_k(y_k) = a \ (y_k)^b \qquad (8)$$

where a and b represent parameters obtained through statistical analysis of existing treatment plants or through a preoptimization analysis similar to that used for collectors. In this cost function, the exponent b takes a value less than 1 (0.7 ... 0.9), reflecting the economies of scale usually observed for this type of equipment. In the present study, the following cost function is used :

$$C_k(y_k) = 961\ 170\ 000\ y_k^{0.76112} \tag{9}$$

where C_k is the total discounted treatment cost (capital + operation and maintenance), in 1975 BF, based on an interest rate of 9.5 % and a lifespan of 20 years, while y_k is the treated flow rate in m^3/sec.

Optimization method

The method used for solving the problem in equations (1) - (5) is a highly efficient local optimum algorithm based on searching for adjacent arborescences in the network and exploiting a shortest path routine. The method was thoroughly developed and described in previous references (Daeninck and Smeers, 1977; Smeers and Tyteca, 1983).

Taking slope and depth as variables

Discussion

In the aforementionned model the only decision variables are the flow rates collected, conveyed and treated in each pipe and plant of the network, as reflected in the objective function (1). In particular, for each pipe the excavation depth is supposed fixed (taken as an average of 3 m in previous studies) and the slope is taken as parallel to the soil slope, while the diameter and the pumping power are implicitly given least-cost values for each possible flow rate value.

In situations where the natural relief is completely or practically absent, taking the slope as parallel to the soil may result in considerable overcosts, because of the many pumping stations required for coping with the absence of free gravity flowing. This may lead to very decentralized solution networks, where treatment plants are numerous and of rather small size, while the collection paths are made as short as possible. As an example, for the case study presented subsequently, results not shown in this paper indicate that when each pipe is considered as parallel to the ground, the least-cost solution includes nine treatment plants, with a global discounted cost 27 % cheaper than the least-cost solution including only one treatment plant. As will be seen later, this percentage can be reduced to about 4 % when considering the possibility of acting on each pipe's slope and depth, while the least-cost solution appears much more centralized, with longer collection paths allowed by the larger possibilities for gravity flowing.

New collection cost function

Thus in lowland areas it is more advantageous to incorporate the slope and excavation depth of each wastewater collector as variables in the model. The collection cost function, primarily given as in equation (7), has therefore to be modified to incorporate the effect of the depth of excavation on the collection cost. Using the belgian data available, the best regressions led to the following cost function :

$$C_{ij}/L_{ij} = a_{ij} + b_{ij}\ D_{ij}^2 + c_{ij}\ P_{ij}^2 \tag{10}$$

in which P_{ij} represent the average depth of excavation and the parameters a_{ij}, b_{ij} and c_{ij} incorporate the same influences as previously (field characteristics). This cost function is quite identical to those obtained by Dajani and Gemell (1971) from american data.

New model formulation

Taking into account the possibility of optimizing each pipe's slope and depth multiplies by 3 the number of variables to include in the collection network. For that reason, and due to the fundamentally different natures of the two problems to be faced (i.e., the problem of optimal plant location and collection layout, which is rather combinatorial, and the problem of optimal selection of diameter, slope and depth, which is a nonlinear design problem), the analysis was divided into two steps :

Step 1. Use of the optimal location model as previously

with an average excavation depth of 3 meters, under the following assumptions: (1) free gravity flowing is possible where the slope is negative or less than 1m/1000m; (2) the successive stretches of a given pipe satisfying the first assumption are aggregated into one stretch by adding their lengths and soil height differences; (3) whenever free gravity flowing is made possible through the first two assumptions, the collector slope should be at least -0.6m/1000m: this may yield slight modifications in the basic data on the original stretches. These three hypotheses result from a preliminary analysis of the physical problem. They can imply, on the one hand, an underestimation of actual pumping costs, and on the other hand, an overestimation of actual pipe costs due to the 3 meters excavation depth assumption. However, as will be seen, the total network cost obtained under these hypotheses is very close to the cost of the optimal solution obtained in the second step.

This first step indicates the location and size of treatment plants, as well as the collection network layout and the flow values. In this manner, "subnetworks" are identified, sketched and quantified, each corresponding to one treatment plant.

Step 2. Postoptimization of subnetworks

The goal of this second step is to optimally design each set of collectors ("subnetwork") leading to one treatment plant, given the flow value in each collector, and taking into account the possibility of selecting the slope and and depth, as well as the diameter and the pumping power (when required) for each collector. For this sake an algorithm was implemented, based on the generalized reduced gradient technique (Abadie, 1978). The algorithm accomplishes three functions : (1) search of possible gravity systems in the subnetwork, (2) optimal selection of diameter, slope and excavation depth of each collector appearing in the gravity systems, and (3) identification of required pressure systems and computation of optimal pumping power and diameters, with the possibility of installing one or two pipes in parallel. In this postoptimization step, all stretches aggregated in the first step are again considered individually.

The optimization of pressure systems has been described in previous references (Tyteca, 1976). The optimization of gravity systems is an original part of this research. Room is lacking here to give its complete mathematical description. Let us simply mention that the gravity system model includes four variables for each collector : the diameter, the slope, the average excavation depth and the depth at the beginning of the pipe. The flow rate, the length and the field altitude appear as data to the model. The constraints of the model express (1) the conditions for free gravity flowing, (2) the critical conditions for abrasion and settling, (3) the diameter range of commercially available collectors (between 0.15 and 2 m), (4) the range of possible excavation depths (between 1.5 and 5.5 m), (5) two more equality constraints defining the collector' slope and average depth.

CASE STUDY

The aforementionned model was applied to a lowland region of Belgium, an important part of which has practically no relief. The region extends over about 500 km^2 (25 km x 20 km). Twenty-seven wastewater discharge points were identified, including domestic as well as biodegradable industrial wastewaters, ranging from 2000 to 120 000 inhabitants-equivalents, and amounting to 308 300 inhabitants-equivalents. Twelve sites were available for building treatment plants. Figure 1 indicates these discharge points and potential plant locations, as well as all possibilities for conveying wastewaters from the discharges to the plants : 176 possible collectors (divided into 458 stretches) and 30 possible intermediate collect points have been enumerated.

Results

The optimization procedure allows the user to specify an initial solution network, which in his mind should be near the optimal solution. When no such initial solution is provided, a default solution is computed by the program. Thereafter the program proceeds from that initial solution to a locally optimal solution. Table 1 and Figures 2-5 show the results obtained under the following conditions. The "Reference" solution and "Solution 4", depicted in Fig. 5, represent a network designed by a specialized engineering firm. The figures in the "Reference" column of Table 1 indicate the costs computed by the firm by the time of the project, while the figures in the "Solution 4" column are the cost values obtained for the same network after undergoing the second step of the optimization procedure described above. For Solutions 1, 2 and 3, the figures of Table 1 correspond to optimal cost values obtained through both steps of the optimization procedure. Solution 1 (Fig. 2), with 2 treatment plants, was generated from a centralized initial solution with only one treatment plant (plant # 64). It can be noted here that a neighbouring solution, with discharge # 24 also connected to the unique plant # 64, represents an overcost of only 425 220 BF with respect to Solution 1. Solution 2 (Fig. 3) was generated from the default initial solution provided by the program. Solution 3 (Fig. 4) was generated from the "Reference" solution as the initial one.

Discussion

Considering the total discounted cost, the results obtained after step 2 show little difference with respect to step 1, except for Solution 1, which requires more pumping power than the other solutions. This rejoins our previous comments, where it was recognized that pumping costs can be underestimated when running step 1.

A balance between collection costs and treatment costs clearly appears in Table 1 : a decentralized solution, that is, a high number of plants (Solutions 3 & 4) implies high treatment costs and low collection costs, and conversely. The relative cost differences between solutions are low : with respect to the most economical solution (Solution 2), these differences amount to 1.4 % and 4.2 % for the most decentralized solution (Sol. 4) and the most centralized one (Sol. 1), respectively. The absolute differences are of course much more significant (on the order of 10^7 to 5.10^7 BF). The cost difference between "Solution 4" and "Reference" (corresponding both to the same network in Fig. 5) can be explained mainly by the fact that the cost of Sol. 4 accounts for the postoptimization step, which will tend to decrease the required number of pumping stations. This clearly appears in the second part of Table 1 (investment costs), where the pumping costs of the "Reference" solution are more than three times those of Solution 4.

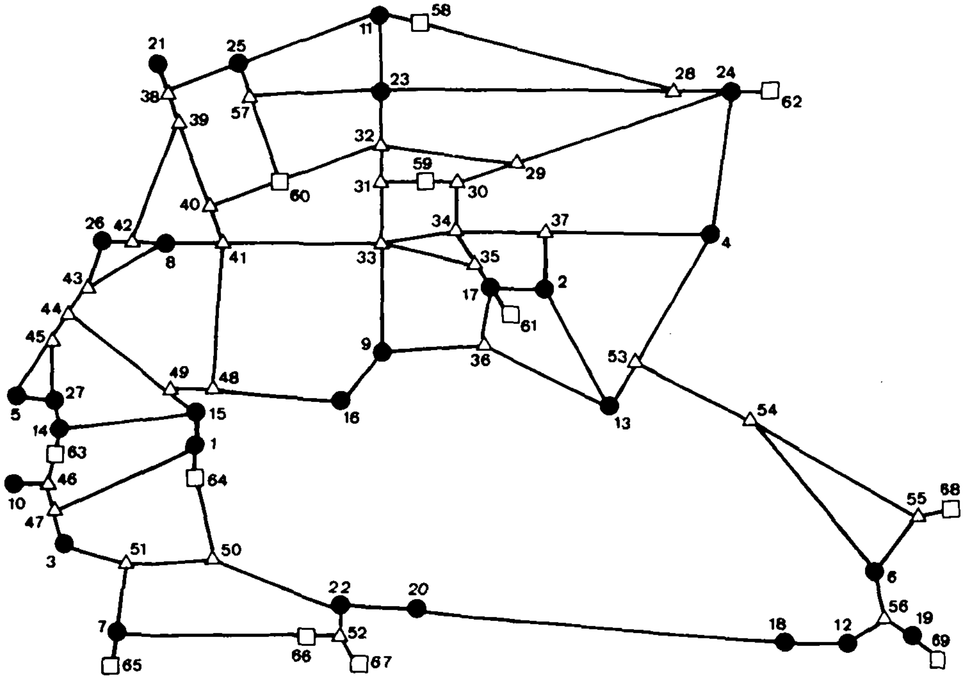

Figure 1.- The basic network for the case study. ● (no. 1 to 27) wastewater discharge; △ (no. 28 to 57) intermediate collect point; □ (no 58 to 69) available location for treatment plant; ———— possible collector layout.

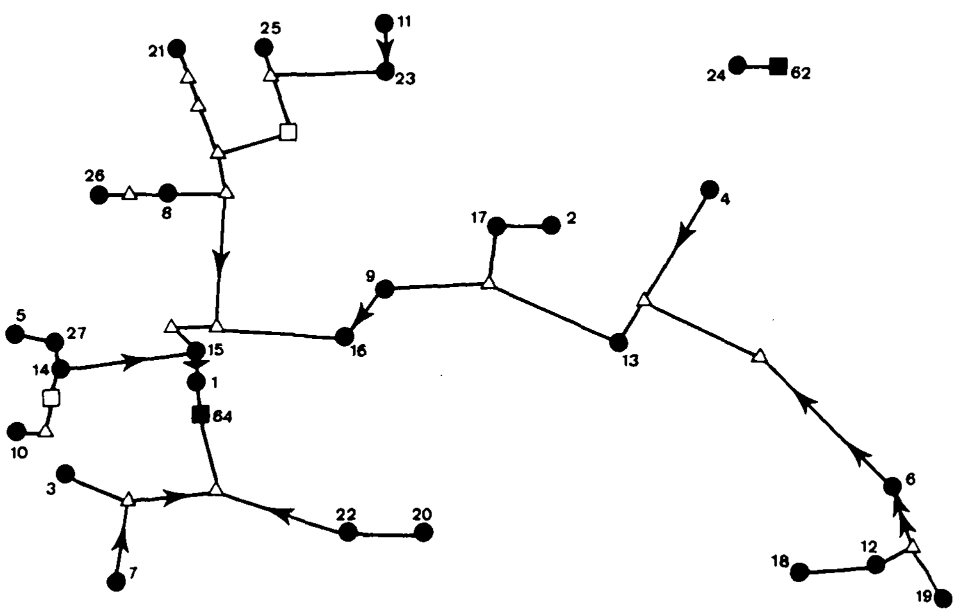

Figure 2. - Solution 1 of case study (2 treatment plants). Legend : see Fig. 3.

Table 1 - The four solutions obtained in the case study (costs in MBF)

			Solution 1	Solution 2	Solution 3	Solution 4	Reference
Number treatment plants			2	4	6	7	7
Total discounted cost	Cost at step 1		1217.5	1205.1	1205.7	1226.6	-
Total discounted cost	Cost at step 2	Gravity systems	371.4	252.4	213.5	194.8	-
		Pressure systems	195.9	98.6	33.3	13.4	-
		Treatment plants	688.3	853.8	965.5	1012.7	-
		Total	1255.7	1204.7	1212.3	1221.0	1258.4
Investment cost	Collectors		473.7	298.2	230.8	199.5	170.1
	Pumping stations		69.3	39.0	11.9	6.5	20.7
	Treatment plants		492.9	611.3	691.3	725.2	748.9
	Total		1035.9	948.6	934.0	931.2	939.7

Considering the investment costs alone, Table 1 indicates a change in the ranking of solutions : Solution 4 becomes the most economical, while Solution 1 now exhibits a cost supplement of 11.2 %. This can be explained by the fact that the operation and maintenance costs of collectors alone are quite unsignificant. Therefore, a ranking based on capital + operation and maintenance costs will tend to favour more centralized solutions, where more collectors are used, while accounting only for investment costs gives more weight to the collectors in the total cost : thus, decentralized solutions with less collectors will be more economical.

CONCLUSIONS

Mathematical programming techniques can be useful as a decision aid for solving problems where the number of possible solutions appears tremendously high. The optimization procedure briefly presented in this paper allows one to generate least-cost solutions to the problem of locating treatment plants and defining the associate wastewater collection network. In this research, special emphasis was placed on lowland situations, where the absence of natural relief leads to adjust both the slope and excavation depth of collectors, allowing for free gravity flowing and for reduction in the required number of pumping plants. A two steps efficient method was presented and illustrated, the first step defining the layout of the network, the second step designing the gravity and pressure systems. Three locally optimal solutions have been generated for a case study, differing mainly by their centralization degree, and were compared to the solution proposed by a specialized engineering firm. Due to the rather flat relief of the region studied, and due to the very high number of possible solutions, the cost differences between these solutions are relatively low, but quite significant when the absolute values are considered. It was also shown that the ranking of the solutions is significantly different when considering only the investment cost rather than the total discounted (investment + operation and maintenance) cost, which is the basic criterion of the proposed optimization procedure.

ACKNOWLEDGEMENTS

Thanks are due to J. Van Herrewege, who actively participated in the research, and to Prof. Y. Smeers for valuable discussions.

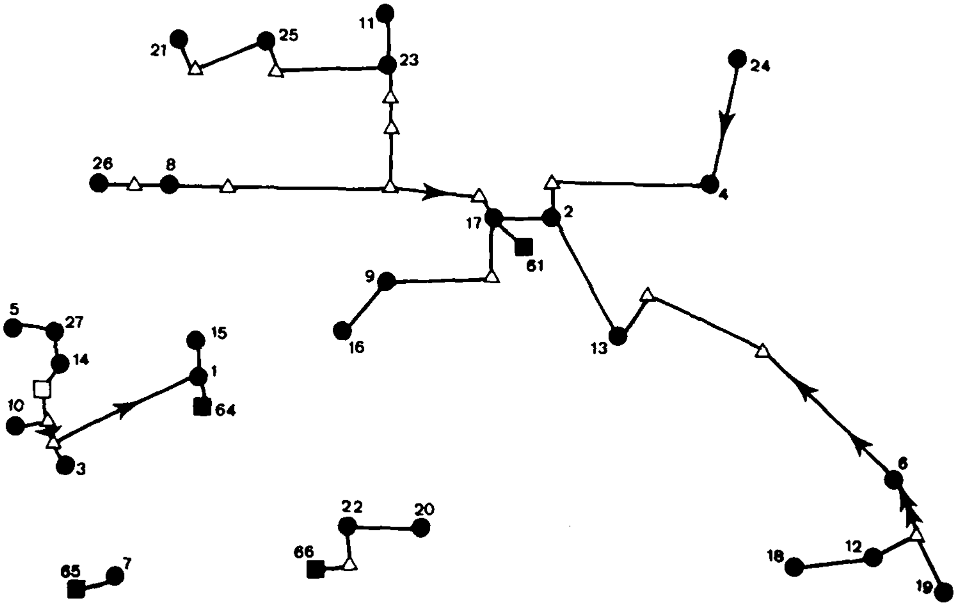

Figure 3. - Solution 2 of case study (4 treatment plants). ● wastewater discharge; △ intermediate collect point; □ treatment plant location used as intermediate collect point; ■ treatment plant in service; ——— collector layout; ➤ required pumping station.

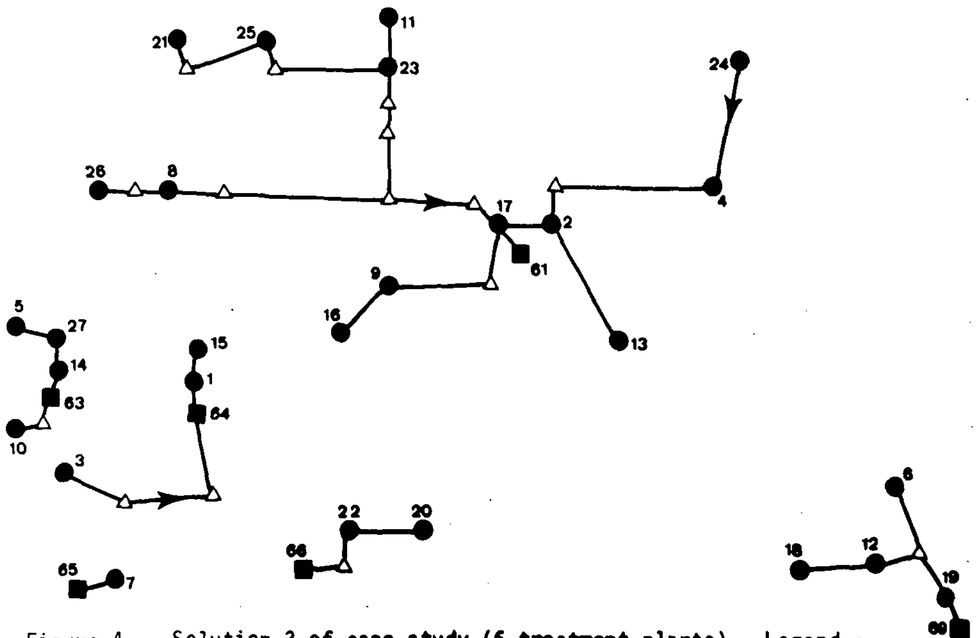

Figure 4. - Solution 3 of case study (6 treatment plants). Legend : see Fig. 3.

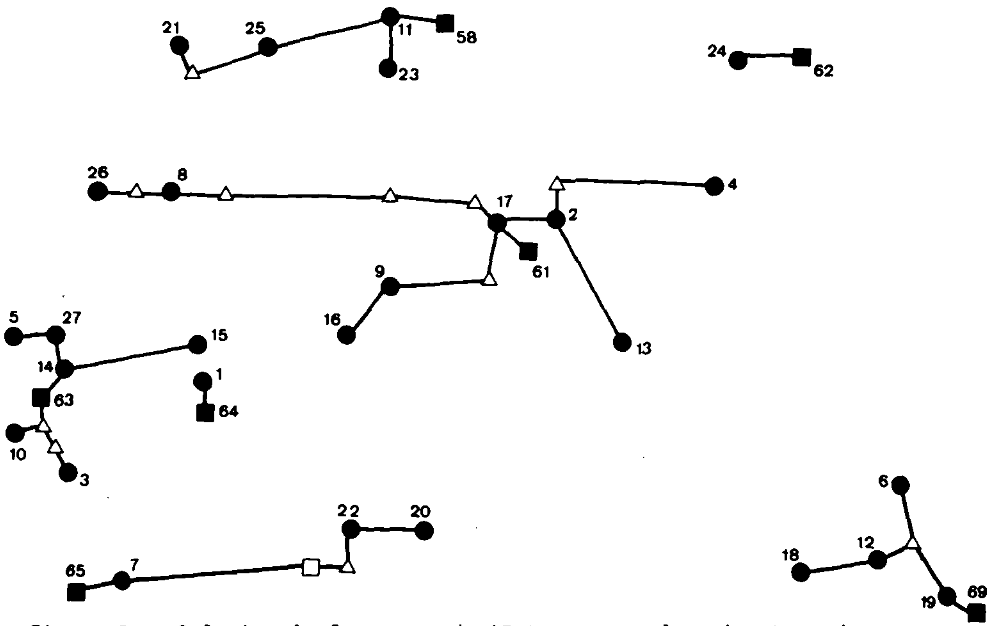

Figure 5. - Solution 4 of case study (7 treatment plants). Legend : see Fig. 3.

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MICROBIALLY MEDIATED FATE OF GROUND WATER CONTAMINANTS FROM ABANDONED HAZARDOUS WASTE SITES

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ABSTRACT

Organic contaminants from abandoned hazardous waste sites threaten ground water supplies in many places. Microbial degradation of the contaminants may be an important mechanism for their removal. The fate of ground water contaminants at hazardous waste sites was investigated in this study to determine rates of biodegradation of toxic chemicals, the factors that limit biodegradation, the general types and numbers of microbes in the subsurface environment responsible for this biodegradation, and the extent of microbial adaptation to compounds present. Much of the work was done at an abandoned wood creosoting operation in Conroe, Texas, U.S.A. The ground water and soil were contaminated with several aromatic organic compounds. Results from these experiments showed that microbes were involved in the removal of the contaminants. Degradation rates ranged from 30 to 100 percent per week for some of the contaminants in ground water samples taken from the site. These results can be used to predict the extent of microbial activity at hazardous waste sites and to help devise microbial techniques for the restoration of contaminated aquifer systems.

Keywords: ground water, aquifer, hazardous waste site, biodegradation, microbial ecology

INTRODUCTION

Ground water is the source for much of the drinking water that is used in the U.S.A.; it has been estimated that approximately one-half of the population relies on ground water for its domestic water supply (Wilson et al., 1983b). Ground water is vulnerable to contamination by organic compounds generated by man's activities such as pesticide application, waste disposal techniques including landfills, land farms or deep well injection, wastewater discharge, spills, and illegal dumping. Many of the organic compounds found in ground water are toxic. For example, ground waters beneath rapid infiltration basins have been shown to be contaminated with organic compounds such as lindane and tetrachloroethane, which may be considered toxicological risks (Tomson et al., 1984). Toxicological data on other compounds found at these sites indicate that they are less likely to be toxic. For some compounds little information is available. Often the levels of the compounds are so low that it is difficult to determine the risk associated with them. Some compounds found in the ground waters beneath rapid infiltration basins are considered to be carcinogenic and there may be no safe levels for carcinogens. One of the worst problems that arises from organic contamination of ground water involves abandoned hazardous waste disposal sites where hazardous compounds which have been discarded into unsecured geological settings can permeate to the ground water. In many cases, it is difficult to determine who is responsible for the site and consequently, its cleanup. The United States recently passed a statute to overcome this problem, the Comprehensive Environmental Response, Compensation and Liability Act of 1980 (better known as Superfund), which authorizes the federal government to oversee the cleanup of abandoned hazardous waste disposal sites (Pye and Patrick, 1983).

Several mechanisms operate to reduce the concentrations of organic compounds in ground water including volatilization, sorption onto soil particles and organic matter, chemical reaction, dilution, dispersion, and biological transformation. Microbial metabolism is probably the most effective process in removing the compounds from the environment. A microbial population of up to 10^6 organisms/g dry weight soil has been found for several shallow aquifers and it is likely that these populations exist elsewhere (Wilson et al., 1983a, Wilson et al., 1983b; Hirsch and Rades-Rohkohl, 1983; Lee et al., 1984; Balkwill and Ghiorse, 1984). A significant portion of the cells (from 0.01 to 50%) are active as shown by comparisons of viable cells from plate counts with direct counts and measures of respiratory activity such as the use of 2-(p-iodephenyl)-3-(p-nitrophenyl)5-phenyl tetrazolium chloride (Balkwill and Ghiorse, 1984).

Microorganisms from ground water aquifers have been shown to be active on a variety of compounds such as styrene, chlorobenzene, toluene, and bromodichlorobenzene (Wilson et al., 1983a; Wilson et al., 1983b). In some cases, adaptation to the compounds may control biodegradation of contaminants (Wilson et al., 1984). Under aerobic conditions, rapid degradation of the contaminants was found in samples from contaminated areas of a site polluted by wood creosoting wastes, but in samples from a background site outside the plume of contamination, no degradation of the compounds was found. Adaptation to novel substrates can occur within as short a period as two days and may last for varying periods (Spain and Van Veld, 1983). Differences between sites and the ability of the microbial population to respond to contaminants may be expected. The microbial ecology of hazardous waste disposal sites is a relatively new research area, which may help to explain some of these differences. Microbial ecology investigations should provide information on the variability of the population, the role of different organisms in the aquifer ecosystem, and the activity of these organisms on organic contaminants. Environmental conditions also play an important role in the biodegradation of organic contaminants. The supply of dissolved oxygen was thought to be the limiting factor in the removal of several aromatic compounds at a site in Texas contaminated by wood creosoting wastes (Wilson et al., 1984; Lee and Ward, 1984). However, many compounds, particularly halogenated aliphatic hydrocarbons, are more likely to be degraded under anaerobic conditions (Bouwer et al., 1981). Anaerobic degradation of phenolic compounds to methane was demonstrated at a site contaminated by wood creosoting and coal tar wastes in Minnesota; but there was no evidence for anaerobic degradation of polynuclear aromatic hydrocarbons such as naphthalene (Ehrlich et al., 1982; Ehrlich et al., 1983). At other sites, inorganic nutrients in addition to oxygen limit biodegradation. Studies at sites where gasoline had been spilled showed that the addition of oxygen, nitrogen, phosphorus, and often other inorganic nutrients to the ground water increased the removal of the gasoline (Raymond et al., 1976; Raymond et al., 1978; Minugh et al., 1983). A spill of several solvents was treated in a similar manner (Jhaveri and Mazzacca, 1982).

RESEARCH AT CONROE, TEXAS, U.S.A., HAZARDOUS WASTE DISPOSAL SITE

One of the few places in which the microbiology of hazardous waste disposal sites has been studied is a site in Conroe, Texas, U.S.A. A wood creosoting plant operated at this site for about 30 years (Wilson et al., 1984). The waste from the operation was routed to a series of unlined pits where it infiltrated through a 5 to 6 m layer of intermixed sand and clay to a layer of poorly sorted sand at a depth of 8 to 9 m. A shallow water table lies at a depth of 7 to 8 m. The flow was estimated at 10 m/year. A number of monitoring wells have been installed at the site and show that organic contaminants including phenanthrene, naphthalene, dibenzofuran, fluorene, and 1- and 2-methyl naphthalene range in concentration up to 3 mg/l. Dissolved oxygen is found in the less contaminated wells, but very little is in the more highly contaminated wells due to microbial consumption.

The microbial population is active against the contaminants found at the site. Lee et al. (1984) found aerobic biodegradation of naphthalene, dibenzofuran, fluorene, anthracene, and pentachlorophenol in ground water samples collected from the site. These samples were collected as aseptically as possible to exclude non-native microorganisms. The samples of ground water were spiked with an aqueous solution of the five test aromatic compounds and incubated at the temperature of the aquifer (22°C). Biodegradation in the test samples and sterile controls were followed by solvent extraction of portions of the sample and quantification by gas chromatography. Degradation rates of up to 100% per week were noted for naphthalene (Table 1). The more highly contaminated wells had faster degradation rates for naphthalene, fluorene, and dibenzofuran due to the presence of an adapted microbial population, but removal rates for anthracene and pentachlorophenol, which were present at lower concentrations, were fairly constant for all the wells. Higher concentrations of the test compounds resulted in longer lag periods for naphthalene and dibenzofuran, but degradation was essentially complete after the same time. Further evidence for the microbial population's activity on these contaminants was shown by the isolation of microbes capable of utilizing four of the five compounds (all except pentachlorophenol) as sole carbon sources. Agar plates containing the compounds as sole carbon sources were prepared by adding 0.5 g of the compound to 1 L of a mineral salts medium, stirring overnight, adding 1.5% noble agar, and autoclaving.

Wilson et al. (1984) found more rapid degradation rates for aquifer soil samples than those reported in experiments with ground water. These samples were collected aseptically to exclude non-native microbes (Wilson et al., 1983a). These samples were collected by drilling bore holes to the desired depth and then exchanging the auger for a core-barrel. The core-barrel is pushed into the non-disturbed soil and withdrawn. The sample is extruded 3-5 cm and the end of the core broken off to expose a sterile face, and then extruded through a sterile parer into a sterile container. The outermost section and the ends of the core which could be contaminated are removed and can be saved to be used for sterile controls, extraction recovery experiments, and other work not requiring aseptic material. In aquifer samples taken from an area with only low levels of the contaminants at the Conroe site, aerobic microbial activity on naphthalene, 2-methyl naphthalene, dibenzofuran, fluorene, acenaphthene, and 1-methylnaphthalene was in excess of 90% removal per week (Wilson et al., 1984). In a more heavily contaminated area, degradation was more rapid for naphthalene, but slower for 2-methyl naphthalene and acenaphthene. There was no activity against these compounds in a pristine background site. This shows the importance of adaptation to the compounds. Non-biological removal rates of up to 11% per week were found in autoclaved controls.

These biodegradation studies were run on test tube microcosms designed by Wilson et al. (1983b). Aseptically collected subsurface material was slurried with water and sterile test tubes were filled with the slurry until about 1 ml of head space remained. The test tubes are filled with an aqueous solution containing the organic chemicals. The mouth of the tube was cleaned with sterile paper, sealed with a screw cap and the tube mixed on a vortex mixer. Abiotic controls were prepared by autoclaving soil for four hours and were treated in the same manner as the test samples. At the end of the desired incubation period, the subsurface material can be extracted and quantitated by gas chromatography. An alternative method for biodegradation studies is the use of ^{14}C-radiolabeled compounds to follow mineralization of the parent compound by the production of ^{14}C-CO_2. This technique offers the advantage of avoiding time-consuming solvent extractions and gas chromatography.

The previous work showed that there was a potential for rapid metabolism of the contaminants by an adapted population of microbes in the subsurface material. Why then are there still high levels of the contaminants remaining in the ground water? It is likely that the microbial population is limited by inadequate quantities of nutrients and/or dissolved oxygen. Lee and Ward (1984) investigated this problem by treating aseptically collected ground water samples from a moderately contaminated well with nutrients (nitrogen, phosphorus, potassium, carbonate, magnesium, calcium, manganese, sulfate, and iron) and dissolved oxygen and dissolved oxygen alone. Additional samples were incubated under a nitrogen atmosphere to simulate the oxygen limited conditions found in the aquifer, and another sample was filter-sterilized and amended with sodium azide to serve as a sterile control. The control was incubated aerobically. The biodegradation of naphthalene, dibenzofuran, fluorene, phenanthrene, and 2-methylnaphthalene was followed for 21 days in these samples by extractions using Sep-Pak C-18 cartridges and quantification with gas chromatography. Microbial counts were made on the samples using medium prepared by adjusting the pH of ground water to 5.0 (pH of the soil at the site), adding 1% agar, and autoclaving.

In the sample with nutrients and dissolved oxygen, removal of the compounds was rapid with biodegradation rates of about 55% per week noted for each compound (Table 2). There was a large increase in the number of cells/ml from day 2 to day 5 which occurred at the same time as a rapid decrease in the concentrations of the contaminants. During the 21-day test, 98% removal or greater was achieved. The sample with dissolved oxygen addition alone showed as rapid biodegradation rates for naphthalene and 2-methyl naphthalene as in the sample with added nutrients also, but slower rates were found for the other compounds. Total removal of the compounds approached that in the sample with nutrient addition. The biodegradation rates found in this experiment were slower than that found by Wilson et al. (1984) for subsurface material. This is evidence for the importance of the attached microbial population attached to subsurface soil particles. The number of cells/ml in the ground water with dissolved oxygen alone reached 10^4 within 5 days. Removal rates in the oxygen limited sample averaged 10%/week, which was much slower than that found in the other test samples. A microbial population of 10^4 cells/ml was detected, which was similar to that found in the dissolved oxygen addition alone sample, but the activity of the microbes was reduced. There was no microbial growth or activity against the contaminants in the sterile control.

This experiment showed that addition of dissolved oxygen increased biodegradation, but that while the addition of nutrients and dissolved oxygen increased the rates of biodegradation somewhat over simply the addition of dissolved oxygen, the overall removal was not significantly altered. The concentrations of dissolved oxygen in ground water at the Conroe site probably controls biodegradation. Additional experiments to determine the quantity of oxygen necessary for significant biodegradation would be beneficial.

MICROBIAL ECOLOGY

The microbial ecology of the subsurface environment has heretofore been neglected. Recently, the potential for the microbial degradation of xenobiotics in the subsurface has been documented (Lee et al., 1984; Lee and Ward, 1984; Wilson et al., 1983). Therefore, information concerning the microbial ecology of the subsurface is important in predicting how the indigenous populations will respond to contamination by hazardous materials.

The major difficulty in enumerating, isolating, and characterizing the ground water microflora is selecting appropriate methods. Conventional cultural techniques select for those microorganisms that are capable of growing on the chosen laboratory medium. The numbers and types of organisms determined using these methods may not reflect the true biomass and diversity. However, recent advances in methods to enumerate subsurface microbes have used epifluorescence microscopy (Ghiorse and Balkwill, 1981). These investigators reported 5.2 ($\pm$ 3.1) X 10^6 cells/g of oven-dried soil using epifluorescence microscopy, whereas viable counts on solid media were at least two times lower. Viable cell counts on nutrient-rich media were also determined to be lower than in nutrient-poor media (Ghiorse and Balkwill, 1981). Electron microscopy techniques are also available for direct observation of subsurface microorganisms. Methods using transmission electron microscopy (TEM)(Balkwill *et al.*, 1975) and scanning electron microscopy (Gray, 1967) are already used to observe microorganisms in surface soil samples. Wilson *et al.* (1983) reported that thin sections of samples examined by TEM revealed only bacteria in the subsurface. Ghiorse and Balkwill (1983) reported that two-thirds of the subsurface bacteria from Ft. Polk, Louisiana, possessed gram-positive-type cell envelopes when observed by TEM.

Ehrlich *et al.* (1983) conducted an experiment to determine the microbial ecology of a creosote-contaminated aquifer at St. Louis Park, Minnesota. Direct counts using epifluorescence microscopy ranged from 7×10^4 to 1×10^7 cells/ml of ground water. These investigators also determined that the Most Probable Number of aerobes and anaerobes in the aquifer ranged from 9×10^2 to 4×10^6 and 2×10^1 to 4×10^4/100 ml, respectively. Denitrifying, iron and sulfate reducing, and methanogenic bacteria were also detected.

Diversity measurements of the subsurface environment have revealed a limited number of bacterial genera. Ehrlich *et al.* (1983) reported species of *Pseudomonas* and *Alcaligenus* that were isolated from enrichment cultures with aromatic compounds as the sole carbon source. These investigators also isolated methanogenic bacteria, which resembled *Methanobacterium soehrigenii*, *Methanobacterium byantii*, and *Methanospirillum hungatii* from enrichment cultures. Stetzenbach *et al.* (1984) identified species of *Acinetobacter* and *Morexella* from ground water at 200 ft. Evidence for eucaryotes in the subsurface is scarce; however, Balkwill and Ghiorse (1984) have recently reported the presence of a cyst-forming protozoan in subsurface samples.

To investigate the microbial ecology of a hazardous waste disposal site in Conroe, Texas, direct counts using epifluorescence microscopy and viable counts are being performed. Microbial diversity will be measured using numerical taxonomy, an arithmetical method that clusters organisms together on the basis of their overall similarities. The isolates will be examined for characteristics that will provide information concerning important ecosystem functions as well as traits for adaptation to high concentrations of xenobiotics. The diversity measurement will be used as a criterion to predict how an aquifer may cope with contamination by hazardous materials.

CONCLUSIONS

The microbial population native to the subsurface soil of hazardous waste disposal sites plays a very important role in the removal of contaminants from ground water. A site in Conroe, Texas, U.S.A., contaminated by wood creosoting wastes provided an opportunity to study the microbiology of a hazardous waste disposal site. The microbial population was shown to degrade the contaminants found at the site under aerobic conditions. Degradation rates were faster for aquifer soil samples than ground water samples. Microbes from a contaminated area were adapted to the compounds and were able to degrade the compounds at rates in excess of 90% per week, but organisms from a background site were not adapted and could not degrade the contaminants. The microbial population was limited by the quantity of dissolved oxygen available to them. Addition of nutrients (nitrogen, phosphorus, and other essential inorganics) increased the biodegradation rates somewhat above that of dissolved oxygen addition alone, but did not significantly increase the overall removal of the contaminants at this site. Characterization of the microbial ecology of the site is now underway. These studies will help provide the information necessary to predict the extent of microbial activity at hazardous waste sites and ways in which this activity can be increased.

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Table 1. Degradation of Aromatic Compounds in Ground Water from a Hazardous Waste Disposal Site

Experiment #1

Compound[1]	Per Cent Degradation/Week Well #1	Well #1 Autoclaved[2]	Well #3	Well #5
Naphthalene	100	10.7	23.1	100
Dibenzofuran	38.9	5.4	18.2	19.7
Fluorene	29.3	12.7	15.2	29.4
Anthracene	13.2	12.9	15.9	13.3
Pentachlorophenol	14.0	11.4	15.2	12.5

[1] About 0.03 mg/l of each compound was added to each sample.

[2] Became contaminated during course of experiment.

Experiment #2

Compound[1]	Per Cent Degradation/Week Well #5	Well #5 Autoclaved	Well #3	Well #3 Autoclaved
Naphthalene	50.0	14.3	100	8.3
Dibenzofuran	50.0	11.3	12.3	0
Fluorene	31.0	10.0	5.9	0
Anthracene	21.0	0	94	11.9
Pentachlorophenol	15.2	11.3	17.5	14.8

[1] About 0.1 mg/l of each compound was added to each sample.

Wells 1 and 5 were moderately contaminated.

Well 3 was uncontaminated.

Table 2. Biodegradation Rates and Removal Efficiencies for Polynuclear Aromatics Under Various Treatments

Treatment	Naphthalene	2-Methyl Naphthalene	Dibenzofuran	Fluorene	Phenanthrene
		BIODEGRADATION RATES (%/week)			
A	54	52	57	54	57
B	58	56	37	33	29
C	23	16	11	0	17
D	0	0	0	0	*
		PER CENT REMOVAL			
A	98	99	100	100	100
B	99	99	94	89	81
C	69	57	12	0	27
D	0	0	0	0	*

* none detected - probably sorbed onto filter

detection limit = 1 μg/l

Treatment

A. Nutrient and Dissolved Oxygen Addition

B. Dissolved Oxygen Addition

C. Oxygen Limited

D. Sterile Control

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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MICROBIAL INVOLVEMENT IN THE REMOVAL OF TRACE ORGANICS DURING RAPID INFILTRATION RECHARGE OF GROUND WATER

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ABSTRACT

A series of soil column tests and field experiments were designed to evaluate microbial removal of trace organics during rapid infiltration recharge of ground water. Column tests using acclimated soil from an operational system demonstrated good removal of trace organics. Increased concentrations of target compounds in the feed did not always result in corresponding increases in the column effluent. Microbial adaptation was evident for some compounds. Other compounds appeared to exhibit a minimum concentration below which biodegradation did not proceed. Microbial activity was confirmed as a fate mechanism for several target compounds using radiolabels. In direct correlation with field results, the induction of anaerobiosis in the soil columns resulted in increased fractional breakthrough of trace organics.

Keywords: Land application, rapid infiltration, trace organics, soil columns, biodegradation

INTRODUCTION

In recent years, population growth and urbanization have led to increased production of municipal and industrial wastes which must be treated and disposed. One of the management technologies being used to treat these wastes in an environmentally acceptable manner is land application. Land application is generally effective for the removal of many wastewater constituents such as fecal coliforms, heavy metals, phosphorus, and carbonaceous biochemical oxygen demand (Bouwer and Chaney, 1974; Iskandar, 1978; Olson *et al.*, 1980). There have been few comparable studies on trace organic fate in land application systems.

The effects of toxic organic chemicals are the least known among all wastewater constituents (Majeti and Clark, 1981). This is of particular concern in rapid infiltration systems because, unlike other land application practices, rapid infiltration depends primarily on percolation for wastewater disposal and thus has the greatest potential for affecting associated ground waters (Environmental Protection Agency, 1977). Ground water has recently been recognized as a limited and vulnerable natural resource whose protection is requisite to meeting the water needs of the nation. Yet most studies indicate that little attenuation of organic chemicals should occur in rapid infiltration systems (Sills *et al.*, 1978; Jenkins *et al.*, 1983). Furthermore, there is virtually no information on long-term degradation of trace organics in ground water (McCarty *et al.*, 1981).

A survey of several rapid infiltration sites in the U. S. has demonstrated that many wastewater trace organics are completely or mostly removed by the treatment process although detectable concentrations of recalcitrant compounds are still present in the associated ground waters (Tomson *et al.*, 1984). Other research using soil columns has indicated that biodegradation may be partially responsible for this removal (Bouwer *et al.*, 1981). We therefore evaluated trace organic transport and fate using soil columns in a series of tests designed to address the effects of microbial involvement in trace organic removal. It is not possible to present the complete results of the soil column tests and the field study in the text of this short paper. Instead, a summary is presented of the methods used and the results obtained. Factors affecting microbial removal of trace organics were evaluated by correlating the results from the separate tests to suggest strategies for enhancing removal of trace organics during rapid infiltration of wastewater.

MATERIALS AND METHODS

Field study

A field study was conducted to evaluate trace organic removal at an operational site. Ground water from a 40 yr-old rapid infiltration facility at Fort Devens, Massachusetts was sampled for trace organic contamination. A subsequent field study was undertaken to determine the primary region where trace organics were removed in the system. Three Teflon monitoring wells were installed at a depth of 1.2 m in one of the basins at the site prior to flooding. Flooding was initiated and samples of the raw wastewater, the wastewater entering the basin, the infiltrate at 1.2-m depth, and the ground water downgradient of the site were obtained continuously over the 6-d flooding period. Sample streams were pumped through XAD-4 resin columns and the columns were replaced every two days. Resin extracts in this study and the next two tests were analyzed for six selected wastewater organics: p-dichlorobenzene (PDCB), 2-methylnaphthalene (2MN), o-phenylphenol(OPP), p-(1,1,3,3-tetramethylbutyl)-phenol (TMBP), 2-(methylthio)benzothiazole (2MTBT), and benzophenone (BZPN).

Column study

The column test was designed to evaluate microbial removal of wastewater organics as a function of input concentration for an operational system and to compare trace organic mineralization in acclimated and non-acclimated soil. Subsequent to the field study, soil was obtained from the Fort Devens site and packed into eight columns. The soil columns and associated plumbing were constructed of Teflon and glass to minimize adsorption and leaching of trace organics (Fig. 1). The columns received feed solution from reservoirs equipped with Mariotte siphons to maintain a constant level of feed above the soil. After a flooding period ended, the siphon was disconnected and the columns drained until the next flooding period began. A proportioning pump was connected to the column effluent line for controlling infiltration rates. Feed solutions and column effluents were analyzed for trace organics using Amberlite XAD-4 resin and gas chromatography/mass spectrometry as described elsewhere (Hutchins et al., 1983). One resin column was used to adsorb trace organics from each soil column effluent during the entire flooding period of a single inundation cycle. An identical analysis was performed on the feed solution so that fractional breakthroughs (mass output/mass input) of each compound could be calculated. Concentrations of each compound were estimated based on the mass eluted from the resin and the volume sampled. Biocide was added to the sample stream prior to contact with the resin so that biodegradation of sorbed trace organics would be inhibited during sampling.

Primary wastewater was separated into four aliquots, and each aliquot was amended with the six wastewater organics evaluated in the field study so that the aliquots yielded final concentrations of approximately 1, 10, 100, and 1000 μg/l of each compound. Each set of duplicate soil columns received feed from one of the aliquots on a 6-d flooding/16-d drying cycle at an average infiltration rate of 1.4 cm/hr. After the third inundation cycle, one of the soil columns which had received the compounds at 10 μg/l concentration was dismantled. Soil samples from this column and from a site adjacent to the basins at Fort Devens were used to test for mineralization of radiolabeled compounds.

The remaining columns were then modified so that anaerobic conditions could develop during flooding. Each column was initially flooded with unfiltered primary wastewater for six days to deplete the residual oxygen in the system. After the system became anaerobic, feed solutions were prepared and the columns were operated anaerobically for an additional 6-d flooding period.

RESULTS

Field study

Most of the removal occurred in the top meter of the basin soil during infiltration. This removal was consistent for all six selected compounds during the first two days of the flooding period; basin infiltrate concentrations were generally an order of magnitude lower than floodwater concentraations (Table 1). However, basin infiltrate concentrations of all six compounds began to increase at two of the sample points as flooding continued (Table 2). Concentrations of PDCB, 2MTBT, TMBP, and BZPN in the basin infiltrate at one of the sample points exceeded floodwater concentrations during the final two days of flooding.

Column study

The relationship between feed concentration and column effluent concentration was different for virtually each compound studied. Breakthrough profiles for OPP were consistent during the test with column effluent concentrations increasing proportionately with input concentrations, i.e., fractional breakthrough was constant (Fig 2). Consistent profiles were also observed for 2MTBT although fractional breakthroughs were higher at lower input concentrations (Fig. 3). The behavior of PDCB was similar to that of 2MTBT after the first inundation cycle with the exception that increased fractional breakthroughs were observed at the highest input concentration as well (Fig. 4). Microbial adaptation was evident for BZPN and TMBP as indicated by increasing removal efficiencies during successive inundation cycles, especially at the higher input concentrations (unpublished data). Column effluent concentrations of 2MN were independent of feed concentrations during the second inundation cycle, and 2MN could not be detected in any of the column effluents afterwards. Similarly, column effluent concentrations of TMBP stabilized during the third inundation cycle, and the data for BZPN indicated that a similar trend might occur with time (Figs. 5,6). No significant mineralization of PDCB, TMBP, and 2MTBT occurred in the non-acclimated forest soil from the site, whereas mineralization of the compounds in the column soil ranged from 0.4 to 20% of the theoretical input for 2MTBT and OPP respectively during the 16-hr incubation (Table 3). With the exception of BZPN, each compound was more rapidly mineralized in the acclimated column soil than in the non-acclimated forest soil. Rates of removal were 20 to 1000 times faster than rates of mineralization for OPP and 2MTBT respectively.

Fractional breakthrough profiles for OPP, TMBP, and 2MTBT did not change when the soil became anaerobic, although the degree of breakthrough increased for each compound compared to aerobic operation (Figs. 2-6). Anaerobic conditions also promoted increased fractional breakthrough of PDCB, although the effect was most pronounced at the lower input concentrations. For BZPN, the effect was most pronounced at both the lowest and highest input concentrations. In contrast, 2MN could not be detected in the column effluent during either aerobic or anaerobic operation. Leaching was evident for OPP and BZPN, because effluent concentrations from some of the columns were higher than feed concentrations.

DISCUSSION

Microbial activity was evaluated in the column test by examining the effect of concentration of the compound on its removal during infiltration. Observed rates of biodegradation are almost always proportional to substrate concentration (Larson, 1980). This behavior has similarly been noted for mineralization rates of certain compounds in lake and stream water (Boethling and Alexander, 1979a; Rubin et al., 1982; Rubin and Alexander, 1983). For all input concentrations tested, the concentration of the compound in the column effluent was proportional to input concentration only for OPP. The removal of OPP is therefore first-order in kinetics and the removal rate constant is independent of the initial concentration. Because several abiotic removal processes in nature are first-order, the breakthrough profile of OPP is not necessarily an indication that microbial activity is involved. Evidence of this is given by the occurrence of mineralization during the isotope study. For the other compounds, however, evidence of microbial activity is also given by the shape of the breakthrough profiles because removal processes are shown to be less efficient at lower concentrations. Low concentrations do not impose constraints on abiotic first-order removal processes; the data therefore suggest that biotic mechanisms may be involved. This observation is consistent with the concept of a minimum substrate concentration (McCarty et al., 1981), which states that a threshold concentration of a substrate can exist below which no further biodegradation occurs. This would be expected if the energy derived from low concentrations of the substrate was inadequate for maintenance of the bacterial cell, or if higher concentrations were required to activate the cell's transport and metabolic systems (Boethling and Alexander, 1979b). This concept was developed from the study of biofilms and is applicable only to the metabolism of a primary substrate. Concentrations of the test compounds were relatively minor compared to the total organic carbon content of the wastewater which should presumably serve as sources of primary substrates. Theoretical considerations by McCarty et al. have indicated that the presence of additional carbon sources can lower the threshold concentration of the primary substrate, and this has been demonstrated experimentally by other researchers (Law and Button, 1977). In addition, certain oligotrophic organisms can preferentially utilize low rather than high concentrations of substrates (Ishida and Kadota, 1981; Ishida et al., 1982). In each case, however, the primary substrate has been an easily-oxidizable compound, and at least one study has shown that these organisms may not be able to assimilate a wide variety of compounds (Van der Kooij and Hijnen, 1983). The lack of significant mineralization observed for PDCB, BZPN, and 2MTBT indicates that these compounds are probably not being used as primary substrates by the soil microorganisms. If these compounds are being transformed via cometabolic pathways, no threshold concentration should theoretically exist (Bouwer and McCarty, 1983). The fact that threshold concentrations were observed for 2MTBT and BZPN indicates that these theoretical considerations are insufficient to explain their fate in soil. Other factors may exist which could result in either a concentration limit for the removal of the compound or in a controlled release of the compound into the column effluent, although the nature of these other factors is unknown.

Trace organic removal during the field study was comparable to that observed in the column test for the soil columns receiving low concentrations of the compounds (Table 1). However, removals were consistent in the column test whereas they began to decline in the field study for two of the sample points towards the end of the flooding period (Table 2). It was hypothesized that this decline was the result of anaerobic conditions which developed with flooding, and this hypothesis was tested during the subsequent column test. With the exception of 2MN, fractional breakthroughs of each compound increased when the soil columns became anaerobic (Figs. 2-6). Anaerobic conditions generally increase an organic compound's recalcitrance to biological attack (Alexander, 1965). This is especially true for aromatic compounds. Although some aromatics can be degraded or transformed even under strict anaerobic conditions, these compounds generally require the presence of an oxygenated substituent group for this to occur (Rose, 1976; Healy and Young, 1979;

Horowitz et al., 1983). Phenolic compounds would therefore be expected to be somewhat labile under anaerobic conditions (Bakker, 1977; Ehrlich et al., 1982); whether or not this occurs with OPP and TMBP in this sytem is unknown. It is also difficult to assess the role of anaerobic biodegradation in the fate of 2MN, 2MTBT, and BZPN because of the lack of data for these classes of compounds. In agreement with the data for PDCB, however, other researchers have observed marked decreases in rates of mineralization and biotransformation for halogenated benzenes under anaerobic conditions (Marinucci and Bartha, 1979; Schwarzenbach et al., 1983).

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Table 1. Trace organic data for raw sewage (duplicate), basin floodwater (triplicate), and basin infiltrate (triplicate) samples during first two days of flooding period.

Trace Organic	Raw Sewage	Basin Floodwater	Basin Infiltrate
		μg/l	
PDCB	0.67 ± 0.10[a]	0.32 ± 0.11	0.044 ± 0.013
2MN	0.33 ± 0.01	0.22 ± 0.07	0.012 ± 0.015
OPP	0.93 ± 0.06	0.43 ± 0.17	0.037 ± 0.037
TMBP	0.79 ± 0.11	0.44 ± 0.22	0.17 ± 0.07
2MTBT	0.39 ± 0.01	0.59 ± 0.27	0.091 ± 0.025
BZPN	0.73 ± 0.44	0.40 ± 0.29	0.040 ± 0.026

[a] Mean ± standard deviation

Table 2. Trace organic data for raw sewage (duplicate), basin floodwater (triplicate), and basin infiltrate (triplicate) samples during final two days of flooding period.

Trace Organic	Raw Sewage	Basin Floodwater	Basin infiltrate		
			Well 1	Well 2	Well 3
		μg/l			
PDCB	0.92 ± 0.14[a]	0.43 ± 0.29	0.033	0.20	0.60
2MN	0.30 ± 0.06	0.27 ± 0.02	0.014	0.019	0.23
OPP	0.82 ± 0.08	0.92 ± 0.19	0.032	0.47	0.87
TMBP	0.56 ± 0.01	0.75 ± 0.34	0.15	0.29	1.1
2MTBT	0.33 ± 0.02	0.69 ± 0.31	0.084	0.47	0.96
BZPN	0.44 ± 0.41	0.33 ± 0.08	0.20	0.41	0.88

[a] Mean ± standard deviation

Table 3. Mineralization of test compounds in acclimated soil from one of the columns used in the column test and in non-acclimated forest soil.

Soil Sample	CO_2 Evolved (% of Theoretical ± Standard Deviation)[a]					
	PDCB	2MN	OPP	TMBP	2MTBT	BZPN
Forest Soil and Biocide	0.75 ± 0.09 [b]	0.86 ± 0.05	0.29 ± 0.03	0.57 ± 0.25	0.16 ± 0.10	0.47 ± 0.13
Forest Soil	0.72 ± 0.08	6.18 ± 2.24	1.36 ± 0.18	0.86 ± 0.55	0.27 ± 0.11	1.78 ± 0.17
Column Soil and Biocide	2.63 ± 0.06	0.64 ± 0.08	0.24 ± 0.02	0.66 ± 0.12	0.14 ± 0.08	0.45 ± 0.13
Column Soil	6.52 ± 0.63	16.3 ± 3.0	20.7 ± 1.2	7.0 ± 1.9	0.58 ± 0.15	2.38 ± 0.55

[a] Three samples
[b] Brackets indicate that the difference between the values is not significant at the 95 percent confidence level.

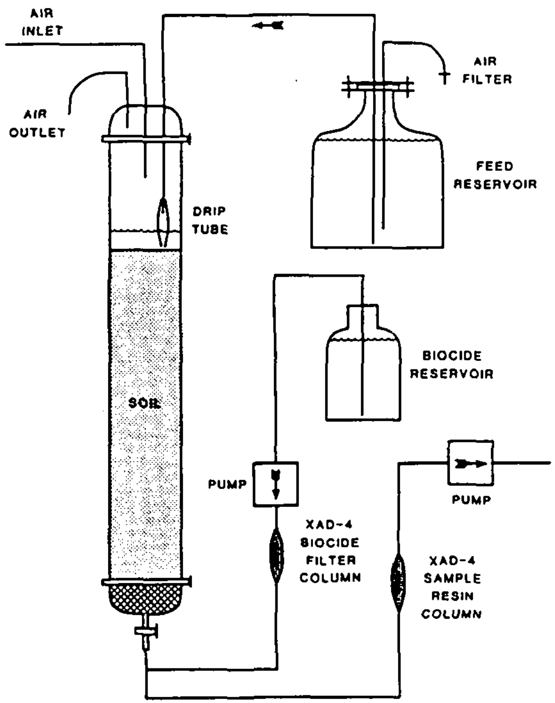

Fig. 1. Column design schematic.

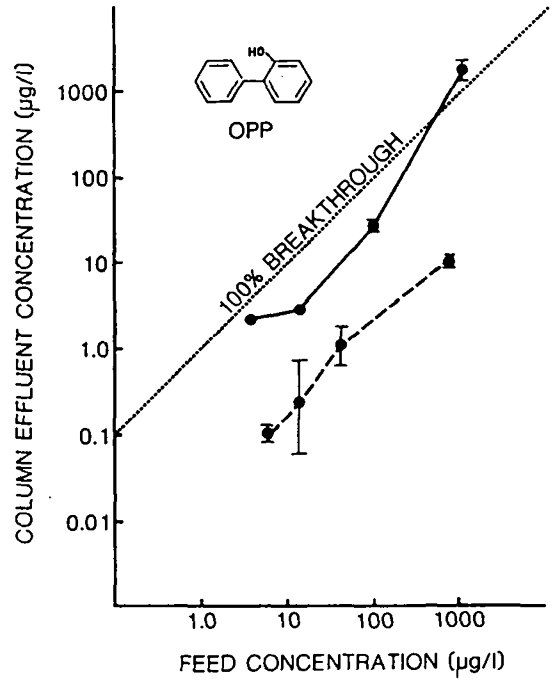

Fig. 2. Breakthrough profile for OPP during aerobic (--) and anaerobic (—) operation of soil columns. Column effluent concentration is average ± standard deviation for two replicates (Hutchins et al., 1984b).

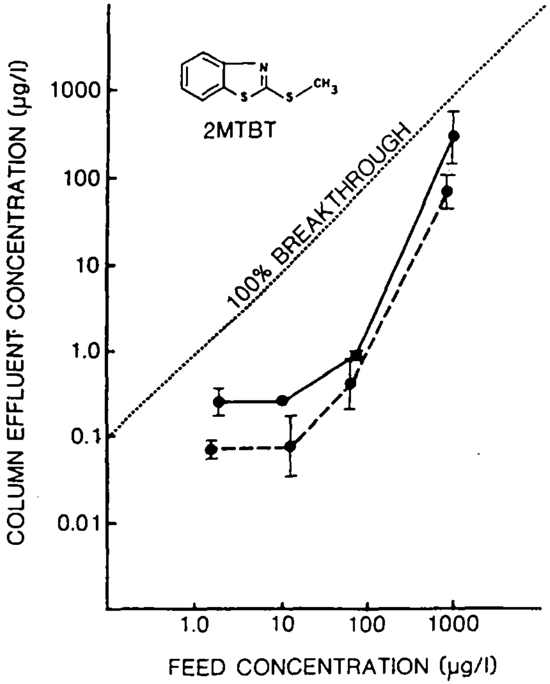

Fig. 3. Breakthrough profile for 2MTBT during aerobic (--) and anaerobic (—) operation of soil columns. Column effluent concentration is average ± standard deviation for two replicates (Hutchins *et al.*, 1984b).

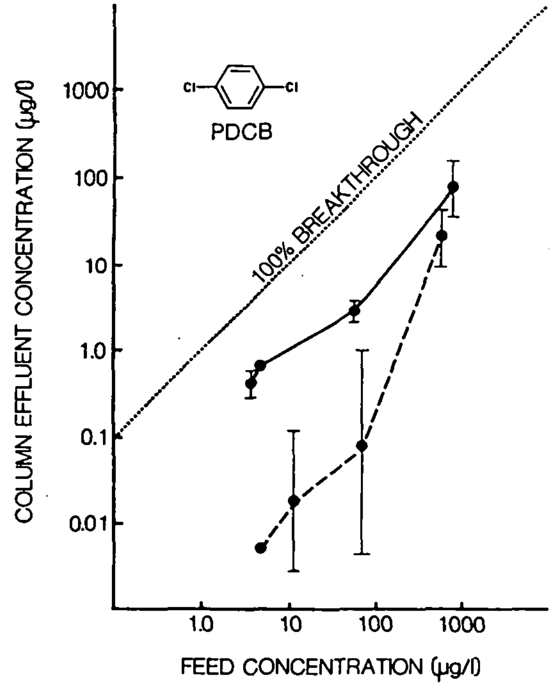

Fig. 4. Breakthrough profile for PDCB during aerobic (--) and anaerobic (—) operation of soil columns. Column effluent concentration is average ± standard deviation for two replicates (Hutchins *et al.*, 1984b).

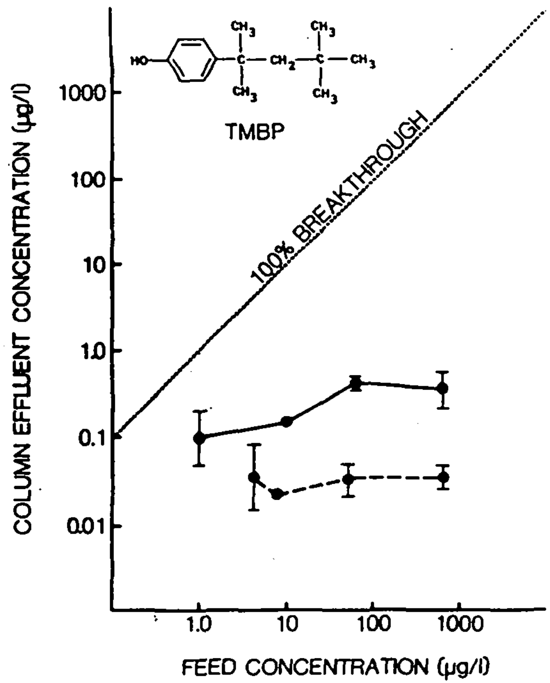

Fig. 5. Breakthrough profile for TMBP during aerobic (--) and anaerobic (—) operation of soil columns. Column effluent concentration is average ± standard deviation for two replicates (Hutchins et al., 1984b).

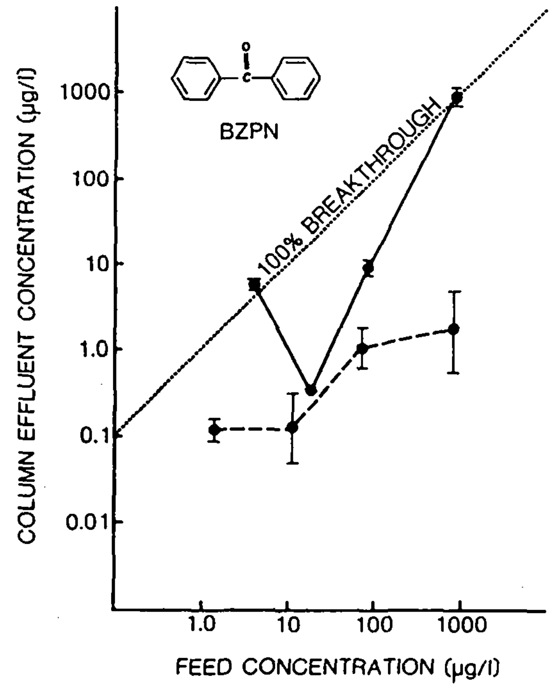

Fig. 6. Breakthrough profile for BZPN during aerobic (--) and anaerobic (—) operation of soil columns. Column effluent concentration is average ± standard deviation for two replicates (Hutchins et al., 1984b).

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PACKED WATER TREATMENT PLANT FOR RURAL COMMUNITIES

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ABSTRACT

The packed water treatment plant has been developed for small rural communities taking raw water from superficial sources. The unit is based on chemical precipitation and separation on flocs in the lamellar settling module and in the buoyant layer of discrete particles having the density less then water. The granular filtration layer is formed by foamed styrene balls, having the diameter of approximately 1 mm with a density of 80 kg.m^{-3}. The depth of the layer is 0,8 m. The inflowing suspension enters underneath the filtration layer and flows upwards at the rate of 6 m.h^{-1}. The washing of the filter is by means of a siphon with automatic operation. The treated water is in the storage space above the filtration layer. During the washing it flows in a downward direction. The expansion of the filtration layer is by 30 to 100 %. The washing lasts approximately 60 seconds. During the washing the pumping of raw water with a dose of the coagulating agent is not stopped.

Keywords : Packed water treatment plant, chemical precipitation, lamellar settling module, sludge blanket, buoyant filtration layer, styrene balls, siphon washing, activated charcoal.

INTRODUCTION

The growth and modernization of agricultural production is placing ever higher demands onto the provision of sufficient supplies of good quality water. There exist ever increasing demands for a plant, the operation of which places no great demands onto energy consumption as well as attendance, which does not require a lot of room, is accessible in price and can adapt itself flexibly to changes in the offtake of the treated water.

The paper comments a small complex water treatment plant designed for performances from 3 $m^3.h^{-1}$ up to 36 $m^3.h^{-1}$ with the possibility of combining several units into blocks. The plant is intended primarily for agricultural settlements.

TECHNOLOGICAL ARRANGEMENT

The water treatment technology is based upon the chemical coagulation by aluminium or iron salts, and with the aid of auxiliary coagulants, respectively.

The block diagram of the water treatment technology is shown in Fig. 1. The raw water flows into the sump (1), from which it is transported by the pump (2) via the homogenizing mixer (3) into the flocculation space (6). The coagulant solution is proportioned by the pump (4) from the bin (5). The flocculating suspension flows downwards through the flocculator (6), passes underneath the submersible wall and through an upflow continues through the sludge blanket layer (7), or the settling module (8) and the filter (9), above which the treated water tank (10) is arranged. The treated water overflows into the AC filter (11) filled with activated charcoal, and is taken off for consumption. The separated flocculated sludge sinks into the thickening space (12), from which it is periodically discharged for dewatering into the bag filter (14). During the washing of the contaminated filter with the buoyant filtration layer (9), the treated water flows from the tank (10) by gravity via the settling tank (8) into the wash water sump (13).

HOMOGENIZATION

For the homogenization of the water and the coagulant solution is utilized a doubled pipeline orifice plate which makes possible the gradual proportioning of the coagulants. The mean value of the velocity gradient G of the rapid mixing is /1/

$$G = (W/\mu)^{1/2} = 1\,000 \text{ sec}^{-1} \qquad (1)$$

where: μ is the dynamic viscosity of the water /Pa.s/
W is the mean value of the dissipated energy /$W.m^{-3}$/.

The ratio of the orifice plate diameter d and the pipeline diameter D, d/D = 0.5.
The doubling up of the orifice plates with a spacing of l = 7 D improves the mixing conditions. The retention period of the homogenization lies under 1 second.

FLOCCULATION

The value of the velocity gradient in the flocculation phase is since G = 500 s^{-1} to G = 80 s^{-1}. These conditions are obtained by the outflow of the mix through a number of openings in the distribution pipe, and by the flow through a layer of buoyant granulated plastic particles (e.g. polypropylene with a density of 850 to 950 $kg.m^{-3}$.
The flocculating space is designed for a retention period of 15 to 20 minutes. A prolongation of the flocculation period by the flow of the suspension through the sludge blanket space is taken into account in some installations, so that the total flocculating period is 20 to 26 minutes.

SETTLING IN THE MULTI-PLATE SETTLING MODULE

For the separation of the overwhelming proportion of the flocs is envisaged the tubular-type multi-plate settling module. The slope of the trapezoidal plates is 60 °. The suspension entering into the tubular micro-spaces is mixed with the settled sludge which flows down along the bottom of the plates, and forms in the bottom part of the tube a sludge blanket which assists the enhancement of the separating efficiency. The concentration of the sludge in the micro-space decreases rapidly in the direction of flow. At the entry into the tube is assumed a turbulent character of the flow which is gradually converted into a laminar flow. In the construction of the described equipment are used plates with a length of 1 100 mm.

The length of the turbulent flow l' was defined /2/ by a relationship derived for cylindrical tubes:

$$L' = 0.058 \frac{v_o . d}{\nu} = \frac{l'}{d} \qquad (2)$$

where: v_o is the axial flow rate in the micro-space /$m.s^{-1}$/

d is the tube diameter /m/

ν is the kinematic viscosity of the liquid /$m^2.s^{-1}$/

It is assumed under practical conditions that the maximum length of the turbulent and transient region will not exceed half the length of an entire plate.

The separation efficiency in the multi-plate module lies for the envisaged range of flow rates between 95 and 65 % and depends on the quality of the flocs. When designing the equipment, we start out from the condition that on the outlet from the multi-plate module the flocs content does not exceed the value of 10 to 20 $g.m^{-3}$. With due regard to this condition (with respect to the sludge capacity of the filtration layer and the economic operation of the filter) we rate the maximum value of the surface loading of the settling tank. For floccular suspensions after the chemical coagulation with iron salts this is a maximum of 3 $mm.sec^{-1}$.

FILTER WITH A BUOYANT FILTRATION LAYER

Instead of the classical sand filter is employed the filter with a buoyant filtration layer /3/, the construction of which has been perfected by further inventions to its contemporary form and function. The granular filtration material are foamed polystyrene balls with a diameter of 0.5 to 1.4 mm. The density of the layer is 40 to 70 $kg.m^{-3}$. The ball layer with a thickness of approx. 500 to 800 mm is held by the filtration bottom below the level of the treated water. The flow through the buoyant filtration layer takes place from the bottom. The colmatation of the filtration layer proceeds in the direction of the flow of the treated water, i. e. from the bottom. The high difference of the densities of the water and foamed polystyrene balls facilitates the formation of a permeable filtration layer of an analogical quality as in sand filters of the classical construction /4/. For the practical application of the colmatation phenomenon was employed the equation :

$$h = h_o + K \cdot c_o \cdot t \cdot v \qquad (3)$$

where: h = head loss during flow through the filtration layer /m/

h_o = the starting value of the head loss for the clean layer /m/

c_o is the mass concentration of suspended materials in the onflow /$kg.m^{-3}$/

K is the coefficient of the given layer /$m^3.kg^{-1}$/

t is the time /s/

v is the superficial flow rate of the liquid /$m.sec^{-1}$/

The separating efficiency of the filter with a buoyant layer lies between 99 and 95 % and the treated water satisfies the requirements of the standard for potable water.
The flow through the filtration layer in an upward direction facilitates the immediate connection of the filter onto the previous separating stage. The overall height of the equipment is practically increased by approx. 1 m and thus the erection of the complex: sludge thickener - settling tank - filter is very simple and non-exacting.

Washing of contaminated filtration layer

The treated water which has passed through the filtration layer, is discharged from the equipment through an adjustable overflow through, the level of which lies approx. 800 mm above the filtration partition wall. The space between the partition wall and the overflow forms a treated water tank for washing the contaminated filtration layers. The washing of the layer takes place against the direction of filtration, i. e. in a downward direction. For a perfect washing of the contaminated layer suffices a gravitational discharge of the water from the tank through the layer and through the multi-plate module.
It was verified experimentally that for the perfect washing of the filtration layer is sufficient a quantity of 0.7 to 0.8 $m^3.m^{-2}$ which flows through the layer being washed at a ra-

te of 10 to 35 mm.sec^{-1}. The expansion of the filtration layer is 30 to 100 % its thickeness. This permits a very quick washing period generally 20 to 60 seconds. With regard to the relationship between the onflow and the quantity of the discharged wash water, it is not necessary to close the inlet of the onflowing water (the losses are slight). Since the washing proceeds without power requirements, the power-saving contribution of this method of filtration and washing of the contaminated layer in comparison with sand filters is readily apparent.

The signal for starting the washing of the layer is its resistance. If we sense the head loss growth of the onflowing suspension, e.g. by means of a float sensor we can control the washing automatically with the aid of a remotely controlled enclosure of the wash water discharge siphon. The discharge of the contaminated wash water is interrupted by an automatic aeration of the siphon and the shut-off fitting is closed by means of a timing relay. The optimum head loss value which is the signal for washing the filter, is adjusted individually in accordance with the quality of the treated water and the employed coagulation technology.

The wash water is retained in an auxiliary sump, from which it is slowly drained during the entire filtration period into the onflowing pumping sump. The proportion of the returned water does not generally exceed 2.5 % of the treated quantity, and thus does not practically influence the power input of the charging pump.

Smaller complex units are controlled by a siphon with a liquid enclosure, whereas in larger units a remotely controlled flap valve is used which makes possible completely regular operation.

ACTIVATED CHARCOAL LAYER

The described water treatment rechnology will make possible the separation of practically all the suspended materials. The raw water which is mostly taken from surface streams, can contain a certain quantity of organic and odour forming materials, respectively, the removal of which by chemical coagulation is not very effective.

In order to improve the quality of the treated water, the complex treatment plant contains an AC filter filled with granulated activated charcoal. The height of the filtration layer is 1.5 to 2 m, the total contents of the filter filling amount to 1 to 4 m^3. A granulate with a grain size of 2 to 4 mm is used. The activated charcoal layer is supported by a bridge screen with openings 1 x 25 mm.
The water discharge from the overflow through in the wash water tank can be alternatively routed via the AC filter or directly to the offtake point.

CONSTRUCTION OF THE PLANT

The basis for the construction of the complex water treatment plant is the link-up of the fundamental separating operations into a common vertical block in a rectangular stand-up tank. A vertical cross section through the equipment is shown in Fig. 2.
The main articulation of the tank space is influenced by the position and shape of the multi-plate settling tank, the plates of which exhibit a slope of 60 °. Thus a space is formed for locating the flocculator and also for creating the AC filter. The actual functional space of the tank is supplemented by an auxiliary space ("machine room"), in which pumps, the electrical switchboard and pipelines are installed. This space is formed by an elongation of the longitudinal tank walls.
The raw water (1) is pumped by the pump (2), in the delivery pipeline on which is inserted a doubled orifice mixing plate (3), into the perforated distributor pipe that is built into the peak of the flocculating space (4). By the screen is separated the layer of buoyant granules for the mixing of the flocculating suspension. In the bottom section of the flocculator space are inserted partition rods (5) for the slow hydraulic mixing of the suspension. The oblique ceiling of this space is formed by the submersible wall.
The flocculated suspension flows under the submersible wall and passes upwards into the multi-plate settling module (6), the plates of which are supported by bars. The water rises from the settling module into the filter with the buoyant layer (7) which is formed by small polystyrene balls. The filtration

screen is mounted on the vertical walls of the tank. The treated water tank (8) is provided at the overflow level with an overflow through. The treated water overflows onto the AC filter (9), the filling of which is supported by the slotted filtration bottom. The treated water is discharged from the AC filter through the branch with a siphon overflow (10).

The settled sludge is thickened in the tapered space (13).

The surrouding space forms a sump for holding back the contaminated wash water.

The wash water is discharged from the space underneath the multi-plate module (8) through the slotted arm via the siphon (11), in the bottom section of which is inserted the servodrive controlled shut-off valve. The auxiliary pipe aerates the upper space of the siphon (11), as soon as the water level in the tank (8) drops to the filtration bottom.

The siphon is operated on the base of head loss by remote controll circuit (12).

The coagulating agent solution is proportioned by the pump before the mixer (3).

Water disinfection is effecteated by dosing of solution of sodium chloride.

The equipment tank, including the built-in partition walls and intermediate walls, is made from steel sheets and provided with a coat of water-resistant polymerate paints. To facilitate handling during loading operations for transport and installation, the tank is provided with suspension eyes.

CONCLUSION

The main advantages of the packed water treatment plant are:

- All separation processes follow upwards,
- all washing processes follow downwards,
- no need of washing pump or blower,
- full automation and little need of supervision,
- high mobility,
- little demands on built-up space,
- production in an industrial manner.

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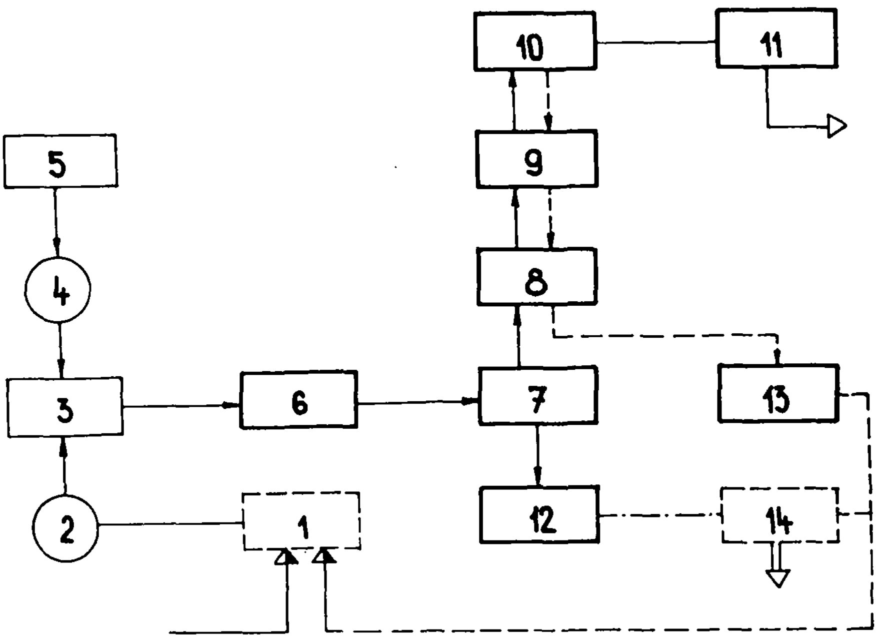

Fig. l.

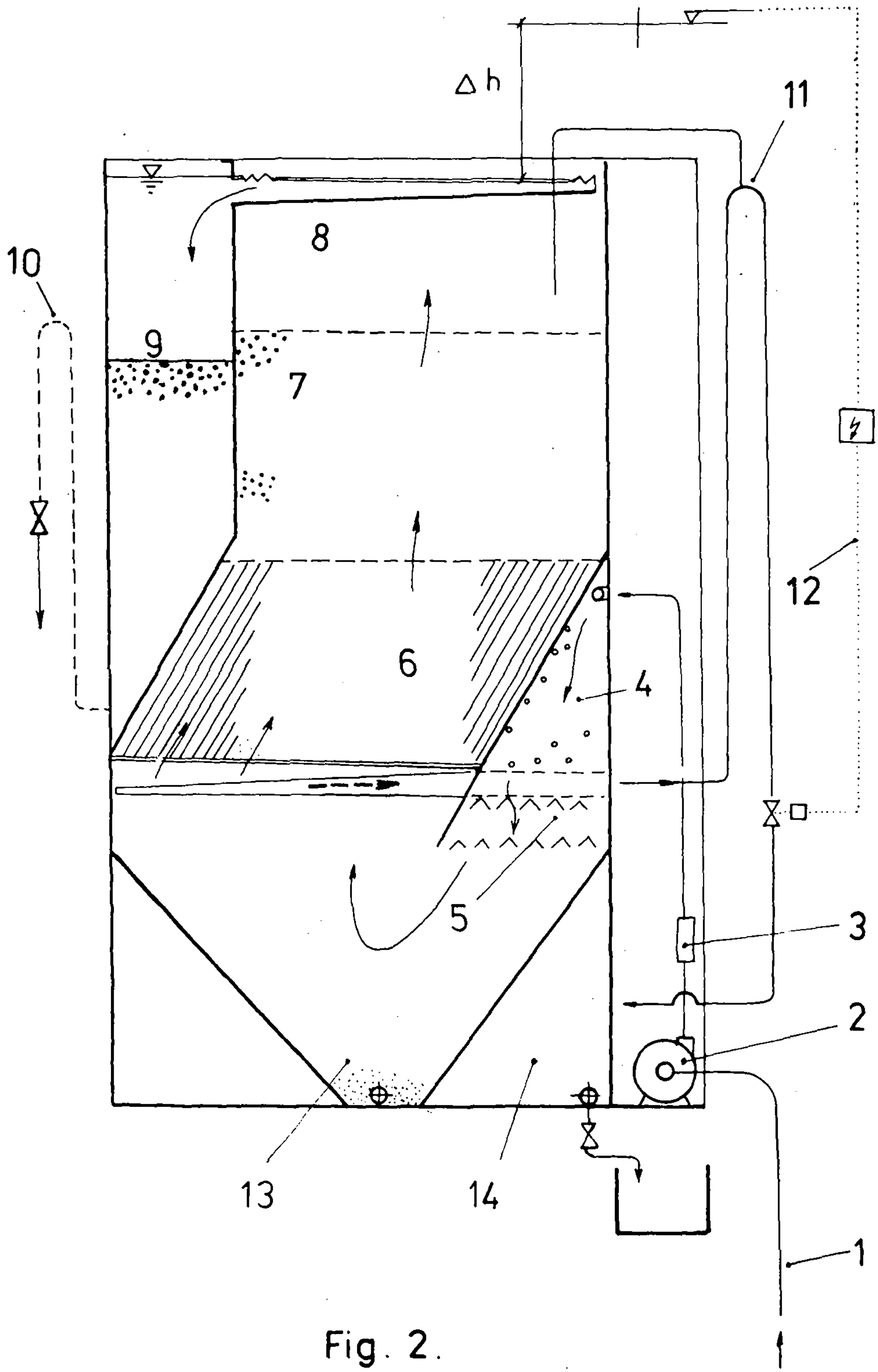

Fig. 2.

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WATER QUALITY IN WEST OF THE NILE DELTA

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ABSTRACT

Samples from water sources and bodies in the West of the Nile Delta Region, including irrigation and drainage water, Maryut Lake, Mediterranean Sea, tap water, industrial wastes and sewage water were regularly analysed during 6 monthes.

Irrigation water is generally of good quality, except in its western part due to discharging the drainage water into the irrigation canals.

The drainage water in the southern part of the area is suitable for irrigation especially after being mixed with irrigation water. In the north and west of the area, the drained water is not suitable for irrigation.

The sewage water and industrial wastes in Alexandria are discharged into the sea, Maryut Lake and in some agricultural drains. The chemical analysis of the sea water at several discharging pumps showed considerable variations; suspended solids from 500 to 15000 ppm, pH 8 to 10, P 0.1 to 3.9 ppm and NO_3^- up to 2.8 meq/L. The total plate count of bacteria in yeast extract-agar incubated at 30°C for water samples was up to 12 $X10^6$ bacteria/ml.

The tap water is of good quality. It has EC 0.68 mmhos/cm, NO_3^- 0.18 meq/L and bacterial count of about 4412/ml.

INTRODUCTION

The West of the Nile Delta Region is about 400,000 ha. The area is fully dependent on the Nile water to satisfy its irrigation and domestic needs. The Nile water is distributed through a net work of canals to provide about 10 billion (B) m^3/yr. The area is equipped with a drainage system. The southern drains discharge their water into a main irrigation canal (Mahmoudiah Canal). Other drains head north. The north western drains discharge their water into the Nubariah canal and its branches. Other drains discharge the water into Idku Lake or the Mediterranean Sea. The Idku Lake -14000 ha- is 30 Km east of Alexandria and connected with the sea by a narrow strait. The Maryut Lake is a brakish shallow basin, -6000 ha south of Alexandria with no free exit to the sea. It is divided by Alexandria-Cairo road and El Omoum Drain into 4 portions:(a) The main portion -2200 ha- adjacent to karmous and El Kabbary In Alexandria. (b) The Fish Farm 400 h. (c) The north western swamp 1200 ha (d) The south western swamp 2200 ha. The part of the sea coast in Alexandria and its vicinity is used for recreation and fishing.

Because the Region contains vast areas of land than can be reclaimed if suitable water is available, drainage water was mixed with the Nile water as stated above. The sewage water and industrial wastes are discharged into the sea, Lake Maryut and in several drains. These causes of water pollution affect the quality of water and its suitability for the purposes of its utilization.

The study reported herein aimed at evaluating the quality of the water sources and bodies in the Region especially where these waters are subject to contamination by variable sources of pollution.

MATERIALS AND METHODS

Samples of the following water sources were taken monthly from fixed points during the period from May to December:

- Irrigation water from several fixed pointson 4 main canalS and their branches.
- Drainage water from 6 main drains.
- Lake Maryut from its 4 main parts.
- Med. Sea from 7 Points.
- Drinking water at the water intake canal and from tap water.
- Sewage water at one discharge pipe.
- Industrial waste water from 11 plants, west, east and south of Alexandria.

Methods of determinations

- The difference between the weight of the filter paper before and after filtering the sample and leaving it to air-dry is equal to the weight of the suspended solids.
- The pH was determined by a pH-meter.

- The electrical conductivity, EC in mmhos/cm at 25 C° by a conductometer.
- Na and K by a flame photometer.
- Ca and Mg by the versenate method (Richards 1954).
- $CO_3^=$, HCO_3^- and Cl^- by volumetric methods (Richard 1954). SO_4 by difference between sum of cations and sum of $HCO_3^- + CO_3^= + Cl^-$.
- Phosphorus by the ascorbic acid method (Murphy an Riley;1982)
- The NO_3^- by the phenol disulfonic acid method for water of low chlorides (Richards 1954). The devarda method was used for determining NO_3^- in water of high Cl^- (Richards 1954).
- Iron, Manganese, Copper, Zinc, Cadmium and Nickel were determined by the Pye Unicam Atomic Absorption Spectrometer.
- Temperature of industrial waste water by a thermometer.
- SAR, Adjusted SAR, residual ($CO_3^= + HCO_3^-$) and pH_c were estimated from appropriate equations.
- Total plate count of bacteria on Yeast-extract Agar incubated at 30°C, and counted after 7 days as described by Stevenson and Rouatt (1953).

RESULTS AND DISCUSSION

The water of the Region is subject to the following sources of pollution: salts, sewage water and industrial wastes.

Salts as a Pollution Source for Irrigation Water

Two drains discharge their water into the Mahmoudiah canal namely Idku and El Khairy Drains. Because the EC values of both drains are relatively low (1.0-1.37 mmhos/cm) and their discharge is small, the EC of the Mahmoudiah water after being mixed with Idku drain water was about 0.6 mmhos/cm. After mixing with El Khairy drain, the irrigateon water did not show significant change in its quality.

The drainage system in the north western part of the Region has no outlet to the sea or to the lake. More than one drain discharge their water back to the irrigation canals. This is one source of increasing the salinity of the irrigation water as in the case of the Mechanized Farm (M.F) Canal which receives the drainage water from Drain No 7. However, the irrigation water of this portion of the Region is subject to other sources of salinity as it is seen in table 1 that the EC value of the M.F canal water before being mixed with Drain No 7 water was about 3.0 mmhos/cm. Because the Nubariah canal and its branches lay at a lower contour relative to the recently reclaimed land and because of other factors, (soil coarse texture and method of irrigation) a portion of the applied water seeps into the soil raising the level of underground water and at the same time flows back to the irrigation canals. In its way, it dissolves the soil content of soluble salts thus increasing the salt concentration in the irrigation canals. The branches of the Nubariah Canal as the M.F., El Tahrir, El Nassr, El Thawrah and others, suffer from this problem. The badly-needed main drain which will discharge its water into the sea, is underway.

Table1A- Chemical Composition of Water from Different Sources.

Locn,	Sus.M. mg/l	EC mmhos/cm	P ppm	Fe ppm	Zn ppm	Mn ppm	SAR	Ad.SAR	pH_c
Canals									
Mahmoud.	216.3	0.55	0.12	0.11	0.01	0.04	1.80	3.73	7.33
Afand.	319.9	0.63	0.19	0.21	0.02	0.03	1.95	4.04	7.33
M. At Kabbary	414.3	0.73	0.19	0.43	0.05	0.04	2.35	4.82	7.35
Mech. Farm	424.8	2.81	0.11	0.11	0.02	tr.	10.49	22.87	7.22
——after Dr.7	440.0	4.43	0.10	0.10	0.01	tr.	11.27	26.49	7.05
Kh.and ak.Sh.	200.0	0.37	0.05	0.09	tr.	tr.	1.50	2.40	7.80
Drains									
Idku	386.9	1.20	0.24	0.23	0.01	3.96	3.96	8.40	7.28
Omum	335	3.60	0.47	0.19	0.04	0.02	9.71	24.86	9.71
El Deshudy	257	6.80	0.39	0.09	0.03	0.02	14.03	34.79	6.92
El Haress	347	15.90	0.27	0.08	0.03	0.02	23.39	-	-
Tabia	411	2.21	0.56	0.25	0.13	0.22	6.99	16.36	7.06
Drain 7	381.4	7.19	0.20	0.14	0.04	0.04	13.41	33.26	6.92
Kalaa	750	3.35	1.36	0.77	0.19	0.11	9.56	23.71	6.92
Awayed	440	2.52	1.61	1.51	0.14	0.22	11.42	28.78	6.88
Edfina	465	6.22	0.57	0.19	0.12	0.09	14.96	37.10	6.92
Maryut Lake									
Shallow part	412.5	9.23	0.07	0.11	0.06				
Fish Farm	396.5	33.24	0.15	0.27	0.03				
Salt Co.	10087.1	488.6	0.26	2.22	0.23				

Table 1b- Chemical Composition of Water from Different Sources.

Location	Na^{+}	Ka^{+}	Ca^{++}	Mg^{++}	$CO_3^{=}$	HCO_3^{-}	Cl^{-}	$SO_4^{=}$
	meq/L							
Canals								
Mahmoudiah	2.42	0.26	1.98	1.64	0.0	4.20	1.25	0.85
Afandina	3.10	0.23	2.08	1.69	0.0	3.60	2.50	1.00
Mah. at Kabbary	3.30	0.35	2.15	1.80	0.40	3.80	3.00	0.40
Mech. Farm	24.35	0.50	5.05	4.74	0.0	4.40	25.0	5.24
" " after Drain 7	38.34	0.72	7.68	6.70	0.0	3.60	35.00	14.84
Khandak Sharky	2.85	0.35	1.53	1.22	0.0	2.60	2.50	0.85
Drink.w. Intake	2.85	0.39	2.05	1.63	0.40	3.6	2.50	0.42
Tap Water	3.00	0.59	1.85	1.61	0.40	2.40	3.00	1.25
Drains								
Idku	6.0	0.45	2.31	2.28	0.80	3.00	5.0	2.24
Omum before (H+Di)	24.60	0.70	6.04	6.81	0.40	6.00	22.5	9.25
El Dishudy	47.00	1.55	8.00	14.46	0.80	3.20	50.0	17.01
El Haress	116.00	3.00	17.00	32.21	0.80	4.20	122.5	40.71
Tabia bef. Racta	13.75	0.87	3.05	4.69	1.20	4.00	12.5	4.66
D 7 drain	46.00	1.28	14.00	9.53	0.80	2.80	30.0	37.21
Kalaa Drain	22.30	0.79	3.96	6.93	0.40	5.50	22.0	6.08
Awaid Drain	20.00	1.48	2.75	3.39	1.60	8.00	17.5	0.52
Edfina Drain	46.0	1.64	6.50	12.40	0.80	3.20	52.5	10.04

Sewage as a Pollution Source of Water

Alexandria sewage system is divided into 3 zones; East, Middle and West of Alexandria. The Eastern and Western Zones sewage water are lift to purification stations and led to Maryut lake. The Middle Zone disposes of its sewage water by pumping to the sea through an outlet 735 m long and 16 m under the sea water surface. Also, there are 18 outlets for sewage water distributed along the sea coast in Alexandria for emergency during winter only. However at present these outlets discharge sewage water all the year round into the sea (El Sharkawy 1977).

The part of Maryut Lake which receives sewage water is that adjacent to karmoose in Alexandria. Samples from this part were not available to the writers. A report from the Institute of Oceanography and Fisheries (1969) stated that the lake has become unsuitable for fish because of the sewage wastes discharged into it, except in the part of the lake called the Fish Farm.

El Kalaa Drain receives the sewage water occasionally from east of Alexandria. The concentration of suspended solids, P and microelements are much higher in El Kalaa Drain water than in the water of the other drains in its vicinity, table 1.

The analysis of samples of sea water from several points where sewage water is discharged are presented in table 2. The concentrations of suspended solids, P and NO_3^- are generally higher at the points of sewage water discharge. The total counts of bacteria on Yeast-extract Agar at 30°C for the samples taken from the sea water 500 m before and after the pipe discharging the sewage water, table 2 were about $12X10^6$/ml. At Sidi Bishr and Miami, 2 and 4 km, respectively from the point of sewage disposal the total plate counts of bacteria were about $2X10^6$/ml. At Camp Cesar and Ibrahimiah these counts were somewhat lower, ranging between 1.3-1.8 X 10^6/ml. Although these counts represent variable species of bacteria and are not indicative of pathogenic species, yet they indicate that the number of bacteria is much higher than normal. Unpublished work by El Sharkawy (1977) had shown that the Coli species counts in sea water receiving sewage in several points at Alexandria varied from 100 to $1X10^6$/100 ml.

Industrial Wastes as a Source of Water Pollution

Alexandria (3 million) is a main industrial center in the country. Its factories discharge their waistes mainly in the streams of water, lakes or into the sea.

In west of Alexandria, the concentration of the suspended solids increased where the wastes of Misr chemicals company were discharged in the sea reaching. 18580 mg/L. Evidently this increase takes place in the close vicinity of discharging pipes. Because the wastes discharged from the Na_2CO_3 factory was excessively alkaline, the pH of the water in the vicinity of the

Table 2- Nitrate content in samples of water from different sources.

Location of Samples		Bacteria count 10^3/ml		NO_3 Average meq/L
1. Beginning of Mahm. Canal		247		0.30
2. Ending Mahm. Canal		2005		0.45
3. Drinking water intake	(122-185)	151	(0.18-0.29)	0.24
4. Tap water from Ibrahimia	(2.7-6.5)	4.4	(0.14-0.25)	0.19
5. Tap water from Agr. Col. at Shatby	(2.8-7.0)	4.7	(0.13-0.21)	0.18
6. Sewage Pipe at Abou Heif before discharge into sea		102,500		2.43
7. Sea water 500 m of sew. Pipe		12,450		0.78
8. Sea water 500 m of ". "		12,425		0.70
9. Sea water 2 Km of at Sidi Bishr		2,075		0.48
10. Sea water 4 Km East of at Miamy		2,050		0.42
11. Medit. Sea at Camp Cezar		1577		0.27
12. Medit Sea at Ibrahimia		1762		0.30
13. Omum D. + Sea W. at El-Max		915		0.43
14. El-Kalaa Drain		n.d.		1.30
15. Mech. Canal at Mech. Farm		n.d.		0.41
16. Drain 7 of the Mech. Farm		n.d.		0.53

N.D = Not determined
Samples from 1-6 were analysed by Phenoldisulfonic Acid Method.
Samples from 7-22 were analysed by Nitrate, Devarda Method.

Table 3- Average chemical analysis of some industrial waste water.

Location	Temp. C	W.of Sus.S. mg/L	pH	E.C mmhos/ cm.	P ppm	Fe ppm	Mn ppm	Cu ppm	Zn ppm	Cd ppm	Ni ppm
(A) West of Alex. at El Max Region:											
Sew.w. of Ch.Co.	20.0	800	7.65	1.13	1.270	0.235	0.090	0.200	0.130	0.200	0.450
Was.w. of Ch.Co.(a)	42.5	2500	11.20	86.50	1.135	1.420	0.130	0.250	0.203	0.300	0.500
Was.w. of Ch.Co.(b)	39.2	13940	11.50	13.50	1.117	0.640	0.215	0.470	0.270	0.250	1.500
Was.w. of Oil Co.	37.0	380	8.40	8.20	0.370	0.300	0.090	0.135	0.110	0.150	0.000
Was.w. of Cement Co.	22.3	680	8.00	35.90	0.163	0.560	0.315	0.410	0.175	0.200	0.500
Was.w. of Elec. Co.	25.0	450	8.45	6.90	0.255	0.340	0.215	0.115	0.125	0.350	0.170
(B) East of Alex. at El Tabia Region:											
Was.w. of Ahl.P.Co.	26.5	725	9.00	0.82	0.243	0.420	0.090	0.310	0.125	0.210	0.180
Was.w. of Kaha Co.	25.1	900	8.35	0.68	0.403	0.280	0.125	0.220	0.130	0.100	0.420
Was.w. of Racta Co.	36.5	800	8.50	1.85	0.240	0.375	0.425	0.475	0.145	0.300	0.500
(C) South of Alex. at El Raas El Souda and Awaid Region:											
Was.w. of Edf. Co.	24.8	460	8.50	10.75	1.400	0.535	0.200	0.315	0.180	0.100	0.250
Was.w. of Sik. Co.	22.9	660	7.15	1.81	3.250	0.105	0.085	0.090	0.115	0.080	0.300
Mixed was.w.of) El Raa. ElSo.Co.)	37.9	739	11.25	2.35	2.400	3.650	0.425	1.350	0.320	0.095	0.550
Was.w. of Chi. Org.	22.5	1150	8.45	3.40	2.860				N.D		
Was.w. of Cu. Co.	28.5	367	7.75	0.90	1.430	2.635	0.315	2.920	0.475	0.120	0.900
Was.w. of Text. Co.	39.0	460	9.30	2.15	1.950	2.000	0.375	1.220	0.550	0.250	0.350
Was.w. of Fe Co.	33.0	300	7.90	0.69	1.285	5.220	0.215	2.150	0.630	tr.	tr.

N.D = not determined
(a) Waste water pipe of Chemicals Company; (b) Another waste water pipe of Chemicals Company.

Table 4- Substances and characteristics affecting the acceptibility of water for Domestic.use.

Substances	Highest desirable level
Total solids	500 mg/L
pH range	7.0 - 8.5
Mineral oil	0.01 mg/L
Phenol	0.001 mg/L
Total hardness	100 mg/L as $CaCO_3$
Ca	75 mg/L
Cl	200 mg/L
Cu	0.05 mg/L
Fe	0.1 mg/L
Mg	30-150 mg/L
Mn	0.05 mg/L
SO_4	200 mg/L
Zn	5.0 mg/L
Cd	0.01 mg/L

No_3^- should not exceed more than 4 5 mg/L in drinking water (Standard Methods, 1965).

pipes discharging these wastes rose to 9-10. The EC of the sea water adjacent to the pipes discharging wastes from the Oil, Cement and Electricity companies dropped to 10 mmhos/cm. Changes in the cation and anion, P and microelements constituents in the sea water in the sea water in the vicinity of the discharging pipes were slight except where Oil, Cement and Electricity companies discharge their wastes.

The industrial wastes of East of Alexandria are discharged into agricultural drains. The salts are important constituents of the industrial wastes of the factories in this part. The main changes which had taken place in the characteristics of water in the drains of th s zone were the increase in P and microelements concentrations.

Quality of Drinking Water

A branch of Mahmoudiah canal at its end close to Alexandria supplies Alexandria and Matrouh with drinking water. The total consumption was about $316X10^6$ m^3/yr in 1978. The other branch of the canal is a polluted stream due to the wastes and refuses thrown into it before it reaches the sea.

Table1 shows that the Mahmoudiah Canal water contained lower concentrations of cation, anions, P and microelements than the limits required for domestic use, table . The NO_3^- concentration in the streamsupplying the purification plant was about 0.24 meq/l. The purified tap water samples taken at Ibrahimiah and the College of Agriculture contained 0.13 and 0.25 meq/L, (8-15 ppm), of NO_3^- which is far below the International Standards (45 ppm). The bacterial count in the drinking water intake was $122-185X10^3$/ml. After purification the tap water samples contained $2.75-7X10^3$/ml.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 48 Aspect number 10

LONG TERM ECOLOGICAL RESEARCH AND MANAGEMENT OF THE UPPER MISSISSIPPI RIVER SYSTEM

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ABSTRACT

The commercially navigable Upper Mississippi River extends 1370 km from its confluence with the Ohio River at Cairo, Illinois to Minneapolis, Minnesota. Above St. Louis, Missouri (river km 293), 27 lock and dam structures provide a channel depth adequate for 2.75 m draft barge tows. The 1000 km of Mississippi River between St. Louis and Minneapolis is managed by two federal agencies and five states for multiple use. System management objectives include commercial navigation, commercial fisheries, minimum flood damage, energy development, water based recreation, water quality, aquatic and adjacent terrestrial or wetland habitat preservation. All management decisions require information on economics, public attitudes, physical environment, and biological structure and function. If any of the types of information are not properly used, the plans and decisions will not achieve their objective or achieve one objective to the detriment of other elements of the system.

The basic data are the hydrologic variables of water flux and depth and sediment concentration and transport rate. The water, sediment, and associated nutrient fluxes are being studied at three sites in the Upper Mississippi River System as part of a program of Long Term Ecological Research. This program began in 1982 and is expected to continue into the next century. Water and sediment transport data for Pool 19 are used to describe the riverine environment. Particular emphasis is given to the impacts of tributary sediment loads and to changes in channel border areas.

Keywords: Rivers, sedimentation, hydraulics, waterways, ecology, aquatic habitat, navigation, dams, multiple use.

INTRODUCTION

One of the recommendations of the Master Plan for the Upper Mississippi River System (UMRS) (UMRBC, 1982) is a long-term resource monitoring program. A comprehensive monitoring program for the 850 miles (1370 km) from the confluence of the Upper Mississippi and Ohio Rivers at Cairo, Illinois to Minneapolis, Minnesota is still being debated in Congress two years after the completion of the Master Plan. However, the "Large Rivers" Long Term Ecological Research (LTER) project began long-term monitoring and research in January 1983 on three pools in the system: Pool 19, above Keokuk, Iowa, Pool 26, above Alton, Illinois, and Peoria Lake above Peoria, Illinois on the Illinois River. Twenty-seven lock and dam structures were built in the 1930s to provide for 9 ft (2.75 m) draft navigation system including the lock and dam structures, wing and closing dams, and dredging. Two methods of pool regulation by gate operation are used to maintain the depths required for commercial navigation. The U.S. Coast Guard has responsibility for placing and maintaining navigation aids such as bouys, daymarks, and lights. The U.S. Fish and Wildlife Service manages the federal refuge areas along the river for the benefit of fish, waterfoul, animals, land birds, and clams. The protection and continued productivity of the riverine habitats is their goal. The successful management of the ecosystem supports sport and commercial fishing, hunting, commercial clamming, and recreation that depends on the natural and aesthetic quality of the river and nearby backwaters, marshes, forests, and bluffs.

The states of Missouri, Illinois, Iowa, Wisconsin, and Minnesota also have management functions on the UMRS, primarily through regulation of hunting and fishing, state wildlife and conservation areas, and cooperation in permitting new barge terminals and fleeting areas. When various interests such as tow boat operators, hunters, fishermen, boaters, and preservationists add their views on management practices, the process becomes very complex and cumbersome. Thus there is a great need for accurate and appropriate information on the physical and biological condition of the river system. The Large Rivers LTER investigators intend to develop a long-term data base on their three sites and to conduct research on the environment, structure, and function of the riverine ecosystem.

After competing management objectives are outlined, the hydrology is described using Pool 19 as an example, and finally the interaction of the hydrologic conditions and the practical operating rules are described.

MANAGEMENT OBJECTIVES

The Master Plan (UMRBC, 1982) places objectives in three categories: economic, environmental, and recreational. Economic objectives include energy efficient movement of goods and materials by barge, reduced flood damage, provision of water for water supply, profitable commercial fishing, clamming, and fur-trapping, and the viability of water-related recreation. Environmental objectives are the improvement of the aquatic and nearby terrestrial habitats, provision of water quality suitable for swimming and fishing, preservation of the unique resources of the river system, and protection or enhancement of resources which may be affected by the navigation system. Recreational objectives include preservation and enhancement of physical, biological, and aesthetic qualities, improved access to the river system for all types of recreation, and improved safety for water-based activities.

The primary purpose of the system operation is to maintain the desired depth of water for commercial navigation. This achieved by controlling the flow through the grated portion of each dam. The system contains no main channel flood storage. The dams are operated either two ways. One method maintains normal pool elevation at the downstream dam in a pool until the gates are out of the water. The other scheme maintains a stage-discharge relation at a central point in the middle reach of the pool. For low discharges normal pool level is maintained at the dam. For intermediate discharges, the water level at the dam is drawn down below normal pool to maintain the specified stage at the control point. At higher flows the pool level rises throughout the pool, until the gates are raised out of the water and uncontrolled or open river conditions exist. Typically open river conditions occur 15 to 24% of the time. This is 50 to 90 days per year. Flood stages are essentially the same now as before the navigation structures were built.

One result of the mid-pool control point operation is the drawdown of water levels below normal pool elevation in the downstream portion of the pool This drawdown can be as much as 1.5 m at the dam. This exposes plant beds and the river bottom to the atmosphere. It also increases the total range of depths in the downstream portion of the pool. Operation maintaining the water level at normal pool elevation at the dam results in minimal range of depths in the downstream portion of the pool. However, depths in the upper part of the pool will be deeper at controlled flows with this method of operation. The comparative impacts of these two operating schemes on the biological communities need to be investigated. Such a study would recommend the operating scheme that was more beneficial to the aquatic habitats.

In contrast to this hydrology controlled stage-discharge operation to maintain channel depth for navigation, fish and wildlife managers would consider the seasonal requirements for fish spawning, waterfowl migration, nesting, and hunting, fur-bearing mammals, upland game habitat and feed, and large game animals. Some of these may require high, low, or constant stages and depths which are not obtainable because of the limited control afforded by the dams at high flows and the need to maintain navigation at low flows. Rapid changes in pool levels are detrimental to fish and wildlife, but the rates of change are not scientifically set or regulated. There is no winter navigation above Rock Island, Illinois at mile 483, (km 777) and iceing makes gate operation difficult. However, gate openings are set before freeze-up based on forecasts of winter discharges (St. Paul District, 1981). The goal is to maintain near normal pool levels for fish and fur-bearers.

Preservation of the existing water and wetland habitats is a long-term objective. Construction of the lock and dam structures changed the low and intermediate flow regime of the river and inundated substantial riparian areas. Each pool has three distinct reaches. Immediately upstream from each dam is a reach in which the water level has been raised, increasing the water surface area and creating a low velocity pool-like condition. The reach immediately below each dam is essentially unchanged from before construction and contains islands and side channels. The middle reach in each pool is a transition between the free flowing reach and the pooled reach. The extent of each type of environment in a given pool depends on the original river geometry and slope and the height of the downstream dam. Sediment deposition occurs in the downstream reach, mostly in channel border areas or backwaters. Sediment also deposits at bifurcations in the river and below tributary months. If deposition occurs in the navigation channel, dredging may be required to maintain the channel depth and width. The dredged material needs to be placed in an environmentally appropriate place. Most channel reaches downstream of the dams have been degraded by scour since the dams were built.

WATER AND SEDIMENT TRANSPORT IN POOL 19

The discharge of the Mississippi River at Lock and Dam 19 has averaged 1780 m^3/sec since 1878 from a drainage area of 294,200 km^3. The extreme daily discharges are 9740 m^3/sec in 1973 and 142 m^3/sec in 1933. In terms of flow duration analysis, 650 m^3/sec is exceeded 90% of the time, 1360 m^3/sec is exceeded 50% of the time, and 3680 m^3/sec is exceeded 10% of the time. The area tributary to Pool 19 is 14,000 km^2. Two tributaries drain 91% of this area. The Skunk River in Iowa has a drainage area of 11,200 km^2 and an average discharge of 66 m^3/sec and Henderson Creek in Illinois has a drainage area of 1550 km^2 and an average discharge of 11 m^3/sec. Ungaged local tributaries are estimated to contribute 9 m^3/sec from 1250 km^2. The variability of these streams is much greater than that of the Mississippi. One example of variability is the ratio of flows that are exceeded 10% and 90% of the time. This ratio is 37.2 for Henderson Creek, 48.5 for the Skunk River, and 5.65 for the Mississippi at Keokuk.

The sediment transport characteristics of these streams have not been measured long enough to be as representative as the water discharges. Some suspended sediment data has been collected by the Corps of Engineers since the 1940s. Daily samples have been collected since 1968 and provide a good record. Suspended sediment has been measured on the Skunk River beginning with water year 1976. Daily data was collected on Henderson Creek for water years 1979, 1980, and 1981 and weekly data has been collected since January 1982, or part of the LTER sampling program. Average annual suspended sediment loads are 10.7 $(10)^9$ kg for the Mississippi at Keokuk, 2.5 $(10)^9$ kg for the Skunk River, and 0.3 $(10)^9$ kg for Henderson Creek. The remaining drainage area to Pool 19 is estimated to contribute as much suspended sediment as Henderson Creek. The range of sediment loads is large. Monthly sediment loads on the Skunk River have ranged from 1.54 $(10)^4$ kg to 1.28 $(10)^9$ kg. Monthly sediment loads on the Mississippi have ranged from 1.89 $(10)^7$ kg to 8.94 $(10)^9$ kg.

Suspended sediment load is the product of concentration and discharge. Concentrations of suspended sediment are about 10 mg/l during low flow periods, especially in the winter when surface runoff is lowest. During floods, which are the result of surface runoff, suspended sediment concentrations may be as high as 10,000 mg/l. Higher concentrations limit light penetration and inhibit photosynthesis. This raises the issue of resuspension of sediment by two boat propeller jets and wave-wash along the shore. Studies by Bhowmik, et.al. (1981) indicate increases in average concentration up to 24% on the Illinois River. The Master Plan (UMRBC, 1982) presents estimates of suspended sediment concentration increases due to increases in the number of barge tows on the rivers.

Two problems are affected of the suspended sediment concentration and load. The scour or deposition of sediment in the navigation channel, side channels, channel border areas, or backwaters is a function of the local ability of the water flow to transport sediment and the sediment already suspended in the flow. Over 50% of the original volume in Pool 19 has been filled with sediment since the dam was built in 1913 (Thomas, 1977). The rate of sediment deposition has stabilized at 3 $(10)^9$ kg/ per year which is approximately 28 mm per year. There was no spring flood in 1977 and the suspended sediment concentrations remained below 50 mg/l through the spring and summer. Rooted aquatic plants established new beds and expanded existing beds on a wide, 1 to 1.5 m deep channel border area extending 4.8 km upstream from Montrose, Iowa.

Subsequent floods have not removed the plants by scour and higher sediment concentrations have neither buried the plants nor reduced light levels enough to prevent regrowth each year since 1977. The 1977 water year had an average discharge of 870 m^3/sec, third lowest in 104 years. In the flood year of 1973, the discharge averaged 3370 m^3/sec, the second highest on record. No change was noted in plant beds, and approximately one year's sediment volume was washed from Pool 19. At the present rate of deposition, large areas of open water will become too shallow for power boating, filled with aquatic plants, and converted to willow-covered islands within 50 years. For example, an island downstream of Nauvoo Point has lengthened about 240 m since 1975. In fact, the 19 km scenic drive along the Mississippi from Nauvoo to Hamilton, Illinois may have its view of the wide river replaced by a 600 m wide brushy island area.

According to a study of sedimentation in the 505 km of the Mississippi from Lock and Dam 22 to Lock and Dam 10 (Nakato, 1981), 9 of these 12 pools have sediment deposition rates between 3 mm and 91 mm per year. The other three pools have net degradation. This assumes that material dredged from the navigation channel is removed from the river, which is not always the case. The amount of material dredged is generally smaller than the sediment deposition by a factor of 10. Note that dredging is required even in pools which are degrading.

Most of the net deposition of sediment occurs outside the navigation channel and other major channels. This means that the deposition is taking palce in channel border areas, side channels with low velocities, and backwater areas. These are generally the most productive biological habitats. The continued shoaling and deposition of fine sediments will cause changes in the benthic organisms and plant communities. An area may be suitable for certain fish to spawn only when depth, bed material, and velocity are suitable.

Summary

The Upper Mississippi River System is in dynamic change following the construction of 11.6 m high Lock and Dam 19 in 1913 and many low-head structures in the 1930s. The system is operated to provide for commercial navigation and is managed to protect and enhance fish and wildlife in and along the river. The lock and dam system provides no flood storage. Thus, the system can be controlled for low and normal discharges, but cannot be controlled for high discharges. Two methods of control are used, but no advantage for navigation or aquatic organisms has been identified for either method.

Suspended sediment transport has two major impacts on the system: deposition in areas outside main flow channels, and decreased light penetration at increased concentrations. The continued deposition is changing the aquatic habitat in areas outside the main channels and will result in some conversion to wetland and terrestrial habitats within 50 years. Low flow and sediment concentrations in Pool 19 in 1977 resulted in an expansion of plant bed which has persisted through several significant floods. The net removal of sediment from Pool 19 during the 1973 flood was localized near the dam and the material was rapidly replaced. A projected increase in barge traffic is expected to increase average suspended sediment concentrations. This will reduce photosynthesis and may limit plant growth. Only with better data on the physical environment and its impact on fish plants, and animals of the UMRS can we make the best overall decisions for management ofthe system. We must also recognize that the present system has a limited capability to control or modify the naturally occuring water discharge and sediment load.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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REVERSE OSMOSIS DESALINATION IN SEMI-DEVELOPED CONDITIONS

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ABSTRACT

In semi-developed conditions simple operation is deemed to be important, specially for small plants. In the presentation comparison will be made between different types of Reverse Osmosis membranes, their need of pretreatment etc. and the feasibility of different kind of improvements of the membranes.

REVERSE OSMOSIS DESALINATION
IN SEMI-DEVELOPED CONDITIONS

Introduction

The development of reverse osmosis membranes has been in progress for some 20 years. It started with membranes for demineralization of brackish water but around 10 years ago membranes for desalination of seawater were introduced. From very small plants (a few m^3/h) in the beginning mainly for industrial purposes the plants have grown to large plants (> 50,000 m^3/d) for domestic water supply.

There are a number of manufactures mainly in the U.S.A. and Japan. The membranes are of different type and they have different data on specific production, pretreatment requirement, life length, salt retention, etc.

In semi-developed conditions simple operation is deemed to be important, specially for small plants. In our presentation we intend to starting from three types of raw water (slightly brackish groundwater, highly brackish groundwater and Arabian Gulf seawater) compare capital and operation costs for plants based on different types of membranes. This will include different degree of pretreatment, different building arrangements such as in- and outdoor installations, different material in pipework and vessels etc.

The aim is to get an idea of what kind of improvement of the different membranes that are most feasible i.e. will result in the biggest savings in capital/operating costs. Below follows some general information on membrane and module selection and pre- and posttreatment.

Membrane and module selection

The choice of membrane type and membrane module affects the extent of the pretreatment more or less, depending on the raw water composition. After the pretreatment, the following limits are applicable:

Parameter	CA/CTA Spiral RO	Composite Spiral RO	CTA HFF RO	PA HFF RO
Turbidity, JTU	≤ 0.5	≤ 1 (NTU)	≤ 1.0	Not specified
Silt Density, Index (SDI)	≤ 4.0	≤ 5.0	≤ 4.0	≤ 3
Iron, mg/l Fe	≤ 0.5	N.A.	< 0.7	≤ 0.1
Manganese, mg/l Mn	≤ 0.5	N.A.	< 1.3	≤ 0.1
Strontium, mg/l Sr	Not specified	Not specified	Not specified	15
Barium, mg/l Ba	-"-	-"-	-"-	0.1
Silica, mg/l SiO_2	≤ 150 (brine)	≤ 150 (brine)	< 100 (brine)	< 150 (brine)
Organic compounds	Not specified	Not specified	Not specified	Not specified
Residual chlorine, mg/l Cl_2	≤ 0.5	0[1]	≤ 1.0	0
Dissolved oxygen, mg/l O_2	Not specified	Not specified[2]	Not specified	Not specified
Max. feedwater temp., °C	35	45[2]	30	35[3]
Max. feed pressure, psi	570	1000	900	1000[3]
Feed pH range	3-7.5	3-11[2]	3-8	5-9
Langliers index; Stiff & Davis index	< 0 (brine)	< 0 (brine)[2]	< 0 (brine)	< 0 (brine)
Solubility product ($CaSO_4$)				
without SHMP	$<1.9\times10^{-4}$ (brine)	$<1.9\times10^{-4}$ (brine)[2]	$<1.9\times10^{-4}$ (brine)	$<1.9\times10^{-4}$ (brine)
with SHMP	$<10^{-3}$ (brine)	$< 10^{-3}$ (brine)[2]	$< 10^{-3}$ (brine)	$< 10^{-3}$ (brine)

1) < 0.1 for FT-30

2) For Toray PEC-1000:

Dissolved oxygen	$\leq$ 0.5 mg/l O_2
Max.feedwater temp.	40°C
Feed pH-range	1-12
Langliers index	$\leq$ 0.5 (brine)
Solubility product ($CaSO_4$)	without SHMP < 3.4×10^{-4}
	with SHMP < 2×10^{-3}

3) An improved B-10 permeator (Du Pont) is now available in the market:

Max. feedwater temp. 40°C
Max.feed pressure 1200 Psi

The development of membranes has been in progress for two decades and the efforts have primarily been concentrated in trying to make the membranes less sensitive to physical and chemical parameters, and thus enabling less pretreatment, and on trying to lower the energy and operation costs by a higher salt retention, energy recovery, lower pressure and with a higher flux. In this connection the choice of module type is also important.

Of the list above it is clear that the demands for pretreatment varies depending on the membrane and module type. For instance the cellulose acetate membranes are more pH-sensitive and can be destroyed by microorganisms but when it comes to chlorine they are not at all as sensitive as the polyamide membranes (Du Pont) and the composite membranes. It is extremely important to dechlorinate the feedwater before it enters the latter membranes. The spiralwound composite membranes have undeniably some advantages since they manage with less pretreatment compared with what the hollow fine fibre polyamide membranes demand.

In a bigger plant with qualified labour the economy becomes the most important matter in contrast to a small plant with demands on a minimum of pretreatment and a simple operation. In the latter case it could even be better to have the plate and frame configuration of composite membranes. According to DDS, Denmark, this system can be used with feedwater temperatures up to 80°C and in the pH-range 1-12.5. The pretreatment can be limited to multimedia filtration and cartridge filtration, but the cleaning of the membranes must be done more often and according to CIP-procedure (CIP = cleaning in place). In all this still means a saving in chemical costs. The membranes can stand a low chlorine concentration and in contrast to other membranes they can be handled dry.

When desalting seawater with high TDS-levels, such as in Saudi Arabia and Bahrain, it is favourable to use membranes with as high salt retention as possible, so that it is not necessary to desalt the permeate from the first stage, or some of it, to get the desired water quality.

Pre- and posttreatment

To reduce the extent of required pretreatment beach wells or Ranney collectors are used when desalting seawater. The infiltration then becomes part of the pretreatment.

During recent years methods have been developed to reduce the risks for membrane scaling and fouling. This can be done mechanically by rotation ("Hydro-fuge RO-system") or by seed- crystals ("RCC-method").

An insufficient pretreatment causes higher water costs. The raw water pretreatment is therefore extremely important at medium sized and big RO-plants. The contaminations of the raw water can be as follows:

- Suspended solids, e.g. silt, clay and organic material (> 1 u), which in general are removed by filtration.
- Colloidal matter (0.2-1.0 u), which can be flocculated with chemicals before the filtration (in line coagulation).
- Microorganisms, e.g. bacteria and marine organisms, which can be killed by chlorine or other desinfectants.
- Dissolved organic matters, e.g. oil, fatty acids etc., which can be separated with activated carbon filtration to prevent membrane fouling and scaling.
- Dissolved inorganic matters, which can cause membrane scaling by the precipitation of sparingly soluble salts ($CaCO_3$, $CaSO_4$, $MgCO_3$, $MgSO_4$, $BaSO_4$ etc.) or metal oxides ($Al(OH)_3$, $Fe(OH)_3$, $Mn(OH)_2$, etc.)

The use of chemicals, especially those who are difficult to handle, can be minimized to make the operation of the plant as simple as possible. This might cause a higher cost. Sometimes filtration, which is easier to operate, can be used instead of chemicals, e.g. sand filtration, dual or multimedia filtration, iron and manganese filter, softening filter (ion exchange), oil filter and activated carbon filter. The last-mentioned filter shall be the final filter and separate dissolved organic matters, e.g. hydrocarbons from oil. Furthermore chlorination, which must be followed by dechlorination for some of the membranes, might be replaced by UV-irradiation or intermittent chock treatment of the membranes with bisulfite or comparable.

There has also been a development during recent years when it comes to chemicals used in desalination plants. It might be more convenient to use a chemical in less quantity even if it is a bit more expensive than for instance sulphuric acid. To prevent the corrosion of the distribution net, the combination of pH-adjustment with acid before RO and lime injection after RO to increase the temporary hardness is still the most economical.

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ZERO OPERATION COST AND LEAST INVESTMENT WATER SUPPLY MODEL FOR TRANSMIGRATION PROJECTS

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ABSTRACT

This paper explains the procedures and philosophy for a water supply model to meet the requirements for a strategy of zero operation cost and the least initial investment in Transmigration projects for the mass rural population.

The Republic of Indonesia is currently carrying out a Transmigration program to provide new settlement areas for 500,000 farming families or a population of 2,500,000 in a 5-year implementation period. The program offers limited funds for the initial cost of the water supply system, but no funds for operation. Therefore, a water supply model needing the least initial investment and a zero operation cost is essential for the success of the system.

The project sites are presently scattered, covered by primary forests or secondary growth, and located far from existing settlement areas. The hydrometeorological data are extremely limited. The studies were conducted through the analysis of the limited rainfall data, the interpretation of aerial photographs, the field reconnaissance, and the assessment of physical models.

A Model using rainwater as the main water source and groundwater as a supplement was developed. This Model is considered satisfactory as far as economical, technical, and time factors are concerned. Therefore, it possesses potential benefit for rural settlement projects in humid regions.

TRANSMIGRATION PROJECT

The Republic of Indonesia, a South-Eastern Asian Couhtry comprises several big islands, i.e., Java, Bali, Madura, Kalimantan, Sumatra, Irian Jaya, Sulawesi and numerous of small islands. The central regions of the Country, Java, Madura and Bali are over-populated, while the outer islands such as Kalimantan, Sumatra, Irian Jaya and Sulawesi where there are extensive natural resourrces, are still under-populated and under-developed. The purpose of Transmigration Project is to select those families who are poor living conditions from the over-populated islands, and to resettle them in the bigger outer islands in order to achieve a balanced nation-wide development by improving the standards of living of the settlers. During the third five-year development programme (1979-1984) of the country the Government intends to resettle 500,000 families under the Transmigration Project. A basic unit under a Transmigration Project is called a SKP which is composed of about 2,000 families in a rural area of 10,000-12,000 hectares of lands cleared from the primary forest area for houselots, and the cultivation of arable crops and tree crops.

DOMESTIC WATER SUPPLY FOR A TRANSMIGRATION PROJECT

The domestic water supply system is one of the major infrastructures of the project. It is essential to provide water supply to the transmigrants at all times to ensure survival of the settlers. Although the water supply systems vary from site to site, however, this paper will confine itself to introduce the study of domestic water supply for a SKP of a transmigration project in West Kalimantan Province, Kalimantan Island.

Appreciation of the study

The distribution of households, the constraints of the project, and the habits of using water consitute the major consideration for the entire planning.

Household distribution

The households in a SKP are grouped into several villages with each village ranging from 200 - 500 families. This village, then to be sub-divided into several hamlets depending on the distribution of suitable lands free from flooding hazards. The houselot is a land with rectangular shape, 25m x 100m with the 25m side faces a village road. Houses are built at the center of the front part of the lot. The distance between two houses along a village road is 25m, hence, the households are stretching as long as several kilometers.

Major constraints

Constraints to be concerned are: (1) least initial investment and operation and maintenance cost, (2) lack of power supply system, (3) insufficient flow of river water for drinking during dry season (from June to September), (4) households are built on high lands to avoid hazards from flooding, river water is usually far away from the village sites, (5) groundwater is scarce at the houselot areas, (6) major head-work and intake facilities on rivers are not allowed because of navigation requirement, (7) poor accessibility for transporting large quantities of contruction materials within a limited time to the spots, and (8) limited time available for contruction.

Habits of using water

In the primary forest areas, nearly all the local inhabitants are living along the riversides and using river water for their daily water demands. In Pontianak, the Capital of West Kalimantan Province and other major townships on the island, the majority of the local residents collect rainwater as their daily supplementary water supply (see photos 1 and 2). According to FOK et al (1981) 1/, during the era of the Roman Empire, the city of Venice and later on in Australia, eastern American States, Hawaii, and other Pacific islands, local inhabitants had been collecting rainwater to supply their daily necessities.

Water resources

Due to the limited time and manpower, only a generalized analysis and screening on water resources were made. The results are shown in Table 1. According to this assessment, rainwater and shallow groundwater are main sources of water supply for human being under any condition when rainfall amount and distribution are favourable.

Table 1 Generalized Screen Study in Water Resources

Water resources / Item	Surface/river water	Ground water	Rain Water
Availability	Limited and unreliable during dry period (June-September)	Subject to the aquifer distribution	Depends on rainfall distribution and the amount
Distribution to user	Provisions for head-works, piping system, electricity supply, and pumping station are essential	Provisions for piping system, electricity supply, pumping station are essential for a public supply system	Roof catchment area is ready upon the completion of a house
Service areas	Suitable in areas near the rivers	Subject to the availability of ground water	Optional
Water quality	Prone to be polluted, treatment may be necessary	Good and easy to be maintained	Good and easy to be maintained
Initial cost	High	Medium for a public supply system	Low
Operation and maintenance cost	High	Medium for a public supply system	None

THE WATER SUPPLY MODEL

The model comprises a rain-catchment cistern as the main water supply for each family and a dug shallow well with a hand pump as the supplemental water supply for every 4 families.

Rain-catchment cistern system

Rainfall amount, roof catchment area, storage capacity in a cistern, and water demand for each family are the four important elemments to be considered in the design.

Rainfall amount

The daily rainfall data at Sanggau Rainfall Station (a central township of West Kalimantan Province) is adopted as the basis for this study. The data contain a rainfall record from 1959 to 1983 with the exception of 8 years of insufficient records. The accumulative rainfall amount of the durations of 1-week through 17-week for the past 17 years are analysed by using computer based on the daily rainfall data. The results in increasing magnitude order are shown in Table 2. The low rainfall frequency analysis is conducted by following the Gumbel extreme-value distribution method. The magnitude of low rainfall for different duration with a 20-year reccurrence period is depicted from Table 3.

By doing so, rainfall distribution of the 17-week was plotted and presented in Fig. 1 to determine the critical low rainfall period for the capacity design.

Table 2 Results of accumulated rainfall in various consecutive weeks (Recorded year 1959-1983 with 8 years omitted)mm

1 week	2 weeks	4 weeks	6 weeks	8 weeks	11 weeks	13 weeks	15 weeks	17 weeks
0	0	4	31	31	103	223	304	355
4	2	9	62	93	146	245	313	398
	3	12	64	140	185	257	379	458
	5	16	97	150	236	315	402	461
	8	20	106	153	263	340	407	471
	9	33	108	174	295	346	408	578
	17	36	113	179	298	354	442	643
	57	43	114	196	315	362	453	647
		44	117	197	344	439	538	668
		53	120	200	390	461	568	693
		56	131	207	415	468	594	699
		57	155	215	441	526	663	756
		71	168	218	450	541	674	779
		75	204	268	459	600	681	863
		97	211	275	583	614	800	979
		110	216	300	619	677	832	993
		114		316		787	956	1224

Table 3 Low rainfall distribution

Rainfall distribution		Incremental Rainfall	
Duration week	Total amount mm	Duration week	Amount/week mm
4	16	-	4
6	52	2	18
8	132	2	40
11	144	3	4
13	205	2	31
15	275	2	35
17	352	2	39

Catchment area

Utilizing the roof of a transmigrant's house to collect rainwater as the water supply for his family. The roof area of 36 m^2 has been predetermined by the transmigration Agency.

Domestic water demand

The following system is designed to meet the water demand during a drought season which occurs once every 20 years. Thus, water consumption is limited to drinking, washing, dish washing and personal hygine as recommended by Culy (1977)2/ as:

Minimum demand for cooking and drinking	6	liters/day/capita
Dish washing	6	"
Personal hygine	15	"
	27	"

Assuming that an average family has 5 members, the total water demand for each family is 135 liters/day.

Storage capacity of the water tank

The amount of daily water demand can be converted to an amount of daily rainfall required as follows:

$0.135m^3/36m^2 = 3.75 \times 10^{-3}m = 3.75$ mm/day or 26 mm/week, this amount is also shown in Fig. 1 to constitute a critical dry period of 11 weeks. The storage capacity of water tank can be designed as shown in Fig. 1 by measuring the shade areas with the conversion factors of roof catchment area. The storage capacity is calculated as follow:

$$V = [(11 \times 26 - 2 \times 40) - (4 \times 4 + 2 \times 18 + 3 \times 4)] \times 36/1000 = 5.10m^3$$

The cistern can be constructed with reinforced concrete, bricks, wood, or plastic inaterials (see photos 1 and 2).

Dug shallow well

From field reconnaissance and dug well pumping tests together with aerial photographic interpretation, it is found that shallow groundwater is scarce in the village sites though deeper aquifer may exist in some low-lying areas near rivers. For the purpose of achieving zero operation and maintanence costs, a system consist of a 1-meter diameter shallow dug well with concrete casing and a hand pump was designed to draw groundwater from the limited shallow aquifer. It is believed that this system is considered practical and useful as a supplemental water supply. It is also recommended that several test wells be dug to provide aquifer information prior to constructing the wells. And, the location of the wells should be determined by referring to the test wells information.

Cost study

A cistern and shallow well of this model is constructed by the Government as an integrated part of the houses. The transmigrants enjoy the water supply without maintenance cost. As new settlers with low income this is important to them. The costs in comparison among various water supply systems using power or other kinds of modern facilities are considered impractical due to the constraints mentioned before. However, the cost for this system is believed to be the least.

CONCLUSION

This model could be used as an effective water supply system in the newly developed remote rural areas in tropical or temperate humid regions, where modern water supply system is not available or not practical.

NOTICE

This paper represents a consultant's view to the Government of Indonesia. It does not reflect the policy of the Government.

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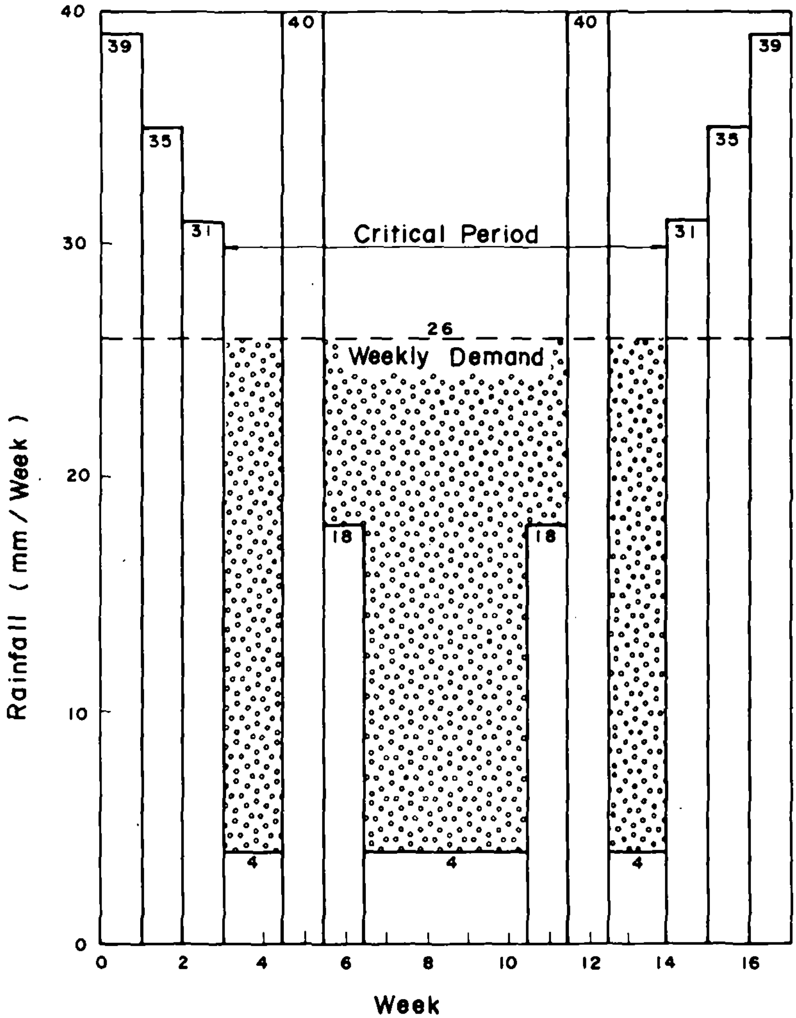

Fig. I Determination of the Critical Period of low rainfall (20 Year return Period of drought) for storage capacity design.

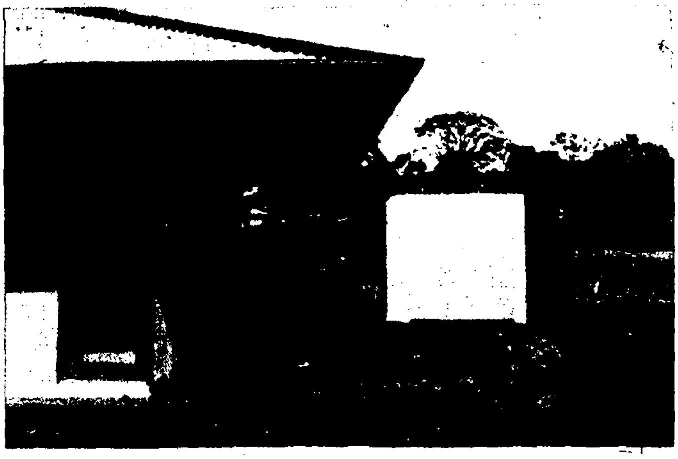

Photo 1 A cistern and a well for water supply constructed on project site.

Photo 2 A cistern for collecting rain water in Pontianak city, the capital of West Kalimantan.

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CHARACTERIZATION OF DAILY STREAMFLOW DATA USING AN APPROXIMATE PARTITIONING METHOD

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ABSTRACT

Adequate water resources planning in any region is based upon proper characterization of the respective hydrologic processes. Knowledge of storm runoff and baseflow may be necessary for developing flood control practices and estimating flows for water supply or pollution abatement. Often, in rural areas, only limited data are available on individual components of the hydrologic cycle. While the traditional separation of streamflow hydrographs into flow components require data for each event, daily data such as daily rainfall and streamflow amounts, may be the best information available to planners.

This study describes a procedure for partitioning daily total streamflow volumes into storm runoff and subsurface flow components. A method is also presented for estimating time base for storm runoff using topographic data. Constant and seasonally-variable rainfall threshold amounts were tested to represent an initial abstraction for soil water storage. The seasonally varied initial abstraction was determined from 12 years of soil moisture data and was represented by a third-degree polynomial function.

The method of partitioning was tested on daily streamflow data from rural agricultural watersheds with drainage areas up to 1,494 km^2 in the Coastal Plain of the Southeastern United States. Estimated volumes obtained for these watersheds were representative of observed stormflow and subsurface flow volumes from a small upland drainage area, 0.34 hectares in size. Average ratios of subsurface flow to total streamflow for the watersheds ranged from 0.58 to 0.83. Results indicate that the procedure gives reasonable estimates and may be useful for application in areas where available streamflow and rainfall data are limited to daily values.

Keywords : Streamflow partitioning, surface flow, subsurface flow, stormtime base, rainfall threshold value (initial abstraction).

INTRODUCTION

Subsurface flow constitutes an important component of the hydrologic cycle, and any method that would permit the water resource planner to better estimate the relative magnitude of this component in regions where hydrologic data are limited, would assist in the assessment of subsurface water quantity and quality inputs to stream channel systems. Previous hydrograph separation techniques (Linsley, et al. 1958) require detailed storm event data, as well as laborious and subjective procedures. For the many real-world applications, especially for rural areas where detailed storm event data are not available, a method which would use commonly available daily hydrologic data to make estimates of subsurface flow components would be a valuable tool for the water resource planner. Shirmohammadi et al. (1984) proposed an approximate method for partitioning streamflow. Their method utilized a constant rainfall threshold value as a partitioning criteria. Knisel and Sheridan (1983) investigated a simple procedure for partitioning daily streamflow into storm runoff and subsurface contributions. The method utilized a subjective estimation of storm runoff duration (time base).

Separately measured surface and subsurface flow from a 0.34 ha upland drainage area in the Georgia Coastal Plain revealed that about 79 percent of the total streamflow was subsurface (Hubbard and Sheridan, 1983). Since the Coastal Plain is a composite of upland drainages, it is reasonable to assume that subsurface flow could represent approximately three-fourths of total streamflow when evapotranspiration by riparian vegetation is considered. The purpose of this paper is to present such an approximate procedure for partitioning daily streamflow and the results of testing of the method.

CHARACTERISTICS OF THE AREA AND DATA AVAILABLE

The U. S. Department of Agriculture, Agricultural Research Service (USDA-ARS) maintains research watersheds on Little River which lies within the Tifton Upland physiographic area of the southeastern Coastal Plain of the United States. Stratigraphically, the area is described as being in the outcrop area of the Miocene series, Hawthorn Formation. The parent material, Hawthorne Formation, which is overlain by Quaternary sands, is continuous and serves as an aquiclude in the Tifton Upland (Stringfield, 1966). The soils are characterized by high infiltration rates (Rawls et al., 1976), which are conducive to subsurface flow of water from valley flanks to the streams.

The Southeast Watershed Research Laboratory facility consists of a network of 55 recording raingages and nine streamflow gaging stations with continuous water-level recorders (Yates, 1976). The gaging stations provide streamflow data from contiguous watersheds ranging in size from 2.62 km^2 to 334 km^2 (Table 1).

Weighted daily rainfall data are available for each research watershed. The Georgia Coastal Plain Experiment Station (GCPES) raingage located at Tifton, Georgia, provides another set of rainfall data for use in these analyses. Mean annual rainfall and streamflow volumes are shown in Table 1 for the period of record for each watershed. Annual precipitation data in Table 1 indicate the record period is near normal when compared with the 1,200 mm long-term point rainfall at Tifton, Georgia.

Table 1. Little River Watershed Drainage Areas, Record Period, Mean Annual Rainfall and Streamflow, and Storm Runoff Base.

Watershed	Drainage Area	Record Period	Mean Annual Rainfall	Mean Annual Streamflow	Storm Runoff Base
	km		mm	mm	days
M	2.6	1968-81	1,242.9	290.2	1
K	16.7	1968-81	1,241.1	371.2	2
J	22.1	1968-81	1,249.7	375.9	2
I	49.9	1968-81	1,265.0	391.4	2
F	114.9	1969-81	1,262.9	366.3	3
O	15.9	1969-81	1,245.9	359.7	2
N	15.7	1971-81	1,250.9	363.9	2
B	334.3	1972-81	1,250.4	328.5	4

METHOD OF ANALYSES

Hypotheses and Criteria

The principal hypothesis for the study is that daily rainfall and streamflow volumes can be used to partition total streamflow into direct runoff and subsurface flow. The method of streamflow partitioning was tested with data from research watersheds in the Coastal Plain of Georgia. It was further hypothesized that: (a) upland unit-source area total surface and subsurface flow data are representative of large mixed-cover watersheds, (b) use of point rainfall data versus weighted rainfall data affects the resultant streamflow components predicted by the procedure, and (c) using variable rainfall threshold value (IA) would increase the performance accuracy of the partitioning method.

A cursory examination of rainfall, runoff, and streamflow records was made to establish criteria for estimating both when direct surface runoff might occur and the probable duration of the surface runoff event. This, along with field observations, revealed that surface runoff may occur from small amounts of rainfall in the winter and early spring when evapotranspiration is low and the phreatic groundwater is generally recharged. Coastal Plain streams flow over a broad alluvial riparian zone. This flow system and the adjacent seepage face along the valley flanks results in significant areas of wetlands that respond quickly to small amounts of rainfall. In summer and autumn periods, the wetlands are not as extensive and greater amounts of rainfall are necessary to initiate storm runoff. This is the partial area concept of surface runoff (Dunne and Black, 1970).

Both constant and variable rainfall threshold values were used as partitioning criteria. First, a constant threshold value of 5 mm was arbitrarily selected as the amount of rainfall necessary for wetland response (surface runoff). Second, the Soil Conservation Service method (1972) was used to determine the seasonally varying rainfall threshold value (initial abstraction, IA) from 12 years of available soil moisture data in the uplands of the Georgia Coastal Plain. This relationship was represented by a third degree polynomial function as follows:

$$IA = -(11.5 \times 10^{-7})\ J^3 + (4.9 \times 10^{-4})\ J^2 - (3.0 \times 10^{-2})\ J + 7.91 \quad (1)$$

where, IA is the rainfall threshold value (initial abstraction) and J is the Julian day.

The duration of surface runoff is a function of watershed characteristics, including soil type, slope, flow length, and drainage area. From detailed hydrograph analyses and cursory examinations of hydrograph shape, travel times between tandem gaging stations were also estimated for selected storms. Since daily values of rainfall and streamflow were used, time increment or storm runoff base was restricted to a multiple of days. The time base for storm runoff duration was estimated as the day on which rainfall exceeds the threshold value plus multiples of days dependent upon size and characteristics of the watershed. The time base estimated for each watershed is given in Table 1.

Method of Streamflow Partitioning

Preliminary streamflow partitioning into storm runoff and subsurface flow was performed by Knisel and Sheridan (1983). Although plottings or bargraphs were not used, the method is demonstrated in Figure 1. The values shown in Figure 1 are not real data, but are merely for demonstration of the method as used for Watershed F (storm base = day of rainfall plus 2 days).

In Figure 1, 4 mm of weighted rainfall are shown on day 4, but since this is less than the constant threshold value of 5 mm, no surface runoff is estimated. On day 6, 22 mm of rainfall is shown. Since this is greater than the threshold value, surface runoff is assumed to occur on that day plus on the following 2 days. A straight line is drawn from day 5 to day 9 and the streamflow above that line (hatched area) is assumed to be surface runoff. Rainfall in excess of the threshold value occurred on days 11, 12, and 13, thus surface runoff is assumed on days 11-15. The dashed line is drawn from day 10 to day 16 and the hatched area is the estimated surface runoff and that below the dashed line represents subsurface flow. Rainfall greater than the 5-mm threshold occurred on days 21 and 23. Since storm runoff is assumed to occur on the 2 days following the day of rain, the dashed line is drawn from day 20 to day 26. The same partitioning principle was used when variable rather than constant threshold value was used as a daily partitioning criteria. This procedure was followed for the entire period of record for each watershed. Relative volumes were not estimated for each storm or by months, but on an annual basis only.

RESULTS AND DISCUSSIONS

Table 2 presents the observed annual precipitation and total streamflow as well as the observed and predicted ratios of subsurface flow to total flow for the 0.34-ha Watershed Z obtained using the partitioning method. The data indicate considerable variability in the ratios of subsurface to total flow for the 12-year record period.

Figures 2a and 2b show a comparison of predicted and observed surface and subsurface flow components. Although it is evident that the prediction method overestimates surface runoff and underestimates subsurface flow for 11 out of 12 years for constant threshold value (Figure 2a) and 8 out of 12 years for variable threshold value (Figure 2b), the results of a t-test showed that on a probability level of $\alpha = 0.05$ there was no significant difference between the predicted and observed flow volumes. The average difference between observed and predicted annual ratios are 0.055 and 0.035 with a standard error of 0.068 and 0.054 for the 12-year record period when constant and variable threshold values were used as partitioning criteria, respectively. Ratios of observed subsurface flow to total streamflow ranged from 0.45 to 0.93 compared with predicted ratios of 0.43 to 0.87 with constant threshold value and 0.45 to 0.87 with variable threshold value (Table 2). The 12-year average predicted ratios of 0.70 (with constant threshold value) and 0.72 (with variable threshold value) are comparable with the average observed ratio of 0.755 (Table 2). There was about 2 percent average increase in performance accuracy.

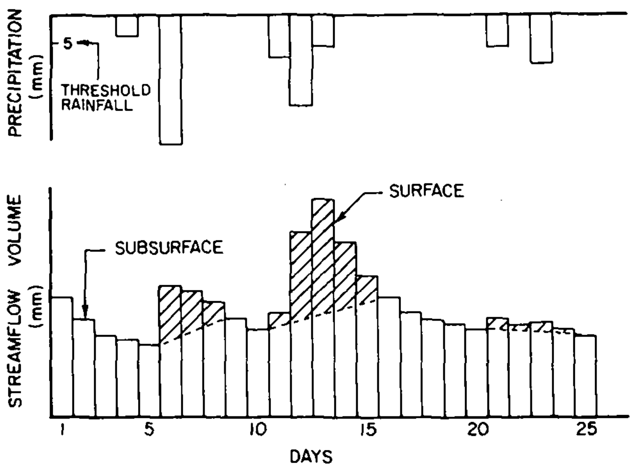

Figure 1. Schematic representation of streamflow partitioning.

Table 2. Observed Annual Precipitation and Total Streamflow Along with Both Observed and Predicted Ratios of Subsurface Flow-to-Total Flow, Watershed Z, 1970-1981.

Year	Precipitation mm	Total Discharge mm	Ratio: Subsurface/Total Observed	Predicted Constant IA	Predicted Variable IA
1970	1,387	532.7	0.803	0.784	0.762
1971	1,278	330.3	0.668	0.620	0.621
1972	1,161	349.3	0.801	0.753	0.831
1973	1,222	487.7	0.848	0.746	0.768
1974	1,275	254.8	0.788	0.625	0.625
1975	1,331	418.4	0.847	0.765	0.764
1976	1,369	456.8	0.778	0.749	0.813
1977	1,166	239.9	0.453	0.435	0.447
1978	1,171	192.0	0.683	0.610	0.627
1979	1,209	334.2	0.933	0.873	0.867
1980	1,232	561.5	0.718	0.681	0.678
1981	1,064	173.0	0.742	0.757	0.831
Average	1,239	360.9	0.755	0.700	0.720

Total streamflow measured on the Little River watersheds was also partitioned using the approximate method. For comparison, data from Watershed K are shown in Table 3 for a constant rainfall threshold value and a variable rainfall threshold value. Values are considerably different between Watersheds Z and K, but the annual trends are very similar.

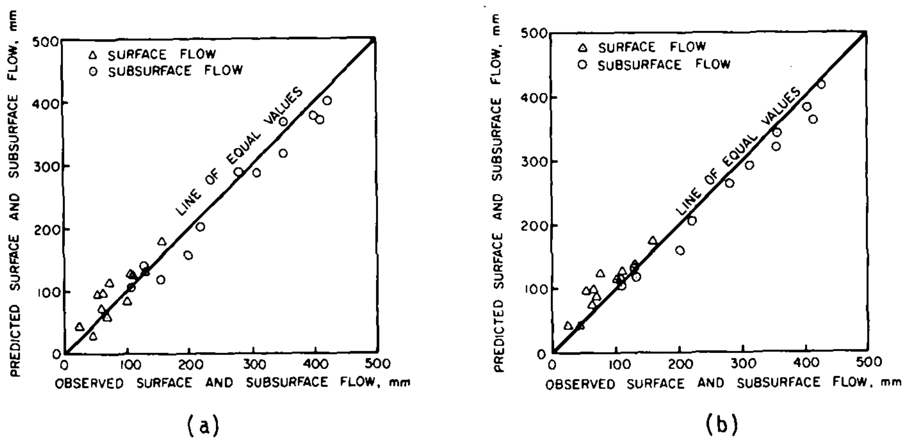

Figure 2. Predicted surface and subsurface flow based on constant (a) and variable (b) rainfall threshold value versus observed surface and subsurface flow for Watershed Z, 1970-1981.

Table 3. Observed Annual Weighted Precipitation and Streamflow, and Predicted Ratios of Subsurface-to-Total Flow, Watershed K, 1968-81.

Year	Precipitation mm	Observed Streamflow mm	Ratio: Subsurface/Total Constant IA	Ratio: Subsurface/Total Variable IA
1968	849	37.4	0.864	0.878
1969	1,122	222.4	0.649	0.621
1970	1,480	535.9	0.585	0.576
1971	1,384	511.3	0.683	0.724
1972	1,125	343.4	0.732	0.735
1973	1,338	544.8	0.632	0.635
1974	1,351	364.1	0.609	0.634
1975	1,410	491.4	0.652	0.662
1976	1,462	476.7	0.663	0.753
1977	1,178	358.4	0.773	0.812
1978	1,135	375.9	0.727	0.711
1979	1,364	437.5	0.739	0.714
1980	1,122	426.9	0.688	0.689
1981	1,055	38.9	0.830	0.904
Average	1,241	368.9	0.702	0.718

Table 4 shows the effect of using precipitation from a single raingage to partition streamflow for Watershed I. The partitioning method was used to estimate surface and subsurface flow components based on observed daily streamflow with watershed weighted and GCPES (single-gage) precipitation amounts. Constant and variable rainfall threshold values were used in partitioning streamflow with weighted and single-gage rainfall. The ratios obtained for all four cases have annual trends similar to those for Watersheds Z and K. Results of a t-test showed, there was no significant difference (α = 0.05) between the ratios of subsurface flow to total streamflow when weighted and GCPES precipitation data were used in the partitioning for 14 years. Similar results were obtained for Watersheds J and K.

Record periods for all of the watersheds studied are not the same, therefore a common 10-year period (1972-1981) was selected for purposes of comparisons (Table 5). Average annual ratios range from 0.582 and 0.621 for Watershed O to 0.818 and 0.834 for Watershed M when constant and variable rainfall threshold value (IA) was used as a partitioning criteria, respectively. Watershed O exhibited a low ratio of estimated subsurface to total flow relative to the other research watersheds. The hydrologic characteristic is that Watershed O typically responds more quickly to rainfall and has a shorter storm runoff time base than nearby Watershed N, for example. Higher ratios of subsurface to total flow for Watershed M reveal that the 1-day storm base probably is too short for this watershed, but in view of the restriction of time base to daily multiples, the 1-day stormbase is a better estimate than two days. The low average total streamflow for Watershed M (Table 6) indicates there may be a significantly different rainfall-streamflow relationship as a result of different geologic and soil conditions within that drainage area. Data also indicate that the streamflow partitioning method is less sensitive for variable rainfall threshold value when used for larger watersheds (Watershed B) as compared to smaller watersheds (Watersheds M-F).

Table 4. Annual Weighted Precipitation, GCPES (Georgia Coastal Plain Experiment Station) Single-Gage Precipitation, Total Streamflow, and Estimated Ratios of Subsurface Flow to Total Flow, Watershed I, 1968-1981.

Year	Observed Streamflow mm	Weighted Precipitation mm	Ratio: Subsurface/Total		GCPES Precipitation mm	Ratio: Subsurface/Total	
			Constant IA	Variable IA		Constant IA	Variable IA
1968	55.1	869.2	0.918	0.912	974.9	0.855	0.870
1969	268.8	1,125.5	0.682	0.648	1,178.3	0.756	0.639
1970	589.3	1,477.3	0.594	0.579	1,379.0	0.654	0.661
1971	503.7	1,373.9	0.714	0.743	1,354.1	0.686	0.717
1972	336.5	1,107.2	0.752	0.749	1,138.7	0.675	0.693
1973	542.1	1,333.5	0.664	0.660	1,239.3	0.591	0.606
1974	363.6	1,321.8	0.652	0.679	1,241.6	0.628	0.642
1975	486.7	1,391.4	0.683	0.683	1,329.4	0.620	0.626
1976	511.1	1,457.7	0.666	0.757	1,349.8	0.597	0.731
1977	382.2	1,211.3	0.773	0.818	1,126.2	0.754	0.769
1978	399.9	1,159.5	0.716	0.707	1,114.8	0.608	0.603
1979	484.6	1,397.0	0.738	0.735	1,156.0	0.630	0.665
1980	468.2	1,121.4	0.661	0.662	1,238.8	0.638	0.652
1981	75.7	1,110.0	0.827	0.914	1,032.3	0.851	0.938
Avg.	390.5	1,247.1	0.717	0.732	1,203.8	0.682	0.701

Storm-runoff time base used in these analyses was estimated from cursory examination of observed runoff hydrographs (Table 6). Regression analyses were performed to relate observed storm time base, SB, to drainage area for Little River watersheds (Figure 3). The resulting relationship is

$$SB = aA^b \text{ or } \ln(SB) = \ln(a) + b \ln(A) \qquad (2)$$

Table 5. Watershed Mean Annual Precipitation, Streamflow, and Ratios of Subsurface Flow to Total Flow, 1972-81.

Watershed	Drainage Area (km^2)	Precipitation (mm)	Streamflow (mm)	Ratio: Subsurface/Total Constant IA		Variable IA
Z	0.0034	1,220	346.8	0.777	(Observed)	0.777
M	2.62	1,257	307.4	0.818		0.834
N	15.7	1,242	362.8	0.628		0.660
O	15.9	1,231	347.2	0.582		0.621
K	16.7	1,254	385.8	0.704		0.725
J	22.1	1,268	393.7	0.650		0.683
I	49.9	1,261	405.9	0.713		0.736
F	114.9	1,243	355.9	0.652		0.676
B	334.0	1,250	328.5	0.808		0.810

where, SB is storm-time base, A is the drainage area in square kilometers, a and b are the intercept and slope of the regression equation. The resultant values for a and b are 0.856 and 0.270, respectively, with a correlation coefficient (r) of 0.981 (r^2 = 0.962).

While this functional relationship would facilitate the selection of the appropriate storm-time base for any size of drainage area in the Little River basin and similar basins in the Coastal Plain, an empirical procedure is suggested to determine the storm-time base for watersheds where avail-

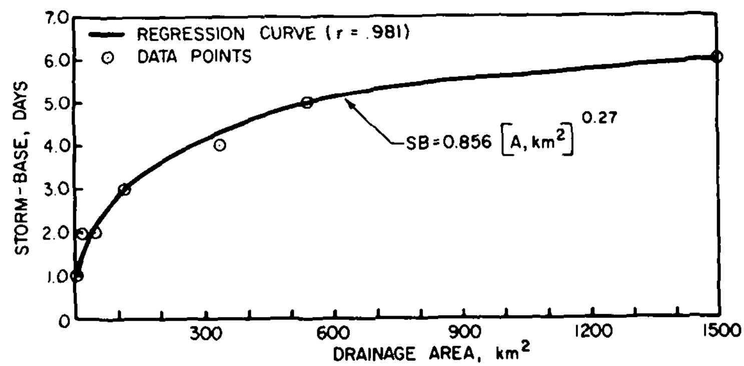

Figure 3. Functional relationship of storm-base and drainage area for Little River watersheds.

ability of hydrographs is limited. To do so, the empirical storm-runoff time base for Little River Watersheds was determined by using a combination of methods suggested by Linsley, et al. (1958) and Kirpich (1940). Time of concentration, Tc, in hours given by Kirpich (1940) and modified for metric units is

$$Tc = 0.000325\ L^{0.77}\ S^{-0.385} \tag{3}$$

where, L is channel length, m, and S is channel slope, m/m. Linsley, et al (1958) related the time from the hydrograph peak to an arbitrarily selected point on the recession to size of drainage area. The relationship is

$$N = 19.84\ A^{0.2} \tag{4}$$

where, N is time in hours and A is drainage area in square kilometers. Then, the two time components Tc and N were added to determine the empirical storm-runoff time base. Table 6 shows the corresponding results compared with the storm-time base estimated from cursory examination of observed runoff hydrographs. Although the general agreement between empirical (Equations 3, 4) and observed time bases is good, the empirical procedure tends to underestimate the storm base for large watersheds when compared to observed durations (Table 6). This could be due to the error involved in the selection of an arbitrary recession point on the runoff hydrograph at the time of developing the proposed time base from peak of hydrograph to the recession point (Linsley, et al, 1958). However, the power function relating storm-runoff bases to drainage area developed in this study seems promising.

The comparative results of analyses are good considering the simplified approach. The ratio of subsurface flow to total flow for Watershed Z is greater than for all Little River watersheds except Watershed M. The data for Watershed Z are exact since surface runoff and subsurface flow are measured separately. Watershed Z represents the high-lying upland areas of the larger drainages and does not include near-stream saturated zones.

Table 6. Little River Watershed Physical Characteristics, and Empirical and Observed Storm Bases.

Watershed	Area	Channel Length, L	Slope, S	Tc (equ.3)	N (equ.4)	Storm-Base Empirical	Storm-Base Observed
	km^2	m	$(x10^{-3})$	hrs	hrs	Days	Days
M	2.6	3,423	6.86	1.16	24.05	1.05	1
N	15.7	6,572	5.29	2.13	34.40	1.52	2
O	15.9	6,366	5.70	2.02	34.51	1.52	2
K	16.7	9,377	4.29	3.03	34.82	1.58	2
J	22.1	10,240	3.93	3.35	36.86	1.68	2
I	49.9	12,594	3.46	4.13	43.37	1.98	2
F	114.9	23,658	2.28	7.88	51.23	2.46	3
B	334.3	40,440	1.605	13.63	63.44	3.21	4

This results in greater rainfall infiltrating on Watershed Z, with consequently higher observed subsurface flow amounts and lower quantities of surface runoff. In larger watersheds that include the saturated alluvial zones (typical of the Coastal Plain), rainfall on the saturated surface results in approximately 100 percent storm runoff, which tends to lower the overall estimated subsurface-to-total streamflow ratios. The data indicate that the approximate method of streamflow separation results in reasonable estimates considering two potential problems. One problem is the time of occurrence of storm runoff within the day, and the second is the selection of threshold rainfall amounts for storm runoff.

Convective storms generally occur in the late afternoon, which may result in only small amounts of storm runoff on the day of rainfall and the largest daily runoff volume occurs on the second day. This causes a shift in the actual partitioning line relative to the apparent line (Figure 1) thus resulting in considerable differences between actual and estimated storm runoff volumes and subsurface flow components. However, the largest errors of this type would be restricted to the smaller drainage areas with the shorter time bases.

CONCLUSIONS

Analysis of results led to the following conclusions.
(1) Daily rainfall and streamflow data can be used to obtain reasonable estimates of storm runoff and subsurface flow amounts.
(2) Use of variable rather than constant rainfall threshold value in partitioning slightly increased the performance accuracy.
(3) Use of single-gage precipitation data instead of weighted precipitation data as input into the partitioning method resulted in more variability of annual ratios of subsurface flow to total streamflow, but the average difference between the ratios was insignificant for the record period.
(4) The available empirical relationship to compute the storm-runoff time base underestimates the time base for large watersheds. A power function was developed to estimate the storm-runoff time base for Little River and similar watersheds in the Coastal Plain physiographic region.

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GROUNDWATER OCCURRENCES IN WEATHERED BASEMENT COMPLEX AREAS

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ABSTRACT

In deeply weathered Basement Complex areas, the occurrence of groundwater is closely related to the in-situ weathered overburden, the saprolite. It has been found that groundwater is almost exclusively present in the lower part of the saprolite which exhibits a high degree of hydrogeological uniformity. Well yields from the saprolite aquifers are predictable using simple probability methods. The landforms play a significant role and a geomorphological approach proves itself a useful method in describing the hydrogeology. In less weathered areas, groundwater is located in fractures which renders the groundwater occurrences less predictable, but still, geomorphological rules apply. The geomorphological approach is particularly useful in rural areas where the use of hand pumps requires a successful borehole rather than a high yielding one. Data from Tanzania, Sri Lanka and Niger are used to demonstrate the approach and explain the Basement Complex hydrogeology.

Keywords : Crystalline rocks, hydrogeology, geomorphology, groundwater occurrence, recharge, groundwater level, landforms, erosion surface, saprolite, weathering, boreholes, specific capacity, rural water supply, hand pumps.

INTRODUCTION

Rural and urban water supply in many developing countries are based on mainly surface water. This has to do with traditions as well as technology. Technically, a surface water supply is easier to establish than a groundwater supply, especially in crystalline Basement Complex areas where borehole yields are limited. Rivers and streams are being polluted by activities of man and industrial society resulting in the spreading of water borne diseases. This conflict is already now a problem and may be even more so in the immediate future.

Basement Complex rocks form a major geological unit in developing countries, especially in Africa. Recently, there has been a widening, if somewhat reluctant recognition that the groundwater across the Basement Complex is an attractive and readily exploitable water resource in rural areas. An understanding of the general hydrogeology of these areas and simple means of locating suitable sites for drilling is, therefore, important. Here, an attemt is made to relate the geomorphological history of an area to hydrogeological and hydrological data in order to establish the relationship between weathering, erosion and rainfall on one side and the groundwater occurrence on the other.

GEOMORPHOLOGY

Much of East and Central Africa has been exposed as continental land above sea level for over 200 mill years. The landscape is, therefore, amongst the oldest found on earth. Topographically, the landforms of the region are plateaus of exceptional extent and notable elevation, interrupted by isolated mountainous highlands and broad linear valleys of tectonic origin. Willis (1936), Dixey (1945 and 1956), Ruhe (1954), and King (1962) have mapped, classified and analysed these plateaus and established the cyclic erosion processes that have moulded the landscape since the late Jurassic.

The accepted erosion cycle concept involves periodic scarp retreat and landscape pedimentation on an almost continental scale. Each erosion cycle is initiated by a moderate vertical isostatic uplift caused by the unloading of the continental surface by the preceding erosion cycle. The continental erosion cycles identified in Africa since the Jurassic are:

Gondwana Cycle - late Jurassic
elevation 2200-2600 m

Post-Gondwana Cycle - early Cretaceous
elevation 1500-1800 m

African Cycle - late Cretaceous to late Oligocene
elevation 1000-1200 m

Post-African Cycle - Pliocene
elevation 600-900 m

Coastal Plain and Congo Cycles - Pliocene to Quaternary
elevation up to 400 m

Geologically, the plateaus and the isolated highlands are underlain mainly by crystalline plutonic and metamorphic rocks belonging to the Basement Complex. The Basement rocks across the plateaus are covered by in-situ weathered rock, the saprolite. The saprolite cover of the African and post-African surfaces is continuous except where recent movements have reactivated faults causing erosion of the saprolites, or where isolated knolls of very resistant bedrock outcrop through the ubiquitous saprolite. Erosion has removed most of the saprolite cover from the Gondwana and post-Gondwana highlands and now only isolated pockets remain. Across younger erosion surfaces, time has been insufficient for the saprolites to become well developed. The erosion of landscapes has taken place on other continents as well (Australia, the Americas, Asia) but it is in Africa that the plateau development is most striking.

WEATHERING AND EROSION

Weathering is the physical breaking down of rocks and can take place in two ways, mechanically and chemically. Erosion is the process by which weathering products are removed either mechanically or in solution. Mechanical and chemical weathering and erosion of a landsurface given sufficient time produces a well developed pediplain which ultimately becomes mechanically stable. From then on only chemical weathering and solution transportation can take place. The main agent of chemical weathering is circulating groundwater and, therefore, well developed saprolite profiles are found almost exclusively across mature pediplains which have been mechanically stable for millions of years.

Chemical rock weathering depends on several parameters other than time, the most important ones being mineral stability and composition of the parent rock, groundwater circulation and degree of jointing of the parent rock. The mineral stability or instability of the original minerals comprising the parent rock controls the abundance of clay particles in the weathered horizon. The best and most uniform saprolite aquifers are found where the parent rock contains abundant quartz and some readily weatherable feldspar. The poorest aquifers are found over ultrabasic rocks which produce predominantly clay minerals limiting groundwater circulation and hence, these rocks produce thin saprolites. The groundwater circulation depends among other factors on the rainfall. Humid regions normally develop thicker saprolite covers than dry regions, other factors being equal.

The degree of jointing of fissuring of the rock is important during the initial stages of the chemical weathering process as a freer circulation of groundwater is allowed with increasing fissuring. As chemical weathering proceeds, however, the joints and fissures become blocked with residual clay weathering products and they lose definition and become absorbed in the developing saprolite mass.

The development of the saprolite profile shown in Figure 1 can be readily plotted. The upper clayey zones, having been exposed longest, are completely weathered. The lower zones with less exposure are less weathered, and in medium or coarse grained quartzofeldspatic rocks this zone almost appears as a gravel with good intergranular porosity. This is the active part of the saprolite profile where the aquifer is found.

A geomorphological description of a weathered Basement Complex plateau gives the age of the plateau and, therefore also an indication of the saprolite thickness. Further, the landforms are given: old stable plateaus are flat to gently undulating, younger plateaus are rolling and irregular to different degrees. The plateau elevation is given and, therefore, in a given region, the rainfall. River run-off can be sketched. All the decisive factors controlling the hydrogeology of an area are included in a complete geomorphological description, and this makes a geomorphological approach a very useful hydrogeological tool.

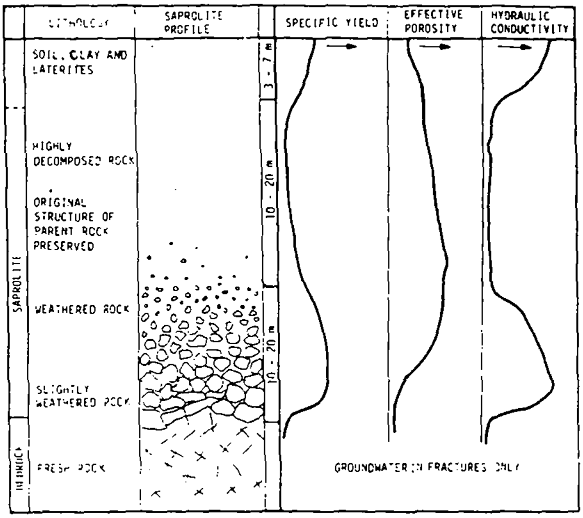


Figure 1. A typical saprolite profile and its hydraulic properties as found on a mature pediplain (from CCKK, 1982).

The geology is merely a single parameter. Once the mineral composition and degree of fissuring of the original parent rock is known, weathering and weathering products are chiefly known, and the geology has more or less played its part.

HYDROGEOLOGY

The saprolites form a composite intergranular aquifer covering the vertical variations from completely unweathered bedrocks to the completely weathered clayey sub-soil zone, as shown in Figure 1. The groundwater in the lower saprolite zones is semi-confined and the saprolite aquifer acts as a leaky artesian system.

The unweathered crystalline Basement Complex rocks may have some fracture and intergranular porosity of tectonic origin. These fracture zones do result in local areas of high hydraulic conductivity which, however, decreases rapidly with depth. The available storage is limited, and long-term yields of wells depend on the storage available in the overlying weathered saprolite.

Across an old deeply weathered plateau having a humid climate, a regional phreatic water table exists in the upper clayey saprolite zone. In elevated areas, this water table is above the piezometric level of the saprolite aquifer. Here, groundwater recharge takes place. In low lying areas, the piezometric surface is above the water table. Here, artesian discharge to a river or discharge in the form of springs occurs.

Across a young plateau with a less developed saprolite cover, the aquitard may not be perennial. Wells drilled in elevated areas may run dry during the dry season because of gravity drainage, and groundwater occurrence is limited to fracture zones. During the dry season, the groundwater catchment area diminishes, water levels become deep, and rivers are likely to run dry.

The oldest deeply dissected high plateaus are during the dry season discharged mainly through springs and seeps. The base-flow component of the dry season run-off will typically exhibit two modes of discharge, one from fracture systems exposed on the slopes and one from the saprolite cover. The fracture system contribution quickly fades out due to the limited storage capacity, while the saprolite contribution feeds the rivers throughout the dry season. This is illustrated in Figure 2 which shows typical hydrographs from the high Basement Complex plateaus in the Southern Highlands of Tanzania.

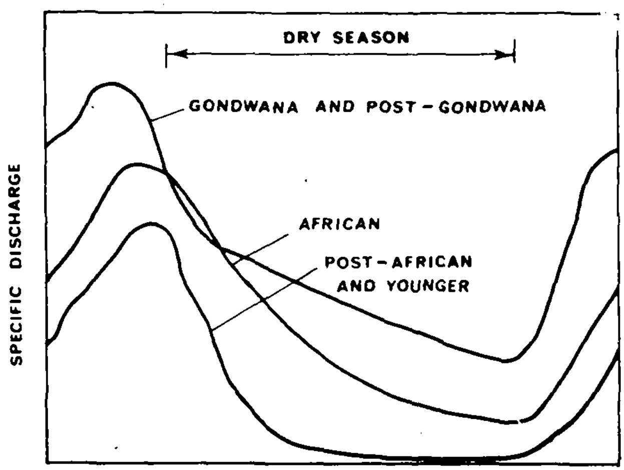


Figure 2. Typical time hydrographs from the high plateau catchments in the Southern Highlands of Tanzania.

Omorinbola (1982) stated that the conditions for perennial aquifers to exist in crystalline rock areas are thick and widespread saprolites, a flat or gently undulating landscape with limited overland flow and subaerial erosion of saprolites, and sufficient rainfall to provide water needed for groundwater recharge. These conditions are met across old stable plateaus, across young plateaus often they are not.

CASE STUDIES

Tanzania

Data presented are all taken from CCKK (1981, 1982) and represent information from boreholes situated across the African and post-African erosion surfaces. A major part of the boreholes are drilled for village water supply and are, therefore, situated in or close to villages, i.e. in the watershed areas. The study areas are spread widely and, therefore, the mean annual rainfall varies from 900-1600 mm. The landscape is that of mature pediplains, i.e. flat to gently undulating, except for limited areas where Neogene tectonics have established new base levels of erosion (Rift Valleys).

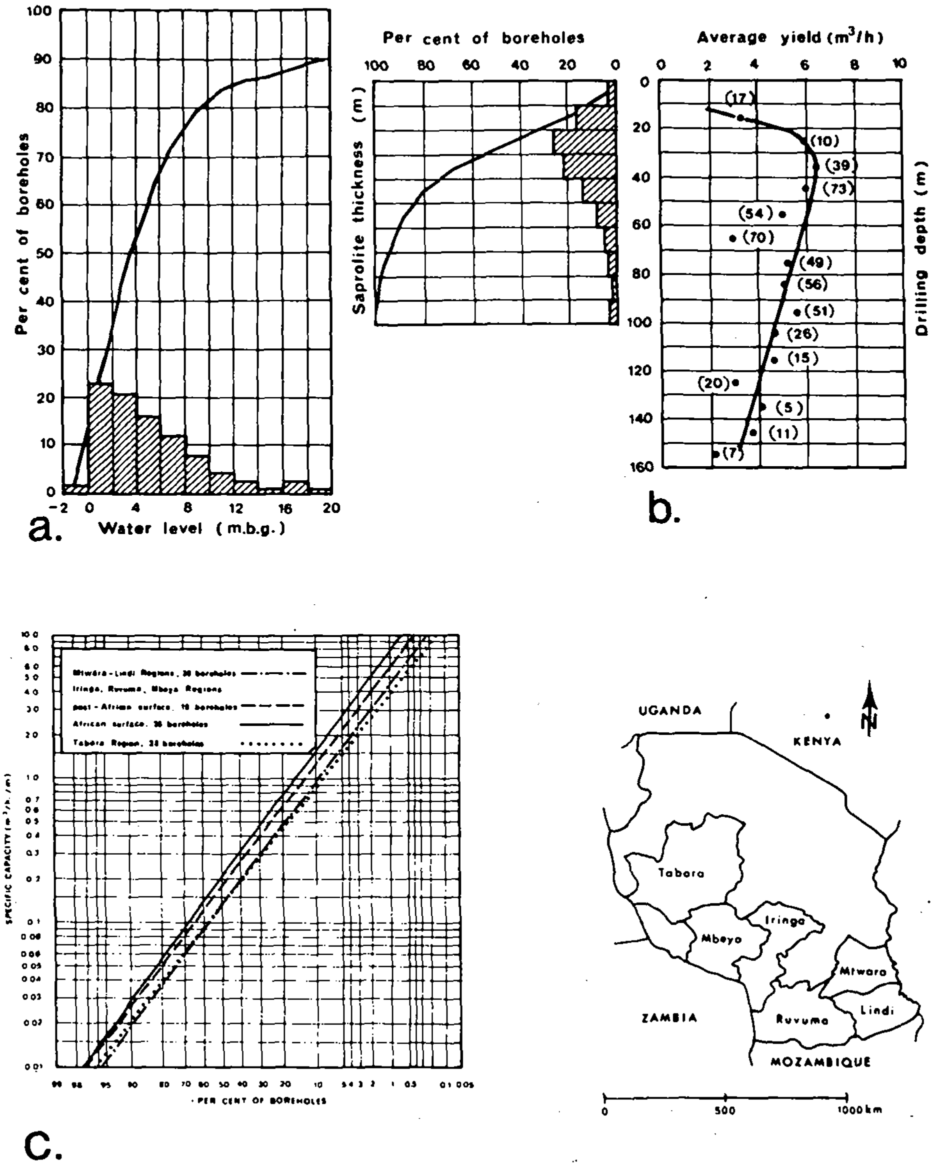


Figure 3. Data from African and post-African erosion surfaces in Tanzania:
a) Water level frequency distribution from 239 boreholes.
b) Saprolite thickness frequency distribution from 236 boreholes and average yield as a function of drilling depth from 503 boreholes. Numbers in brackets denote number of boreholes averaged.
c) Specific capacity frequency distributions.

Figure 3 shows the water level and saprolite thickness frequency distributions, average yield as a function of depth and the specific capacity frequency distribution. In 75% of the boreholes the water level is less than 6 m below ground. 90% of the successful boreholes have specific capacities larger than 0.016 $m^3/h/m$ (0.5 m^3/h with 30 m drawdown). This makes the saprolite aquifer across the African and post-African erosion surfaces an attractive groundwater resource for rural water supply using boreholes equipped with hand pumps.

The saprolite aquifer is situated 30-60 m below ground where the highest average yields in the range 5-7 m^3/h are obtained. Comparing this depth interval with the saprolite thickness distribution it appears to coincide with the lower saprolite zones. Below this zone, yields decrease with depth. The specific capacity frequency distribution graphs are almost parallel and very close indicating that the geohydraulic conditions are uniform over large areas in the weathered crystalline Basement Complex.

Sri Lanka

Data originate from drilling campaigns in the central and southern part of Sri Lanka, Figure 4 (Kampsax-Krüger 1982, Klitten 1984). The mean annual rainfall in the study areas is high, 1600-1900 mm. The Matale and Anuradhapura districts are situated on old now heavily dissected crystalline Basement Complex plateaus. Erosion is active and the saprolite cover is located in isolated pockets in some places, in other places very thin or totally missing. The Polonnaruwa and Hambantota districts fall on a young Basement Complex erosion surface, similar to the Coastal Plain surface in Africa. The plateau is flat and chiefly stable, but because of its youthfulness, the saprolite cover is thin.

Figure 4 shows an average saprolite thickness around 10 m. Therefore, groundwater is to a large extent located in fractures, but still the lower saprolite zone contributes significantly to the borehole yields, as does the upper zone of the bedrock. Again, yields decrease rapidly with depth. Since fracture systems are important here, a large variation in depth to the groundwater level would be expected. However, this is counterbalanced by the high rainfall. The borehole yield frequency distribution graphs are parallel and close, again indicating the geohydraulic uniformity of Basement Complex plateau areas.

Niger

The area of investigation is Damagaram-Mounio in southern Niger (Figure 5) which falls on the post-African erosion surface. The rocks underlying the area belong to the crystalline Basement Complex. Data are from a drilling programme conducted by I. Krüger AS based on a hydrogeological study (1977) supplemented by the results of satellite data processing (Zafiryadis, 1982).

The mean annual rainfall in the area is normally around 500 mm but during the last decade rainfall in parts of the area has been below 150 mm. This means that no perennial rivers exist, and only groundwater sources may provide water throughout the year. The topography of the area is extremely flat. Because of the dry climate, the saprolite thickness is expected to be low, and the groundwater levels deep. It appears from Figure 5 that this is indeed so. The mean depth to groundwater is about 18 m and the mean saprolite thickness is about 10. Therefore, the saprolites are often dry, in which case groundwater is taken from fracture systems at depth. The highest

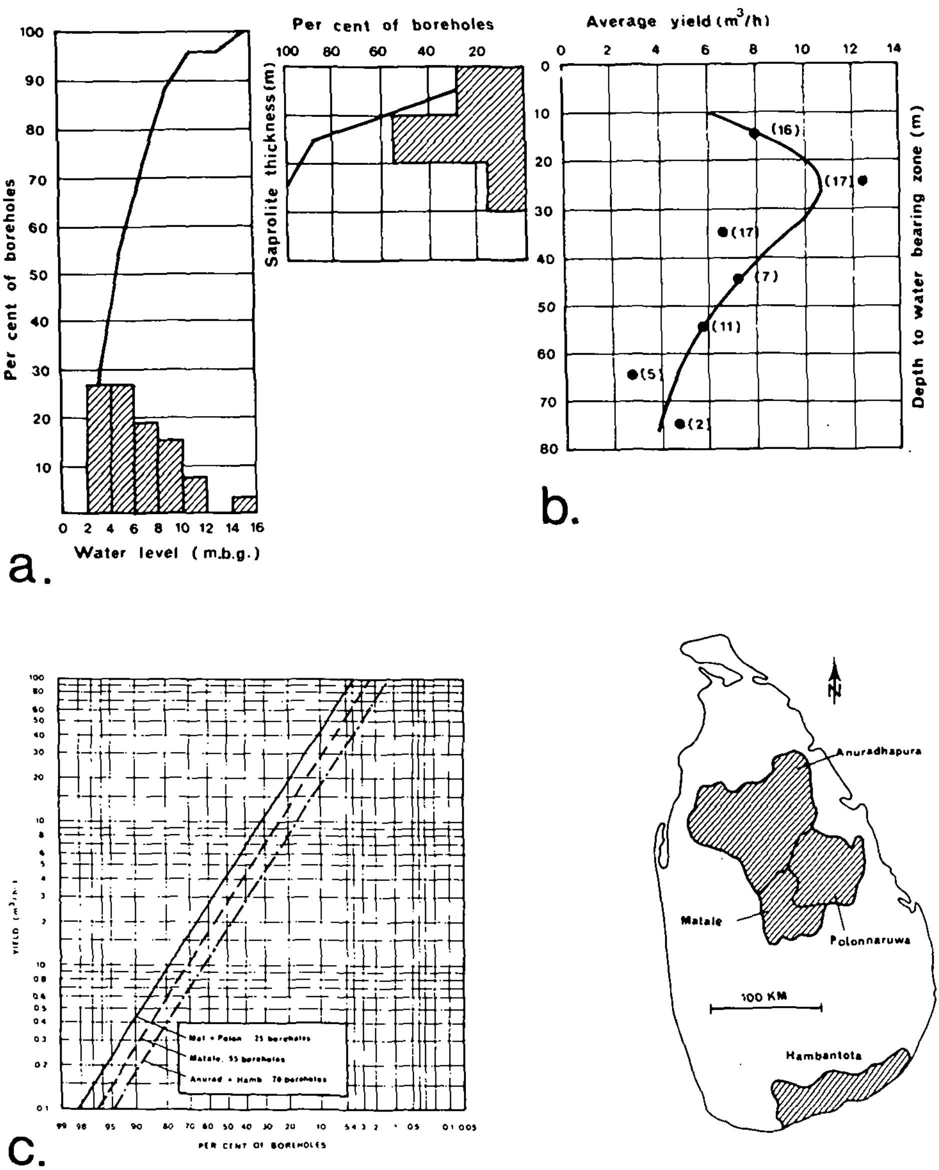
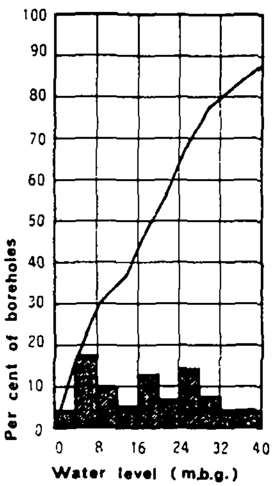


Figure 4. Data from 4 Districts in Sri Lanka.
a) Water level frequency distribution from 26 boreholes, Matale and Polonnaruwa.
b) Saprolite thickness frequency distribution from 25 boreholes and average yield versus depth to water bearing zone from 75 boreholes, Matale and Polonnaruwa.
c) Yield frequency distributions.



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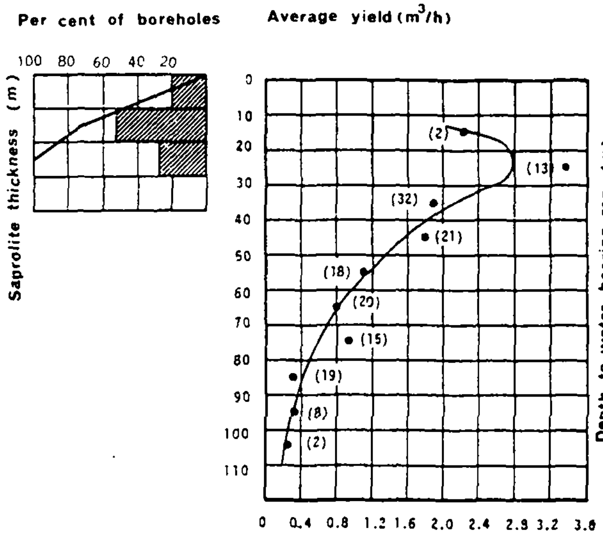

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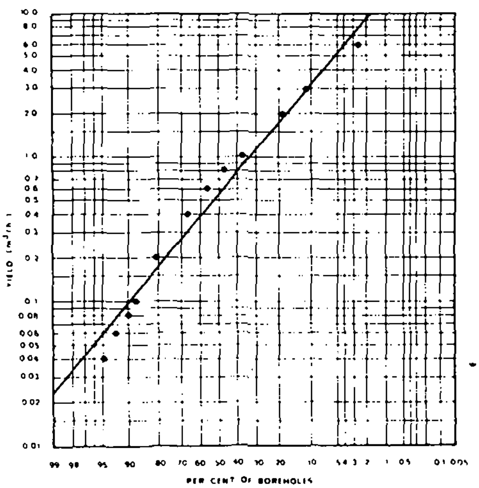

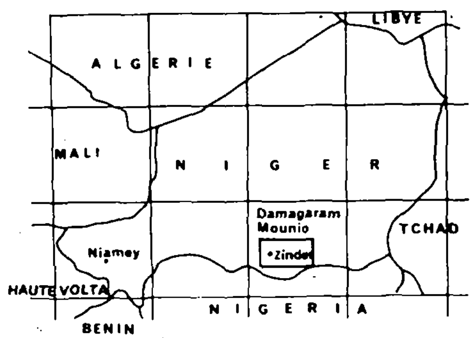

c.

Figure 5. Data from the post-African erosion surface in Niger.
a) Water level frequency distribution from 121 boreholes.
b) Saprolite thickness frequency distribution from 104 boreholes and average yield as a function of depth from 150 boreholes.
c) Yield frequency distributions from 169 boreholes.

are again found in the lower saprolite zones below which yields decrease rapidly with depth. As would be expected, yields are small, about one order of magnitude lower than found in Tanzania and Sri Lanka.

Although the saprolite is of minor importance in the area, groundwater occurrences and well yields are in agreement with what could be expected from the geomorphological characteristics.

CONCLUSIONS

From data presented, a close relationship between the geomorphology and the hydrogeology of a weathered Basement Complex erosion surface has been demonstrated. Combining meteorological, hydrological, geological and hydrogeological data with a geomorphological description of a Basement Complex plateau will provide an almost complete assessment of the groundwater occurrences and availability. The variation of physical parameters of the study areas in Tanzania, Sri Lanka and Niger demonstrates that the Basement Complex plateaus comprise an attractive groundwater resource in rural areas, using boreholes equipped with hand pumps. This type of water supply will greatly reduce the incidence of water-borne diseases and must be considered one of the obvious means of improving the living conditions of rural populations in the Third World.

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CONCEPTUAL MODEL OF EROSION AND SEDIMENTATION PROCESSES

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ABSTRACT

Erosion and sedimentation have been occurring since prehistoric times. Indeed, these natural processes have resulted in topographic features such as peneplains, rearrangements of glacial materials, and the sinuous nature and maturity of rivers and streams. Erosion and sedimentation are not processes that can be completely stopped--water moving on the land surface and in streams and rivers will always move sediments. In recent times man's activities have drastically increased the rate of these processes. In order to better define the interrelationships between various components and processes that impact erosion and sedimentation, a set of "conceptual models" has been developed to provide a sound and comprehensive framework for future plans of action.

Two levels of models were developed. The Level I model serves the important function of identifying the major subdivisions of the environment including natural and human factors that influence the erosion and sedimentation processes. Within the Level II models, the total environment has been divided into ten systems or subsystems and a model for each system or subsystem has been developed. The ten Level II models are for Agriculture, Grassland, Forest, Mining, Urban, Construction, Streams and Rivers, Lakes and Reservoirs, Permanent Wetland, and Seasonal Wetland. Generalized descriptions of the Level I model and two of the Level II models is presented in this paper.

Keywords: Erosion, sedimentation, conceptual model(s), agriculture, grassland, forest, mining, construction, urban, streams and rivers, lakes, wetland.

INTRODUCTION

Soil erosion is the process by which soil particles are dislodged and carried away by water or wind. These particles may then be deposited as sediment in lakes, rivers, or streams. Erosion and sedimentation processes are natural events which have been in effect over geological time periods. Both processes are partially counteracted by other natural events: erosion by the chemical and biological process of new soil formation and sedimentation by the renewed erosion and removal of sediments during floods or high wind. However, the balance of these natural events results in a slow net transfer of soil particles by erosion from landscape surfaces and by sedimentation, into lakes, streams, and the ocean.

Though erosion and sedimentation are natural processes, human activities have drastically increased their rates. As a result, topsoil is lost from farms, lakes are accumulating deep layers of sediment, and rivers and streams frequently carry heavy loads of eroded soil particles which impact water quality and may settle as layers of sediment. In Illinois current average annual soil losses are 2.5 to 6.0 times greater than the natural rate. This soil loss is at the rate of 1.5 Kg per m^2 per year. In parts of Illinois, about 60 to 70 percent of the original topsoil has been lost due to wind and water erosion.

Sedimentation rates have also increased such that, for example, a billion kilograms are deposited each year in the Illinois River Valley lakes, and about 11 billion Kg are transported into the Mississippi River. Extensive lake sedimentation surveys in Illinois have shown that the Illinois lakes are losing their capacities at the rate of about 0.1 percent per year to a maximum of 3.5 percent per year. This indicates that some lakes will be almost useless in 25 to 30 years. As an example, it is estimated that Upper Peoria Lake in Illinois will lose approximately half of its volume by the year 2000.

Because the erosion and sedimentation processes by nature are complex, and because governmental agency jurisdiction and responsibilities are complicated, this research was initiated to determine the existing information on erosion and sedimentation processes and to develop conceptual models to describe the impact of these processes on the natural environment.

CONCEPTUAL MODELS

Level I model

Because of the complexity of the soil erosion and sedimentation processes, an organizing framework was needed. The most useful framework is a conceptual model or diagram that clearly shows the important components and the interactions among the components. To demonstrate the general framework for organizing information on erosion and sedimentation in any watershed or country, a single general model was constructed as shown in Figure 1. This Level I model is very general in nature but serves the important function of identifying the major subdivisions of the environment and the important natural and human factors which influence erosion and sedimentation processes. In this Level I model, the watershed or the country is represented by the large box

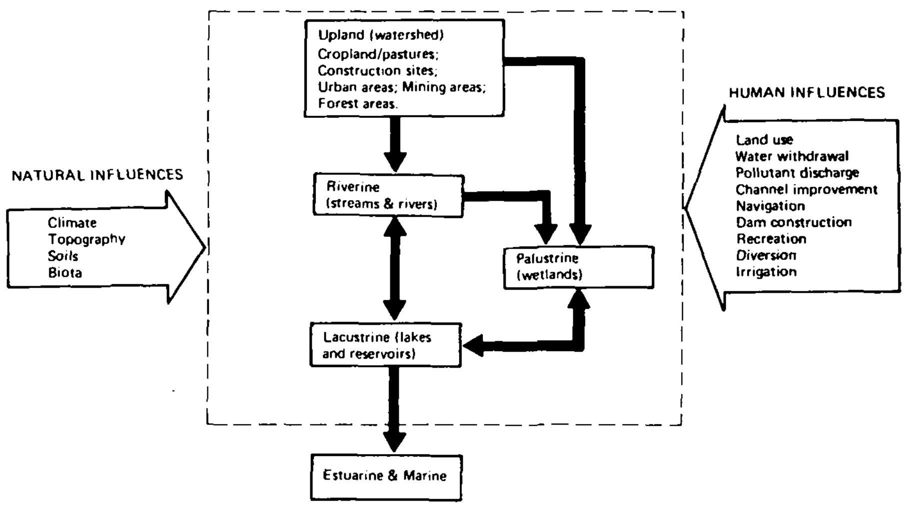

Figure 1. A conceptual model for the transport of sediment, biota, nutrient, and chemicals by water

(dashed lines) which has been divided into four major systems: Upland, Riverine (Streams and Rivers), Palustrine (Wetlands), and Lacustrine (Lakes and Reservoirs).

Two other systems, the Estuarine and Marine Systems, are represented by the lowermost box outside the dashed lines. It should be noted that most of the sediment and adsorbed materials are eventually deposited in these two systems. The classification shown in this model generally follows the U.S. Fish and Wildlife Service's classification system for wetlands and deep water habitats (Cowardin et al., 1979).

The Upland System consists, by definition, of land surfaces which are not inundated by water from a river, lake, or reservoir even during extreme high water periods. This system includes most of the land use patterns such as agriculture, urban, mining, construction, pasture, and forest.

The Riverine System consists of stream and river channels and their border areas. Here there is either a continuous or periodic flow of fresh water in the channel.

The Lacustrine System generally refers to natural lakes and man-made reservoirs including most of the border areas flooded during periods of high water. In a Lacustrine System it is assumed that there will be some water in the deepest part of the lake or reservoir even during periods of low water.

The Palustrine System consists of most nontidal wetland including some tidal wetland where salinity is below 0.5 percent. Palustrine wetlands may be situated on the fringes of Riverine, Lacustrine or Estuarine Systems. This system also consists of all temporary or permanent water bodies often called ponds. Wind and water movement has a limited effect on the erosion potential of a Palustrine System. The Palustrine System can also be temporarily flooded.

The movements of sediment, nutrients, biota, and chemical pollutants from one system to the other are indicated by the arrows between the different boxes in the model. The primary directions of material movement are shown by wide arrows. This is generally from the Upland to the Riverine, from the Riverine to the Lacustrine, and finally to Estuarine or Marine Systems. Double arrows indicate the potential of two-way movement of material. For example, in areas where the Palustrine System is found, water, sediment and adsorbed materials may be exchanged between the Palustrine System and the Lacustrine or Riverine Systems. If a dam is built on the main channel of a river or if a river originates from a lake or reservoir, material may move from the Lacustrine to the Riverine System. This is shown by the double arrow between the Riverine and Lacustrine Systems.

The processes of soil erosion, sediment transport, and sedimentation within each system and the interactions between the systems are controlled by natural and human influences exerted on the entire environment. These natural and human influences are represented in the model by two switches acting on the large box, which represents the whole watershed. Among the natural factors which influence the erosion and sedimentation processes are the soil characteristics, topography, vegetation, natural biota, and climate, including seasonal and long-term changes in climate.

Human factors which influence the erosion and sedimentation processes may be classified into two broad categories: land use and water use patterns. Among the land use patterns are agriculture, pasture, urban, mining, construction, and forestry. The water use pattern may consist of water withdrawal, nonpoint and point sources of pollution, recreation, diversion, and others.

Some studies describe erosion or sedimentation processes on the basis of just one component or habitat, such as upland cropland. However, other studies concern more than one model component -- for example, upland watersheds and the impacts on bottomland lake habitats. Similarly, many relevant questions address more than one component and also call for information about one or more of the natural and human influences.

Level II models

Representation of the complex interrelationships between various systems and subsystems required the development of specific models designated here as Level II Models. These models are for agriculture, grassland, forest, mining, urban, construction, riverine (rivers and streams), permanent wetland, seasonal wetland, and lacustrine (lakes and reservoirs). These models are fairly complex and contain a significant amount of information. They are also interrelated with keywords, bibliography, and model interactions.

In order to demonstrate the complexity and the important contributions of Level II models, brief descriptions of two models and figures showing the respective models are included here. A detailed description of these models, their various interactions, and other associated materials are given in the publication by Bhowmik, et al (1984).

Before a description of the model is presented, it should be pointed out that within each model component, the parameter(s) shown are also the keywords utilized in the literature search and for the identification of various related references for specific interactions.

Agriculture subsystem

Figure 2 shows the Level II model for the agriculture subsystem. It includes all row-cropped acreage, cultivated nurseries, truck crop, and small grain crops. The model shown can be broadly subdivided into 5 categories from left to right. The first group of parameters consists of the economic factors that are imposed upon the agriculture. The second group of parameters can be put loosely under "management influences" or strategies needed at the farm level. The physical and natural characteristics of the watershed or the farmstead form the next group of parameters within this subsystem. The external physical constraints and the resultant erosion and sedimentation because of natural or man-made influences form the fourth group of parameters. Finally, the export of materials consisting of both the adsorbed and nonadsorbed types forms the last group of parameters.

The interactions between and among various parameters are shown by arrows. It must be emphasized here that the arrows do not necessarily indicate the flow of materials such as soil and water. Rather, they show the cause and effect of one parameter on other parameters including the movement of materials.

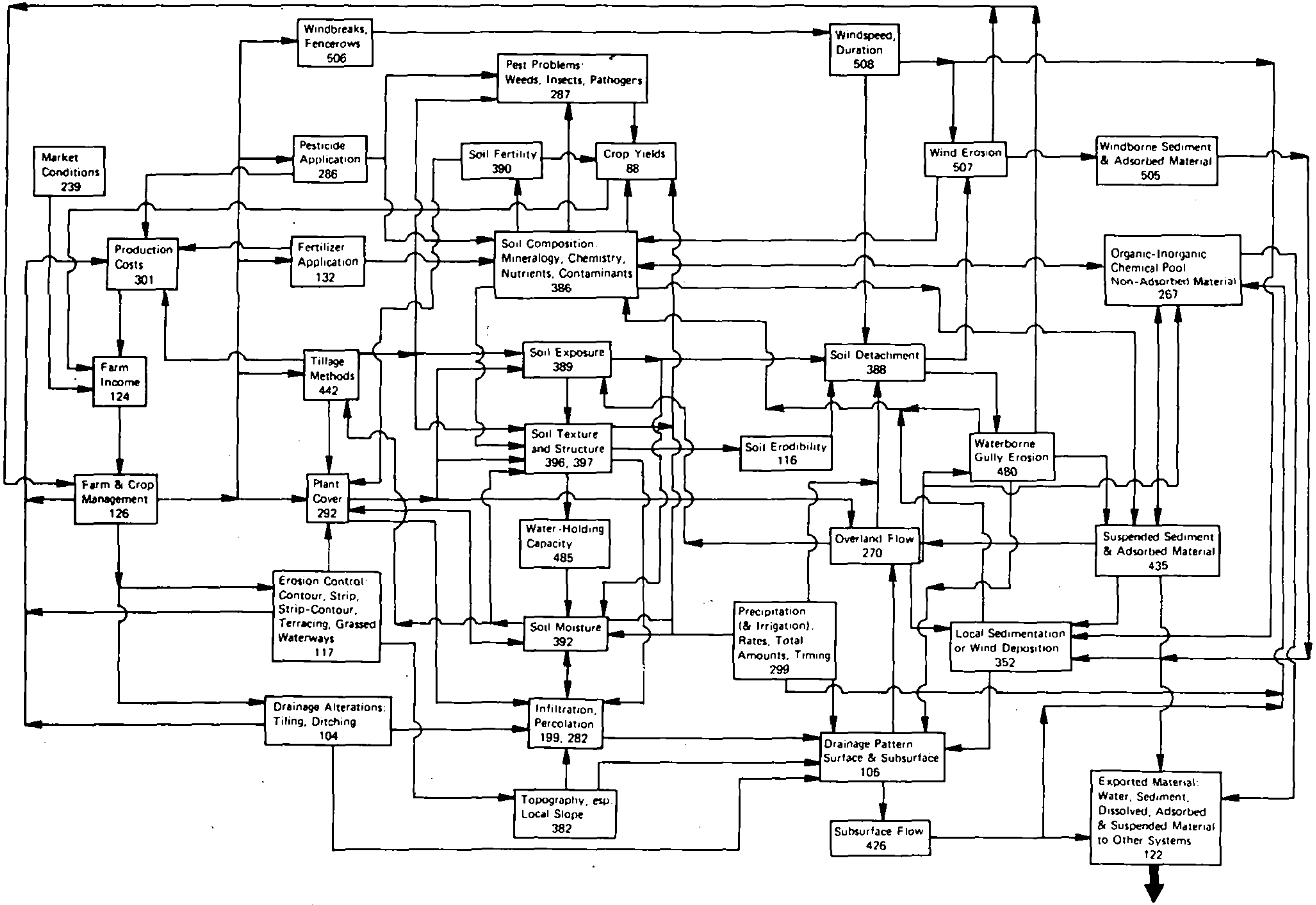

Figure 2. Conceptual Level II model for the Agriculture Subsystem

The interaction of various components can be explained by considering one component of the model at a time. The production costs shown in Box 301 are influenced by many variables including pesticide application (Box 256), fertilizer application (Box 132), tillage methods (Box 442), farm and crop management practices (Box 126), erosion control techniques (Box 117), and drainage alterations (Box 104). Similarly, an analysis of other parameters such as soil fertility (Box 390) can be performed by tracing the arrows to and from this box. The second-, third-, fourth- order interactions and so on can be traced starting with any one of the variables given within a box.

Riverine system

Figure 3 shows the Level II model for the Riverine System. This model encompasses the system which is primarily responsible for the transport and movement of eroded sediments downstream from the source areas. Moreover, transport and movement of sediment becomes visible within this system.

The model can be subdivided into four major categories from left to right. These are: hydraulic and hydrologic controls including modifiers, physical characteristics of the system, factors controlling the movement of water and sediment, and quantity and composition of the materials imported into or exported out of the system.

Most streams and rivers in Illinois are not significantly controlled except for man-made dams, and locks and dams on the three major rivers. Most of the rivers are in a state of dynamic equilibrium where erosion and sedimentation processes are constantly taking place. The dynamic equilibrium is generally altered when an excessive amount of sediment or water is delivered to the system or when man-made changes such as dams, channelization, or diversion take place.

An example of first order interactions between some parameters is as follows. Erosion including bed, bank and mass wasting (Box 120) is impacted by flow geometry (Box 150) and erodibility of bed and bank (Box 116), whereas erosion impacts bed-bank stabilization and bank protection (Box 29,37), channel morphology (Box 63), and bed-bank composition (Box 35). Two-way interactions exist between erosion with bed-bank stability (Box 36), suspended sediment load (Box 435, 434), and bed load (Box 40,41).

An example of second-, third-, and fourth-order interactions is given in Figure 4. Here the economic condition (Box 109) will have an impact on whether or not channelization (Box 56) should be done. However, channelization will impact flow geometry (Box 150), which in turn will impact sedimentation (Box 352), erosion (Box 120), and riverine fauna (Box 328). Sedimentation will determine if dredging or removal of sediment (Box 107) should be done. On the other hand, erosion (Box 120) will impact channel morphology (Box 63), bed-bank composition (Box 35), and bed-bank stabilization and bank protection (Box 29, 37). Thus starting from economics, following one of the routes through channelization, the importance of economics will be felt on channel morphology and other parameters. In figure 4 only one-way interactions have been shown. However, there are many two-way interactions that could be traced and impacts explained by selecting a specific area within the model diagram shown in figure 3.

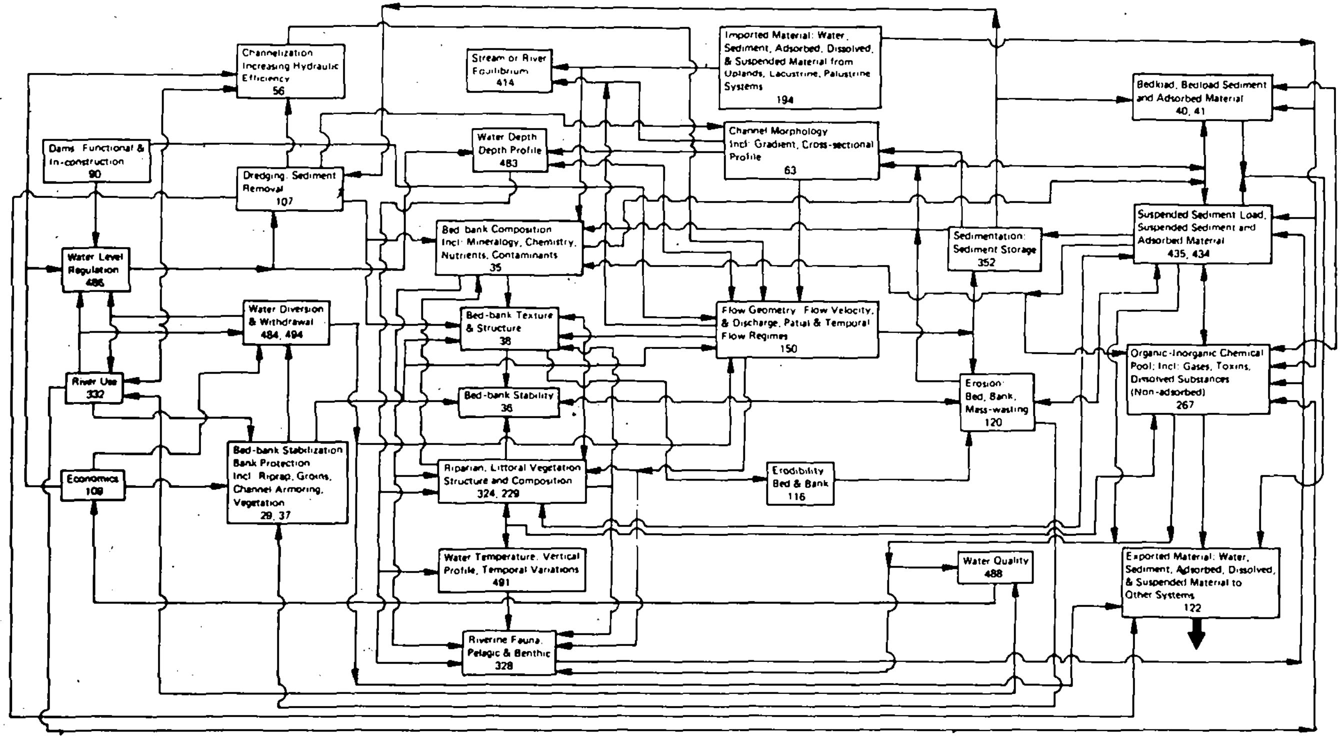

Figure 3. Conceptual Level II model for rivers and streams system

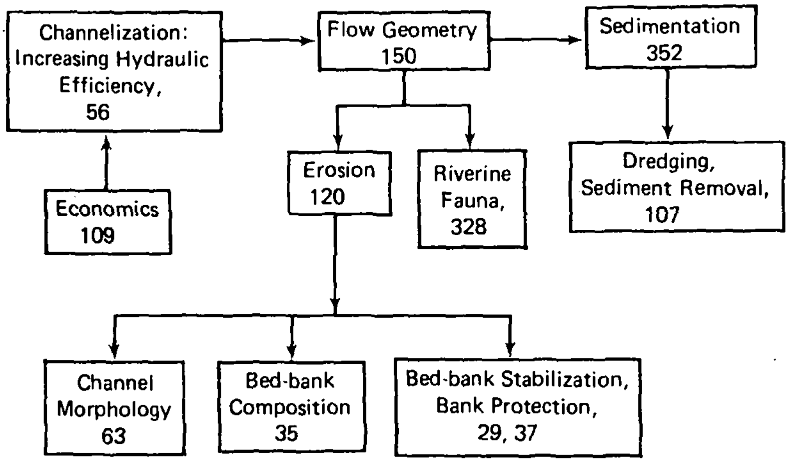

Figure 4. A set of second-, third- and fourth-order interactions for rivers and streams model

REMARKS AND CONCLUSIONS

A brief summary of the various conceptual models developed for the erosion and sedimentation processes within the total environment has been presented in this paper. It should be noted that the detailed description of each of the models, the explanation of the interactions between components within each model, the related references corresponding to each interaction, an identification of the information and data gaps, and a list of 800± references are all given in the original publication by Bhowmik et al. (1984).

Erosion and sedimentation processes are complex in nature and they impact the total environment. An understanding of the interactive nature of these processes can best be analyzed through conceptual models. Such models enable the reader to identify important components and then follow through these components to arrive at a logical conclusion for the relative significance of one parameter compared to the other parameters. Even though only two of the models have been described briefly in this paper, they show the complex nature of the models that have been developed.

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SEDIMENT MOVEMENT IN NATURAL RIVERS IN ILLINOIS, U.S.A.

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ABSTRACT

The hydraulics of flow in a natural stream and its sediment transport characteristics are the two basic phenomena that determine its geometric and planform shape. There are many variables that affect the hydraulics of flow and the nature of sediment transport in a river. The materials through which a river flows, the characteristics of the watershed, the rainfall-runoff pattern from the basin, the constraints imposed by humans, and the geology of the watershed are some of the factors that determine the hydraulic and sediment transport characteristics of the river.

This paper presents a comprehensive analysis of the suspended sediment data collected within the State of Illinois. Analyses of the data have shown that most of the yearly sediment load moved in a short period of time with storm events, and simple regression relationships between water discharge and sediment load can be developed for prediction purposes. It has also been shown that generalized analyses of the yearly sediment load in streams can be utilized to delineate the region or areas of the state or of a nation where instream sediment load may be excessive. It has been observed that some correlations do exist between geomorphological characteristics of the basin and the average annual sediment load.

Keywords: Sediment transport, suspended load, streams, Illinois, climate, regression equations, discharge.

INTRODUCTION

Most of the major rivers of the world flow through alluvial materials consisting mainly of sand and silt. The flow of water in these alluvial channels has been studied by various researchers for many years. In a sand bed channel, the flow velocity, the turbulence associated with the flow velocity, and the patterns of the secondary circulation all have the capability and the opportunity to mold the shape of the channel. Researchers have tried to express the characteristics of flow in alluvial channels in terms of theoretical relationships. Some of their attempts have been successful, whereas others have met with failure. The flow in a natural channel, however, is obviously affected by so many variables that a clear and straightforward analysis is not possible unless one resorts to some acceptable simplifications and assumptions.

As a result of all the constraints in an alluvial channel, a velocity distribution with both lateral and vertical components is developed. These velocity components vary in time and space. The longitudinal water surface slope, or the hydraulic gradient, also constantly adjusts to reflect the constraints of the channel geometry on the flow in a natural channel. This variability of the water surface profile is more pronounced for flow around a bend than it is for a straight reach of the river.

Resistance to flow in an alluvial channel is a function of many variables, which in turn determine the bed form in an alluvial channel flowing on a sand bed. Simons and Richardson (1971) have classified the bed form in two categories: "low flow regime" and "upper flow regime." Most stable alluvial sand bed channels generally flow on a bed where the bed forms are either dunes or dunes with superimposed ripples. Many times bed forms change from one cross section to the next and even within the same cross section. Even though the turbulent flow in a rigid boundary channel is mostly independent of viscous drag, on a sand bed channel this may not be true. With fine sediment in suspension the flow in a sand bed channel during turbulent flow is affected by a change in viscosity with a change in temperature or a change in the concentration of suspended sediment load.

Motion of the bed materials begins when the hydrodynamic forces exerted on the individual particles are large enough to dislodge the particles from the bed. There are three modes of transport: 1) translation, 2) lifting, and 3) rotation. Incipient motion of bed particles has been analyzed either theoretically or on a semi-empirical basis by many researchers including Shield, as cited by Simons and Senturk (1977).

For the purpose of analysis, the total sediment load is often split into two parts: bed load and suspended load. Bed load is defined as that sediment in the bed layer moved by saltation (jumping), rolling, or sliding. The bed layer is a flow layer several grain diameters thick immediately above the bed. Its thickness is usually taken as 2 grain diameters. Suspended load is defined as that sediment load which is moved by upward components of turbulent currents and which stays in suspension for a considerable time.

When attempting to determine the suspended load one must remember that only the suspended load due to bed material can be calculated from available equations. Wash load is determined by available upslope supply rate.

The total load can be obtained from the sum of the bed load and suspended load. If the hydraulic and suspended sediment load data are available, the total suspended sediment load can be computed. In many instances, especially in the case of streams flowing on sandy beds, it is easy to measure the suspended sediment load. However, present instrumentations are not yet well enough developed to measure the bed load. For cases such as these, an empirical relationship is needed to determine the total load based on the hydraulic data and the measured suspended sediment load. Simons and Senturk (1977) have indicated that for a large and deep river, the amount of bed load may be about 5 to 25% of the suspended load. Bed load on the Kankakee River was found by Bhowmik et al. (1980) to be small. Although total bed load may be small in these rivers, it is nevertheless important since bed load influences the bed stability and determines the bed and grain roughness of the channel.

DATA COLLECTION AND ANALYSES

Data from 22 to 50 stations around the state of Illinois were collected over the last 3 years. All the suspended sediment data were collected following the procedures outlined by Guy and Norman (1970). Laboratory analyses followed the technique given by Guy (1969). These are essentially the same techniques used by the U.S. Geological Survey.

For any generalized analyses, in-stream sediment data for a period of 5 to 10 years are needed. However, when data for such a period of time are not available, some standard analyses can be performed to determine the trends in sediment load within a river basin. The data collected and analyzed by the Water Survey's sediment network consist of the instantaneous water discharges and the corresponding sediment load. Through use of these instantaneous values of suspended sediment load, Qs, and water discharge, Qw, regression equations for each station were developed. Figure 1 shows such a plot for the Mazon River near Coal City, where data for the 1981 water year (October 1980 through September 1981) have been plotted. Based on the regression equations similar to the one shown in Figure 1, and the gaged daily flows, daily sediment loads for each station were calculated. These daily sediment loads were then utilized to determine yearly sediment loads for each station.

Additional data collected by the U.S. Geological Survey were utilized. These data represent average daily mean values of sediment discharge and the daily mean flows. Figure 2 shows a relationship between the daily Qs and Qw with time for the 1979 water year for the Wilmington gaging station on the Kankakee River in Illinois (Bhowmik et al., 1980). It is obvious from this plot that at some time during the year there exists a good correlation between the water discharge Qw and the sediment load Qs. However, about 165 days after October 1, 1978, the sediment load was quite low even though the water discharge remained fairly high. Thus an exact correlation between discharge and sediment load did not exist throughout the whole year.

The correlation between the daily or instantaneous water discharges and the sediment load varied from station to station and year to year. In general, sediment load increased with increasing water flow (figure 1), but on a one-to-one basis, this was not always true (figure 2).

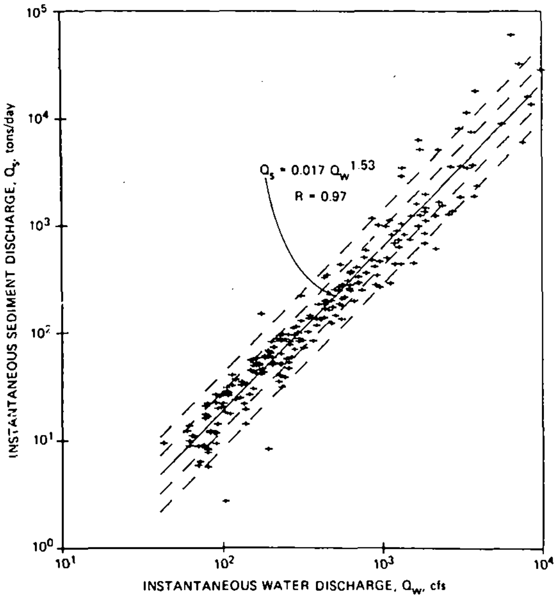


Figure 1. Sediment transport curve for the Mazon River near Coal City, Illinois, Water Year 1981

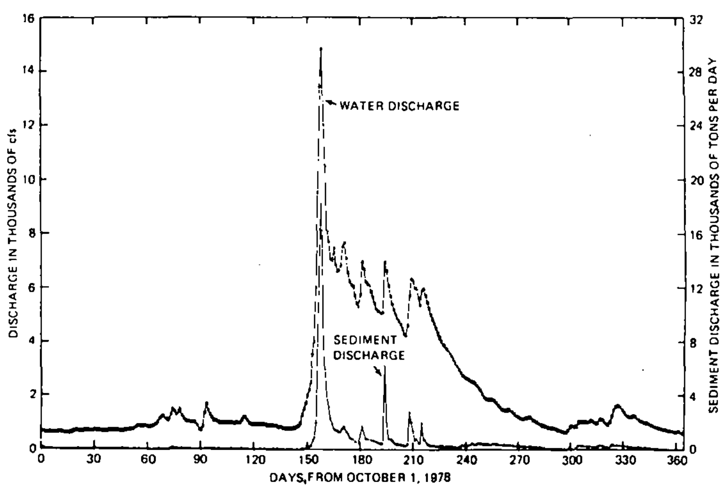


Figure 2. Relationship between the water discharge and sediment load with time in days for the Kankakee River at Momence, Illinois

ANNUAL SEDIMENT LOAD

Analyses were performed to determine the correlation between the yearly sediment yield and the total yearly discharges. Data from three river basins in Illinois (Spoon, Sangamon, and Kankakee) and from a major tributary of the Kankakee River (the Iroquois River) have been analyzed and are presented in figure 3. An examination of the figure indicates that excellent correlations do exist between the total yearly sediment load and the total annual discharges. As a matter of fact, the correlation coefficient R varied from 0.97 to 0.99. Considering the extreme variabilities in sediment yields for natural rivers, these variations should be considered quite satisfactory.

A comparison between the river basins shown in figure 3 indicates that for sediment yield in the Spoon River basin is the highest. All three river basins are located within the glaciated part of Illinois. However, the Spoon River is located within the Galesburg Plain (Leighton et al., 1948), which contains soils that are easily erodible. The Sangamon River basin is situated within the Springfield plain which oontains glacial deposits but where soils are not highly erodible. On the other hand, the Kankakee River flows on an alluvial valley containing mostly sand-sized particles on its beds, resulting in a relatively small suspended sediment load in this river basin compared to the others. Thus relatively high percentages of fine soils present on the Spoon and Sangamon River basins contributed to the higher suspended sediment loads on these river basins. It is apparent that the physiographic factors played an important role in the determination of the sediment yield from each one of the basins.

The relationships shown in figure 3 are extremely important and can be utilized to determine the yearly sediment yield at any location on the basin from known water discharge, Qw. Research conducted at the Water Survey (Bhowmik et al., 1980) has shown that a good relationship exists between the water discharge Qw and drainage area D_A. Therefore, regression equations can be developed between Qs and Qw, once the regression equation between Qs and Qw and Qw and D_A are known. It is much easier to determine the drainage area D_A within a basin. With known D_A, the total yearly sediment load Qs can easily be computed.

CUMULATED SEDIMENT LOAD

An examination of figure 1 will indicate that the bulk of the suspended sediment moved during storm events. Since the number of storm events in a year is small and the durations of the storm events are generally short, the bulk of the suspended sediment moves past a station during a relatively small number of days during the year. This is illustrated in figure 4 for the Chebanse Station on the Kankakee River (Demissie et al., 1983). In this figure, the percentage of the annual suspended sediment moving past the gaging station in a given number of days is shown. The three curves in the figure represent the three water years, as indicated.

From figure 4 and the computations performed, it became apparent that at the Chebanse station, 50% of the total sediment load in 1980 moved past the station in only 4 days. For 80% of the total sediment load, the corresponding

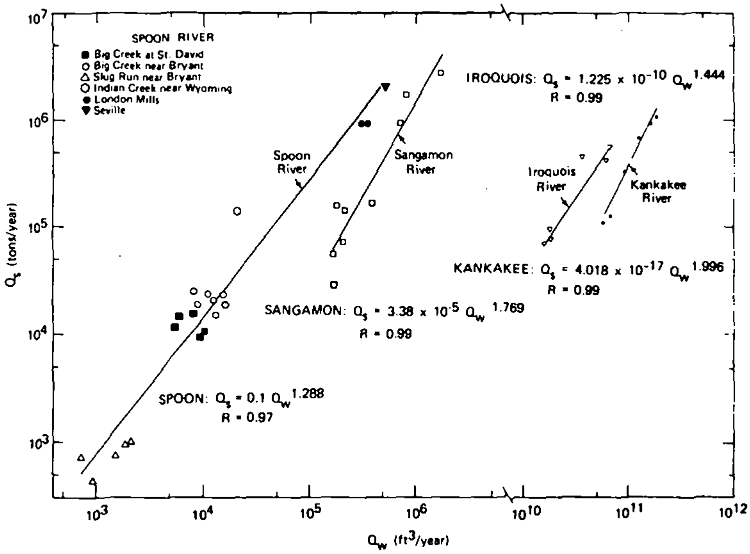


Figure 3. Relationship between annual sediment load and annual discharge for the Spoon, Sangamon, Iroquois and Kankakee Rivers in Illinois

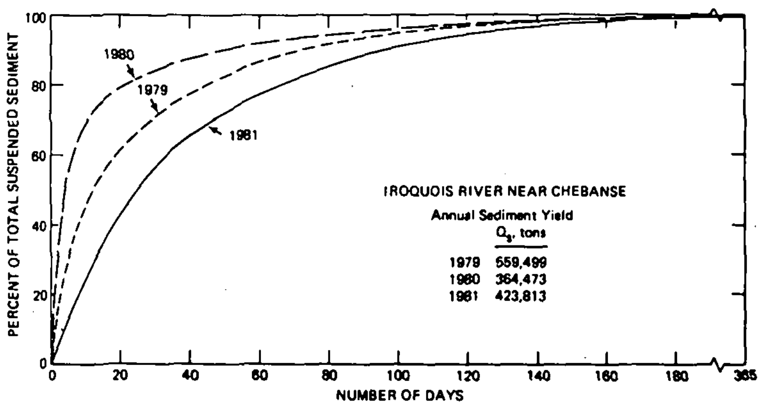


Figure 4. Annual suspended sediment load carried in a given number of days, Iroquois River near Chebanse, Illinois

number of days in 1980 was 21. It is obvious, therefore, that most of the suspended sediment load is transported during the storm events which take place in a relatively short period of time during the year.

Another important observation from figure 4 is the variation of the cumulative sediment transport curve from year to year. The curve for Water Year 1980 has a steeper slope in the initial zone of the cumulative curve than the 1979 water year curve, which in turn has a steeper slope than the 1981 water year curve. The 1980 water year was the driest year among the three years considered, and during this year the highest percentage of the suspended sediment was transported in the shortest period of time. The 1981 water year was slightly wetter than the 1979 water year, and during this year, it took more time for the same percentage of suspended sediment to pass the station than in 1980 or 1979. Thus, there seems to be a trend of a higher percentage of suspended sediment moving in a shorter number of days during drier years than in wetter years.

AVERAGE ANNUAL SEDIMENT YIELD

The sediment load calculated for each station for each year is the total yearly sediment load for that particular station. The amount of sediment generated within a basin from its sub-watersheds will probably not be uniform. Some areas may contribute two or three times more sediment than other areas. However, if it is assumed that the drainage basin above each gaging station contributed uniformly toward the total sediment load, then the sediment load per unit area of the basin can be computed and a statewide comparison of the sediment yield can be made.

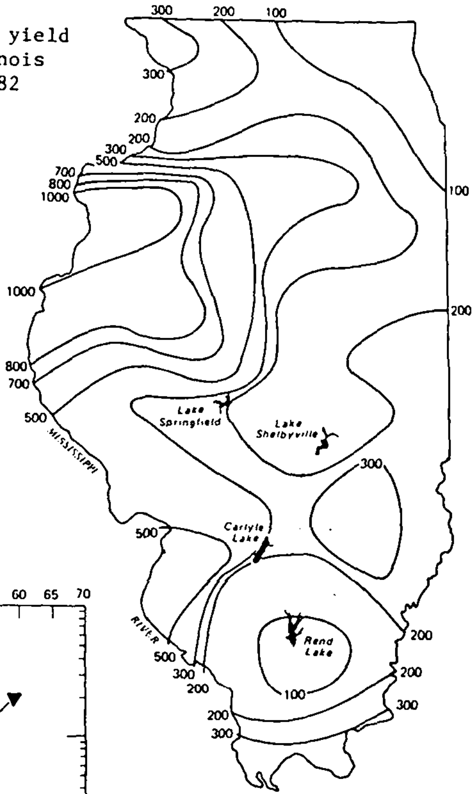
The total calculated and measured annual sediment load from each station was divided by the drainage area above each gaging station, and the average annual suspended sediment yield in tons per square mile was obtained for Water Years 1981 and 1982. These values have been plotted and were utilized to draw lines of equal sediment load, as shown in figure 5.

The isoline plots in figure 5 show a clear and unmistakable trend of heavy sediment loads in certain areas of the state. It is obvious from this map that the west-central part of the state, mostly in the Galesburg and Springfield Plains (Leighton et al., 1948) and the bluff areas of the Mississippi River, contributed the maximum sediment loads in Water Years 1981 and 1982. In the westernmost part of the state, the maximum value was more than 1,100 tons per square mile per year.

It should be pointed out that these isolines were drawn on the basis of the average annual sediment yield per unit drainage area at the measuring station. There are many subwatersheds where the average sediment yield could be different than the values shown in figure 5. However, an analysis such as this affords an opportunity to make generalized comments about the trends in sediment yield within a particular watershed.

An examination of figure 5 shows that there is an increasing gradient of high sediment yield from east to west and south to north. There is a dip in the trend line from south to north in and around Rend Lake. Similar dips in the sediment yield values were also observed at gaging stations downstream of

Figure 5. Average annual sediment yield in tons per square mile for Illinois streams, Water Year 1981 and 1982



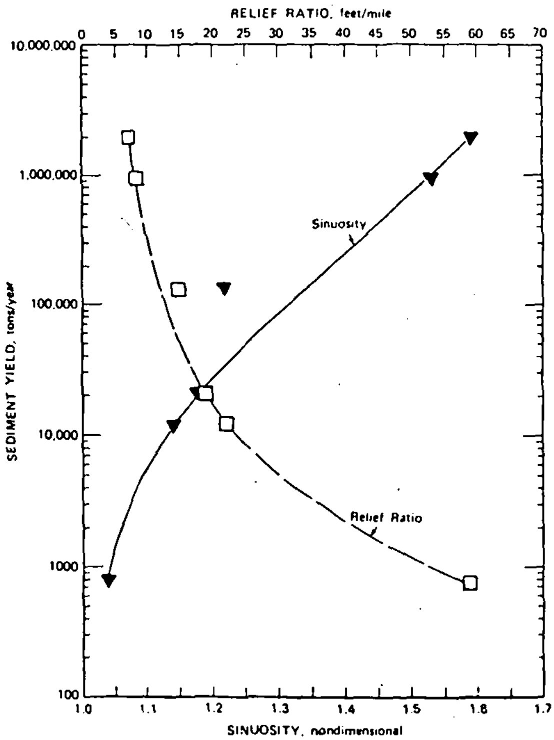


Figure 6. Relationship between average annual sediment yield and sinuosity and relief ratio, Spoon River, Illinois

other large man-made lakes. It is apparent that the lakes by their nature have been trapping a significant amount of sediment load and this in turn is being reflected at the downstream gaging stations. Thus, even though excessive sediment fills up the lakes at a faster rate, still this does help in reducing the sediment load within the downstream reaches of the river.

Most of the trend lines shown in figure 5 probably have been in existence for a long period of time. The implications of this assessment are, of course, many and varied. It appears that the high yield areas shown in figure 5 are the areas where resources should be targeted for reducing the soil erosion. In the process of evaluating watershed protection strategies and priorities, planners and administrators must direct their limited resources to the areas where the greatest potential benefit may be realized.

SEDIMENT YIELD AND BASIN CHARACTERISTICS

An analysis was performed to determine the relative importance of morphological characteristics of the basin and the sediment yield. Parameters such as drainage area, basin shape, basin length, relief ratio, total relief, incision, sinuosity, stream frequency, mean stream length, total stream length, and floodplain area were considered. Analyses between the sediment yield and the parameters mentioned above for the Spoon River Basin indicated that the average annual sediment yield shows an increasing trend with an increase in the values of total stream length, main stream length, total relief, basin length, sinuosity, incision, and floodplain area. However, average annual sediment yield decreased with an increase in the relief ratio, and no significant relationship was observed between sediment yield and either stream frequency or basin shape.

Figure 6 shows the relationships between the average annual sediment yield and both sinuosity and relief ratio for the Spoon River Basin. Here sinuosity is defined as the ratio of the total stream length to the down valley length, and relief ratio is defined as the ratio of the total drop of the stream to the length of the main stream. Obviously, the variations of the sediment yield with increasing values of these two parameter are not the same. Sediment yield increased with an increase in discharge and drainage areas in the downstream direction. Figure 6 also indicates that the sinuosity of the river not only increased in the downstream direction but also correlated well with the sediment yield. Spoon River is a highly incised river with a narrow basin. Analyses performed for the Spoon River Basin are now being extended for other basins in Illinois.

SUMMARY AND CONCLUSIONS

Instream sediment data collected from Illinois streams and rivers have been analyzed and are presented in this paper. Even though sediment data for a 2- to 3-year period are not sufficient to make a generalized statement, nevertheless the analyses have shown that some definite trends do exist and that regional analyses of instream sediment data are quite feasible.

It was observed that regression equations could be developed between daily or instantaneous discharges and suspended sediment load. These regression equations can be utilized to determine the annual sediment loads.

In most alluvial streams, most of the suspended sediment load is transported by the stream during storm events with high flows. In a relatively dry year, about 50% of the yearly sediment load is transported within a period of 20 days or less.

Total annual sediment load correlated fairly well with the total annual discharges. Variations from basin to basin are present and reflect the physiographic and geologic setting of the river basins. On the basis of the average annual sediment load, isolines of sediment yield within the State of Illinois were developed which identify the regions of the state where excess sediment load is a problem. Analyses such as this are invaluable in targeting resources to reduce excessive soil erosion from the watershed.

Average annual sediment yields were found to correlate positively with various geomorphological parameters of the basin. Additional research is needed to expand these analyses for other river basins.

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AMENAGEMENTS HYDRAULIQUES DANS LES CAS DE CATASTROPHES NATURELLES

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Direction de l'Hydraulique Urbaine
au Ministère de l'Equipement

S U J E T

- Les inondations et leurs causes : Des phénomènes hydrologiques exceptionnels dans un urbanisme en pleine expansion. L'auteur fera notamment des remarques et observations sur le degré de précisions des données hydrologiques.

- Les inondations et leurs conséquences : Perturbation du milieu physique, du milieu humain et aussi de la Cité.

- Les inondations et les remèdes : Vu leur soudaineté et leurs conséquences, les solutions sont à plusieurs niveaux : amont (aménagement urbain) et même au niveau social et civique...

- Le calcul du dimensionnement des ouvrages : Les méthodes utilisées couramment, posent certains problèmes : elles sont expérimentales et en général, adpatées à la région et au pays où elles ont été mises au point. Elles demandent aussi une certaine connaissance de l'hydrologie de la région et aussi beaucoup de bon sens.

- Commentaire : En dernier lieu, on trouvera un état sommaire récapitulatif des interventions de l'Etat à travers la Tunisie.

1. - Les inondations et leurs causes

Les causes des inondations sont d'abord des phénomènes naturels exceptionnels ; ainsi à Sfax, en 1982, 70 % de la pluie annuelle est tombée en moins de six heures...

L'oued Zéroud a vu passer en 1969, 17000 m^3/s à une vitesse de 11 m/s dans le site de Sidi Saâd.

a) L'hydrologie

Nous parlerons donc au début, de l'hydrologie... Cette science basée sur des mesures parfois ingrates et sous nos latitudes fort onéreuses. Entretenir une équipe de jaugeage pendant de longues années, représente une charge lourde avec le risque de louper les crues, lesquelles sont très soudaines ; parfois les équipements n'ont pas le temps de réponse adéquat.

L'hydrologie en Tunisie est actuellement assez développée, le stock d'informations disponibles est assez conséquent, notamment pour la pluie et ses intensités. Pour les mesures de débit, l'un des problèmes est que plus le bassin est petit, plus il est nécessaire d'avoir une série de mesures plus longue, afin d'en approcher les débits d'une manière plus cohérente. Or, ce sont ces bassins qui concernent la protection contre les inondations.

Ce problème est encore plus complexe en site urbain, où le terrain est régulièrement remanié par des travaux de terrassement.

La méconnaissance de l'hydrologie des petits bassins et des urbains, constitue un handicap sérieux pour le projeteur qui se réfugie dans l'utilisation de "modèles" le plus ancien (1851) est l'aquation Q = C I A qui a été "reprise" par Caquot où les différents paramètres ont été affectés d'exposants.

A ma connaissance, en Tunisie par exemple, il y a eu des essais et mesures en site urbain, mais qui n'ont pas été sanctionné par des résultats tangibles, exploitables par le projeteur. Car le passage de l'hydrologie "hydrologienne" à l'hydrologie pratique, n'est pas assurée et un important effort est à faire, pour aider le projeteur à exploiter le stock de données disponibles en hydrologie.

J'ai personnellement exploité 60 années d'hyétogrammes et actualisé les équations intensités - durées de la ville de Tunis, qui se sont avérées fort utiles, lors de l'établissement du plan directeur d'assainissement de Tunis.

Il serait utile de disposer du minimum d'informations suivantes :

- Les débits spécifiques par taille de bassin, notamment les petits bassins (de 1 à 10 Km2).
- La forme si possible des hydrogrammes de crues et la liaison entre temps de montée et temps de concentration, afin de dimensionner plus correctement les bassins d'amortissement.

On prend couramment un hydrogramme isocèle avec un temps de montée égal au temps de concentration.
Il reste aussi le problème du temps de concentration dont les formulations sont aussi nombreuses que variées et donnant des résultats très disparates. Nous avons dénombré une quarantaine de formules des temps de concentration sous différentes latitudes...

b) L'urbanisation

Au delà des phénomènes hydrologiques exceptionnels, l'urbanisation est une cause d'inondation.

Sous nos latitudes, où les villes voient leurs populations urbaines évoluer, et la superficie urbaine augmenter rapidement, il y a un phénomène d'urbanisation centrifuge lié aux renchérissements des terres, donc affectation presque implicite à des couches sociales défavorisées des bas-fonds et terres inondables...

Les terrains qui ne risquent rien, sont surévalués par rapport aux terrains réputés inondables, et les terrains inondables qui ne le sont plus, voient aussi leurs prix monter et il y a un déplacement de couches sociales défavorisées que n'arrête pas le fond du thalweg (nous avons vu des maisons construites en travers du lit d'oued).

La DAT (Direction d'Aménagement du Territoire) essaie d'établir des cartes de risques, mais ce travail de longue haleine est difficile à mener à terme, vu le manque de données et aussi la vitesse avec laquelle les villes évoluent. Une zone étudiée d'un certain risque, est urbanisée avant même que cette constatation ne soit une forme technique et juridique.

Il serait peut être utile que nos aménageurs tiennent compte de l'expérience du passé : rarement les vieilles villes sont inondables ; les ruines romaines, sauf cas exceptionnel, sont toujours hors d'eau et le flot d'inondation qui a envahi en 1982 la ville de Sfax, s'est arrêté religieusement aux portes de la Médina (vieille ville) laquelle a été épargnée par l'eau.

On peut donc déjà prêcher cette évidence : l'eau n'inonde que les zones inondables...

2. - Les inondations et leurs conséquences

Plusieurs types d'inondations ont différents types de conséquence :
- l'inondation bénie des Pharaons, qui apporte joie et bonheur aux populations.
- l'inondation catastrophique qui apporte mort et désolation ; en 1969, 60 $\overline{M}$ \$ de dégats ; en 1982 plusieurs morts à Sfax et Annaba.

Si le premier type d'inondation est en voie de disparaitre, vu la rationalisation des équipements hydrauliques où l'on cherche à créer un réseau de transport et de distribution des eaux jusqu'à l'utilisateur, malheureusement, les inondations catas-

trophiques ou telles, semblent devenir plus courantes ; ceci est à confirmer :
Est-ce la même pluie tombant dans les mêmes conditions climatologiques, cause plus de dégats aujourd'hui que dans le passé?
Cette question reste posée et on peut affirmer que si on prend le cas de la Tunisie, sur les 160 communes qu'elle compte, une centaine au moins a des problèmes d'inondation, et dont les conséquences ne sont pas négligeables.

- dégats directs : coupures de route, maisons démolies, infrastructures touchées, pertes humaines.
- dégats indirects : perte de temps, retard dans les circuits économiques et remise en cause de certains projets...

Nous avons constaté dans certaines villes, une psychose de l'inondation, qui rejaillit même sur le fonctionnement de l'Administration où déjà, le moindre nuage gris est accueilli avec appréhension...

Une autre conséquence plus indirecte, c'est que le fait d'être exposé aux inondations, favorise le rapprochement et la coordination des différents services administratifs et techniques travaillant en général séparément.

Même le fait de protéger les villes, entraine une conséquence non négligeable. La création d'endiguements autour d'une ville entraine une survaleur des terrains protégés, donc encore une fois, des populations en exode rurale, viennent s'installer derrière la zone protégée et il faut recommencer le travail...
Au point de vue équilibre hydrique et même si les spécialistes estiment que les inondations ont aussi l'inconvénient de déplacer d'énormes quantités de sol et favorisent la paupérisation des sols agricoles et peuvent boucher le réseau d'assainissement (transport solide). Par exemple, pour une ville dominée par un bassin de 1 Km2 (100 ha) et que le transport solide est de 1 mm, les apport solides sont de 1000 m3 donc nécessitent 1000 journées de travail, pour enlever ces apports (1 journée d'ouvrier = 1 m3). Si un emploi coûte 5 $ 1 mm/Km2 revient à l'entretien à 5000 $...

3. - <u>Les inondations et les remèdes</u>

L'inondation coûte cher et les remèdes aussi. Les remèdes classiques qui consistent à calculer des ouvrages dits de protection, offrent toujours l'inconvénient d'avoir été dimensionnés sur des bases aléatoires... hydrologie inconnue, formules inadéquates, positionnement sur le terrain parfois incertain.
Ainsi par exemple, la crue millénaire de l'oued Zéroud est passée de 8000 m3/s avant 1969, à 30000 m3/s après 1969.

Que de digues emportées, que de canaux ensablés, que de bassins érodés, que d'oueds coulent à côté de dalots en béton... L'analyse de l'état de l'ensemble de ces ouvrages montre qu'à ce jour, il n'y a pas encore de remède systèmatique à ces problèmes.

C'est dans cet esprit que nous proposons les démarches suivantes :

- <u>Traitement amont</u>

Une inondation est causée par un écoulement généré par une pluie sur un bassin versant : dans certains cas (centre de la Tunisie), le bassin versant est difficile à délimiter; les lignes de crêtes sont si peu présises que certains écoulements se déplacent au gré des lames d'eau et des oueds sont "captés" par d'autres oueds.
Ce phénomène a été observé en 1969, dans la région de Sfax.

Donc, la première tâche - parfois délicate - consiste à délimiter le bassin versant qui génère les écoulements...Une fois ce bassin versant délimité, il s'agit de rechercher déjà des moyens de réduire ces apports; les techniques de traitement de bassin versant en font leur preuves et aussi sauvegardent l'environnement.

Le principal inconvénient de ces traitements, est *<u>le facteur temps</u>*, car un traitement même s'il se fait avec des moyens intensifs, a une vitesse de réaction limitée : un arbre met plusieurs années à pousser et à jouer son rôle efficacement. Dans ce domaine, les erreurs sont fatales : par exemple, sous nos latitudes, l'eucalyptus pousse vite, mais est fragile et peut être décimé par le philocondra.

Au niveau des bassins versants, un repérage systématique des sites de retenues et d'amortissement, est utile et il serait nécessaire pour les petits bassins de mettre au point, un guide de l'amortisseur de crues de capacité inférieure à 1 million de mètres cubes.
Ces ouvrages rencontrent parfois des réticences de la part de l'administration, surtout s'ils sont installés à l'amont des villes.

D'ailleurs, sauf cas particulier, leur conception générale entre dans le cadre de plans directeurs et obéit à différents critères techniques, financiers et politiques.

Donc essayer tout d'abord, dans la mesure du possible de freiner la dégradation des bassins versants amont; nous avons calculé que la remise en état d'un bassin versant peut réduire les apports de plus de 50 %.

- <u>Les remèdes avals</u>

Les remèdes avals partent tous du même principe, c'est à dire, évacuer le flot d'eau le plus rapidement possible, sans qu'il y ait la possibilité d'attaquer le site urbain; soit on dévie l'écoulement, soit on le canalise, ou bien on endique la zone à protéger...

Ce traitement appelle les remarques suivantes :

- Calcul des débits d'équipements : en général, on cherche à évacuer les débits de fréquence centenale.

Dans les équations utilisées en Tunisie, la fréquence T intervient à la puissance 1/5 environ : la crue centenale est de 50 % plus élevée que la crue décennale.

- Le calcul des ouvrages de transport (canaux, dalots...) utilise les formules classiques (Strickler...). Ces formules considèrent le régime comme permanent, ce qui n'est pas toujours le cas.

  Ainsi, si l'on garantit l'écoulement de pointe, les faibles écoulements se font à des faibles vitesses entrainant d'importants dépots qui réduisent la section, donc l'efficacité de l'ouvrage...

- Dans le choix de la section, intervient le revêtement : un canal revêtu coûte cher, mais son emprise est de 30 à 50 % plus faible qu'un canal en terre, qui aura aussi tendance à se creuser ou se remblayer, vu que le cours d'eau recherche son profil d'équilibre.

- Par ailleurs, dans le dimensionnement des ouvrages, nous avons constaté par un calcul d'erreur, que pour les conduites, la connaissance du débit à 50 % près, ne faisait que changer la gamme commerciale du diamètre.

- Ces ouvrages fonctionnent 50 à 100 heures par an (coefficient d'utilisation : 10 % par an), et sont exposés pendant le reste du temps : érosion des bajoyers, dépots d'ordures.
  Il serait utile de les dimensionner en fonction des moyens mis en oeuvre, pour les entretenir ; par exemple, un dalot enterré, doit avoir comme dimension minimale 2 X 2 pour permettre à un ouvrier, d'y pénétrer à l'aise.

Donc :

La partie purement hydraulique du dimensionnement de ces ouvrages de protection, est relativement secondaire par rapport au choix des tracés, à la nature des matériaux mis en oeuvre (revêtement) et aux contraintes d'entretien.

Le problème de choix de tracé, nous amène à parler d'urbanisme. Nos villes s'étendent à une vitesse vertigineuse ; les plans d'aménagement deviennent rapidement caduques, et parmi les problèmes que l'on rencontre, il y a l'inondabilité ; en général, comme l'histoire de la toile de Pénélope, le projeteur protège, puis l'urbanisme dépasse le site protégé et (ou) reprotégé...

La résolution de ces problèmes entre dans un cadre politique et socio-économique ; on ne peut qu'émettre des directives générales.

* Plus que des plans d'aménagement détaillés, définir d'abord des idées directrices d'affectation puis, mettre au point les plans détaillés au fur et à mesure de la viabilisation et la construction.

* Définir des côte-seuils minimum : de nombreux projets d'habitat et touristiques ont été endommagés par l'eau pour avoir été calés 50 cm trop bas.

* Pour la viabilisation, les routes font parfois barrage aux oueds et leur calage est lié à la côte seuil des bâtiments.

Il reste le problème des communes rurales : Ces dernières sont de plusieurs types :

- les localités *"anciennes"* qui sont plus ou moins bien situées et peu exposées ;
- les points de peuplements de la colonisation qui ont évolué ces communes sont mal implantées et donc exposées aux écoulements et posent parfois des problèmes insolubles...

En conclusion, la protection contre les inondations est liée à l'évolution des villes, au problème de l'urbanisme et de son organisation, constitue une importante contrainte dans cette protection.

Commentaire sur les villes à protéger en Tunisie

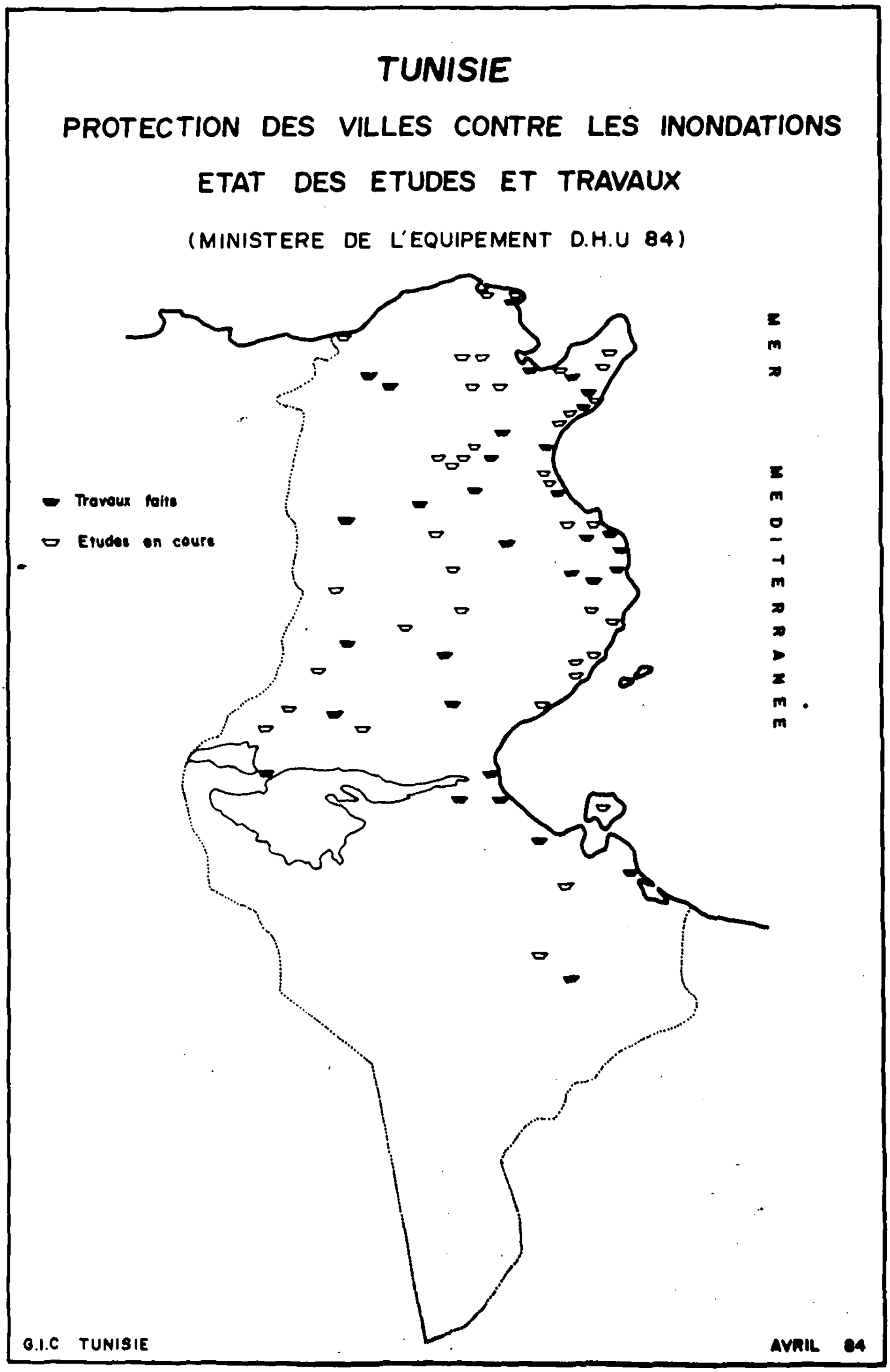
Comme on peut le voir sur la carte de la Tunisie, ci-jointe, il y a environ 80 centres urbains qui ont des problèmes d'inondation.

Dans un pays où l'hydrologie est fort capricieuse (1400 mm au Nord, moins de 200 mm au Sud, des débits spécifiques atteignant jusqu'à 40 m3/s/Km2...), il n'y a pas de région moins exposée qu'une autre, aux inondations. L'ampleur des dégats est liée aussi bien à la taille de la ville, qu'à la situation par rapport aux bassins versants dominants.

Au point de vue investissement, l'Etat dépense deux millions de dollars par an, répartis comme suit :

- 50 % réservés à la protection rapprochée des villes ;
- 30 % réservés à l'aménagement des bassins versants ;
- 20 % réservés à la protection des voies de communication.

Il serait utile de développer les investissements des deux derniers types d'aménagement, mais là aussi, l'intervention sur les bassins versants et sur les voies de communication entre dans un cadre plus général de l'équipement du pays.


TUNISIE
PROTECTION DES VILLES CONTRE LES INONDATIONS
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THE «PULSA» WATER OSCILLATION SYSTEM FOR HAND PUMPING

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ABSTRACT

The development of the "Pulsa" system for water oscillation hand pumps for deep wells represents a major technological breakthrough which is expected to have a profound influence on the International Drinking Water and Sanitation Decade.

The water oscillation system of pumping presented allows for extreme simplicity of pump construction, the total elimination of all organs of transmission and parts in relative movement below ground level, extreme facility of installation and maintenance, particularly wide possibilities of application, maximum security of water supply including the use of multiple independent unit installations and excellent resistance to sand, adaptability to local social and cultural structures, universal use especially by children, low global long term costs of operation, village level operation and maintenance, and real prospectives for local manufacture of pumps and/or spare parts.

Keywords : water oscillation pumps, hand pumps, multiple independent unit pumps, oscillating water columns, multiple pump installations, village water supply, plastic feed pipe, maximum pump security, parallel technologies principle, progressive investment principle, multiple re-utilisation of parts principle, self-compensating parts, village level operation and maintenance, use by children, local manufacture, standardisation of stocks and parts, resistance to sand, universal application.

INTRODUCTION

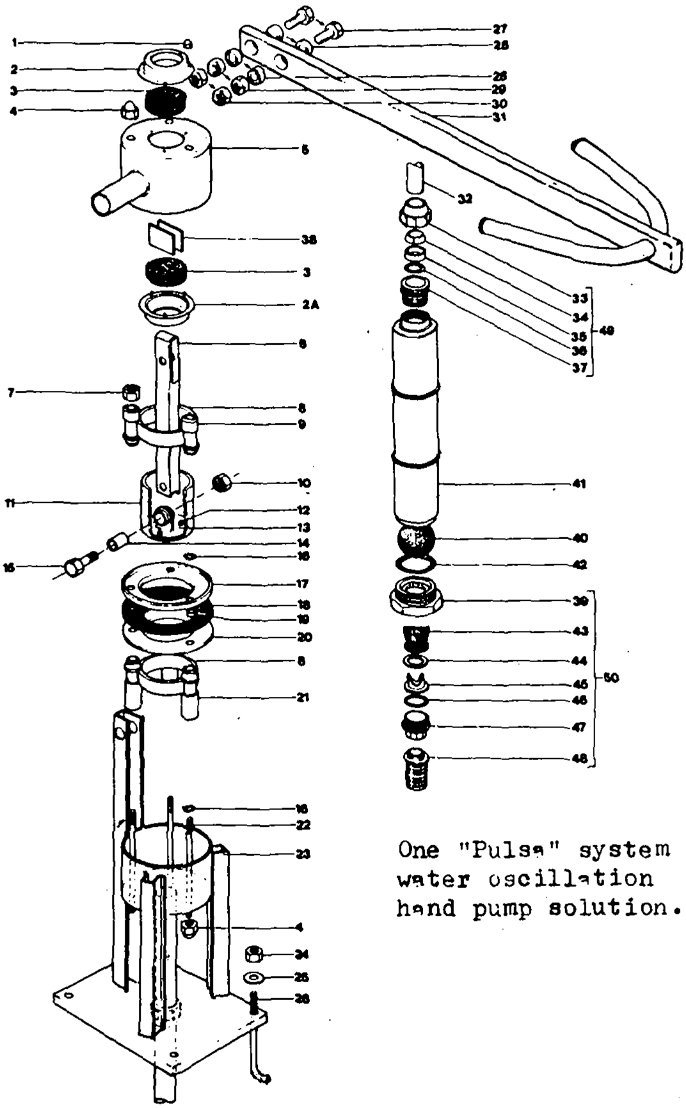
The present paper describes the philosophy behind the development of the "Pulsa" water oscillation pump system, the basic working principles of the said system, the nature of the technical solutions adopted, some consequences of said adopted solutions, and some prospectives resulting from the development.

THE PHILOSOPHY BEHIND THE DEVELOPMENT OF THE "PULSA" WATER OSCILLATION PUMP SYSTEM

The pumping of water from great depths cannot be considered a natural human activity. People, like animals, generally looked for surface water and gradually developed methods of capturing, storing, and moving surface and rain waters. The need to sink deep wells and drill bore-holes to find drinking water for communities is therefore relatively recent in human history. The political, social, and economic reasons for the increasing dependence of peoples in many areas of the world on ever deeper below ground water sources do not fall within the scope of this paper. What must be stressed, however, is that traditional systems of hand pumping were originally developed for purposes other than systematic community water supply from deep wells. Hence the problems associated up to now with deep well hand pumps.

Most modern research has been directed to rationalising traditional systems by proposing, for instance, flexible cables or light weight below-ground parts. Progress has of course been made, but the pumping systems involved have known inherent defects and limitations rendering them essentially unsuitable for community water supply. Solution of one problem has often aggravated or even caused others. The world was thus desperately in need of an alternative system which did not have such limitations, which would ensure security of water supply with a long global duration, which would not break down even where maintenance were needed, which guaranteed ease and economy of maintenance and a potential for local manufacture of pumps and/or parts. The new system would have to be universally applicable, and fit in with the social and cultural conditions of local communities.

A bicycle can be ridden, after a minimum of preparation, by anyone, young or old, sick or tired, at a chosen speed which is determined by variables such as the distance to be ridden,



One "Pulsa" system water oscillation hand pump solution.

the time available, the physical characteristics and the current state of mind of the user. In the same way the new pumping system would have to be susceptible to use by anybody according to his wishes needs and physical possibilities at the moment.

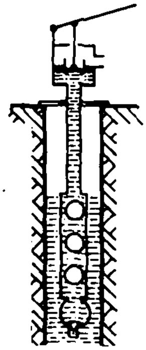
It was with the above objectives in mind that the "Pulsa" water oscillation system of pumping was born.

THE BASIC WORKING PRINCIPLES OF THE PULSA WATER OSCILLATION SYSTEM

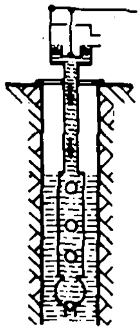
The "Pulsa" water oscillation hand pumps comprise an upper pump body with a plunger actioned by a lever, and a lower cylinder containing a series of special elastic elements closed at the bottom by a no-return valve. The two parts are joined together by a single flexible plastic pipe of suitable diameter and pressure resistance. Once only initial priming of the system is necessary. The handle and upper pump group forms one oscillatory system, and the lower cylinder group together with the water in the column a second oscillatory system. We therefore have two oscillatory systems which are indepedent of one another and which have to be appropriately synchronised.

The handle is moved downwards and upwards more or less as with other types of lever pumps. On the downwards stroke, energy is transmitted from the lever system to the plunger and thence to the mass of water present in the column. The water in turn transfers the energy at origin supplied by the person/s actuating the lever to the special elastic elements in the lower cylinder. These, contracting, store the energy in the form of potential elastic energy. At the lower dead point or close of the downwards stroke, the elastic elements spring back to their original volume, returning the energy stored to the column of water in the feed pipe. The kinetic energy of the water is such that as it tends by inertia to continue its upward movement in the feed pipe, a small vacuum is created at the bottom of the pipe so that a small quantity of water is sucked into the cylinder. The pump is designed in such a way that at the same time the plunger in the upper pump frees itself from its seal, so that an equal quantity of water simply spills out of the pipe at the top, thus concluding the pumping cycle. The actual pumping is thus done by the water itself as it oscillates in the feed pipe.

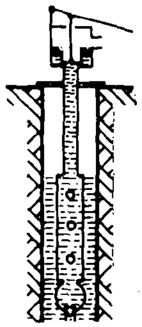
Illustration of the pumping cycle



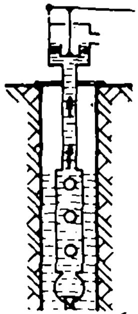
1. Start of cycle
Piston against seal. Water in pipe still. Elements at rest Foot valve closed.



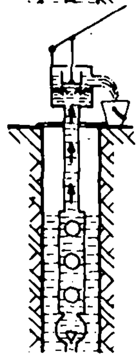
2. Piston descent
Seal blocks water at top. Water pushed down. Elements compressed progressively. Foot valve closed.



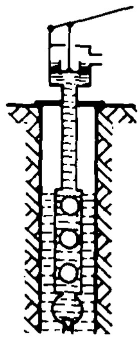
3. Lower dead point
Elements at maximum compression. Water in pipe still. Foot valve closed.



4. Piston return
Piston returns. Pressure gets lower. Elements expand. Energy passes to water. Foot valve still closed.



5. Upper dead point
Piston released from seal. Water continues upward course. Elements take original volume. Foot valve open due vacuum in pipe.



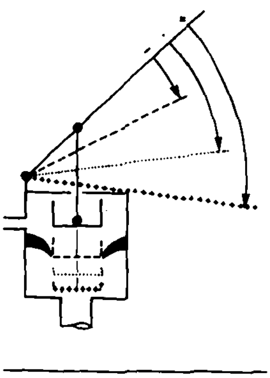
6. End of cycle
Piston returns to rest position in contact with seal. 1. Start of cycle.

THE NATURE OF THE TECHNICAL SOLUTIONS ADOPTED

Variability of stroke length

It is the pump and not the user/s which determines the rhythm of work with a water oscillation pump. The necessary rhythm is easily learned. The length of the stroke, and therefore the amount of force applied, may, however, be varied according to the wishes and/or physical possibilities of the user (as with the bicycle already mentioned).

Illustration of variable stroke lengths



Short, slightly more rapid stroke -----
For one child.
Average stroke ....
For one adult or two children.
Longer, slightly less rapid stroke +++++++
For two adults or a group of children, or maximum capacity for one adult.
Average frequency of pulsation - approximately 65 strokes per minute.

The capacity of the pump will increase with the length of the stroke only for so long as the special elastic elements continue to have elastic compression possibilities. Beyond such point, the surplus energy used is wasted.

The Pulsa system water oscillation pumps have been designed to yield optimum efficiency with shorter lighter strokes, and, generally speaking, efficiency decreases as the handle stroke becomes longer.

Security

It is of fundamental importance to ensure continuity of a healthy drinking water supply in beneficiary communities. This problem has been faced in several ways acting in parallel : careful selection of materials to lessen risk of stoppage; the use of multiple independent unit pumps; village level operation and maintenance; pluralisation of elastic elements.

As at present, up to three separate pumping units can go into a single four inch internal diameter bore hole, while four such units can go down a five inch bore hole. This ensures that should one unit break down the others will continue to supply water. Multiple unit pumps also offer economic advantages deriving from improved exploitation of investments in bore-holes and wells.

Pulsa system pumps will generally continue working even where parts require maintenance. In particular, they will continue working even when several or most of the elastic elements are out of function, and it is for this reason that pluralisation of elements has been deliberately preferred to more economic single elements, which are also more efficient.

Foot valve rubbers and piston seals are self-compensating to wear and tear.

Extremely long overall duration

Water oscillation pumps, if manufactured in appropriate materials (for instance stainless steel) have a practically unlimited life-span and can be brought back to new condition at ten yearly intervals by systematic replacement of parts as provided in the standard maintenance programme mentioned later.

Ease and economy of maintenance

The extreme simplicity and light weight (34kg per unit and 15kg for 50m. of suitable feed pipe) of the Pulsa system pumps enables them to be lifted from the well, opened, if necessary dismantled, and re-assembled in the space of just a few minutes. In particular, the principle of multiple re-utilisation of parts subject to wear and tear has been applied. Thus, the connecting rod guide plaques can be turned upside down and then inverted so as to present a new face to the source of wear and tear, and can therefore each be used four times. The piston guide rings and sleeve groups can be revolved several times (without the need to dismantle the upper pump system) so as to present a new face to the piston. The guide rings can then be inverted, while the sleeve groups can each be used at least six times.

A ten years' estimate of maintenance costs including amortization of all parts with the consequent possibility of bringing the pumps back to new working condition at the close of each ten years' cycle, assuming an average of ten hours' effective use per day at ten litres per minute and thirty litres per day pro capita for two hundred persons per pumping unit, is, at current prices, a maximum of US$ 4,75 or US$ 0,475 per head per annum, excluding the costs of the work and transport of local artisans. These figures are based on European products and costs, and will certainly be lowered with a) technical improvements across the years and b) local manufacture in alternative materials.

Potential for local manufacture of pumps and parts

Pulsa system water oscillation pumps are (with the present exclusion of the special elastic elements) susceptible to manufacture with even the most primitive technologies. In line with basic philosophy, the best available materials should always be used, so as to extend to a maximum the life span of the pumps and limit maintenance operations. The European manufactured Pulsa therefore benefit from the application of the most advanced pump production technologies in the world.

Lower and upper Pulsa system pump bodies can however be made with simple iron piping, while parts subject to wear and tear such as connecting rod plaques and piston guide rings have been deliberately reduced to the most elementary geometric forms so that they can be made in any material whatsoever, for instance, hard wood, at village level. The piston seals and the foot valve rubbers can be made, if necessary, at village level with discarded recovery materials suchas motor vehicle tyre tubes, and locally made leathers.

The only equipment necessary for local quantity manufacture of Pulsa system bodies is a) a lathe, b) a metal cutter c) simple welding equipment and d) a drill for metals. For emergency or village production parts as abovementioned, a single cutting implement such as a knife or a pocket knife is sufficient.

Notwithstanding the professional appearance of the European made versions, the Pulsa system pumps therefore represent probably the only and in any case by far the most advanced application of the principle of village level operation and maintenance in the world, including the most restrictive defintion of VLOM concepts requiring parts subject to wear and tear to be made at village level.

The application of the principle of parallel technologies to enable the manufacture of pumps and/or parts of Pulsa system water oscillation pumps in accordance with the different possibilities offered by the various developing countries is one of the most profoundly innovative aspects of the Pulsa system.

SOME CONSEQUENCES OF THE SOLUTIONS ADOPTED

Wide possibilities of application

Pulsa system water oscillation pumps can be used horizontally and/or vertically with respect to the source of water, and with or without bends or angles in the feed pipe. They can therefore be installed in any bore-hole however shoddily it may have been drilled, in any well, in lakes or rivers, or even used as a mobile unit. It is not necessary for the lower cylinder to be vertical with respect to the water source. As several independent units can comfortably go down the one borehole, it is possible to place one or more of such units at a distance from the bore-hole or well itself, or to pump, for example, on the banks of a river, at a distance from the water. The Pulsa system pumps can therefore be used in any reasonably foreseeable situation, which means that an effective standardisation of pumps and spare parts can accordingly become a reality.

The principle of progressive investment

One characteristic of Pulsa system water oscillation pumps is that they occupy a minimum of below ground level space, with the result that three units can go down a four inch hole and four units down a five inch hole. This facilitates the application of the principle of progressive investment. Where, as is usual, funds are limited, one chooses larger villages and puts a single pump down the bore-hole. From year to year the system can be extended. A medium-sized village receives a bore-hole and installs the single pump from the larger village, which installs a double unit pump. Later on, a small village receives a bore-hole and installs the single pump from the medium-sized village which installs the double pump from the larger village, which intalls a triple pump. This is possible because any of the multiple unit pumps fits onto the same base as its predecessors.

Adaptation to social structures

The Pulsa system pumps have been designed to fit in correctly with social structures and cultural traditions of the communities for which they are destined. The standard handle length, for instance, represents a compromise solution planned for two persons (particularly women) pumping together and for groups of children (who in practice give the best pumping results) as these are the more common combinations for pumping in most communities. The rhythm and the type of musculer work required are adapted for peoples accustomed to rhythmic activities still generally performed collectively in developing countries, such for example, as rowing, separating chaff from grain, spinning, hoeing by hand. The ergonomics have been studied expressly for the users, and not with a "western" mentality with industrialised markets in mind.

The infinite number of parameters available for the utilisation of the pumps makes them ideal for universal use.

SOME PROSPECTS OPENED BY THE DEVELOPMENT OF THE PULSA SYSTEM WATER OSCILLATION HAND PUMPS

With the know-how now available, Pulsa system pumps can in future be dimensioned and built in accordance with specific market requirements. The range of depth applications can be extended to the 0-15m and 45-70m+ areas. Capacities in given depth ranges will be progressively improved. Models to fit down 2" internal diameter holes for private applications can be developed.

The parallel technologies theory will be developed (with the exclusion for the time being of the elastic elements) for interested parties in accordance with the materials available locally.

Further progressive simplification of the Pulsa system and the use of still more resistant materials to extend the principle of multiple re-utilisation of parts subject to wear and tear and consequently to further reduce the need for spare parts and maintenance costs are to be expected.

Research will also be directed to the application of the Pulsa system to alternative energy supplies such as wind and solar energy.

CONCLUSION

The Pulsa system water oscillation pumps together with their consequential developments can be expected to have a profound influence on hand pumping in developing countries up to and beyond the end of this century.

REFERENCES

The development of the Pulsa system of water oscillation hand pumps is in all respects the original work of the authors.

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LA GESTION PATRIMONIALE DES EAUX

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RESUME

L'eau apparaît de plus en plus comme une ressource rare dont la qualité doit être préservée ou améliorée. L'Etat a été conduit par des préoccupations d'intérêt public ou général à intervenir de plus en plus activement dans sa gestion, essentiellement par la voie réglementaire. La complexité des systèmes de relations ainsi mis en jeu a conduit, pour ce faire, à un éclatement de la vision des problèmes de gestion des eaux sous la forme de filières sectorielles dont la coordination est au moins malaisée. Le rôle prééminent de l'Etat a induit parallèlement un transfert de responsabilités qui ne sont plus assumées convenablement par les multiples acteurs directs et réels impliqués dans la gestion sur le terrain. Il s'en suit au mieux une indifférence vis-à-vis des interactions entre parties prenantes, et fréquemment des situations conflictuelles inévitables en pareil cas.

Sans mettre en cause le rôle décisif de la puissance publique, responsable de la politique de l'eau, il apparaît possible de développer une nouvelle approche fondée sur la gestion d'un patrimoine commun par la voie d'un dispositif de concertation fondé sur la diffusion de l'information mutuelle, le développement de processus de négociation impliquant les acteurs réels, individuellement ou par représentation de communautés d'intérêts, la mise en oeuvre subséquente d'engagements contractuels venant se combiner aux dispositions réglementaires qui représentent à la fois dans cette perspective un cadre de référence et une voie de recours. Il peut normalement en résulter une restauration d'une attitude de responsabilité véritablement assumée par les acteurs réels, et à terme un infléchissement positif de leur comportement.

Les dispositions relatives aux grands bassins adoptées en France dès 1964 ont constitué une première démarche sur cette voie, à l'échelle de vastes ensembles hydrographiques. Plus récemment des études approfondies sur l'instauration d'une gestion patrimoniale ont été réalisées à propos de la nappe phréatique d'Alsace, et du bassin de la Sèvre Nantaise pour les eaux superficielles.

AVANT PROPOS -
Les bases d'un diagnostic en matière de gestion des eaux

L'impact des activités humaines sur les milieux aquatiques s'est profondément modifié au cours des 150 dernières années, dans sa nature aussi bien que par l'intensité des phénomènes mis en jeu, tandis que l'identité des acteurs concernés se diversifiait largement en accordant une place de plus en plus large à des systèmes de relations indirects entre l'eau et ses usagers. Les facteurs initiaux de cette évolution sont bien connus dans leur enchaînement causal : maîtrise de l'énergie et de ses transformations ; développement industriel générant une diversification toujours plus étendue des technologies et une croissance rapide de la taille des unités de production ; extension et densification des concentrations urbaines de population exigeant la création et l'adaptation permanente de services collectifs ; intensification de la production agricole sous toutes ses formes.

Les quarante dernières années ont été marquées par une accélération considérable de cet ensemble de processus entre lesquels la croissance économique des pays industrialisés a développé des synergies toujours plus accentuées, et dans le même temps des situations de concurrence vis-à-vis des usages de l'eau. La prééminence de leurs aspects quantitatifs, redevables de solutions techniques relativement simples sinon peu coûteuses, a progressivement cédé la première place à une demande de qualité dont la satisfaction s'est avérée beaucoup plus malaisée. Le rapide développement des activités de loisirs a conféré une dimension nouvelle à cette demande de qualité en étendant un système de relations déjà complexe à de nouveaux partenaires, dont la force et la capacité d'intervention ont pu se manifester très vite avec vigueur, grâce au développement des mouvements associatifs.

La réponse des sociétés centralisées et hiérarchisées a pris la forme d'une intervention croissante de l'Etat fondée sur des moyens législatifs et réglementaires afin de promouvoir l'intérêt public tout en préservant l'intérêt général. La collectivité nationale s'est ainsi érigée progressivement en partenaire privilégié face à la multiplicité des autres acteurs concernés, en s'appuyant sur un système de plus en plus complexe de déclarations, d'autorisations et de sanctions économiques ou pénales.

Ce mode de gestion des eaux s'est avéré efficient aussi longtemps que les aspects quantitatifs ont pu prévaloir tandis que le nombre d'acteurs d'importance significative demeurait limité. L'émergence de formes multiples de la demande de qualité alors que les facteurs de dégradation de celle-ci se multipliaient, et le nombre rapidement croissant des partenaires ont rapidement affaibli l'efficacité de ce dispositif fondé sur la prééminence du rôle de la collectivité nationale, c'est-à-dire de l'Etat.

Les raisons en sont multiples. Par sa nature même, un tel système implique un transfert de responsabilité des nombreux acteurs réels en direction de l'Etat, et donc une attitude ou même une conviction d'absence de responsabilité de ces derniers, qu'il s'agisse de particuliers, de personnes morales ou même de collectivités publiques locales soit territoriales soit fonctionnelles. Tel est l'aboutissement d'un véritable processus de désengagement vis-à-vis de la responsabilité de leurs actes. Ce comportement n'est pas exclusif de vigoureuses réactions de défense si ces mêmes acteurs se tiennent pour lésés par des décisions prises au bénéfice d'autres acteurs. Ainsi est engendrée une autonomie des comportements tous orientés vers une relation unique et directe avec le partenaire privilégié que constitue l'Etat, dans un contexte d'ignorance des besoins et contraintes propres aux autres acteurs directs, et souvent même de totale indifférence à leur égard.

La conséquence inévitable en est l'apparition de situations conflictuelles parfois aigues. La puissance publique, seule à même de prendre en considération les argumentations antagonistes, se verra contrainte d'imposer un arbitrage sous la forme d'un acte administratif. Dans bien des cas le conflit demeurera latent, faute d'avoir pu en maîtriser les causes profondes, et ressurgira un jour ou l'autre sous une nouvelle forme. Ainsi obtiendra-t-on au mieux l'instauration d'un équilibre précaire au sein d'un système de forces antagonistes.

De plus l'Etat, multipliant les décisions contraignantes et les arbitrages imposés, devra se donner en permanence les moyens de les faire prévaloir. La multiplication des actes élémentaires et des partenaires mis en jeu implique une omniprésence de fonctionnaires spécialisés dont la charge pour la collectivité nationale devient rapidement insupportable, au fur et à mesure que se multiplient les besoins d'intervention de leur part. Dès lors l'exercice de cette fonction ne peut plus être que partiel au gré de choix guidés par l'importance des problèmes, les priorités résultant de facteurs externes, l'acuité des conflits qui se déclarent. L'efficacité globale de la gestion des eaux ne peut qu'en être affectée.

Une telle approche paraît du reste mal adaptée à la prise en compte d'une nécessaire cohérence dans la gestion des eaux, dans la mesure où la hiérarchie des priorités induites par les aspects d'intérêt public et les contraintes économiques d'une part, les structures administratives d'autre part conduisent inévitablement à une gestion par filières sectorielles : alimentation en eau des collectivités, assainissement public, équipements producteurs d'énergie secondaire, rejets industriels, gestion piscicole, prévention des inondations en sont des exemples. Les milieux aquatiques étant caractérisés en premier lieu par leurs continuités spatiale et temporelle, une cohérence aussi poussée que possible des interventions par filières est indispensable. Elle se heurte à de multiples difficultés tout en exigeant des procédures de coordination complexes, souvent lentes et qui parfois tendent à transférer certaines situations conflictuelles au sein même des différents services de l'Etat appelés à intervenir. La pratique d'une gestion globale, fortement intégrée, peut ainsi devenir un exercice difficile ou aléatoire.

Il convient enfin de compléter ce diagnostic en constatant qu'un système de gestion fondé sur une délégation de responsabilité à la collectivité nationale et sur la mise en oeuvre d'une réglementation s'avère mal adapté pour prendre en compte certaines données nouvelles de la gestion des milieux aquatiques. Un tel système peut répondre efficacement aux phénomènes durablement réversibles. Mais les cas d'irréversibilité potentielle de la dégradation de la qualité de certains milieux se font désormais plus nombreux, notamment par le fait de l'eutrophisation. L'intervention administrative est moins efficace dans ce cas. Il apparaît ainsi qu'elle appréhende mal le long terme dont la prise en considération est étroitement liée aux problèmes d'irréversibilité. Ce type d'intervention fait difficilement face, d'autre part, aux phénomènes diffus dont le rôle est croissant en matière de dégradation de la qualité des eaux. Une approche fondée sur une réglementation suppose un système de référence, faisant appel à des paramètres quantitatifs. Les causes et les mécanismes des pollutions diffuses ne s'y prêtent guère, en l'état actuel des connaissances, et ce domaine de la gestion des milieux aquatiques appelle nécessairement une démarche différente.

Les différents termes de cette rapide analyse montrent le besoin d'une approche renouvelée des problèmes de gestion des eaux, capable d'intégrer la nécessité et les avantages certains des interventions de la puissance publique d'une part, des concepts nouveaux qui permettent de mettre à profit les solidarités inhérentes aux interactions entre opérateurs intervenant dans la gestion des eaux d'autre part.

SOURCES ET PRINCIPES D'UNE APPROCHE RENOUVELEE.

L'eau, "res communis"

Le droit civil français reconnait au propriétaire d'un bien foncier le libre usage de l'eau courante qui le traverse ou le borde, dans la limite d'une juste appréciation de ses besoins. Initialement dédié à l'irrigation, ce droit d'usage s'est progressivement étendu aux utilisations les plus diverses et trouve ses limites externes dans le respect des droits identiques des riverains d'aval. Le respect de ces derniers a induit le caractère de chose commune attribué à l'eau d'un cours d'eau non domanial, donnant lieu à des pratiques de gestion locale au sein d'une communauté rurale. L'irrigation en a été l'objet le plus habituel à l'origine. Ce mode de gestion correspondait très directement à un système économique rural fortement autarcique. Très localisé, fortement unitaire, il se fondait sur une information réciproque permanente caractéristique des communautés suffisamment petites et sur une négociation globale et immédiate entre les membres de cette dernière. Ainsi s'élaboraient de véritables réglements locaux d'usage de l'eau mis en oeuvre grâce à une discipline procédant de l'interaction entre les membres de la communauté. Les ajustements entre communautés voisines s'opéraient sur des bases comparables. L'arbitrage indispensable dans le cas de conflits irréductibles par le moyen de la négociation interne directe relevait de la justice civile, ainsi qu'il sied à propos de litiges entre particuliers ou entre communautés de droit privé.

Ce mode de gestion, dénommé "autarcique unitaire" en raison des caractères dominants des communautés qui en étaient le siège, était souple, efficace et fortement intégrateur vis-à-vis des préoccupations de l'ensemble des acteurs concernés, en exprimant leur solidarité vis-à-vis du meilleur usage d'une ressource d'autant plus précieuse que les arrosages étaient plus nécessaires.

Le démembrement des formes primitives de gestion communautaire

Le contexte technique, économique et social auquel pouvait répondre la gestion de type "autarcique unitaire" ne lui permettait pas d'évoluer et de se transformer afin de s'adapter aux contraintes nouvelles qui se sont progressivement imposées à lui : le système a été mis dans l'obligation de prendre en compte des acteurs nouveaux, externes vis-à-vis de la communauté, sans pouvoir les intégrer dans son propre schéma du fait même de son caractère autarcique. Son démembrement progressif était inévitable, faisant place à une intervention de plus en plus active de la puissance publique afin de promouvoir tout d'abord l'intérêt public et par la suite de façon de plus en plus large pour préserver l'intérêt général.

Ce démembrement a pris la forme de limitations imposées par la collectivité au libre exercice du droit de propriété et donc à l'usage de l'eau "chose commune" dans toute la mesure où le système autarcique unitaire était incapable de s'ouvrir à de nouvelles nécessités. Pour les cours d'eau les plus importants, il a abouti depuis longtemps à un transfert de propriété au profit de la collectivité nationale, sous le forme des cours d'eau domaniaux.

Cette intervention de l'Etat dans la gestion des eaux, fondée sur une action réglementaire, se traduit dans les faits par une démarche discontinue, faiblement intégratrice, qui s'organise par filières. Ainsi est négligé le caractère spatio-temporel des "systèmes-eaux". Le caractère unitaire de la réglementation s'oppose à la diversité des situations locales, nuisant à l'efficacité des interventions administratives.

On peut désigner ce mode de gestion, capable d'aborder les aspects les plus variés de sa pratique mais condamné à les sérier, voire à les isoler de leur contexte, par la dénomination "ouvert parcellisé".

Le passage du contexte autarcique au contexte ouvert constitue une nécessité toujours plus évidente dans les conditions présentes. L'objectif paraît dès lors être l'identification des moyens de retour d'une démarche "parcellisée" à une démarche "unitaire" suffisamment intégratrice en recherchant les possibilités de participation directe ou indirecte de tous les acteurs concernés à une gestion globale des eaux dans des domaines d'espace appropriés. En résumé, le but à poursuivre est très schématiquement le passage du système "ouvert parcellisé" à un système "ouvert unitaire".

L'eau, patrimoine commun

L'eau est fréquemment citée comme un élément du "patrimoine naturel" sans qu'un effort s'en suive pour expliciter cette notion, ni pour en tirer les conséquences. Tout au plus lui reconnaît-on ainsi une certaine importance sans pour autant avoir conscience qu'il puisse en résulter une responsabilité particulière de chacun de ceux qui adhèrent à cette esquisse de relation commune avec un cadre de vie perçu de manière essentiellement qualitative.

Pour dépasser cette perception confuse et abstraite, il y a lieu tout d'abord de préciser la notion de patrimoine afin de pouvoir apprécier dans quelle mesure une valeur patrimoniale peut être attribuée à un milieu aquatique.

Un patrimoine est "un ensemble d'éléments matériels et/ou immatériels qui concourent à maintenir et à développer l'identité et l'autonomie de son titulaire dans le temps et dans l'espace par adaptation en milieu évolutif".

Cette définition très générale et fort abstraite peut s'appliquer aux concepts les plus variés : il pourra tout aussi bien s'agir du "patrimoine culturel" d'un groupe ethnique ou d'une communauté linguistique que de biens matériels détenus par une entité familiale ou au contraire une collectivité, une personne morale. Dans tous les cas la qualité de patrimoine procède de l'existence d'une relation stable entre son objet et un titulaire dans le but d'en obtenir un juste avantage immédiat tout en privilégiant sa conservation à long terme par l'effet d'une gestion "en bon père de famille".

Cette relation implique nécessairement une composante fonctionnelle d'usage, ainsi que des liaisons "métafonctionnelles" dès qu'intervient une multiplicité des acteurs - c'est-à-dire la prise en compte d'interactions entre relations fonctionnelles propres à chacun de ceux ci dans le cadre d'un "système régulé". Mais la relation patrimoniale va bien au delà en imposant de véritables relations d'identité prenant en compte la stratégie des titulaires vis-à-vis du long terme, dans le but de maintenir un potentiel de réversibilité, de fonctionnalisation et d'adaptation, un état de relations entre acteurs.

Un "systéme-eau", quelles que soient son étendue et sa plus ou moins grande complexité, met en jeu un faisceau de relations d'identité attachées à un ensemble d'acteurs qui prévilégient pour des raisons variées la pérennité des satisfactions qu'ils entendent en obtenir, qu'elles résultent d'usages ou de préoccupations qualitatives voire même esthétiques, et à la prévention des nuisances qu'ils pourraient avoir à subir de son fait. En d'autres termes, la relation patrimoniale conduit en matière de gestion des eaux à l'identification d'un ensemble d'acteurs qualifiés par des préoccupations identiques ou interdépendantes vis-à-vis d'un même "système-eau".

Cette approche appelle un engagement direct et personnel de tous les acteurs : une collectivité, nationale ou plus restreinte, ne pourra être que l'un d'entre eux puisque procédant d'une délégation de responsabilités, quelle que soit l'importance privilégiée qu'elle pourra prendre au sein du système relationnel. Les relations d'identité devront au contraire s'établir au sein d'une communauté de l'ensemble des acteurs concernés, individuels ou collectifs, publics ou personnels.

Un "système-eau" pourra donc, par la prise en compte de l'ensemble des relations d'identités réputées capables d'intégrer les liaisons fonctionnelles et métafonctionnelles, constituer le "patrimoine commun" de l'ensemble des acteurs concernés. Son titulaire sera ainsi la communauté de tous ceux qui sont impliqués à des titres et degrés divers dans sa gestion.

L'adoption de ce concept peut paraître, en première analyse, passablement utopique : la complexité des sociétés est devenue telle qu'il peut sembler peu réaliste d'impliquer tout un chacun dans une procédure de gestion patrimoniale d'un bien commun omniprésent. Il serait effectivement inconcevable que puisse être ressuscitée la pratique communautaire d'antan. Mais il s'avère par contre tout à fait possible de développer des modes d'information, des instruments de représentation et d'expression, des procédures de négociation capables de rendre compte d'une approche patrimoniale de la gestion des eaux dans un contexte de communauté.

LES PERSPECTIVES D'APPLICATION D'UNE GESTION PATRIMONIALE.

La principale difficulté rencontrée pour mettre en oeuvre une gestion patrimoniale des eaux résulte de la multiplicité des acteurs et de la variété des relations d'identité par lesquelles ils s'intègrent à la communauté de fait au sein de laquelle devront s'élaborer objectifs, stratégies et décisions en découlant.

Constatations préliminaires

Traduit en termes pratiques, le concept de gestion patrimoniale s'organise à partir de la reconnaissance par les multiples acteurs concernés des solidarités qui doivent les lier au sein d'un certain domaine d'espace, et implique à terme un infléchissement réciproque des comportements dans le but de régler au mieux les affaires d'intérêt mutuel s'inscrivant dans le temps présent tout en prenant en considération les effets à long terme dans un souci de préservation du patrimoine commun.

La condition première du succès d'une telle démarche sera nécessairement une information suffisamment complète et objective des partenaires concernés. Or bien souvent l'auteur d'un usage de l'eau - prélèvement ou pollution - ignore ou sous-estime les conséquences de son acte à l'égard d'autres ayant-droit généralement situés en aval, tout comme ces derniers sont dans l'incapacité de percevoir les motivations et contraintes qui peuvent conduire le premier à agir en toute bonne foi. Aucune recherche de compromis n'est possible sans que la confrontation de ces points de vue ait pu intervenir, sur la base d'une information réciproque qui permette à chaque partie de bien comprendre les fondements de l'argumentation "adverse".

Toute institution visant à promouvoir et à animer une gestion patrimoniale des eaux doit donc avoir pour première préoccupation la collecte et la large diffusion d'une information appropriée, la réalisation des échanges d'informations réciproques nécessaires chaque fois que devra intervenir une négociation.

Par ailleurs l'efficacité des mécanismes qui pourront être mis en oeuvre sera subordonnée dans tous les cas à une identification correcte des acteurs concernés. Si les relations de type amont-aval s'y prêtent en général assez aisément, les relations d'identité peuvent être bien moins évidentes lorsque doivent être considérées les interactions entre eaux souterraines et superficielles, les effets du drainage, l'érosion diffuse ou les conséquences de l'imperméabilisation des sols résultant d'aménagements liés à une urbanisation.

L'identification de l'ensemble des acteurs impliqués à des titres divers dans la gestion de l'eau au sein d'un domaine d'espace fonctionnel vis-à-vis de celle-ci constitue donc un préalable déterminant pour le succès d'une approche patrimoniale même si certains d'entre eux peuvent en première analyse paraître indistincts ou éloignés, ainsi que sont souvent perçus par les usagers les plus directs les tenants de certaines activités de loisirs, par exemple.

Une démarche de type patrimonial doit enfin situer la préparation de décision aussi près que possible de son lieu d'application, quel que soit le niveau imposé par la réglementation et les procédures en vigueur pour la prise effective de décision en général matérialisée par un acte administratif. La modification attendue des comportements, l'adhésion des acteurs aux dispositions à mettre en oeuvre, la pénétration convenable de l'information supposent un engagement aussi direct que possible des acteurs directement impliqués sur le terrain, pour bénéficier des conditions psychologiques les plus favorables à la réussite. La solution des problèmes posés au sein d'un petit bassin versant ne bénéficiera pas des avantages espérés de la démarche patrimoniale si la préparation de décision, ou plus précisément la négociation d'un consensus, se situe à un niveau d'intégration éloigné tel que le département ou le grand bassin. Rapprocher le plus possible le "lieu" de la concertation du domaine qui en est l'objet doit être tenu pour un facteur important de réussite.

Principes d'organisation et de mise en oeuvre

L'extrême diversité des situations aussi bien que la nature même d'une démarche de gestion patrimoniale excluent tout schéma théorique, tout modèle d'organisation dont la rigidité et l'inadéquation vis-à-vis des situations réelles iraient à l'encontre des conditions du succès.

Seules peuvent être formulées des lignes directrices à partir desquelles devra être élaboré, dans chaque cas d'espèce, un véritable projet de gestion patrimoniale fondé sur une étude de diagnostic approfondie. Cette étude devra nécessairement comporter une identification appropriée des acteurs, de leurs agrégations en communautés particulières d'intérêts susceptibles de participation unifiée, de leur hiérarchisation spatiale ou fonctionnelle. Elle devra ensuite mettre en lumière les principales contraintes de gestion, identifier les antagonismes exprimés ou latents, les situations conflictuelles dominantes, les facteurs externes capables de favoriser certaines actions ou au contraire susceptibles de limiter la capacité interne de choix dans les processus de préparation de décision aux différents niveaux. Une telle étude devra identifier, à partir du diagnostic, les principaux objectifs susceptibles d'être adoptés. Mais elle ne devra en aucun cas préjuger d'un choix quelconque, laissant aux processus de la gestion patrimoniale le soin de les déterminer.

Les"lieux de concertation"

Compte tenu de ces constatations préliminaires, le petit bassin versant est sans nul doute un domaine privilégié d'expression des solidarités et de réduction des antagonismes, pourvu que soit respecté le principe d'une intégration de toutes les composantes du "système-eau" dans ses limites, afin que puissent être prises en compte toutes les interactions entre eaux courantes superficielles, zones humides, eaux souterraines et ruissellement. Dans ce cadre pourra être mise en oeuvre une instance de préparation de décision, et par là même de négociation, où nombre de problèmes pourront trouver leur solution. Située au niveau le plus élémentaire du dispositif, une telle instance devrait dans son principe réunir tous ceux que la gestion des eaux concerne directement ou indirectement. De fait elle devra être ouverte à tous. Mais en pratique la concertation ou la négociation n'y seront en général possibles que dans la mesure où les multiples acteurs individuels relevant de communautés d'intérêts particulières pourront s'agréger pour constituer des interlocuteurs uniques, sur la base d'un consensus préalable obtenu au sein de ces groupes. Aux côtés d'acteurs individuels d'une importance suffisante pour justifier leur intervention directe apparaitront ainsi les collectivités territoriales, des représentations professionnelles, des organisations ou associations d'usagers. L'Etat, expression de la collectivité nationale, y trouve obligatoirement sa place, dans la mesure où les dispositions susceptibles d'être proposées ne sauraient en aucun cas s'affranchir des réglements et procédures d'ordre général. En définitive il s'agira d'institutionnaliser sous une forme suffisamment souple une expression convenable de la communauté titulaire du patrimoine "eau" dans le petit bassin versant. En son sein devront s'exercer les fonctions d'information et de négociation, dans une perspective de prise de décision si le problème peut être réglé localement, de préparation de décision si l'affaire relève d'un acte administratif. Dans tous les cas l'essentiel est de parvenir à un consensus ou au moins à une solution acceptable par tous en prévenant l'émergence de situations conflictuelles.

Selon qu'il est plus ou moins vaste, un système hydrographique comportera un nombre variable de niveaux d'agrégation de petits bassins,, puis d'agrégats de petits bassins. A chacun de ces niveaux pourra correspondre une instance de concertation de niveau correspondant, constituée selon un mode de représentation dont seules les données locales et régionales permettront de définir les modalités.

L'opportunité d'affirmer un statut juridique des instances ainsi développées, de matérialiser leur lieu de fonctionnement, relève des cas d'espèces. L'essentiel est la mise en oeuvre effective d'une structure de négociation capable, à chaque niveau, d'intégrer tous les facteurs antagonistes aussi bien que les convergences révélées par l'étude de diagnostic.

Les "règles du jeu"

Toute procédure de négociation implique a priori l'acceptation unanime d'une "règle du jeu" fondée sur le respect des intérêts mutuels et sur l'acceptation de concessions réciproques, assorties d'éventuelles compensations bilatérales ou multilatérales, pour parvenir à une solution qui puisse être l'objet d'une adhésion de la part de tous les acteurs, ou au moins d'une acceptation non restrictive de la part des plus réticents, conçue de manière à réduire les antagonismes et à éliminer les sources de conflits ultérieurs. Cette règle du jeu implique d'autre part, dès l'origine, la formulation d'objectifs qui seront le fondement même de la gestion patrimoniale et devront faire l'objet d'une large adhésion. L'acceptation préalable et le respect de cette règle du jeu, la mise en oeuvre d'un langage de négociation suffisamment simple, accessible à tous les acteurs, le refus de toute exclusivité quant à la qualité d'acteur, le respect des décisions conformes à la règle du jeu y compris en ce qui concerne les éventuelles compensations constituent les conditions déterminantes du succès de toute expérience de gestion patrimoniale.

Le médiateur ou modérateur

Toute instance de concertation et de négociation, visant à réduire des antagonismes et à rechercher un consensus sur des objectifs et des solutions de compromis au sein d'une communauté, doit faire appel à un "meneur de jeu" capable d'animer son fonctionnement, d'apporter l'information technique nécessaire aux acteurs pour garantir la validité des solutions qu'ils proposeront, de veiller à la diffusion et aux échanges en matière d'information interne, de guider la recherche des termes d'accord. Cet animateur, que l'on dénomme souvent "modérateur", doit avant tout avoir la compétence technique requise, et ne doit pas être impliqué comme acteur à quelque titre que ce soit. Ces conditions remplies, ses qualités de relations humaines et son acceptation unanime par les acteurs de toutes natures participant à l'instance de concertation devront prévaloir dans son choix.

Le rôle de l'Etat et les relations contractuelles

La mise en oeuvre d'une procédure de gestion patrimoniale ne saurait en aucun cas mettre en cause le rôle fondamental de l'Etat, garant de l'application cohérente d'une politique de l'eau et de la préservation de l'intérêt général. Législation et réglementation demeurent dans cette perspective un cadre de référence primordial par rapport auquel devront se définir tous les accords négociés au sein du système patrimonial, et la base de tout recours que rendrait nécessaire l'échec d'une négociation, le rejet de tout consensus par certains acteurs. L'Etat doit enfin apparaître comme le garant du respect des accords réalisés, tout particulièrement lorsque doivent intervenir des compensations dont il sera souvent partie prenante à travers les mécanismes d'intervention qu'il a mis en place. En contrepartie de cette responsabilité particulière qui en fait un acteur essentiel de toute initiative de gestion patrimoniale, l'Etat y trouvera l'avantage d'un terrain particulièrement favorable pour intervenir sous des formes contractuelles dans tous les cas où son intervention réglementaire s'avère impraticable. A titre d'exemple ce cadre de négociation pourra grandement faciliter la réalisation d'accords avec la profession agricole sur des "codes de bonnes pratiques culturales" visant à protéger les eaux souterraines et plus encore à améliorer l'efficacité des périmètres de protection des captages, en instituant si nécessaire les compensations techniques ou financières que cela implique tout en conservant la plus grande souplesse d'adaptation à la diversité des situations locales. Le développement de la gestion patrimoniale peut ainsi être l'occasion pour l'Etat de compléter son dispositif réglementaire par la promotion d'une politique flexible de relations contractuelles. La condition majeure en est sa propre adaptation à une unicité de son mode d'intervention, par une adaptation de l'organisation de ses services pour opérer comme un interlocuteur unitaire face aux instances patrimoniales ou en leur sein.

L'EXPERIENCE ACQUISE

Par le jeu même de ses institutions fédérales et cantonales, la Suisse pratique depuis longtemps une gestion des eaux qui relève en fait du concept patrimonial . Sans avoir fait l'objet d'une approche théorique préalable, des modalités largement participatives de gestion des eaux se sont élaborées et adaptées aux situations locales selon un modèle aussi simple qu'original. Son analyse approfondie serait riche d'enseignements et mériterait la publication d'une étude.

Des expériences ponctuelles, généralement spontanées, parfois suscitées par des "situations de crise", se sont développées sporadiquement sans pouvoir constituer de véritables références.

Par contre différents pays ont développé depuis la seconde guerre mondiale des législations modernes sur l'eau qui mettent en oeuvre les principes essentiels d'une gestion patrimoniale même si ce concept n'a pas été explicité en vue de leur élaboration. Même s'ils s'adressent à de grands bassins pour résoudre des problèmes d'importance nationale, ces principes ont pu ainsi faire la preuve de leur efficacité et montrer tout l'intérêt que présenterait leur transposition à des contextes de plus en plus décentralisés.

La loi sur l'eau votée par le parlement français en décembre 1964 constitue un exemple caractéristique de cette démarche globale. Elle s'adresse a de très grands bassins, six de ces entités se partageant l'ensemble du territoire national. Sans mettre en cause le rôle de l'Etat et le jeu du dispositif réglementaire, elle a institué pour chacun d'eux un "lieu de concertation" où se situent les négociations entre "acteurs" sous la forme de représentants qualifiés des multiples groupes d'usagers homogènes : les Comités de Bassin. La loi a établi les éléments d'une règle du jeu, à travers la politique d'objectifs de qualité et la définition des responsabilités incombant aux différents acteurs: préleveurs, pollueurs, demandeurs de "qualité". Elle a enfin, par la voie des Agences Financières de Bassin, créé le support et les mécanismes des systèmes de compensation inséparables des solutions de compromis : redevances de prélèvement et de pollution, aides à l'investissement, primes "de résultats". Au total, un système cohérent de gestion globale dont l'ampleur justifiait l'institutionalisation tout autant que la création d'importants moyens logistiques.

Plus récemment, des études approfondies ont été réalisées en vue d'une application locale, très décentralisée, de gestion patrimoniale. La première a porté sur la nappe phréatique d'Alsace. Cette nappe rhénane, d'une importance exceptionnelle, est soumise à des dégradations de qualité alarmantes par le fait d'une multiplicité d'actions "individuelles". La création d'une institution patrimoniale appuyée sur des structures de concertation très décentralisées a été proposée sur la base d'un diagnostic approfondi. La complexité du problème posé par cette nappe n'a pas encore permis de donner suite à ces suggestions très concrètes.

Plus récemment, une étude très complète de diagnostic et de propositions concrètes a été réalisée dans le bassin de la Sèvre Nantaise où les aspects de qualité des eaux s'avèrent très préoccupants dans une région de roches anciennes où les débits d'étiage sont très faibles face à une forte demande estivale de prélèvements. S'appuyant sur une institution associative préexistante, constituée à l'initiative des élus des collectivités territoriales particulièrement ouverts aux perspectives de gestion patrimoniale, ce nouveau projet devrait connaître une mise en oeuvre progressive au cours des prochaines années. Il constituera une expérience riche d'enseignements.

CONCLUSION

Expression d'une nouvelle approche fortement décentralisée et essentiellement participative de la gestion globale des eaux, la gestion patrimoniale vise avant tout à restaurer la notion de responsabilité individuelle vis-à-vis de l'eau, et au delà à faire évoluer les comportements. De ce fait elle ne saurait être un "plaquage" anonyme sur d'autres structures. Elle doit être au contraire suscitée par la capacité d'initiative locale ou régionale pour se développer au sein même des communautés concernées, sans jamais leur être imposée depuis l'extérieur. Il serait enfin bien imprudent d'en attendre des résultats miraculeux autant qu'immédiats : son développement ne pourra être qu'une oeuvre de très longue haleine. Celle-ci apparaît cependant comme la seule voie capable de mobiliser les véritables acteurs de la qualité des eaux.

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ALTERNATIVE SOURCES OF ENERGY

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ABSTRACT

Taiwan is a small island, limited in arable land and meagre in natural resources but densely populated and has to depend on two-way trade for economic development. During the past three decades, the Republic of China on Taiwan has successfully developed a booming export-oriented economy which is characterized by massive manufacturing industries and concomitantly heavy and intensive energy consumption.

However, the indigenous energy resources and energy production in Taiwan area are both highly restricted. Coal is one of the most important indigenous energies in Taiwan, but its total reserve is only 207 million tons and annual production is about 2.5 million tons. Local oil is insignificant. Natural gas is produced at a daily rate of 3.5 to 4 million cubic meters equivalent to 22,000 to 25,000 barrels of oil. Existing hydro power installation of 1,431 Mw represents the most favorable hydro electric potential on the island, leaving about 3,900 Mw of less favorable potential yet to be developed. Because of this background, most of the energy needs must be covered by the imported energy, especially oil. In 1982, 86.17% of our energy consumption was imported as compared with 13.83% locally produced. Naturally, the overdependance on imported energy brought far-reading and negative impact on the economy, particularly in the year after the energy crisis in 1974. Although the oil price reduction by OPEC early this year did herald good signs of recovery for the island economy as well as world economy both hard hit by the world-wide recession, yet it is quite likely that oil price will rebound when the energy demand picks up with the increasing recovery. It is doubtless that any excessive oil price escalation in the future will severely affect and retard the pace of economic development unless a correct energy strategy is mapped out and adequate alternative sources of energy are selected.

In this paper, energy background data is firstly brought up forward for introduction and roots of the energy problems are probed into. Secondly, the economic environment and the energy demand are presented and generally reviewed. Following this, the considerations and principles for mapping the national energy plan are set forth while the strategy for the plan is also recommended. Finally, some alternative sources of energy under consideration are discussed in certain length.

INTRODUCTION

Taiwan is an island province of the Republic of China (ROC), occupying an area of 36,000 square kilometers, or being roughly equivalent to the Netherlands in size. Of its territory, most parts are rugged terrain or steep mountains, only one quarter is arable land. Its population as of the end of 1983 stood over 18.7 millions, with a density of 520 persons per square kilometer, making Taiwan one of the most densely populated areas of the world. Under such circulstances. It has no other choice but to develop the two-way international trade for its economic growth.

During the past three decades, the Republic of China on Taiwan has successfully developed a booming export-oriented economy which is characterized by massive manufacturing industries. As shown in Table-1, ROC's GNP in 1963 and 1983, were US$ 5.073 billions and US$ 29.425 billions respectively, representing an average growth rate of 9.2% per annum. And the per capita income increased by 15 times from US$ 166 in 1963 to US$ 2,444 in 1983. Table-2 shows that the total volume of external trade jumped in great strides from US$ 1.638 billions in 1963 to US$ 30.126 billions in 1983, corresponding to an average growth rate of 15.67% per annum. And the share of industrial production in GNP significantly increased from 28.9% in 1963 to 51.6% in 1983.

TABLE-1 GNP AND PER CAPITAL INCOME 1963-1983

Year	GNP (at 1976 constant price)		Per Capital Income (at current price)	
	Amount(US$ Billion)	Growth Rate	Amount (US$)	Growth Rate
1963	5.073	8.6	166	8.1
1965	6.323	11.6	203	10.6
1967	7.620	9.8	249	10.8
1969	9.059	9.0	320	13.4
1971	11.380	12.1	410	13.2
1973	15.354	16.2	642	25.1
1975	16.164	2.6	888	17.6
1977	20.150	11.7	1,182	15.4
1979	26,153	13.9	1,722	20.7
1981	27,881	3.3	2,360	17.1
1983	29.425	2.7	2,444	17.6
1964-1983 (Average)	-	9.2	-	14.4

To support the continued development of industries and constant growth of economy, energy is undoubtedly one of the most important necessities. However, both indigenous energy resources and energy production in Taiwan area are highly restricted. The up-to-date reserves are 196 million tons of coal and 22.0 billion cubic meters of natural gas. Oil reserve is insignificant, estimated at only 1.8 million kilolitres. The theoretical hydro power potential is about 12,000 Mw, of which 5,300 Mw are technically feasible, and 1,431 Mw has been developed.

In 1983, local energy production included 2.2 million tons of coal, 1.5 billion cubic meters of natural gas, 0.1 million kiloliters of oil and 4,987.7 Gwhe of hydro-electric power, totalling 4.4 million kilolitres of oil equivalent and supplying only 12.4% of the domestic energy demand. The deficit of 87.6% was met with imported energy, including 61.1% oil mainly from the Persian Gulf Countries; 13.0% coal; and 13.5% nuclear. Table-3

TABLE-2 THE EXTERNAL TRADE AND INDUSTRIAL PRODUCTION

(at 1976 constant price)

Year	Exports		Imports		Industrial Production	
	US$ Billion	Growth Rate (%)	US$ Billion	Growth Rate (%)	US$ Billion	Growth Rate (%)
1963	0.718	-	0.920	-	1.468	-
1965	1.111	24.4	1.349	21.1	1.971	15.9
1967	1.513	16.7	1.788	15.1	2.585	14.5
1969	2.358	24.8	2.712	23.2	3.538	17.0
1971	3.990	30.1	2.656	-1.0	4.922	17.9
1973	6.903	31.5	6.355	54.7	7.214	21.1
1975	6.477	-3.1	6.693	2.6	7.553	2.3
1977	9.876	23.5	8.834	14.9	10.132	15.8
1979	13.308	16.1	12.508	19.0	13.625	16.0
1981	15.020	6.2	12.922	1.6	14.535	3.3
1983	16.992	6.4	13.134	0.8	15.185	2.2
1963-1983 (Average)	-	17.1	-	14.2	-	12.4

TABLE-3 THE ENERGY CONSUMPTION AND SUPPLY

Unit: 10^3 K.L. Oil Equivalent

Year	Internal Energy Consumption	Total Energy Supply (100%)			
		Indigenous	%	Imported	%
1963	4,781.3	3,953.4	71.74	1,557.4	28.26
1965	5,973.1	4,614.4	66.94	2,278.8	33.06
1967	7,400.1	4,844.8	57.07	3,644.8	42.93
1969	9,230.3	5,058.1	44.88	6,213.0	55.12
1971	11,266.4	4,875.9	36.60	8,444.7	63.40
1973	14,443.3	4,827.2	28.15	12,322.5	71.85
1975	16,120.8	5,277.1	30.87	11,817.4	69.13
1977	20,938.2	5,281.0	21.33	19,482.2	78.67
1979	26,827.4	5,138.0	17.21	24,720.7	82.79
1981	27,431.4	4,726.2	14.34	28,236.4	85.66
1983	31,097.1	4,408.6	12.40	31,135.2	87.60

* Total Energy Supply=Internal Energy Consumption+Export+Change in Stocks +Loss

shows the annual energy consumption and supply in the past two decades. The consumption increased at an average rate of 9.8% p.a., with the imported energy climbing from 28.3% in 1963 to 87.6% in 1983, while the local contribution dropped from 71.7% in 1963 to 12.4% in 1983. Naturally, the overdependence on imported energy brought far-reaching and negative impacts on the economy, particularly in the years after the two energy crises. Although the oil price was once lowered by OPEC last year, yet it is generally believed that the price will rebound when the energy demand picks up with increasing economy recovery. It is doubtless that any excessive oil price escalation in the future will severely affect and retard the pace of economic development unless a correct energy strategy is mapped out, aiming at diversifying the forms and sources of energy for future needs.

PROJECTED ECONOMIC GROWTH AND ENERGY DEMAND IN THE FUTURE

In the early years, the industries of ROC on Taiwan were mostly labor intensive ones, such as textile, apparels, electronic assembly. Beginning in the late 1950s, heavy industries were subsequently introduced, including petrochemical, steel, electrical equipment and appliances, automobile parts and assembly. Over 20 years of development, these industries have made an amazing growth of domestic economy with the help of cheap energy, low cost labor and rapid expansion of two-way international trade.

Since late 1970s, those advantageous factors have gradually weakened or even vanished owing to the impact of energy crises, the upgrading of living standard and the competition from other countries as evidenced by the interruption of the nation's double-digit economic growth. The undesirable climate will strongly affect Taiwan's economic development as well as prompt significant changes in the industrial structure. Since 1980, two broad categories of industry have been accorded higher priority, namely, machinery manufacturing and information. By virtue of their linkage effects, technology intensiveness, low energy consumption, high value-added, good market potential, and relatively low pollution, these industries are considered as "stragetic industries". Their development will speed up industrial restructuring that is essential to the continued growth of the island economy. According to the government's ten-year (1984-1993) economic development plan, the GNP growth is projected at an average rate of 6.2% p.a., whereas the growth rate of whole industry sector is projected at 6.9% p.a., manufacturing industries at 6.9% p.a. and international trade at 5.8% p.a. The important economic indices for the years 1984-1993 are shown in Table-4.

TABLE-4 IMPORTANT ECONOMIC INDICES OF THE REPUBLIC OF CHINA (1983-1993, at 1976 Constant Price)

	1983	1985	1987	1989	1991	1993	Average Annual Growth Rate 1984-1993
Economic Growth Rate (%)	5.2	6.1	6.3	6.1	6.3	6.0	6.15
GNP (US$ Billion)	29.43	33.16	37.47	42.18	47.62	53.50	6.15
Per Capital Income (US$)	1,437	1,706	1,870	2,048	2,256	2,478	5.60
Export (US$Billion)	16.99	17.50	19.67	22.10	24.95	28.03	5.13
Import (US$Billion)	13.13	14.55	16.70	19.17	22.14	25.44	6.84

* Foreign Exchange Rate (Per US$) 1:40.00

In accordance with the above-mentioned economic development plan for the next decade, it is forecasted that annual energy consumption will increase from 31.53 million kilolitres of oil equivalent in 1984 to 51.59 million kilolitres of oil equivalent in 1993, representing an average annual increase rate of 5.6%. Of the total energy supply, the share of imports will increase from 87.8% in 1984 to 90.4% in 1993. The energy supply projection for the years from 1984 through 1993 are shown in Table-5.

TABLE-5 ENERGY SUPPLY PROJECTION, 1983-1993

(Million Kilolitres Oil Equivalent)

	1983	1985	1987	1989	1991	1993	Average Annual Growth Rate (%)
Indigenous Energy	4.40	4.50	4.75	5.19	5.07	5.12	1.5
Coal	1.54	1.56	1.49	1.49	1.49	1.42	-0.8
Crude	0.13	0.13	0.13	0.14	0.10	0.08	-0.5
Natural Gas	1.47	1.29	1.30	1.35	0.95	0.82	-5.6
Hydro Power	1.26	1.38	1.59	1.81	1.94	1.94	4.4
Others	0	0.14	0.24	0.40	0.59	0.86	25.5
Imported Energy	31.13	32.71	36.38	40.71	45.13	49.73	4.9
Coal	4.64	5.79	6.78	8.30	10.82	11.32	9.3
Crude Oil and its products	21.70	19.88	21.99	24.80	25.48	27.00	2.3
Natural Gas	0	0	0	0	1.22	1.44	-
Nuclear	4.79	7.04	7.61	7.61	7.64	9.97	7.6
Total Energy Supply	35.53	37.21	41.13	45.90	50.23	55.00	4.5

ENERGY POLICY

Energy is not only essential to the motivation of the economic development, but also indispensable for national security. The government promulgated in 1973 a comprehensive energy policy, namely "Energy Policy for the Taiwan Area", which became inadequate after the 2nd oil crisis in 1979 and was therefore revised with emphases on accelerating development of indigenous energy, strengthening energy conservation and further diversification of energy imports. The policy is formulated on basis of the characteristics of energy demand and supply together with the consideration of the principles of economy, security and welfare. The highlights of the policy are:

A. Assure the stability of energy supply
   1. Continuing to strengthen the exploration and development of indigenous energy.
   2. To seek the diversification of imported energy forms and sources.
   3. To encourage the investment in the development of energy resources in foreign countries.
   4. To intensify the construction of energy transportation and storage facilities.

B. Accelerate the rationalization of energy price
   The pricing of various energy forms should be based on such factors as their mutual structure, cost, utilization efficiency, purposes, and the future development, etc.

C. Promote the efficiency and benefit of energy utilization
   1. Intensifying energy management and conservation.
   2. To improve industrial structure and promote the electrification of industry.
   3. To develop the systems of mass transportation and rapid transit.
   4. To upgrade the utilization efficiency of energy consuming equipment and appliances.

D. Prevent the environmental pollution of energy
   Environmentalpollution should be avoided, natural scenery should be conserved and ecological balance should be maintained in the process of the exploration, development, production, transportation, storage and use of energy.
E. Enhance energy research and development
   1. To establish the system of coordination and cooperation in energy research and development.
   2. To push forward integratedly the research and development of energy economics and energy technology.
   3. To step up international cooperation in the research and development and the communication of information.

ALTERNATIVE SOURCES OF ENERGY AND STRATEGIC PLANS FOR ENERGY SUPPLY

As mentioned previously, the share of indigenous energy in Taiwan's energy supply has been declining and the increasing consumption of imported energy has led to deepening dependence on imported oil and growing vulnerability to a possible supply interruption. Any interruption of the supply could possibly be catastrophic to the economic development and the security of the nation. To minimize the probability of supply interruption of imported energy, various alternative sources of energy have to be carefully evaluated and suitable strategic plans for energy supply should be worked out.

In planning imported energy, due consideration should be given to such factors as reliability in supply; stability in price, easiness, safety and economy in utilization, storage and transportation, and special attention should be paid to the diversification of the forms and sources of energy. In the meantime, the development of indigenous energy sources should not be overlooked. The following is a brief description of the various possible alternative sources of energy and their corresponding supply plans for the foreseeable future.

Oil

Oil is the most convenient form of fuel for industries and electric utilities, and has been the major primary energy consumed in Taiwan. In 1983, the total energy consumption amounted to 35.5 million kilolitres of oil equivalent, of which 56.9% was in the form of oil. However, the proven reserve of oil in Taiwan area is insignificant, and the oil rpoduction in 1983 was 0.1 million kilolitres, representing only 0.7% of the total oil consumption, leaving 99.3% to be covered by the imports mainly from the Persion Gulf Countries. Since the two oil price crises of the 1970s, the government has taken measures to encourage power utility and industries to shift the use of fuel from oil to coal. As discussed in the later section, Taipower, the sole electric power utility in Taiwan, has proceeded with conversion of some existing oil-fired units to coal fired ones, and almost all of the new fossil base-load units are contemplatively designed for coal-firing or dual-firing. Table-6 shows the composition of Taipower's electricity generation by various forms of primary energies, the share from oil will decline from 32.4% in 1983 to 15.7% in 1993. Consequently, the nation's oil consumption as a percentage of total energy consumption will decline from 56.9% in 1983 to 49.2% in 1993, although the annual oil demand will continue to increase from 21.83 million kilolitres to 27.08 million kilolitres during the same period as shown in Table-5.

It is realized that oil could continue to be a political weapon in the coming years, and its availability would become more uncertain. Yet, the government has successfully maintained a very friendly relationship

TABLE-6 THE COMPOSITION OF TAIPOWER ELECTRICITY GENERATION (1983-1993)

	Hydro		Coal		Oil		Nuclear		Total
	Gwh	%	Gwh	%	Gwh	%	Gwh	%	Gwh
1983	4,976	10.9	7,661	16.8	14,751	32.4	18,138	39.9	45,517
1985	5,092	10.0	12,495	24.6	8,422	16.6	24,785	48.8	50,794
1987	6,865	11.6	18,496	31.4	5,583	9.5	27,985	47.5	58,931
1989	7,313	11.1	18,297	27.9	11,361	17.3	28,666	43.7	65,637
1991	7,970	10.9	26,122	35.6	10,571	14.4	28,730	39.1	73,393
1993	9,432	11.4	27,658	33.4	13,075	15.7	32,765	39.5	82,930

with some Middle East countries, particularly the greatest oil exporting country, the Kingdom of Saudi Arabia. As a result, oil supply from the Mid-East probably be maintained at an adequate level to meet the demand. In the meantime, the Chinese Petroleum Corporation is actively engaged in exploration and exploitation for oil in Taiwan as well as in some foreign countries.

Coal

In spite of being the most important indigenous energy in Taiwan, the proven reserve of coal is estimated at only 196 million tons. The annual production of local coal reached its maximum of 5.1 million tons in 1967, but since then has declined erratically to about 2.2 million tons in 1983 due to the poor physical conditions in the mines and the rapid increase in mining cost. It is expected that the production can hardly exceed 3 million tons per annum in the future.

However, coal, being the most abundant and widely distributed conventional energy resource in the world, is recognized to be a major source that ROC will have to rely on for its energy supply during the remainder of this century. The government policy encourages the expanded use of coal, combining with nuclear power and renewable resources to fill the growing energy needs. Electric power utility and some industries have switched from oil to coal for some of their facilities. For example, Taiwan Power Company has converted some of its existing oil-fired units, with a total 1,205 MWe of installed capacity, to coal fired ones. In addition, all additions of fossil base-load generating facilities are designed for coal-firing or dual-firing use.

By the end of 1984, the total installed capacity of Taipower's coal-fired units was 2,505 Mw, representing 19.5% of its system total. Taipower has formulated its Long-Range Power Development Program mainly covering the period from 1984 to 2,000, with special emphases on the installation of nuclear and coal-fired units to diversify forms as well as sources of primary energy. As a result, the total coal requirement for electricity generation would increase from 3.1 million tons in 1983 to 8.9 million tons in 1990 and 14.8 million tons in 2,000. Together with efforts by other industries, ROC's total coal demand would increase from 8.7 million tons in 1983 to 15.9 million tons in 1990 and 26.2 million tons in 2000. Of the total demand, 13.8 million tons would have to be imported in 1990, and 24.2 million tons in 2000. The potential sources of coal for imports are: the United States, Canada, Australia, South Africa and Indonesia.

Nuclear Power

Nuclear power development is thought to be the most effective way of energy diversification and crucially important to the future economic development of ROC on Taiwan. Nuclear power has been successfully integrated into the power system. By the end of 1982, four nuclear units with a total installed capacity of 3,242 MWe were put into operation. In 1983, these four units generated 18,138 Gwh of electricity, representing 39.9% of total electricity generation in Taiwan. Nuclear units 5 and 6, 951 MWe each, will be synchronized in 1984 and 1985 respectively. Upon the completion of the sixth unit in 1985, the electricity generated from nuclear plants will amount to 24,785 Gwh, equivalent to 5.9 million kiloliters of oil, representing 48.8% of the then total electricity generation or 15.9% of the total energy consumed.

According to our operating experience, nuclear power generation is not only safe, clean, and reliable but also highly economical. The unit production costs of the various types of power generation in the Taipower system in the recent years are tabulated as follows:

TABLE-7 UNIT PRODUCTION COST (U.S. MILLS/KWH)

Year	Overall Producation Cost	Hydro		Oil-fired		Coal-fired		Nuclear	
		Cost	Ratio (%)	Cost	Ratio (%)	Cost	Ratio (%)	Cost	Ratio (%)
1980	35.4	14.1	129.4	43.0	394.5	30.7	281.7	10.9	100
1981	38.6	10.9	83.8	52.7	405.4	41.9	322.3	13.0	100
1982	39.6	12.2	75.8	56.3	349.7	44.0	273.3	16.1	100
1983	34.1	11.4	71.3	52.8	330.0	44.0	275.0	16.0	100

From the above table, it is clear that nuclear power has distinct economic advantage over either coal-fired or oil-fired power generation. The economic advantage of nuclear power can also be evidenced by the Table-8, which shows the comparative costs of electric energy produced from new additions of generating facilities fueled by nuclear, coal and oil, respectively, at 1983 price level. The development of nuclear power is indeed an important measure to cope with ROC's energy problem.

It is expected that the weight of electricity (Kwh) generated by nuclear energy will be increased from 39.9% in 1983 to a high of 55.3% in 2000.

TABLE-8 ECONOMIC COMPARISON OF ELECTRIC ENERGY COSTS, NUCLEAR-COAL-OIL

	Nuclear	Coal	Oil
1. Construction Cost US$/Kwh	1,388	882	725
2. Fixed Cost of Capital (%)	12.42	12.36	12.36
3. Fixed Charge (1)x(2) US$/Kw	172.4	109.0	89.6
4. Fuel Cost ¢/Kwh	1.02	2.83	4.64
5. O & M Expenses ¢/Kwh	0.31	0.43	0.21
6. Total Fuel and O & M Exep. (4)+(5) ¢/Kwh	1.33	3.26	4.85
7. Kwh/Kw @65.8% CF for Nuclear @70% CF for fossil	5,476	5,703	5,887
8. Total O & M and Fuel Cost (6)x(7) US$/Kw	72.8	185.9	285.5
9. Total Annual Cost (3)+(8) US$/Kw	245.2	294.9	375.1
10. Total Production Cost (9)÷(7) ¢/Kwh	4.48	5.17	6.37
11. Index	1	1.15	1.42

It is the government's policy to continue the development of nuclear power. The proposed nuclear power development program until 2,000 is as follows:

	Unit Size in MWe	Accumulated Nuclear Installed Capacity in MWe	Year of Commissioning
Nuclear Units 7 and 8	2x1000	7,144	1993, 1994
Nuclear Units 9 and 10	2x1000	9,144	1996, 1997
Nuclear Units 11, 12 and 13	3x1000	12,144	1998,1999,2000

Natural Gas

Proved recoverable reserves of natural gas have not materially changed since 1971 as new discoveries compensate the withdrawals. The reserves in 1971 were 28.6 billion cubic meters as compared with 22.0 billion cubic meters in 1983.

Production of natural gas in Taiwan has been decreasing, since 1977 when 2.0 billion cubic meters has produced and furnished 8.0% of the total energy supply. By 1983, the production decreased to 1.5 billion cubic meters representing only 4.1% of the total energy supply. It has been the government's policy since 1972 to give top priority to use this premium fuel to residential and commercial sectors and to prevent its use for power generation.

The gas production is intended to be maintained at a level of 3.3 million cubic meters (equivalent to 31,500 bbl of oil) per day, if no new gas field is found. Daily consumption in 1991 is projected to be 4.9 million cubic meters, possibly in excess of this estimate if no restriction is made. The gap would be made up by imported liquefied natural gas (LNG) and synthetic natural gas (SNG).

Due to its high cost, use of imported liquified natural gas for power generation appears economically unjustifiable for the time being. However, in the wake of the oil crises of 1970s, the government has changed its policy of LNG. It is conceded that LNG is a long-term secure substitute for domestic natural gas and the importation of LNG is an important step toward the further diversification of imported energy.

Hydroelectric Power

The theoretical hydro power potential in Taiwan is estimated to be 12,000 Mw of which about 5,300 Mw are considered technically feasible. By the end of 1983, 1,431 Mw have already been developed, leaving about 3,900 Mw yet to be exploited. Though the remaining hydro potential to be developed is not capable of furnishing large blocks of continuous power, this indigenous energy resource can be advantageously developed to provide the system with peaking power and to generate as much energy as flow conditions permit.

By the turn of this century, Seven conventional hydroelectric projects with a total of 960 Mw in installed capacity are under construction or design or planning. Two huge pumped-storage projects namely Minghu and Mingtan are not included in the potential estimate as well as in the above conventional hydro development. The former with four 250 Mw reversible units is at the final stage of construction and will be totally completed by January 1986; the latter with six 267 Mw reversible units of which basic design is already completed, will be put on line from December 1991 to

September 1995. With addition of the hydro projects the system operation will become more and more flexible in any period of time.

Geothermal Energy

An island-wide geothermal exploration program is under way. The results of explorations at some geothermal fields in northern Taiwan indicate certain possibility for power development. Two pilot power plants of 1.5 megawatts and 3.0 megawatts were installed in 1977 and 1981, respectively, to determine the characteristic and reserve of this energy resource. Geothermal power development would be favorably considered if and when an economic geothermal field is definitely located.

New Energy Resources

Research and development of the so-called new energy resources such as solar energy, biomass, wind energy, tidal power, OTEC, etc., is considered to be an important approach to reduce the demand for imported energy. A few diverse solar, biomass, wind energy and other technologies are in various stages of development. However, for one reason or another, these resources will not make a significant contribution to the future energy supply at least before 2000.

CONCLUSIONS

A. The indigenous energy resources in the Republic of China on Taiwan are inadequate and energy production is highly restricted. Becuase of this background, energy consumed has been increasingly dependent on imports and the heavy dependence on imported oil could possibly lead to catastrophes to the economic development and the security of the nation.

B. Aware of the vital importance of diversification of energy both in forms and sources, the government has been endeavoring to achieve a reliable supply and is seeking to reduce the dependence on oil through the exploitation of indigenous energy resources, the development of nuclear power and the conversion of oil-fired operation to coal-firing.

C. The justification of nuclear power has well been proved and recognized technically and economically. With the special conditions of Taiwan today and the present costs of oil and coal, the advantages of nuclear power are very great. Thus a nuclear predominant power system will unquestionably make the greatest contribution to ensuring the availability of the cheapest possible electricity to ROC on Taiwan.

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PLANNING AND OPERATION OF RURAL WATER SUPPLY IN DEVELOPING COUNTRIES

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ABSTRACT

This paper discusses some of the problems associated with the planning and operation of water supply schemes in rural areas. In so doing emphasis has been placed on developing countries since rural population constitutes a disproportionate large fraction of total population in such countries. To illustrate points given in the paper some experiences of the authors' own country Tanzania have been cited. It is shown that the major problem in the planning phase of a rural water supply scheme is insufficient and unreliable data. Examples of the data of population, water consumption, and water quality have been used to illustrate some of the problems with reference to planning of a rural water supply. Operation of rural water supply in developing countries suffers from a problem of limited technical know - how. Involvement of the rural population in the operation process has not been very successful. Specific recommendations to improve planning and operation of rural water supply in developing countries have been presented in the paper.

INTRODUCTION

In most developing countries particularly in the African region the rural population constitutes a disproportionate large fraction of the total population. To give but one example the rural population of Tanzania was reported to be about 90 percent in 1978 according to the national census that was carried out in that year. Naturally in developing countries greater emphasis has always been placed on the development of rural water supply schemes. This paper looks into key problems pertaining to the planning and operation of rural water supply schemes and in so doing the author has drawn some examples from his own country, Tanzania. Brief demography of Tanzania could be cited from Gondwe (1983). The experiences obtaining in Tanzania as far as rural water supply is concerned are likely to be similar to those of other developing countries.

In Tanzania a rural center is described as that settlement which has a population not exceeding 5000 people and whose main occupation is agriculture (MAJI, 1982). Normally a rural center would be provided with the essential social services. A settlement of lesser status is termed a village. Often times rural water supply in a developing country refers to water supply for villages. Indeed the definition of rural may differ from country to country.

RURAL WATER SUPPLY

Basically the essential components of an improved rural water supply do not differ from those of an urban water supply. The components are namely the source, intake, storage and transmission. However in some cases depending on the quality of the water at the source and the size of population to be supplied an additional component of treatment is introduced. This component is often placed between the two components namely intake and storage. Rural water supply systems are divided into two main types namely pumped type and gravity type. Groundwater is normally preferred to surface water because of its lesser susceptibility to contamination which makes it relatively safe for domestic use with minimum - or nil - treatment. In developing countries still many rural communities enjoy their water supply from unimproved sources. Village data collected by the Dutch consulting engineers DHV (1980) from their survey of villages in the Morogoro region in eastern Tanzania (see fig.1) has been used to compile Table 1 showing the distribution of types of rural water supplies in that region.

Table 1. Rural water supplies in Morogoro Region, Tanzania

Type	Number	Percentage
Hand-dug and shallow wells	103	32.9
Streams and rivers	165	52.7
Pumped schemes	22	7.0
Gravity schemes	23	7.3
Total	313	100

Morogoro region has relatively better potential of water resources than other regions in Tanzania and yet from Table 1 it is seen that only about 14 percent of all the villages had improved water supplies (pumped and gravity schemes) by 1980. Rural population in Tanzania enjoy improved water supply from public taps with each public tap serving between 200 -

400 people on average.

Efficiency of a rural water supply may be judged from the distance people have to cover to the water source. Village data from Morogoro region, Tanzania (DHV), indicate that only about 20 percent of the villages sampled in that region have their potential water sources within radius of one kilometre. This is more or less consistent with the situation in Mtwara region in southern Tanzania (Gondwe and Msimbe, 1982). According to the World Health Organization (WHO) the radius of reasonable access to the collection point of water is given as 200 metres (Saunders and Warford, 1976). Following the WHO definition then a limited proportion of rural population of a developing country appears to be within radius of reasonable access to water supply. Construction of improved water supplies and provision of public taps shall improve the situation as regard access to water. The quality of the water is an important criteria in the planning of rural domestic water supply. The water should be safe to human health. It is for this reason that international accepted standards for quality of the water have been set by the WHO. Individual nations have set their own standards which are close to the international accepted standards. However, in developing nations the lack of long term investigations and inadequate potential of water resource have lead to adoption of temporary standards which are several times higher than the international accepted limits. With more feed back from rural areas on the impact of water quality on human health the standards shall be improved.

In many contries rural water supply programmes come under the authority of one government Ministry or another. For instance while in Tanzania rural water supply comes under the Ministry of water and energy, in Lesotho in Southern Africa it comes under the Ministry of rural development. The national authority governing the rural water supply may have an impact on the planning and operation of rural water supply.

PROBLEMS

Planning

Data of population is necessary in the planning of any water supply scheme in that it helps in the assessment of the water requirement. One common characteristics of many developing countries is the high growth rate of population which makes estimation of population projections some what difficult. The growth rates of population are highly variable even within a country. For instance in Tanzania the growth rate has been found to vary between 2.0 - 6.0 percent with a national average growth rate of population estimated at 3.3 percent according to the national census of 1978. The growth rate of population could also vary in one rural center due to the nomadic life style of its inhabitants. In addition to this the villagisation programmes that are practised in a number of developing countries have not been well planned at times resulting in great variation of population from one year to another. Thus good plan for a rural water supply can be made only for short term periods of say up to five years.

Data of livestock population is important in planning a rural water supply. In Tanzania the rate of consumption of water for one unit of livestock is given as 25 litres (1) and is given as high as 50 - 75 l/animal for high grade dairy cattle (MAJI, 1982). Often traditional methods of animal husbandry are used by the rural population; animals are made to follow areas with good grass. Consequently the population of livestock for one village may change greatly within a short period. Economic conditions and various animal diseases may force livestock owners to send large number of their herds to the slaughterhouse.

In 1978 about 165 villages out of 360 villages in Morogoro region, Tanzania were indicated to have any livestock. It is indicated by Table 2 that about 50 percent of those villages with livestock had more than 800 heads each. The number of livestock per village is very much variable. This is examplified in Table 3 compiled from the village data of Morogoro region, Tanzania (DHV, 1978). Coefficient of variation of livestock population for 1978 was about 271 percent ! Tindiga village had the maximum ratio of livestock population to human population of approximately 10. According to rates in use in Tanzania the classical computation of daily water requirement for Tindiga village would be as follow;

Population 3941 at 30 l/capital		= 118,230 l
Livestock 39394 at 25 l/head		= 984,550 l
	Total	= 1103,080 l

It is noted from this simple calculation that 89 percent of estimated total daily demand would be due to livestock population. In Tanzania population of livestock is expected to increase by 50 percent in twenty years. That means by the year 2000 Tindiga village would have a daily water requirement of over 2 million litres. But it is likely the livestock population of the village of Tindiga may decrease several time and hence straight population projection could lead to gross overdesigning of a rural water supply.

Having known the population of a community one needs to determine the rates of water consumption for different categories of people and livestock to be able to compute the water requirement. Unfortunately water consumption rates are not determined easily due to the fact that people tend to adjust their consumption of water to the available yield and hence even in a country there is bound to be a marked variation of the rates. For planning purposes the rate of rural water consumption in Tanzania is given as 25 - 30 lcd (litres per capita per day) MAJI, 1982). Many researchers on rural water supplies in the East African region have given estimates of water consumption between 15 - 35 lcd and thus it would appear that Tanzania is using reasonable estimates in planning her rural water supplies.

The author is of the opinion that the estimate of water consumption of 25 - 30 lcd is on the low side. Definitely if a kiosk or public tap having a reliable supply was provided within a reasonable access the consumption rate would increase. The containers used to fetch water in rural areas have capacities ranging from 15-20 litres. For one proper bath a person may use water from a full containter. Considering the climate prevailing in most developing countries people would prefer to take at least one bath daily and also they would drink a lot of water. And if sanitation is to be maintained in the rural areas a lot more water than 30 litres per person per day would be needed. The low estimates of water consumption adopted for rural areas in developing countries are due to the method of their estimation which is based on the ratio of yield to the population. This method could give very misleading estimate as Finnwater (1977) clearly pointed out for some areas of Mtwara region in South-East Tanzania where values as low as 8-10 lcd were determined. For a village with abundant water supply one could get much higher estimate of consumption of water using the yield - population ration method.

The water quality of the source has an impact on any water supply system. The two substances commonly occuring in water and that have harmful effects on human health are fluorides and nitrates. Often times the quality of water from the rural water sources is way out of international accepted standards. If these standards were to be strictly adhered to, a number of sources would have to be abandoned. For instance it was found that approximately 45 percent of the sources in Arusha region in northern Tanzania have flouride content of 1.5 mg/L (Bardecki, 1974) whilst the international accepted limit, is 0.8 mg/L.

Thus the international accepted limits are a constraint in planning of rural water supplies. Like many other countries, Tanzania has set its own temporary limits of domestic water quality. For example the Tanzanian limit for flouride is 8 mg/L (MAJI, 1974) which is ten times the international accepted one. The temporary standards have some limits several times higher than the international accepted limits in order to allow exploitation of more potential water sources. Usually a water resources engineer in a developing country has very scanty hydrologic data for the evaluation of potential water resource of a certain rural area. And for this reason a good number of completed schemes fail shortly after their commissioning. In many places in Africa a number of wells and of reservoirs are reported to have dried up. Several hundred shallow wells which were sunk by Finnish experts in Masasi district in south-east Tanzania between 1973-1975 were reported to have dried up in 1981.

Finally in planning of a rural water supply it is preferable that the people themselves be involved. Often time the villagers just see a team of technicians moving about with strange instruments and equipment over their land. The people are rarely informed on schemes which are supposed to serve them. If a livestock keeper sees that a pipeline passes through his village without any provision of watering troughs for animals, he or she is likely to damage the pipe in order to get some water. This has happended in some places especially in areas which have many livestock units.

Operation

The policy of involving the rural population in the operation of a water supply system has met with mixed success. Many people take for granted that a rural water supply technology is necessarily a very simple one. Some rural centers with a population of over 1000 people may involve scales of technology similar to those utilized in urban centers complete with a treatment plant. A rural person with no sound basis in hydraulics, pump mechanics and treatment processes shall be unable to operate such a water supply system. The experts or technicians of the water department or water authority in many country are based in district centers and due to limitation of logistics they are not able to visit an appreciable number of rural water supply schemes in their district. Consequently poor operation by the villagers result in early breakdown of a completed water supply scheme.

The wide assortment of technologies available for rural water supply systems also cause some operational problems. In most developing countries you normally have a situation where a number of consulting engineering firms from different countries are involved in drafting of regional water master plans and in some cases, in the implementation of the plans. Each consulting engineering firm introduces its own technology with a result that the country ends up with a number of alternative technologies. This assortment of technology may be too much for rural population to grasp. Tanzania has over five regional water master plans and each plan is managed by a different consulting engineering firm. Thus it is desirable that the training of operators for rural water supply be loclised. Economic situation of developing countries do not allow for proper operation of rural water supply. The budget allocated for water supply development is normally relatively small although many developing countries have set a target to supply clean water to all their people by the year 2000. Tanzania annual budget allocation to water resource development is approximately half of what it should be if the target of supplying clean water to all the people by 2000 is to be met (Kassum, 1981).

RECOMMENDATIONS

It is necessary to give a new definition of the term 'rural' as is applied on developing countries. The definition should be more practical in that it should indicate the size of population, level of social services and income level. As it is some of the centers termed rural may have water consumption levels similar to urban centers. Planning should not be based on estimates of population growth of people and livestock alone. Anticipated increase of affluence of the rural population must be taken into account when making projections of future population.

Mere transfer of estimates of rate of water consumption from one region to another region should be discouraged. Transfer of such a variable should only be permitted for areas with similar climatic and social patterns Realistic estimates of water consumption or a region must be determined. Nearly all developing countries could mount research programmes to determine the water consumption rate. Training institutes, colleges and universities could be used to conduct such research.

Villagisation programmes need to be pursued coutiously to avoid possibility of moving people and their livestock to a place where the water resource potential is very limited. Otherwise people tend to wander about looking for water and hence assume nomadic lifestly. It is normally better planning to move people nearer to the water than vice versa. People should be encouraged to keep manageable sizes of livestock herds so as to limit the water requirement and maintain it at a more or less steady level. Adequate number of watering troughs for animals have to be provided in order to avoid deliberate tempering of pipes by irate herders.

Developing countries should research into the effect of water quality on the health of their people. Many such countries have set their own temporary water standards and it appears the main criteria of setting up these standards was to allow exploitation of greater number of water source than would be possible otherwise. Research would be necessary to clear any lingering doubts on the suitability of the adopted temporary standards. All the same it is to be recommended that the national temporary standards should not vary very much from the WHO international accepted standards.

In the case of operation it is desirable that a country should have a uniform technology for its rural water supply programme. This will limit confusion on the rural population most of whom can not interpret a technical mannual properly. Low level training institutes should be established to train craftsment selected from different villages to operate and maintain rural water supply schemes. Tanzania in collaboration with Seden through its International Development Agency (SIDA) proposed such zonal institutes for water staff (Daily News, February 24, 1981) be set up.

Lastly developing countries should allocated realistic budget for water development improvement to water supply. This is a good national investment since a healthy nation implies a cut of health bill and an increased productivity.

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Table 2 Number of Villages in Morogoro Region, Tanzania, with the Indicated Population

Population Range	Villages Human	%	Livestock	%
200		100	82	99.9
200 - 600	19	99.9	16	50.3
600 - 1000	81	94.7	19	40.6
1000- 14000	83	72.2	4	29.1
1400- 1800	73	49.2	10	26.7
1800- 2200	37	28.9	30	20.6
2200- 2600	31	18.6	7	18.8
2600-3000	16	10.0	5	14.5
3000- 3400	6	5.5	2	11.5
3400 -3800	7	3.9	0	10.3
3800	7	1.9	17	10.3
	360		165	

Table 3 Population Extremes and Population Statistics of villages in Morogoro Region, Tanzania

Village	People	Livestock	Ratio
Ruaha	7369(max)	14	0.002
Seregate - A	380(min)	Nil	0
Tindiga	3941	39394(max	10
mean population	1550*	1650**	
standard deriation	880	4470	
6n - 1 ±			
Coeff. variation%	57	271	

* Data from 360 villages

** Data from 165 villages. Mean becomes 760 if all 360 villages considered.

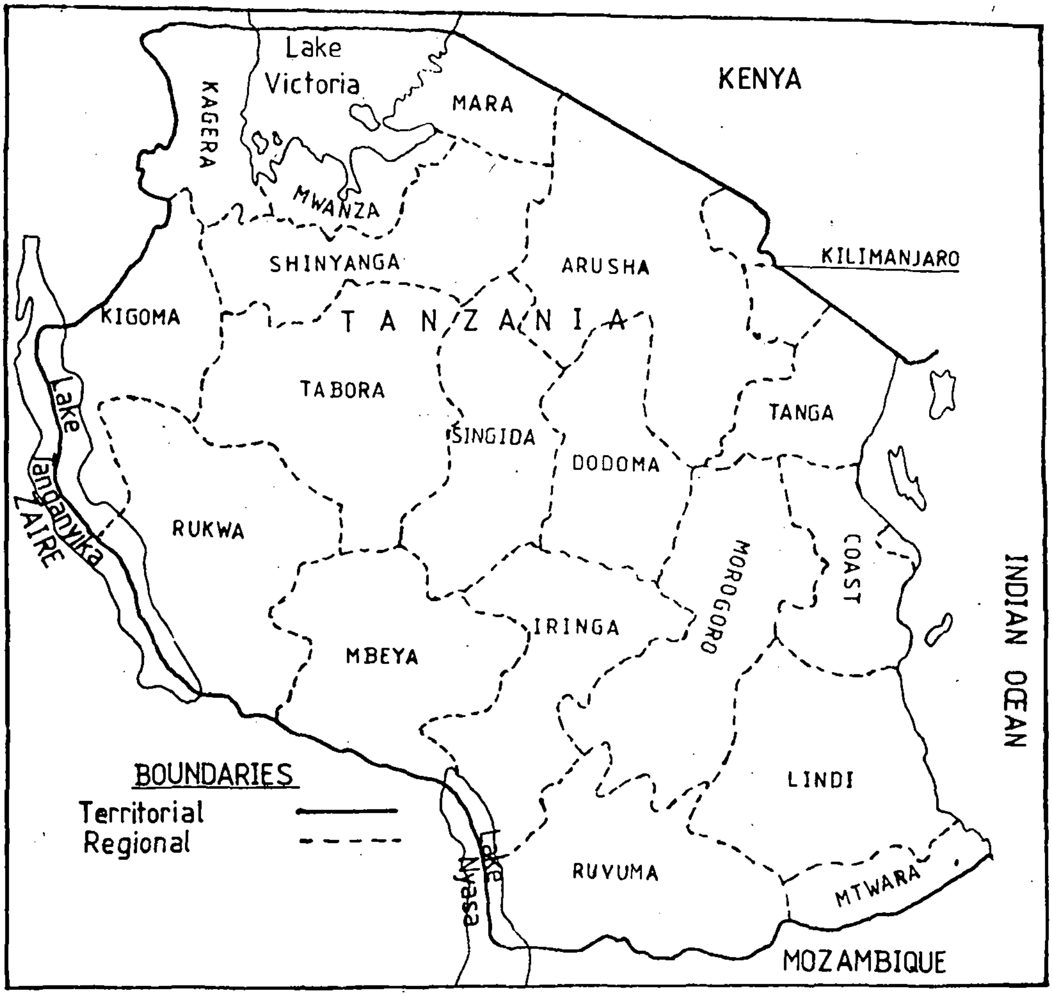

Fig. 1 United Republic of Tanzania

Administrative regions

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L'ACTION DE RECHERCHE PLURIDISCIPLINAIRE DU GROUPE PIREN-EAU EN ALSACE: CONTRIBUTION D'UN PARTENAIRE SCIENTIFIQUE A LA DEFINITION D'OBJECTIFS REGIONAUX DE GESTION DE L'EAU

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RESUME

Dans le cadre de son programme interdisciplinaire de recherche sur l'environnement (PIREN) le Centre National de la Recherche Scientifique (CNRS) développe depuis 1979 des actions de recherche appliquées à la gestion écologique des ressources en eau.

Le Groupe PIREN-Eau en Alsace inscrit son action de recherche dans les objectifs suivants :

- aide à la décision en matière de gestion des ressources en eau ;
- connaissance des mécanismes d'hydrosystèmes spécifiques en Alsace à partir d'une analyse approfondie des divers éléments et de leurs relations évolutives mettant en jeu des transferts de matière, des organisations de vie, des comportements humains.

Sur un modèle représentatif (couple rivière Fecht - nappe phréatique autour du Ried Central de l'Ill) les acquis se situent au niveau de

- la constitution de la ressource Eau au plan quantitatif et l'alimentation de la nappe phréatique ;
- la qualité physico-chimique des eaux, les flux de matière au niveau du sol, les mouvements et échanges dans l'aquifère et les transferts de polluants ;
- la qualité biologique des eaux, les fonctions bioindicatrices d'espèces aquatiques témoins, les impacts sur l'environnement et sur la santé ;
- les usages et enjeux, l'approche analytique pour une gestion globale des ressources en eau.

INTRODUCTION

Dans l'objectif de préserver la qualité de l'eau en Alsace, compte tenu de ses fonctions dans le cadre régional, un groupe pluridisciplinaire recouvrant les aspects physico-chimiques, biologiques, agronomiques, sociologiques, économiques et juridiques, s'est constitué avec l'aide du Centre National de la Recherche Scientifique (Programme Interdisciplinaire de Recherche sur l'Environnement : PIREN) et le soutien du Ministère de l'Environnement et de la Région Alsace.

Le système alluvionnaire rhénan confère à l'Alsace une richesse quasi inépuisable en eau souterraine (record national d'utilisation avec un prélèvement de plus de 14 m^3/s). La ressource est cependant peu protégée car la nappe phréatique est proche du sol et les terrains de couverture peu perméables sont rares. De plus, les interférences entre réseau hydrographique et nappe ajoutent à la vulnérabilité, le système interactif eau de surface - eau souterraine conduisant à une accélération des transferts (transferts en masses d'eau et transferts de polluants).

L'atout économique essentiel que constitue la nappe phréatique pour la Région fournit, du fait de sa facilité d'accès notamment, tous les ans une rente de situation à préserver dans le cadre d'une gestion qui prend en compte le système EAU dans son ensemble : approvisionnement et constitution de la ressource, mouvement et échanges dans le réservoir, transferts de polluants, impacts sur l'environnement et sur la santé.

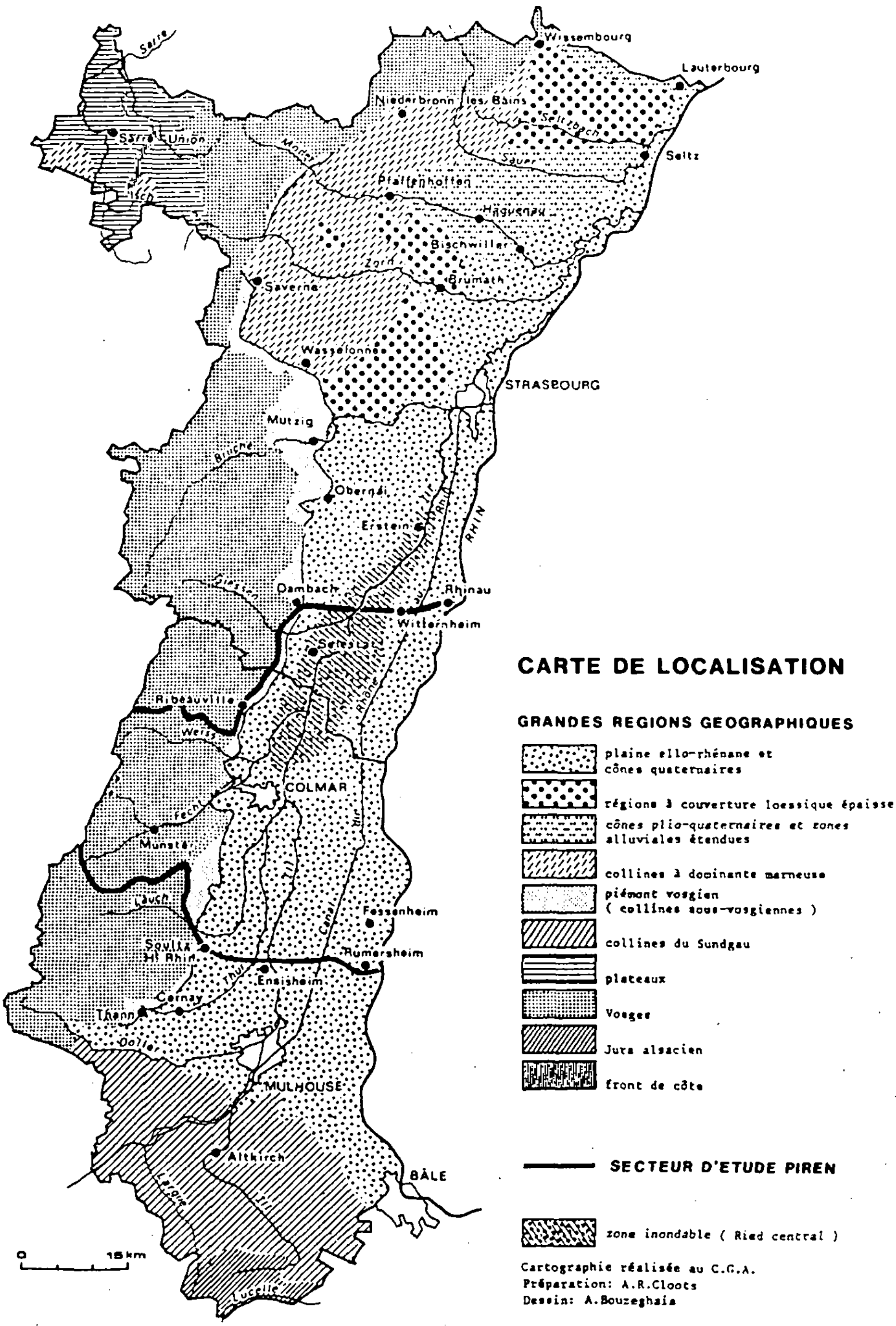
L'IMPORTANCE DU CHOIX D'UN MODELE D'ETUDE - CONNAISSANCE DE L'HYDROSYSTEME

L'ensemble choisi "rivière Fecht - nappe phréatique", secteur de moyenne Alsace (voir carte de localisation), recouvre l'essentiel du "bassin de l'Ill domaniale", englobant le bassin de la Fecht qui, lors de crues, apporte jusqu'à 30% des débits de l'Ill à Illhaeusern (confluence de la Fecht et de l'Ill).

Ce secteur, bien représentatif dans le contexte alsacien par la variété de ses milieux, comprend la gamme quasi complète des "zones d'intérêt écologiques":

(I) domaines d'intérêt climatique sous influence de variations de relief et d'espaces boisés, favorables à des circulations d'air locales dans une situation générale dominée par des conditions atmosphériques critiques (tels : forts gradients de précipitations de la crête des Vosges à la Plaine ; inversion des températures dans le "fossé rhénan" ; ...) concentrant une pollution atmosphérique aux impacts multiples ;

(II) milieu naturel d'intérêt particulier avec de précieux biotopes où il convient de ne pas "trop perturber" les conditions écologiques (niveau de la nappe, exploitation agricole et forestière...) et où coexistent des zones intensément cultivées et des zones proches de "l'état naturel". Suite aux variations locales de la nature du sol (et du sous-sol), de ses conditions hydriques, de la couverture végétale, il s'y est maintenue une importante variété d'espèces animales et végétales ;



CARTE DE LOCALISATION

(III) domaines d'intérêt hydrologique, hydrogéologique et hydrodynamique avec trois grandes unités hydrologiques et l'importante nappe phréatique :

- l'unité "vallée vosgienne" où le substrat granitique est recouvert de formations superficielles généralement assez épaisses et perméables ;

- l'unité "cône de déjection", au matériel fluvio-glaciaire très filtrant ;

- la "zone humide" (Ried) de l'ancien lit majeur exceptionnel de l'Ill et du Rhin où les alluvions rhénanes (bien perméables) sont recouvertes de limons et où l'eau souterraine est proche de la surface du sol ;

- la nappe phréatique rhénane - étroitement liée au réseau hydrographique (transferts et mécanismes d'inondation) - dont les niveaux et la qualité sont également corrélés avec la végétation en général (cultures, prairies, forêts), les pratiques culturales (modes d'irrigation), les aménagements des cours d'eau (à l'exemple de la frange rhénane où est particulièrement mise en relief l'incidence de la médiocre qualité de l'eau par de multiples "indicateurs").

Sur ce secteur, un constat de croissance des "situations d'atteinte à l'écosystème" - notamment en ce qui concerne le "vecteur Eau" - conduit à évoquer des problèmes comme :
. la pollution atmosphérique (pluies acides, pluies chargées en sels) ;
. les risques pouvant résulter d'aménagements hydrauliques (atteintes au régime des eaux, diminution de l'infiltration vers la nappe avec abaissement des niveaux et altération de la qualité par réduction des dilutions, gravières devenant des décharges...)
. les atteintes aux sols (imperméabilisation de grandes surfaces, érosion des sols nus, irrigation avec des eaux de mauvaise qualité...)
. les perturbations de biotopes (impacts des pollutions, destruction de berges et de fonds de cours d'eau, incidence d'épandages de crues sur la faune et la flore...).

Ces situations, qui sont déjà conflictuelles pour certaines et peuvent le devenir pour d'autres, nécessitent une analyse approfondie qui puisse :

* mettre à jour les influences réciproques entre activités humaines et données de l'environnement naturel,

* permettre de déterminer les "seuils acceptables" des atteintes à l'environnement,

* développer prioritairement la connaissance des interrelations multiples de l'hydrosystème et des facteurs naturels (relief, climat, végétation, sol, sous-sol...).

Pour progresser dans la connaissance de l'hydrosystème choisi, permettant :

- d'approfondir sa valeur économique et le coût social et écologique de sa détérioration,
- de mettre en oeuvre des méthodes de protection et de gestion de la ressource eau, compatibles avec "l'optimum économique" et "l'impératif écologique",

le groupe PIREN-Eau/Alsace a réussi, petit à petit, avec d'autres partenaires régionaux à montrer qu'il n'est plus possible d'oublier les conséquences à long terme dans tout projet technique d'aménagement (les travaux réalisés et les résultats déjà obtenus qui sont présentés ici, très succinctement, ont fait l'objet d'un important effort de publication dans des revues spécialisées et de communications à des colloques).

CONSTITUTION DE LA RESSOURCE EAU AU PLAN QUANTITATIF - ALIMENTATION DE LA NAPPE PHREATIQUE.

L'importance de recherches hydrologiques poussées dans le secteur choisi a été illustrée par 2 évènements récents (2 crues exceptionnelles en avril et mai 1983) montrant parfaitement la complexité des crues et des facteurs qui les contrôlent dans le bassin de l'Ill, soumis par ailleurs à des étiages souvent marqués.

Après 3 années, les travaux effectués ont déjà permis de préciser certains mécanismes en jeu, leur régime, leur distribution spatiale ainsi que leurs interactions.

Un premier ensemble de recherches a été consacré à une meilleure connaissance du contexte climatique et hydromorphologique du secteur, et notamment des variations spatiales des précipitations, de l'évapotranspiration et des propriétés hydriques des sols très importantes dans cette région fortement contrastée.

A partir de ces recherches sur les variations spatiales des composantes du cycle hydrologique (NAJJAR et al. 1981, AMBROISE et al. 1982, AMBROISE et VIVILLE 1984), une approche par bilan hydrologique détaillé, facile à informatiser, a été testé dans le haut bassin. Elle a permis de montrer qu'environ 90% de l'écoulement de crue se produit en saison froide, de confirmer le rôle fondamental des premiers horizons du sol dans la redistribution des précipitations et de conclure à une évaporation réelle proche de l'évaporation potentielle en année "normale" dans ce milieu.

Tirant parti de ces résultats, un deuxième ensemble de recherches a porté plus spécifiquement sur les processus hydrologiques encore très mal connus que sont, d'amont en aval, la formation et la propagation des crues, les échanges rivières-nappe dans les cônes et les interactions Ill-nappe-rivières phréatiques, notamment lors des inondations.

Pour l'analyse des débits, une chaîne de traitement informatisé a été spécialement mise au point, permettant de saisir, stocker et traiter les données hydrologiques et d'établir des fiches synthétiques et des graphiques pour tout épisode caractéristique. Cette analyse en cours porte surtout sur les fortes crues.
Les volumes de crues importants fournis par le haut bassin sont rapidement évacués par un réseau de drainage très développé. Une partie non négligeable s'infiltre dans la nappe au niveau des cônes de déjection : globalement il s'infiltre vers la nappe au travers de $1m^2$ de lit fluvial environ 1000 fois plus d'eau qu'au travers de $1m^2$ de sol dans ce secteur.

Le Ried central de l'Ill correspond à une cuvette subsidente (zone d'affaissement lent) à l'entrée de laquelle la rivière se divise en diffluences. La cartographie des différents éléments qui composent le contexte hydromorphologique (terrasses, levées, chenaux, ...) a permis d'établir une typologie des drains naturels qui provoquent et propagent les inondations.

Une carte des inondations a été dressée pour le Ried central de l'Ill à partir de trois évènements caractéristiques : une inondation d'automne (1978), une inondation d'hiver (1982) et une inondation de printemps exceptionnelle (1983).

Enfin pour analyser plus finement les échanges complexes Ill-nappe-rivières phréatiques et tenter de les modéliser mathématiquement un secteur test, situé au début de la zone inondable, a été choisi.

QUALITE PHYSICO-CHIMIQUE DES EAUX - FLUX DE MATIERE AU NIVEAU DU SOL - MOUVEMENTS ET ECHANGES DANS L'AQUIFERE - TRANSFERTS DE POLLUANTS.

Dans l'aquifère rhénan, l'eau souterraine est marquée par les différents types d'apports liés à l'activité humaine (apports ponctuels ou diffus, accidentels ou chroniques, tolérés ou clandestins). Toutes ces perturbations de ce que serait l'état hydrochimique normal de la nappe, n'ont pas la même importance, mais elles participent à la détérioration progressive de la qualité de l'eau, par effets cumulatifs.

Les résultats concernent l'étude de la variation et des incidences de la qualité physico-chimique des eaux dans le système interactif alsacien, d'amont en aval au sens large : des eaux de pluies aux eaux de nappe, du bassin vosgien à la plaine d'Alsace et des données de terrain aux modèles expérimentaux et théoriques.

Dans le secteur vosgien, les eaux de pluie sont nettement marquées par une pollution acide et nitrée. En plaine, l'analyse des eaux de pluie sur un axe Sud-Nord a mis en évidence un phénomène d'ampleur inattendue quant aux retombées de sels en solution (FRITZ et SCHENCK, 1984).

L'évaluation des flux d'entrée et de sortie (sources et cours d'eau) des éléments chimiques dans le bassin amont (granitique), a montré qu'il y a superposition de deux types de chimismes : altération normale des minéraux, pollution acide des précipitations (FRITZ et al., 1984).

Dans la nappe en plaine, au-delà d'une confirmation du phénomène de salinisation déjà connu, l'étude des caractéristiques physico-chimiques des eaux phréatiques a montré que cette salinité n'est pas inerte. La fixation de sodium par échange ionique avec les minéraux argileux de l'aquifère est conforme aux prévisions réalisées par une modélisation de l'interaction eau phréatique "dopée" en sodium-argiles sur une base thermodynamique (FRITZ, 1981). Le phénomène est d'ampleur régionale et doit concerner actuellement des millions de tonnes de sodium.

Les observations et les mesures effectuées in situ dans le cadre d'un réseau permanent de 15 puits d'irrigation en plaine, ainsi que les données expérimentales réunies de 1977 à 1982 (cette recherche avait démarré au plan régional avant le programme PIREN), ont conduit à énoncer des résultats d'ordre hydrologique, agrologique et phytotechnique (SCHENCK, 1984).

En laboratoire, les expériences réalisées ont permis de caractériser les sols testés par leur "courbe de sodisation".

Sur un secteur-test, les résultats d'analyses des teneurs en nitrates des eaux au toit de la nappe phréatique, ont nettement montré que l'infiltration des nitrates est maximale sous les zones mises en exploitation agricole (20-25 mg/l NO_3^-) et minimale sous les forêts (5-10 mg/l NO_3^-).

La diffusion des nitrates dans l'aquifère semble devoir affecter peu à peu toute la nappe : cette évolution est inquiétante.

Si la vulnérabilité du réservoir phréatique alsacien a été très nettement mise en évidence par la dynamique des grandes perturbations quantitatives de type salines, le risque que représentent les micropolluants toxiques, même si leurs impacts restent localisés, ne peut être oublié. Dans le cadre du programme de recherche et compte tenu des risques potentiels dans la région, les travaux portent actuellement sur 2 éléments (métallique et organique) : le mercure (MARTAUD et DINH, 1983 ; BEHRA et al., 1984) et les hydrocarbures (ZILLIOX, 1984).

La modélisation des propagations de polluants dans l'aquifère est réalisée à partir de modèles déjà existants (VANÇON, 1984) et qui subissent, dans le cadre des recherches PIREN, des ajustement qui leur permettent de mieux rendre compte de la complexité de l'aquifère des alluvions du Rhin.

QUALITE BIOLOGIQUE DES EAUX - FONCTIONS BIOINDICATRICES D'ESPECES AQUATIQUES TEMOINS - IMPACTS SUR L'ENVIRONNEMENT ET SUR LA SANTE.

Le périmètre d'étude choisi présente l'intérêt exceptionnel d'aborder l'étude qualitative de la partie supérieure de la nappe phréatique par les biais d'écosystèmes complets et de révéler, grâce aux fonctions intégratrices et amplificatrices d'indicateurs biologiques (faune, flore), des pollutions non détectées directement par des analyses d'eau.

Les variations hydroécologiques du "modèle naturel" rivières phréatiques et secteurs corrélés de la nappe se traduisent en effet dans les modes d'agencement dans l'espace et l'évolution dans le temps de séquences de groupements végétaux, dans la composition et la répartition de populations microbiologiques (l'accent étant mis sur des taxa potentiellement pathogènes) et dans les réactions physiologiques d'animaux aquatiques témoins (situés à un niveau élevé des chaînes trophiques).

L'étude des peuplements de macrophytes des rivières phréatiques (CARBIENER 1983) a montré l'eutrophisation diffuse de la nappe sous influence de l'Ill. Dans certaines rivières, elle a clairement mis en évidence les effets d'eutrophisation brusques, suite à des déversements d'effluents polluants. A proximité du Rhin, les groupements végétaux (en tant que "traceur hydrologique") témoignent de la contamination des eaux phréatiques par les filtrats rhénans. Ces bioindicateurs participent à la caractérisation des subdivision géochimiques de la nappe et des rivières : la sensibilité très grande du biorévélateur phytosociologique a été démontrée par CARBIENER et KAPP (1981).

La recherche de micro-organismes dans l'environnement aquatique (avec étude de l'influence sur la santé de l'homme) concerne essentiellement les eaux phréatiques superficielles ; elle est complétée, en profondeur, dans des forages sous diverses influences (urbaines, industrielles ...). Ces travaux font l'objet d'une communication au Congrès AIRE de Bruxelles (DERR-HARF et al., 1985).

La perception de la qualité biologique des eaux passe par une connaissance intime des mécanismes écophysiologiques. Les résultats obtenus concernent l'étude (sur le terrain) de différentes caractéristiques physico-chimiques du milieu aquatique et (au laboratoire) de leurs conséquences physiologiques chez la truite et l'écrevisse. L'analyse des caractéristiques respiratoires des eaux phréatiques a permis de montrer leur évolution circannuelle sur un transect nappe phréatique (résurgence) - Rhin. (MASSABUAU et FRITZ,1984).

L'étude des effets de la composition des eaux en sels minéraux sur la truite a montré
- que, malgré des différences importantes dans la minéralisation de l'eau, l'équilibre ionique aussi bien au niveau branchial qu'au niveau intestinal (DI COSTANZO et al. 1983 ; LERAY et FLORENTZ, 1983) est remarquablement stable;
- que la prolifération de cellules "à chlorure" (LAURENT, 1984) est fortement stimulée dans les eaux faiblement minéralisées ; lorsque les eaux sont enrichies en NaCl, on observe une régression du nombre de ces cellules, ce qui montre que ces ions jouent un rôle important sur la structure branchiale.

Parmi les facteurs susceptibles de modifier l'équilibre acide - base du sang chez l'écrevisse, la recherche a porté sur les mécanismes d'action de l'oxygénation (DEJOURS et ARMAND, 1983 ; MASSABUAU et al., 1984), de la température (DEJOURS et ARMAND, 1983), des variations de l'équilibre acide - base de l'eau ainsi que de la chlorinité (DEJOURS et al., 1982).

Les travaux sur la régulation respiratoire de l'écrevisse ont montré que la ventilation branchiale joue un rôle fondamental dans le maintien du métabolisme aérobie : elle augmente avec une baisse de l'oxygénation et semble ajustée de façon à maintenir sensiblement constant le niveau d'oxygénation dans les cavités branchiales (MASSABUAU et BURTIN, 1984).

USAGES ET ENJEUX - APPROCHE ANALYTIQUE POUR UNE GESTION GLOBALE DES RESSOURCES EN EAU

Une première série d'investigations concerne des données de portée générale sur l'Alsace (Moyenne Alsace surtout) confrontées aux résultats obtenus dans les autres régions rhénanes (Pays de Bade,Palatinat méridional et Suisse du Nord Ouest).

L'affinement de l'analyse à la lumière de cette expérience et de la définition des variantes d'un périmètre relativement vaste, favorise une démarche visant à faire ressortir les aspects humains, sociaux et économiques de l'usage de l'eau dans une portion de territoire alsacien qui répond grosso modo au profil d'évolution de ses principaux types de relief : massif vosgien, vallée de pénétration multimodale, piedmont, plaine, façade rhénane. Le périmètre d'études PIREN-Eau/Alsace (cf.carte de localisation) est spatialement d'autant plus intéressant qu'il s'insère dans le mouvement de consommation de l'eau avec un profil conforme aux diverses étapes de mise en valeur des paysages alsaciens. Sur un secteur test, une comparaison entre situations juridiques et situations réelles est entreprise.
L'apparition des signes de mutation dans la nature des exigences socio-économiques en matière d'eau modifie petit à petit les relations sociales et la dynamique économique, secouant les structures matérielles et mentales, bousculant les habitudes culturelles prises. L'eau finit par être englobée dans l'appropriation ou la revendication générale. Les agriculteurs veillent toujours davantage aux réserves d'eau pour des raisons économiques, surtout dans la plaine où certains associent déjà à ce souci des préoccupations écologiques. L'école et la vie associative jouent un rôle déterminant dans tous les domaines écologiques, y compris celui de l'eau.

Si l'eau, à travers sa disposition et sa distribution, permet d'être à la portée de tous, elle permet davantage encore de mettre en valeur les conflits qui opposent des usagers aux intérêts divergents, conflits qui trouveront réponse dans un compromis politique : parler de la gestion de l'eau c'est parler des solidarités et, à travers elles, de la gestion politique de ces solidarités (FROEHLICHER et al., 1983).

L'élaboration d'une politique régionale de l'eau exigerait la définition d'un système de compensations sur la base d'une comptabilité précise du milieu. Dans ce sens, la reprise et la réactualisation de l' "Etude économique des inondations dans le bassin de l'Ill" réalisée par le Bureau Central d'Etude pour les Equipements d'Outre-Mer (BCEOM) en 1972 est effectuée et complétée par des enquêtes sur le terrain.

CONCLUSION

Compte tenu du programme défini en juin 1983 dans le cadre du schéma régional d'aménagement des eaux - Alsace et de la concertation, en février 1984, avec le Comité Technique de l'Eau, la Direction Régionale de l'Industrie et de la Recherche et le chargé de mission "Environnement" du Conseil Régional d'Alsace, il a été établi un recouvrement entre les compétences du Groupe pluridisciplinaire de recherche PIREN-Eau/Alsace et les préoccupations de la Région.

Pour s'assurer la maîtrise de la dynamique des transferts "atmosphère-sols-eaux de surface-eaux souterraines" (en quantité et en qualité) et de leurs impacts sur l'environnement, il est fondamental de développer la connaissance des mécanismes complexes en jeu au niveau des hydrosystèmes.

Des études techniques (constats de situation, réalisations localisées d'équipements d'exploitation, de valorisation, de protection ...) doivent être effectuées en général à court terme ; mais progressivement, l'éclairage scientifique aidant, ces études doivent évoluer vers des projets d'aménagement prenant en compte un maximum d'intérêts, y compris ceux garantissant à très long terme la conservation du milieu EAU.

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REVISION DE LA GESTION D'UN RESERVOIR DESTINE A IRRIGUER DES TERRES ET A LES PROTEGER CONTRE LES DEGATS DES CRUES

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RESUME

Le cas concret présenté concerne une région du Sud-Est de la France, qui est très souvent touchée par la sécheresse pendant les mois d'été. Tout un système de canaux très anciens assure l'irrigation de 7000 ha de vergers et de cultures maraîchères. Pour remédier au manque d'eau, il a été créé un réservoir à objectif multiple : écrêtement des crues et fourniture d'eau pour les besoins d'irrigation. L'aspect contradictoire de ces deux objectifs impose pratiquement une gestion annuelle, de façon à ce que la retenue soit vide en hiver et pleine au début de la saison sèche. Pour résoudre ce problème, on a tenu compte de différentes valeurs des débits réservés et de ceux nécessaires aux canaux, d'un apport supplémentaire éventuel provenant d'une retenue plus en amont et de coefficients mensuels pondérateurs du risque de crue ; ceci conduit à une courbe d'objectifs de remplissage que le gestionnaire du barrage doit s'efforcer de suivre en fonction des apports journaliers de la rivière, afin de satisfaire les besoins d'irrigation avec un taux acceptable de défaillance - inférieur à 20 % - tout en laissant le creux le plus intéressant possible pour l'écrêtement des crues. Ultérieurement, une prévision des crues sera mise en place pour améliorer la gestion du creux.

La méthode proposée pour déterminer les courbes d'objectifs de remplissage se veut résolument plus simple que les méthodes habituelles issues des applications de l'algèbre linéaire à la recherche opérationnelle.

Mots cles : ressources en eau, gestion de réservoir, réservoir.

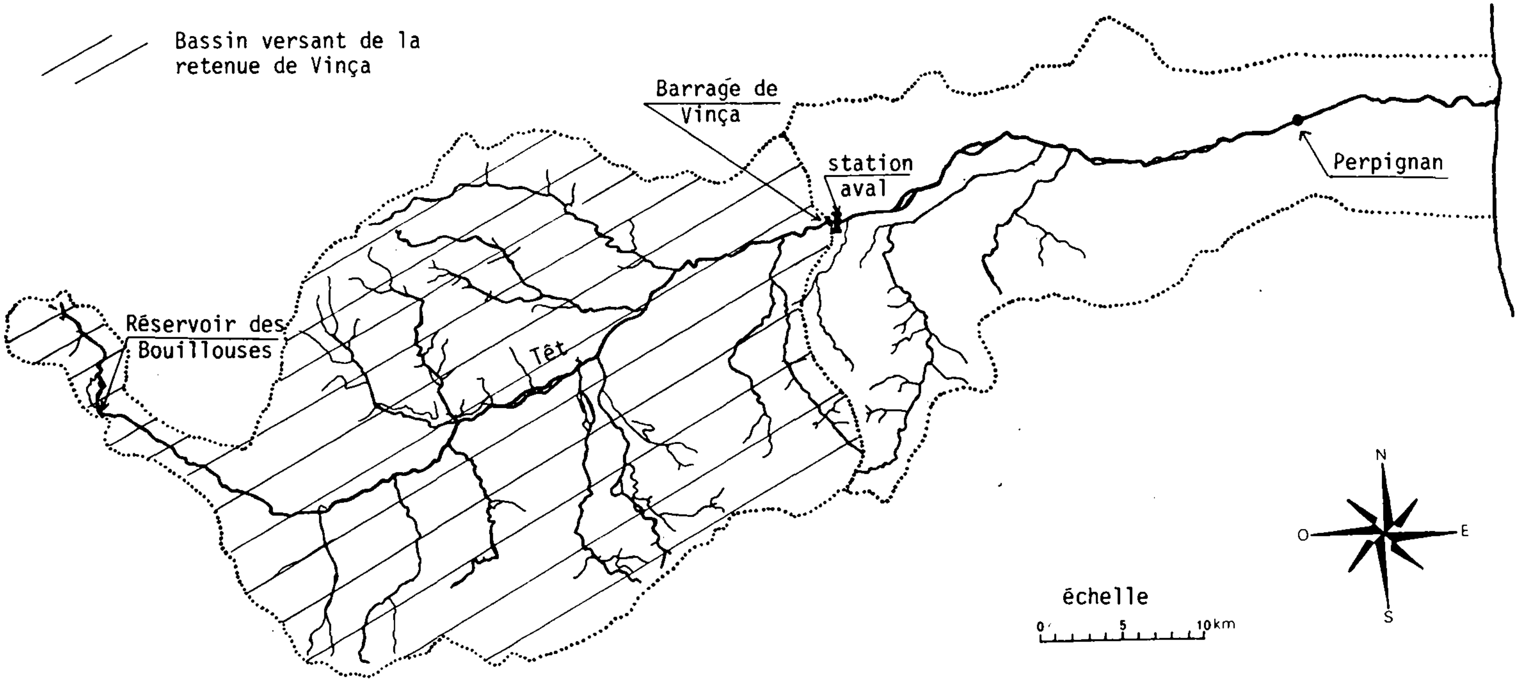
INTRODUCTION

Le barrage-réservoir de Vinça, situé sur la Têt, en amont de PERPIGNAN dans les Pyrénées Orientales (France) (voir fig. 1), est à *objectifs multiples : satisfaire en priorité les besoins d'irrigation* des cultivateurs de la vallée de la Têt,et écrêter les crues de moyennes fréquences /BENECH, 1968/. La gestion de ce barrage distingue trois périodes dans l'année : remplissage du 1er janvier au 30 juin, soutirage du 1er juillet au 30 septembre, vidange du 1er octobre au 31 décembre. Pendant la période de soutirage, la retenue reste pratiquement vide pour l'écrêtement des crues. Actuellement, pendant la période de remplissage, on ferme pratiquement les vannes de fond pour remplir la retenue en maintenant cependant un débit réservé dans la rivière et certains canaux ; pendant la période de soutirage, le réservoir permet de compléter le débit naturel pour satisfaire les besoins mais il n'y a pas vraiment de consignes très précises fournies au gestionnaire pour ces deux périodes. Nous avons proposé de préciser la gestion de ces deux premières périodes en établissant une courbe d'objectifs de remplissage - destockage qui indique au gestionnaire les niveaux d'eau à maintenir dans la retenue en fonction de la date. Plusieurs possibilités d'utilisation de l'apport disponible de la retenue des Bouillouses, située plus en amont, ont été testées. Enfin, le cas problématique des années de faible enneigement a dû être étudié à part.

DONNEES

Rappelons que l'on s'intéresse essentiellement aux périodes de remplissage et de soutirage. A la demande des agriculteurs, on a envisagé l'extension de la deuxième période jusqu'au 15 octobre. Pour quantifier les apports, nous avons utilisé les chroniques de débits mensuels sur 40 ans et journaliers sur 7 ans à la station hydrométrique de Vinça (reportée un peu plus loin en aval depuis la mise en eau). Quant aux besoins, ils sont de deux sortes : débit réservé et besoins d'irrigation. Le débit réservé comprend la dotation due au canal de Corbère qui sort de la retenue même (1.4 m3/s d'octobre à février, et 1.8 m3/s de mars à septembre) et le débit minimal que l'on doit laisser couler dans la rivière (3 m3/s) ; ce dernier débit sert aussi en partie pendant l'été à l'alimentation de certains canaux sans préjudice pour la rivière parce

Fig. 1 : Carte de situation des ouvrages



que les fuites de ce système d'irrigation l'alimentent suffisamment. Les besoins d'irrigation définis par la Direction Départementale de l'Agriculture sont donnés au tableau I.

Tableau I : Besoins d'irrigation

Mois	J	F	M	A	M	J	J	A	S	O
Débits en m3/s	1.4	1.4	1.8	2.4	2.9	3.5	4.	4.	4.	3.3

Les années de faible enneigement ne sont prévisibles qu'à partir de février, on a alors admis de ne fournir que 70 à 90 % des besoins précédemment cités, lorsqu'on se trouve donc en année de pénurie (critère à définir plus précisément en fonction d'une évaluation du stock neigeux).

La retenue de Vinça emmagasine 24 millions de m3. Le réservoir des Bouillouses apporte un supplément de 15 millions de m3 moins les pertes du trajet que l'on estime au 1/3. L'utilisation de cette réserve n'est possible que pendant la période de soutirage. Il a été étudié l'influence de la limitation du volume utilisable par mois sur le taux de défaillance des besoins.

METHODOLOGIE

La contradiction entre les deux types d'utilisation de la retenue de Vinça impose l'établissement d'une courbe d'objectifs de remplissage : on détermine une règle de gestion qui assure le remplissage de la retenue avec une certaine fréquence de défaillance acceptée, tout en maintenant le volume le plus petit possible dans la retenue. Pour cela, nous avons utilisé deux méthodes qui s'appliquent à des gestions annuelles.

Première méthode /CEMAGREF 1978, 1981/ :

On cumule, pour chaque année de données, les apports mensuels ainsi que les besoins. La différence entre les deux donne une courbe d'apports nets cumulés sur une année ; une translation de l'axe des abscisses jusqu'à la tangente au point maximum de la courbe donne à la fois une courbe de remplissage qui permet de satisfaire strictement les besoins en ne retenant que le volume nécessaire, et la date de remplissage pour l'année considérée (fig. 2).

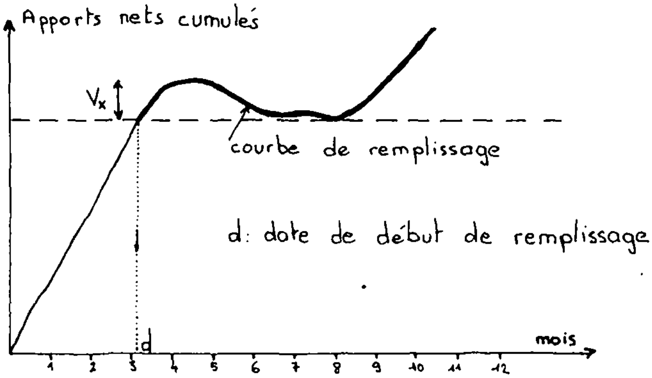


Fig. 2 - Détermination théorique de la courbe d'objectifs de remplissage.

En pratique, on choisit une date TF à partir de laquelle les besoins sont généralement satisfaits par les apports naturels et on calcule les apports nets cumulés en remontant le temps à partir de TF pour chaque année. Les valeurs négatives représentent un déficit en eau et donc un objectif de remplissage. Pour chaque mois, on détermine le quantile expérimental de la fréquence F de défaillance retenue. L'ensemble de ces quantiles donne la première courbe d'objectifs de remplissage. En fait les besoins sont sous-estimés parce qu'on n'a pas tenu compte des lâchures effectuées par le barragiste lorsqu'il cherche à rejoindre la courbe parce que la retenue est trop remplie. Pour les calculs on simule la gestion de la courbe choisie sur les N années disponibles et on effectue à nouveau le premier calcul en soustrayant les lâchures aux apports nets cumulés. Le processus est alors itératif et s'arrête lorsque la dernière courbe obtenue est très peu différente de l'avant-dernière (seuil à se fixer).

Deuxième méthode : type MONTE-CARLO /MICHEL, 1983/

On tire plusieurs fois au hasard les volumes objectifs au début de chaque mois en imposant bien sûr certaines contraintes de dates et de volumes ; on simule la gestion des courbes obtenues pour chaque année de données disponibles et on ne garde que celles qui n'admettent pas plus d'un nombre maximal autorisé d'années de défaillances. Parmi celles-là, on ne retiendra finalement que celle dont le volume du creux disponible dans le barrage, pondéré par des coefficients mensuels de risque de crue, sera le plus grand. Ainsi on satisfait les besoins d'irrigation avec un pourcentage fixé de défaillance et on maximise la lutte contre les crues.

Le volume maximal du barrage est insuffisant pour satisfaire seul les besoins en eau de la période d'étiage. Nous avons donc considéré que la retenue devait, de toute façon, être pleine au 1er juillet. A l'examen des volumes disponibles fournis par les apports, nous avons opté pour une date de début de remplissage au 1er mars. On remplit donc, entre le 1er mars et le 1er juillet, les volumes tirés au hasard sont ceux d'avril, mai et juin, dans un ordre croissant. Pour le déstockage, on tire au hasard les valeurs d'août, septembre et octobre.

Détail de la gestion suivie :

Pendant le remplissage, de mai à juin, on donne priorité à la satisfaction des besoins sur le remplissage. De juillet à octobre, on vide la retenue en déstockant d'abord de la retenue de Vinça, ce qui est indispensable aux besoins, en se règlant au départ sur les valeurs objectifs puis, si nécessaire, on fait appel à la réserve amont des Bouillouses en limitant a priori le volume mensuel prélevable aux valeurs suivantes :

juillet : 2.9 Mm3 Août : 2.9 Mm3 Sept : 2.8 Mm3
Oct : 1.4 Mm3 (total : 10 Mm3).

Les quantités non utilisées un mois donné sont reportées au mois suivant. On considère bien entendu qu'il y a défaillance si les besoins ne sont pas satisfaits un mois donné. Pour les années de faible enneigement, on ne peut pas suivre de courbe de remplissage ; on gère de la même façon en stockant tout l'apport excédentaire et en diminuant les exigences des besoins (70 à 90 %).

RESULTATS

La première méthode donne des courbes d'objectifs qui s'orientent vers une gestion presque interannuelle ; on remplit le réservoir dès le mois de décembre pour le vider mi-octobre. De plus, le régime de la rivière est très influencé par la fonte des neiges au printemps, ce qui donne le résultat curieux de remplir la retenue au début de l'hiver pour la revider ensuite, comme le montre le tableau II.

Tableau II - Courbe objectif avec la 1ère méthode

Mois	J	F	M	A	M	J	J	A	S	O	Taux de défaillance
Volumes objectifs en Mm3	4.	4.3	0.9	1.9	0.9	22.	20.	16.	9.7	3.8	16 %

Cette méthode ne convient donc pas vraiment au cas qui nous intéresse. C'est pourquoi nous avons mis au point la deuxième méthode.

C'est cette deuxième méthode qui a en fait donné les résultats qui ont été retenus. La chronique des débits mensuels utilisée pour l'établissement des courbes comporte plus de 40 années de relevés ; la station limnigraphique au droit du barrage a été doublée dès le début de la construction de l'ouvrage par une station un peu plus à l'aval. Pour des raisons pratiques, nous avons dans un premier temps établi séparément une courbe de remplissage et une courbe de déstockage. Pour la periode de remplissage, les données utilisées ne permettent pas d'obtenir moins de 5 années défaillantes sur les 44 qui servent à la gestion. On appelle année défaillante toute année dont les apports ne permettent pas de remplir la retenue. Après 20 000 itérations, la courbe obtenue est celle du tableau III.

Tableau III - Courbe objectif avec la 2ème méthode

Mois	1er mars	1er avril	1er mai	1er juin	1er juil.
Volumes objectifs en Mm3	0.9	1.5	7.4	24.0	24.6

Le remplissage ne commence en fait sérieusement que pendant le mois d'avril. Les 5 années défaillantes correspondent à des années de faible enneigement dont les débits de printemps n'ont pu être soutenus.

Pour la période de déstockage, la détermination de la courbe s'est faite sans tenir compte d'un apport supplémentaire éventuel de la retenue amont. Dans ces conditions, on doit admettre au moins 7 années de défaillances sur 42. La courbe obtenue est celle du tableau IV.

Tableau IV - Courbe de destockage avec la 2è méthode

Mois	1er juil	1er août	1er sept	1er oct	15 octobre
Volumes objectifs en Mm3	24.6	23.6	11.4	3.5	0.9

La gestion de cette courbe a ensuite été simulée en prenant en compte l'apport supplémentaire possible de la retenue amont, en se fixant les limites d'utilisation mensuelle pour chaque mois d'été (juillet à octobre) précédemment citées ; si ces volumes sont insuffisants, on déstocke plus du réservoir de Vinça. Le taux de défaillance de la courbe passe alors de 17 % à 7 % avec la réserve supplémentaire.

Il faut cependant bien noter que les taux de défaillance respectifs des deux courbes des tableaux III et IV ne sont pas représentatifs des taux réels lorsque l'on suit la courbe totale sur une année. Il est donc nécessaire de simuler la gestion de la courbe totale (fig. 3).

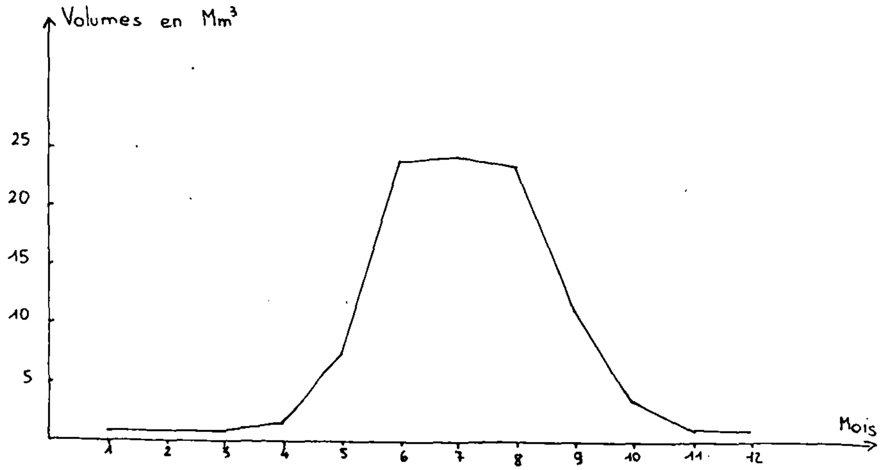


Fig. 3 - Courbe d'objectifs de remplissage proposée

Le taux de défaillance devient alors 20 %, ou de façon plus réaliste 15 %, si on tient compte des valeurs moyennes de débits réellement utilisées par le canal de Corbère, qui sont plus faibles que la dotation qui lui est due.

Ces taux de défaillance comprennent les années où le débit naturel a été insuffisant pour fournir le débit minimal de 3 m3/s plus la dotation du canal de Corbère : si on admet de ramener à 2 m3/s le débit minimal de janvier à mars, les taux de défaillance deviennent respectivement 13 % et 8 %.

Notons par ailleurs que le pourcentage moyen d'utilisation de la retenue amont est de l'ordre de 20 % ; cette valeur moyenne est assez faible mais trompeuse car, si pour certaines années il n'est absolument pas nécessaire de disposer d'un apport supplémentaire, pour d'autres la totalité de la retenue des Bouillouses est indispensable.

Un des problèmes soulevés par cette étude est le rapport réel entre les taux de défaillance obtenus avec une gestion mensuelle ou une gestion journalière. Malheureusement, sur les 11 années de données journalières disponibles, aucune n'est défaillante aussi bien en gestion mensuelle qu'en gestion journalière. Nous avons donc artificiellement multiplié les besoins par 1.2, 1.5 et 1.7 pour estimer un rapport entre les taux de défaillance respectifs ; ce rapport est en moyenne de 1.3 dans les conditions particulières que nous avons précisées ; il faut donc le considérer avec beaucoup de précaution et ne pas le généraliser. Cela permet de dire qu'en gestion réelle, le taux de défaillance (avec débits réels à Corbère) devient alors 20 %.

Les années défaillantes correspondent en fait aux années de faible enneigement. Si on simule la gestion exposée dans la partie méthodologie pour ces années là en diminuant les besoins de façon homogène sur l'ensemble de l'année, on obtient seulement 3 années défaillantes (pas de temps mensuel) en ne fournissant que 90 % des besoins, une année avec 80% des besoins et aucune défaillance pour 70 % des besoins.

CONCLUSION

L'intérêt de cette étude est multiple ; tout d'abord cela nous a permis de comparer 2 méthodes de courbes d'objectifs de remplissage. Toutes deux permettent de satisfaire des besoins donnés avec une fréquence de défaillance choisie. La différence essentielle entre les deux est cependant la maximisation de la lutte contre les crues. Dans le cas qui nous occupe, le régime climatologique méditerrannéen et le régime hydrologique influencé par la fonte de neige font que l'on obtient des résultats totalement différents. La première méthode donne une régulation interannuelle qui laisse en fait peu de creux disponible dans la retenue pour les crues, cette méthode convient mal à ce type de bassin à régime trop irrégulier ; on retiendra donc plutôt les résultats de la deuxième méthode. Il faut cependant bien s'assurer que le nombre de tirages au hasard est suffisant en vérifiant que lorsqu'on l'augmente, le résultat varie peu ou plus du tout.

Un deuxième intérêt est de mettre en évidence l'importance du pas de calcul. De telles études devront se faire dès que cela sera posible au pas de temps journalier car le pas de temps mensuel sous-estime le taux de défaillance /ANDRE et al. 1976/.

Le rapport entre les deux est difficile à estimer parce qu'il faudrait en fait prendre en compte seulement les défaillances vraiment gênantes ; il peut être admissible, par exemple, d'avoir de légères défaillances pendant quelques journées. Dans l'étude présentée, ce rapport était de l'ordre de 1.3 ce qui n'est quand même pas négligeable.

Enfin, il faut retenir l'importance de bien déterminer les priorités parmi les différents objectifs d'un barrage-réservoir et les taux de défaillance admissibles pour chaque objectif. Cela permet de moduler la gestion : par exemple, si on veut absolument fournir un débit soutenu en hiver avec la retenue de Vinça, il faut s'orienter vers une gestion interannuelle en acceptant de moins laminer les crues d'hiver ; à l'inverse, comme on désire se réserver un certain creux pour les crues, on préfère prendre le risque d'avoir des débits réservés plus faibles en hiver.

En conclusion, la méthode proposée permet de définir une courbe d'objectifs de remplissage qui satisfasse les besoins demandés 8 années sur 10 en moyenne et 70 % de ces besoins les autres années, avec une retenue de volume relativement faible et un laminage de crue relativement intéressant pour les crues de fréquence moyenne.

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MULTIOBJECTIVE WATER PLANNING IN SAUDI ARABIA

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ABSTRACT

The development of water resources is never an end in itself, but rather it is undertaken in support of the economic development of a country, and of the well-being of its people.

The Government of Kingdom of Saudi Arabia is faced with the daunting task of guiding a nation through a period of unusally rapid socio-economic growth and it has decided to do this by means of adopting a series of 5-year Development Plans, the most recent of which spans the years 1980-1985. Some of the promulgated objectives of this plan, which have a bearing on water planning, are :

- Industrialization based on hydrocarbon/energy-intensive activities.
- Modernized agriculture
- Housing programs

Obviously, these socio-economic objectives are predicated on the ability of the country to mobilize adequate water resources in their support. Saudi Arabia is unusually short of such resources, and, moreover, the conservation of depletable resources by itself is one of the development goals stated in the current Development Plan. This would appear to rule out an accelerated exploitation of the (non-renewable) fossil waters stored in the country's extensive aquifer systems.

The paper illustrates some of the alternatives available to Saudi decision makers in choosing among various potential water sources in order to satisfy the many competing demands, within a framework of certain economic, geographic and traditional guidelines.

KEYWORDS : Water planning, multiobjective decision-making, Saudi Arabia, water importation, desalination.

INTRODUCTION

Much has been written about the mechanics of decision-making and there appears to be broad agreement that decisions are preceded by the following steps :

1. Identify the problem
2. Identify alternative solutions
3. Quantify the solutions
4. Select the "best" solution.

The last step is in fact "the decision".

In the case of planning for water resources development a similar process is followed, and with increasing demands, and relatively few available alternatives, the importance of identifying, quantifying and evaluating alternative solutions on an objective basis is being more widely accepted. Simple decisions can be grasped by the human mind, which makes them also very subjective. Complex decisions, including those effecting long-term development plans for water resources, require the introduction of decision aids, which sometimes take the form of relatively sophisticated models or mathematical techniques. The subjective element is not eliminated, but it is distributed over more factors which the mind can handle, or it is distributed over more minds.

Recent economic development of the Kingdom of Saudi Arabia has placed serious stresses on the very limited water resources, and the traditional, highly personalized approach to decision-making is yielding to more sophisticated, informed planning processes. A Ministry of Planning has been established, and several Universities now offer programs in planning of natural resources. The present study is part of an ongoing effort at the University of Petroleum and Minerals to put water resources management on a more scientific basis and to familiarize local officials with the opportunities which new techniques may provide.

PLANNING OBJECTIVES

The Government of Saudi Arabia is guided by a sequence of five-year plans, the current one being the Third Development Plan, 1980-1985, which lists six long-term goals, one of which appears to have direct bearing on water planning:

> "To continue balanced economic growth by developing the country's resources, by increasing the income from oil over the long term and by conserving depletable resources, thereby improving the social well-being of all citizens and providing the economic strength to attain all the other fundamental goals of development. (Min. of Planning, 1980)".

The clause "increasing income from oil" puts diversion of oil to desalination plants outside the planning scope, since such diversions decrease income from oil.

Similarly, the exploitation of depletable fossil groundwater resources from deep aquifers runs counter to the promulgated goals.

Planning options appear to become even more limited in the future; the Minister of Planning has indicated that in the Fourth Development Plan (1985-1990) more emphasis will be placed on economic principles (benefit/cost ratios, rates of return etc.), which if strictly applied, would seriously restrict water resources development. (Nazer, 1983). The preceeding leads therefore, to the conclusion that water planning in Saudi Arabia will increasingly become an exercise in multicriteria decision making, with many of the criteria being of a non-technical and non-commensurable nature.

It should be pointed out that, in line with instructions contained in the current Development Plan the Saudi Ministry of Agriculture and Water has undertaken the preparation of a National Water Plan, which is likely to be published and discussed in the course of 1984.

PLANNING TECHNIQUES

The literature on comprehensive water resources planning is rather limited; a recent effort was made by Helweg (1984), a summary of which was presented at a Short Course in Dhahran, Saudi Arabia (see deJong, 1984). One of the steps in the planning process is the evaluation of alternative plans, or of alternative strategies to satisfy the original objectives, and numerous techniques have been developed in the past few decades to introduce a semblance of order, precision and "system" into a traditionally highly personal and subjective undertaking.

Numerous articles and textbooks have been written dealing with multiobjective analysis and decision making (Goicoechea et.al. 1982; Chankong and Haimes, 1983; Major, 1977) and multiple criteria decision making appears to be growing into a new independent discipline (e.g. Fandel and Gal, 1980).

From the standpoint of water planning not all techniques appear to be equally suitable, as was pointed out by Cohon and Marks (1975) in a comparative evaluation of multiobjective programming techniques as they might be applicable to water resources problems. Generally the techniques are aimed at eliminating the non-preferred (dominated) solutions to a specific planning problem, according to preferences which may either have been formulated in advance, or have taken shape in the course of the planning process, as the Decision Maker (DM) obtains a better insight into the consequences of various alternatives. The elimination process leaves the DM with a smaller set of solutions to choose from. Another classification of these techniques identifies them as either "continuous" or "discrete", depending on whether the DM is faced with an infinite number of alternatives (such as choosing the best size of a reservoir), or with selecting the best of a few finite solutions.

In this study the ELECTRE method was used to test several rather arbitrarily conceived combinations of alternative solutions to solve the long-term needs for water in Saudi Arabia. One of these tests is reported, and compared with an application of the Compromise Programming technique.

CONSTRAINTS

Several descriptions of the hydrological setting of Saudi Arabia have found their way into the literature (e.g. Ali, 1983; Noori and Kalthem, 1981).

The most important resources, and the constraints imposed on them, are listed below.

<u>Surface water and shallow groundwater</u>

Some areas of the country, particularly the mountainous southwest and the drainage areas facing the Red Sea, receive rainfall in the range of 200-500 mm/year, sufficient to generate sporadic, violent floods. For centuries local residents have succeeded in diverting some of this runoff in support of small-scale agriculture. More recently, the Government has undertaken to construct more than 50 small dams for flood control and recharge of the local, shallow aquifers. In some cases, such as in the city of Abha, flood waters are stored in surface reservoirs for municipal water supply. Additional capture may be feasible, but remaining damsites are becoming less attractive, and the potential locations tend to be remote from existing development centers. Apart from the fact that sediment concentrations during floods tend to be very high, the surface water and shallow groundwater resources present no qualitative problems.

Deep groundwater

Most of the Kingdom is underlain by a system of deep aquifers which received substantial recharge during an earlier period of different climatological conditions. Current recharge is minimal, but considerable amounts of water are in storage with estimates ranging as high as 6.75×10^{11} cubic meters (Min. of Planning, 1980). The deep, fossil groundwater presents quality problems. Some wells yield water at temperatures in excess of 60°C, which requires cooling before treatment, and typical values for "total dissolved solids" are in the 2000-5000 ppm range. Demineralization, usually by means of Reverse Osmosis plants, is required for municipal uses, and frequently also for agriculture.

Geographical availability presents a major advantage, particularly to the urban conglomerates in the Central and Eastern Provinces, and to the coastal developments along the Gulf.

Artificial resources

In this category the following may be identified :

- Desalination of seawater, which requires the diversion of valuable (and marketable) fossil fuels and relies on very vulnerable technology.
- Importation of icebergs, which has not yet become operational, partly because it violates the aim of self-sufficiency.
- Importation of surface water by pipelines, which has been discussed; the technical, and perhaps even the economic, feasibility may be favourable, but major water quality problems and negative political implications have hampered its development.
- Importation of water by tankers, which is a technique of simple technology and attractive economics; it may hold some promise, even though the reliance on outside sources for a fundamental resource has major political disadvantages; the latter are to some extent offset by the fact that the sources are relatively flexible in magnitude and location.
- Weather modification, which has been studied and discussed, but climatological conditions are not considered favourable in Saudi Arabia.
- Wastewater re-use, which is expected to meet about 13% of projected demands by the end of the century, an expectation which is based on the assumption that all other demands will be satisfied from the other sources mentioned above. In this connection it is of interest to note that a fairly recent development appears to have removed any religious impediments to the reuse of wastewater, provided that stringent quality standards are being observed (Farooq and Ansari, 1981).

Summary

A review of the resources which could be mobilized to satisfy the projected demands indicates that all resources incur certain liabilities, either physical, financial or political. It is, therefore, necessary that the Decision Makers carefully weigh the tradeoffs involved in utilizing the various resources, recognizing that most of the constraints are not easily quantifiable, nor compatible. In this context it appears that the stage is set for the application of multi-objective water planning which would attempt to satisfy most of the objectives.

ILLUSTRATION

This section will illustrate the implications of applying a multiobjective planning technique in the Saudi Arabian setting; such implications refer mostly to the need for choosing among alternatives, or among mixed strategies, and to determining the trade-offs inherent in those choices.

The ELECTRE method, referred to earlier, is aimed at reducing the number of planning alternatives to a small set of nondominated solutions from among which the Decision Maker can choose whatever solution is the "best" in his judgement.

The interested reader can find detailed descriptions of the ELECTRE technique in the literature(e.g. Benayoun et.al., 1966; Roy and Bertier, 1971); only the major steps will be highlighted here. An important intermediate point in the technique is an array of Systems vs Criteria, which summarizes the alternative planning strategies, and the criteria by which they are to be judged. For the purpose of this illustration the following alternatives were postulated, all of which represent mixed utilizations of available water resources. (See Table 1). Alternative No.8 resembles the mix as projected in the Current Development Plan.

A group of students was asked to act the role of a Group Decision-Maker, and they were given the following criteria to formulate their preferences among the various mixes of resources :

- Safety : the relative assurance of non-interference as a result of technical breakdowns, hostile acts, lack of supplies (chemicals) or equipment, political barriers etc.,
- Autonomy : the extent to which water can be totally produced within the country's territorial limits,
- Environmental Impact : the effects a certain strategy may have on the local natural environment.
- Financial Feasibility : the availability of the financial resources to construct the necessary engineering works.
- Economic return : the value of the economic benefits, generated by investments in water development projects.

On the basis of the alternatives shown in Table 1 and the criteria mentioned above it is possible to formulate an Alternatives vs. Criteria array, where the entries represent the subjective value judgements of the Decision Maker as to the relative importance of the criteria as applied to the available alternatives. This array is shown in Table 2, and it confirms a basic assumption in multicriterion decision making, that is that there is not one decision, or alternative, which is preferable on the basis of all criteria. Hence, a compromise is to be reached which will provide a high level of concordance, and a low level of discordance among the alternatives (or their proponents). These levels are determined by treshold values, chosen by the Decision Maker, of the concordance and discordance indexes associated with each two alternatives.

According to the computational procedures proposed by Benayoun et

Table 1 : Definition of Alternatives

Alternative Water Resource	1	2	3	4	5	6	7	8
a) Development of deep groundwater	100%	50%	40%	40%	70%	40%	70%	60%
b) Desalination of seawater	50%	30%	30%		10%			20%
c) Importation of water by tankers or icebergs			30%					
d) Water conservation				30%	30%	50%		8%
e) Wastewater reuse							30%	12%

Table 2 : Alternative vs. Criteria Array

Alternative Criteria	1	2	3	4	5	6	7	8
Safety	High	Medium	Medium	Medium	High	Medium	Low	High
Autonomy	High	Medium	Low	Medium	High	High	Low	Low
Environmental Impact	Pos.	Pos.	Neg.	Pos.	Pos.	Pos.	Pos.	Neg.
Financial Feasibility	Medium	Low	Low	Low	High	High	High	Medium
Economic Return	Neg.	0	Neg.	Neg.	0	Pos.	Neg.	0

al. (1966) a Concordance Matrix and a Discordance Matrix were constructed. Subsequently the Group Decision-Makers arbitrarily chose a Concordance Treshold value (p) of p = 0.85, and a Discordance Treshold value (q) of q = 0.2. These values led to the identification of alternatives 2, 4 and 8 as the "most acceptable" strategies for water resources development.

In an effort to provide a semi-independent confirmation of the earlier findings the same Alternative vs. Criteria array was "scaled" (i.e. all value judgements expressed numerically), and the information thus obtained was analyzed by the Compromise Programming technique (see e.g. Zeleng, 1977). This yielded again alternatives 2 and 4 as the most preferred, with 8 next in a group of equally valued solutions.

It should be stressed that, in spite of the scientific and mathematical procedural image, these techniques depend on subjective judgements at many points in the process, such as in formulating (and scaling) the Alternatives vs Criteria array, in choosing treshold values for concordance and discordance, and even in choosing the alternative mixes.

CONCLUSIONS

The exercise reported herein involved the exposure of a small group of University students to multiobjective water planning, and it should not be taken as representative of the decision-making cirlces. It is of interest to note that :

1. The concepts inherent in multiobjective planning, including the search for a best compromise, were readily understood and accepted.
2. A high degree of reliance was placed on self-sufficiency and on meeting needs using locally available resources, with costs being of secondary concern.
3. Alternatives which have not yet been proven were not preferred, in spite of the relatively attractive economic aspects.

Further efforts are being planned, aimed at refining the alternatives based on the latest findings concerning available resources as well as on current projections of requirements. At the same time the Decision-Makers will be more broadly based and reformulated so as to more closely represent local preferences and opinions.

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UN EXEMPLE D'ETUDES PRELIMINAIRES DES RESSOURCES EN EAU D'UN PETIT BASSIN RURAL et AGRICOLE EN VUE DE SON AMENAGEMENT HYDRAULIQUE : LE HAUT CHAPEAUROUX (Lozère) - France

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RESUME

Il s'agit de fournir aux collectivités locales une méthodologie permettant de prévoir les travaux hydrauliques nécessaires à l'aménagement des terres agricoles du département de la Lozère. Cette méthodologie est testée sur le bassin du CHAPEAUROUX qui a déjà fait l'objet d'aménagements dans sa partie aval (hydroélectricité et réaménagements de cours d'eau). Le contexte climatique implique une prise en compte du facteur nival.

Il s'agit d'abord de tirer le meilleur parti possible de l'information hydro-climatologique disponible car le bassin lui-même (18 km2) est dépourvu d'observations.

Puisque les premiers besoins semblent concerner l'assainissement et le drainage, on décrit les caractéristiques hydro-climatiques afin de délimiter les secteurs dont le comportement hydraulique pourrait être homogène. A ce niveau, on peut déjà concevoir la structure du réseau d'évacuation des eaux excédentaires.

On tente ensuite de préciser les termes du bilan hydrologique et d'étendre l'information pluviométrique régionale aux apports et aux débits grâce à une modélisation ; cette dernière est orientée prioritairement en vue d'une utilisation pour les projets d'assainissement et de drainage.

On estime bien entendu les paramètres habituels descriptifs des fortes crues.

L'ensemble du travail a été mené de façon à ce que la méthodologie puisse être reproduite sur d'autres bassins versants de la région.

INTRODUCTION

La situation peu favorable à une utilisation efficiente des outils de l'hydrologie opérationnelle, faute de données disponibles sur la partie du bassin investigué, nous a conduit à mettre l'accent sur deux aspects.

Le premier a trait à l'étude de la variabilité spatiale des phénomènes hydroclimatiques (pluie, température, debits).

Le deuxième concerne la modélisation pluie-débit qui, hormis l'intérêt suscité par la compréhension et l'analyse des termes du bilan hydrologique, offre la possibilité de simuler les débits en des sites non jaugés. On peut souligner que le deuxième aspect est étroitement lié au premier dans la mesure où les performances des modèles sont extrêmement conditionnées par le degré d'incertitude sur les entrées :(pluie, t° en régime nival), degré que l'on peut réduire si l'on sait apprécier la variabilité des phénomènes hydroclimatiques et en expliquer leur cause.

L'objectif de cette étude étant de fournir tous les éléments hydrologiques nécessaires à l'élaboration des projets de drainage et d'assainissement, il conviendra donc d'élaborer, d'une part, une mini-synthèse régionale des pluies annuales et décennales de 1 à 3 jours consécutifs et, d'autre part, de proposer un faisceau d'abaques débit-durée-fréquence généralisables avec la surface.

Le modèle adapté au régime nival va mettre en évidence les mécanismes de production d'excedents d'eau et nous fournira un ordre de grandeur de ces excédents pour l'année moyenne.

1. ANALYSE ET SYNTHESE DE L'INFORMATION DISPONIBLE

1. Etude des pluies :

Dans ce paragraphe, l'accent est mis sur la volonté d'identifier la variabilité spatiale de la pluie, et ce pour deux raisons en quelque sorte complémentaires :

- la densité des postes pluviométriques en Lozère, ainsi que la qualité (ou l'ampleur) de l'information qu'ils collectent, est médiocre ;

- la pluie conditionne les débits et, là encore plus que partout ailleurs, la densité en stations hydrométriques est faible. Ainsi, dans un contexte d'homogénéité du complexe sol-végétation à l'échelle d'une région (comme c'est le cas ici, à quelques exceptions près), une bonne connaissance de la pluie nous permet de préjuger (appréciation correcte possible des conditions thermiques du bassin) du comportement hydrologique d'une zone (ou bassin non jaugé), référence faite au bassin pourvu d'une information complète (pluie, débit, t°), et sur lequel on aura pu, par exemple, caler un modèle.

L'aspect quantitatif de cette variabilité spatiale est précisé par l'estimation des pluies de 1 à 3 jours de fréquence annuale et décennale aux différents postes. Une analyse des concomitances des pluies extrêmes annuales donne, en outre, une idée des variations extrêmes possibles du champ des précipitations pour 2 postes susceptibles d'être représentatifs d'une fraction de la lame d'eau précipitée sur le bassin étudié.

Pour permettre et faciliter l'interprétation de cette variabilité spatiale, nous avons utilisé l'analyse en composantes principales sur les pluies mensuelles et journalieres. Nous ne présenterons ici que les résultats relatifs aux pluies journalières.

Les matrices employées sont constituées des réalisations des pluies journalières supérieures à un seuil (fixé au poste le plus proche du bassin). On a analysé la configuration du nuage des points variables en fonction de ce seuil, afin d'étudier la représentativité de certains postes (ceux de courte durée d'information) vis à vis des événements exceptionnels susceptibles d'affecter le bassin du Haut-Chapeauroux.

Cette analyse va nous aider à réaliser, d'une part, d'éventuels comblements de lacunes afin de rendre possible la détermination des caractéristiques de pluies de 1 à 3 jours, d'autre part, nous permettre de choisir les postes et leur pondération qui nous garantissent une estimation correcte de la lame d'eau précipitée sur le bassin. En outre, l'analyse des proximités sur le plan factoriel confortera en quelque sorte les estimations des pluies annuales et décennales aux postes de courte information.

Au pas de temps journalier, le pourcentage d'inertie du premier facteur est plus faible qu'au pas de temps mensuel. Il n'excède jamais 50 % de la variance totale et il diminue lorsqu'on augmente le seuil des pluies au poste de référence (F1 : 48 % ; F2 : 22 % pour PS = 30 mm à CHATEAUNEUF).

Sur la figure 1, on retrouve à peu de chose près les mêmes configurations qu'au pas de temps mensuel. Bien que dans l'ensemble les proximités géographiques expliquent la plupart des affinités entre variables, on dénote cependant quelques exceptions qu'il est intéressant de signaler : MENDE, par exemple, sur le plan 1-2 est bien plus proche de RIEUTORD et St SAUVEUR que de BAGNOLS, et GRANDRIEU (pour PS > 30 mm) de CHATEAUNEUF que de St SAUVEUR ou de St JEAN (leurs pluies décennales et annuales, comme nous le verrons, sont d'ailleurs assez comparables).

A partir du seuil PS > 20 mm, on constate une anomalie concernant le poste de BAGNOLS, lequel quitte le groupe des postes Est-Sud-Est pour se joindre à l'autre. On notera par la suite que ce poste a une pluie annuale journalière proche de celle du groupe auquel il se rallie.

<u>Estimation des pluies maximales de 1 à 3 jours de fréquence annuale et décennale sur la saison année et la saison drainage</u> :

Dans un esprit de synthèse régionale, nous avons ajusté sur chaque poste un modèle de renouvellement constitué du binôme loi exponentielle - loi de POISSON.

Le modèle a d'abord été appliqué aux 12 stations de base (1960 - 78).

Les valeurs attribuées aux stations de courte durée l'ont été sur la base d'une comparaison avec les valeurs estimées aux postes voisins et/ou les plus proches au sens de l'acp sur la même période.

Une cartographie des pluies décennales journalières est proposée en figure 2.

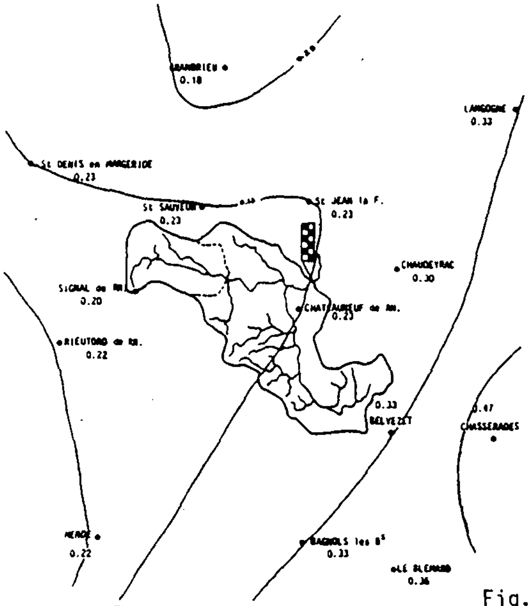


Fig.3 - Isovecteurs

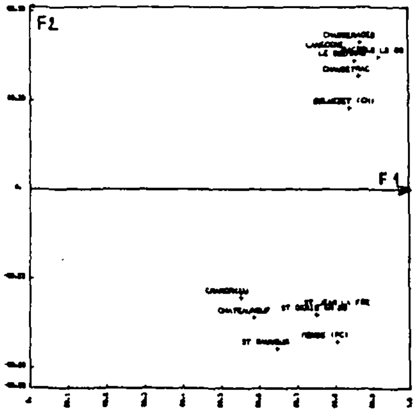


Fig. 1 - PS = 15 mm ; F1 = 68 % VT
F2 = 25 % VT

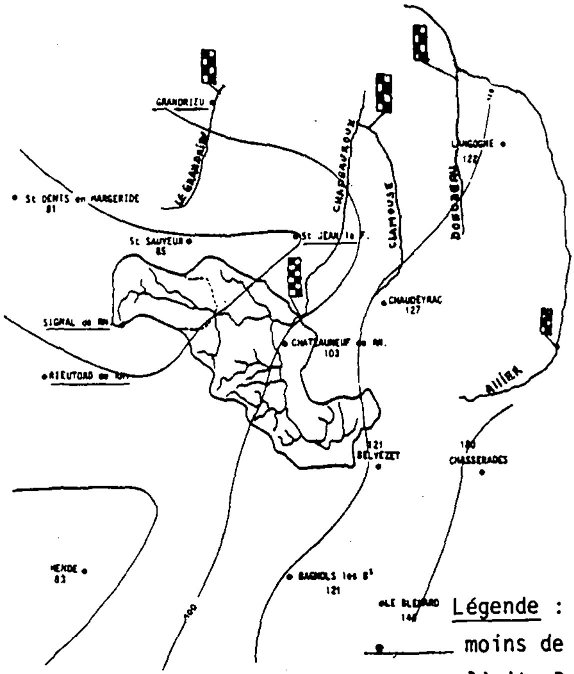


Fig. 2

Pluie décennale journalière

Légende :
moins de 7 ans
------- limite BuV. 18 km2

Conclusion relative à l'etude des pluies :

L'analyse des données et la cartographie des pluies décennales et annuales confirment une certaine homogénéité des pluies à l'Ouest de l'isovecteur 0.33 (cf. fig. 3). Cette homogénéité apparente n'affecte cependant qu'une partie du bassin (partie Nord-Ouest), celle d'ailleurs à l'exutoire de laquelle il s'agira de simuler une chronique de débits.

La formule permettant de calculer la lame d'eau sur le bassin du Chapeauroux à l'Hermet que l'on propose est la suivante :

PL = 0.35 P (CHATEAUNEUF) + 0.30 P (BELVEZET) + 0.20 P (St SAUVEUR) + 0.15 P (MENDE).

Elle résulte d'un compromis entre le modèle de THIESSEN et les conclusions relatives à l'analyse des données. Elle comporte donc une petite part de subjectivité.

On remarquera que les pluies deviennent de plus en plus intenses au fur et à mesure que l'on se rapproche de la barrière qu'exerce la face Nord-Ouest du M^{t} Lozère aux perturbations venant du Sud-Ouest. Le poste de CHASSERADES n'est en outre certes pas à l'abri des perturbations méditerranéennes, lesquelles peuvent remonter sans encombre la vallée de l'Ardèche.

2. Etude des débits :

Une ACP sur les débits pour la région concernée n'est pas ici réalisable faute d'un nombre suffisant de stations hydrométriques. A l'échelle du département, l'ACP sur les débits a pu être réalisée, elle tend à montrer que les débits ont une configuration à peu de choses près superposable à celle des pluies sur le plan factoriel 1-2. L'intérêt de cette constatation réside en ce que l'extrapolation des formules d'estimation des caractéristiques de débits devra être faite en tenant bien plus compte des caractéristiques locales ou régionales des pluies que des types de sols. Les formules auxquelles nous nous intéressons sont, par exemple :

$QCX^{F}n = an^{b}S^{c}$; $QCX^{F}n$: débit atteint ou dépassé pendant n jours consécutifs de fréquence F (S : superficie du bassin versant, $1 \leqslant n \leqslant 30$ j).

$QIX^{10} = R\ (\frac{P^2}{80})\ S^{0.8}$: estimation du débit instantanné décennal par la méthode CRUPEDIX (R : facteur régional, P : pluie décennale journalière).

Au niveau de notre région, en déhors de la station d'HERMET, nous disposons de 4 années d'information à la station de NAUSSAC sur le Donozeau et de 7 années à la station de ROGLETON sur l'Allier. Chacune de ces deux stations sont situées de part et d'autre de l'isovecteur 0.33, lequel délimite des pluies de caractéristiques très différentes. La superposition des chroniques de débits journaliers de chacune de ces stations à celles de l'HERMET, permet l'analyse du comportement hydrologique des bassins représentatifs de ces deux zones que délimitent grosso-modo l'isovaleur 0.33

On vérifie que le bassin du Donozeau à NAUSSAC appartient bien à la même zone que le bassin du Chapeauroux.

Essai de synthèse régionale des débits instantanés décennaux :

Les débits instantanés estimés sur les 3 bassins ont été complétés par des résultats tirés de la Synthèse Nationale des Crues /1/ (1980) relative à 5 autres bassins de la région.

L'application de la formule CRUPEDIX : $QIX^{10} = R(\frac{P}{80})^2 S^{0.8}$ donne lieu aux valeurs de R suivantes :

R = 1 (l'Allier à LAVALETTE (396 km2)
(l'Allier à ROGLETON (48 km2)
(le Langouroux à LANGOGNE (65 km2)

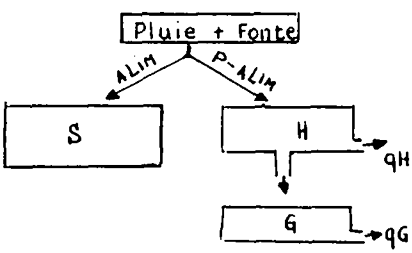
Ces 3 bassins sont situés à droite de l'isovecteur 0.33.

R = 0.67 (le Chapeauroux à l'HERMET (109 km2)
(" à AUROUX (225 km2)
(le Grandrieu à GRANDRIEU (70 km2)
(la Clamouze à CHASTANIER (51 km2)
(le Donozeau à NAUSSAC (55 km2)

Ces 5 bassins sont situés à gauche de l'isovecteur 0.33 et appartiennent à la zone dite des Monts de Margeride.

II - ETUDE DES RELATIONS PLUIE-DEBIT A L'AIDE D'UN MODELE CONCEPTUEL ADAPTE AU REGIME NIVAL /4/ : ESTIMATION DES TERMES DU BILAN HYDROLOGIQUE

Une version du modèle CREC /2/ a été couplée à un modèle de fonte, inspirée de celui du SNOW-HYDROLOGY /3/, qui calcule l'équivalent en eau dû à la fonte sous couvert forestier et en clairière.



- au préalable un modèle simple, calé sur 4 années communes entre CHATEAUNEUF et LANGOGNE, nous a permis de générer une chronique de 11 années de t° moyennes journalières pour le bassin du Chapeauroux à l'HERMET (ΔH MAXI = 360 m).

$$Q_J = qH + qG$$

Les performances du modèle ont été améliorées grâce à une information supplémentaire autre que la température, laquelle était seule utilisée (moyennant l'utilisation d'un seuil de transformation pluie→neige) pour décider si la lame d'eau en entrée du modèle était solide (neige) ou liquide. Cette information supplémentaire consiste en l'utilisation d'une matrice de "0" (pluie) et de "1" (neige) élaborée à partir des renseignements consignés dans le fichier des pluies de la Météorologie Nationale et relatifs au poste le plus représentatif de l'altitude moyenne du bassin : CHATEAUNEUF. Le modèle a été ajusté sur les années 71 à 73.

Malgré les incertitudes liées aux "INPUT", le modèle donne des resultats très acceptables au regard de l'ensemble des critères numériques (plus ou moins objectifs), d'une part, et graphiques, d'autre part (fig. 4). Certainement, la possibilité de modifier le modèle pour qu'il puisse prendre en compte la discrétisation spatiale aurait amélioré la qualité des simulations.

Application du modèle : Simulation des débits journaliers sur le bassin de 18 km2 et estimation des différents termes du bilan hydrologique :

Avant d'entreprendre la simulation sur le bassin de 18 km2, on a tenu à connaître dans quelles conditions on pouvait extrapoler spatialement les paramètres d'un modèle en un site non jaugé. Pour cela, le modèle, avec son jeu de paramètres, a été appliqué aux 2 bassins voisins : NAUSSAC et ROGLETON. On a vérifié que la qualité moins bonne des simulations était due à l'imprécision encore plus grande que pour l'HERMET des "INPUT".

Sur le bassin de 18 km2, en amont de l'HERMET, la bonne homogénéité des pluies que l'on a pu constater, et une bien meilleure connaissance des températures, nous garantit une simulation correcte.

A partir du bilan hydrologique sur les 11 années simulées, on a élaboré le tableau 1, qui représente le bilan de l'année moyenne.

Bien que relevant d'une approche conceptuelle, les différents termes du bilan explicités ci-après (*) ont des valeurs tout à fait réalistes pour la région concernée. On signale à cet égard que l'erreur moyenne sur les bilans annuels (Qobs - Qcal) est à peine de l'ordre de 10 % sur le bassin de l'HERMET.

On notera une forte contribution de la fonte de neige à la recharge en eau du sol aux environs du printemps.

En admettant qu'en moyenne la teneur en eau du sol soit de 200 mm aux environs d'avril, date à laquelle le cultivateur souhaite travailler la terre, et qu'elle soit de 100 mm aux environs de juillet (période où la terre est en principe labourable), on peut alors en déduire une valeur approximative du stock d'eau à évacuer au cours des mois de mars à juin, soit environ 300 mm environ.

(*) PM : lame d'eau mensuelle précipitée sur le bassin (pluie ou neige).
FONTEM : fonte mensuelle.
PLUFON : lame d'eau mensuelle résultant de la pluie et/ou de la fonte associée.
ETRM : évapotranspiration réelle mensuelle.
INFILM : infiltration mensuelle (recharge du réservoir-sol).
INFILMXJ : " maximale journalière du mois.
SOLM : teneur en eau du sol : M = mo. mens, s = écart-type.
SOLMXJ : " " " " " maximale journalière du mois.
QM : lame d'eau écoulée mensuelle.

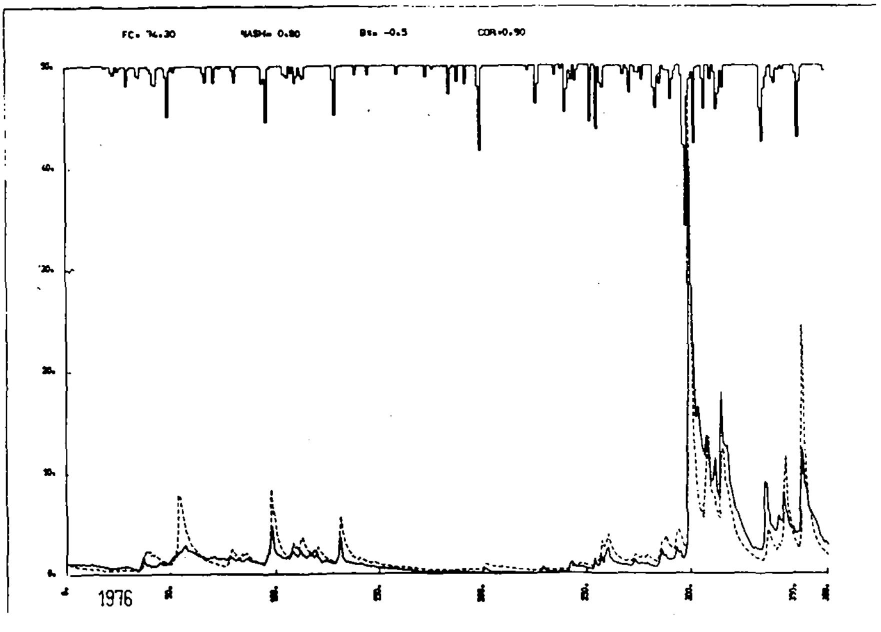


Fig. 4 - Exemple de simulation des débits journaliers.

	J	F	M	A	M	J	J	A	S	O	N	D	Année
PM	104.	83.	89.5	70.5	95.2	75.8	72.7	72.5	95.1	121.5	67.2	122.5	1070.
FONTEM	36.0	70.0	119.0	92.0	29.0	0	0	0	0	6.0	32.0	51.0	435.
PLUFON	57.	95.	150.	131.0	109.	75.8	72.7	72.5	95.1	119.	61.	117.	1155.
ETRM	1.5	4.	8.	15.0	41.0	60.	80.	70.	49.	26.	10.	4.2	369.
INFILM	9.	15.	12.	14.	15.	24.	44.	56.	57.	64.	22.	38.	370.
INFILMXJ	3.	3.	3.	3.	4.	9.	22.	23.	23.	26.	8.	13.	
SOLM M)	173.	183.	192.	196.	185.	148.	105.	79.	82.	112.	131.	154.	
SOLM s)	18.	17.	15.	14.	13.	18.	20.	21.	26.	36.	44.	31.	
SOLMXJ M)	177.	189.	198.	203.	196.	170.	132.	99.	105.	131.	141.	173.	
SOLMXJ s)	18.	14.	16.	12.	14.	13.	13.	21.	29.	44.	43.	16.	
$\hat{QM}$	46.	81.	120.	117.	104.	56.	35.	21.	23.	48.	41.	69.	760.

Tableau 1 - Bilans mensuels (année moy.) sur le bassin de 18 km2.

III - ELABORATION D'ABAQUES : DEBIT - DUREE - FREQUENCE

La variable dont il est question est le QCXn, très utile pour définir les avant-projets de drainage et d'assainissement.

Les abaques débit-durée-fréquence pour le bassin de l'Allier à ROGLETON, représentatif des bassins de la zone située à l'Ouest de l'isovecteur 0.33 et pour le bassin du Chapeauroux représentatif des bassins des Monts de Margeride, situés à l'Est de l'isovecteur 0.33, ont été établis.

On constate que l'abaque, relatif au bassin de 18 km2, obtenu à partir des débits simulés, est tout à fait conforme (même exposant de n) à celui du bassin aval représentatif de la zone.

Formules généralisées avec la surface :

Pour des zones ou secteurs hydrologiquement homogènes, où l'on a pu constater qu'il s'agissait de zones sujettes à des caractéristiques de pluies comprises dans un intervalle déterminé, il est d'un intérêt certain de pouvoir généraliser avec la surface les formules établies pour les bassins étudiés.

	T = 10 ans	: $QCXn = ,^{-0.6}\ S^{0.8}$	(bassins des M^ts (de Margeride : (B.V. représen- (tatif :le Cha- (peauroux à (l'HERMET.
I	T = 2 ans	: $QCXn = 1/2n^{-0.6}S^{0.8}$; $80mm < PJ^{10} < 120$ mm	
	T = 1 an	: $QCXn = 1/4n^{-0.6}S^{0.8}$	
	T = 10 ans	: $QCXn = 4n^{-0.8}S^{0.8}$	(bassins situés (au S-E de l'iso- (vecteur 0.33. BV (représentatif : (l'Allier à (ROGLETON.
II	T = 2 ans	: $QCXn = 2n^{-0.8}S^{0.8}$: $120 < PJ^{10}$	
	T = 1 an	: $QCXn = n^{-0.8}S^{0.8}$	

CONCLUSIONS : SOLUTIONS POSSIBLES A L'ASSAINISSEMENT

La zone étudiée, relative au bassin du Haut-Chapeauroux, représente l'un des secteurs de référence drainage-assainissement de la Lozère. Nous y avons mis en évidence une typologie des mécanismes d'excès d'eau dans la vallée en grande partie basée sur la fonte de neige durant les mois de mars et avril. La mise en place d'un système d'assainissement visant à réduire la contribution à la recharge de la nappe dans la vallée par les apports latéraux émanant pour la plupart de la fonte doit être en conséquence envisagé. Le principe des rigoles (ou goulottes) au pied des versants (ou en bordure de plaine) pour capter le ruissellement issu de la fonte peut être retenu.

Une autre solution, qui peut être plus généralisable, consiste en la pose d'un drain à une distance entre le ruisseau et le pied des versants, et ce longitudinalement à la rivière, dont les études complémentaires liées à l'hydrodynamique des sols et aux possibilités hydrauliques du cours d'eau pourvoieront au dimensionnement. Le système présentera certes des variantes très localisées, liées aux conditions topographiques pour un même bassin, ou régionales selon le particularisme des pluies ou l'importance relative du régime nival, points essentiellement soulignés dans la présente étude.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 82 Aspect number 10

WATER RESOURCE PLANNING: MULTI-OBJECTIVE CONCEPTS APPLIED TO RURAL WATER MANAGEMENT

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ABSTRACT

Multi-objective techniques for the planning and management of large scale river basin development plans are well established. These techniques have also been applied to assist in the solution of complex water supply problems in urban situations. This paper will present information on the potential for the application of these concepts of multi-objective planning and analysis to rural water management problems. This paper will identify a range of rural water objectives which could be considered in a multi-objective planning framework. Preliminary problem formulation will be presented. Examples will be developed for non-point source pollution and nutrient enrichment. It is anticipated that the techniques of multi-objective planning applied to rural water management issues will sharpen the policy trade-off decisions which must be made by responsible decision-makers. It is anticipated that the application of those techniques will assist in the further clarification of data needs required to support multi-objective techniques for rural water resource management.

INTRODUCTION

In the United States, the requirements to undertake multi-objective planning for water resource projects arose from both the provisions of the National Environmental Policy Act of 1969 as well as the Principles and Standards (Now Principles and Guidelines) for Federal Water Project evaluation which resulted from the Water Planning Act of 1965. Basically, it is recognized that present and future studies need to consider the presence of multiple objectives which may in fact be in conflict with one another. It becomes the requirement of the analyst to undertake water resource planning which explicitly considers the multi-objective nature of the utilization of the water resources. This paper presents information on the potential for application of the concepts of multi-objective planning and analysis to rural water management problems.

One rural water management problem of particular interest is the adverse impact on receiving water quality as a consequence of non-point source pollution from agricultural areas. The water quality cited as having deteriorated as a consequence of such non-point source pollution include trophic status (excessive nutrient loadings), suspended solids (excessive erosion), and chemical contamination (excessive application of various chemicals to control insects, weeds, and other threats to agricultural productivity). Research is underway to identify improved means to control or reduce the adverse impact resulting from agricultural non-point source pollution. Techniques of multi-objective planning and analysis may provide useful insights into improved rural water management.

MULTI-OBJECTIVE PLANNING AND ANALYSIS

There are numerous references [1], [2], [3], [4], [5] now available on the theory and development of multi-objective planning and analysis. This literature [2] makes a distinction between multi-objective planning and multi-objective programming. Planning is a process which contains all of the elements of a systems approach to problem solution. For example, planning includes problem identification, formulation of goals and objectives, specification of measures of effectiveness, generation of alternative solutions, evaluation of alternatives, selection of an alternative, and finally implementation of a selected alternative.

Multi-objective programming (MOP) is a highly structured formal mathematical procedure for identifying a range of attractive solutions to the planning problem at hand. In contrast with a single-objective programming model, MOP generally does not identify a single solution for the decision-maker. Rather, it identifies a range of possible solutions which satisfy the constraints in the planning problem and leaves to the decision maker, the choice of a solution point along a frontier of non-dominated solutions. The actual decision point chosen by the decision-maker thus reflects the relative value which the decision-maker attaches to the several objectives included within the problem. Any movement along the frontier of non-dominated solutions means that improvement in one objective - for example improvement in water quality as measured by a decrease in total phosphorous can only be achieved as a result of a trade-off with another objective - for example an increase in cost for nutrient removal.

In order to specify a multi-objective programming model, the problem needs to be formulated in the following way [2]:

Consider a problem with -

N decision variables
M constraints
P objectives

maximize

$$[Z_1(X_1, X_2 \dots X_N),$$
$$Z_2(X_1, X_2 \dots X_N),$$
$$\dots,$$
$$Z_P(X_1, X_2 \dots X_N)]$$

subject to:

$$g_i\ (X_1, X_2 \dots X_N)] \leq 0 \quad i = 1, M$$
$$X_j \geq 0 \quad j = 1, N$$

The solutions to this multi-objective programming problem result in a set of non-inferior solution defined as follows [2]:

"A feasible solution to a multi-objective programming problem is non-inferior if there exists no other feasible solution that will yield an improvement in one objective without causing a degradation in at least one other objective."

To be able to solve multi-objective programming models, one needs to be able to establish the relationships which ultimately will provide the trade-off between competing objectives. For example, in the case of of improvement in water quantity one needs to establish the <u>delta</u> or <u>change</u> in water quality for a unit removal of the pollutant of interest which is causing the water quality problem. The cost of removing each unit of the pollutant should be established as well. Clearly, the cost of pollutant removal increases as one moves towards greater improvements in water quality. The cost per unit of pollutant removed increases as one moves to higher levels of treatment in order to reduce the pollutant quantities to very low levels. For the problem presented in this paper a series of water quality models which predict both concentration of total phosphorus and probability of trophic state have been used to establish the relationships between removal of phosphorous and improvements in receiving water quality.

The paper illustrates a two-step process to utilize multi-objective programming techniques to demonstrate the implications of choice for decision-makers. In the first step, relatively simple water quality models are utilized to develop the anticipated improvements in water quality for each unit of pollutant removed. The information provided by these water quality models is then incorporated into a multi-objective programming model using the epsilon constraint approach for establishing the set of non-inferior solutions. Preliminary cost of treatment data is incorporated into the programming model as well. The two objectives considered are to minimize cost of treatment and to minimize the probability of eutrophic conditions occuring in the receiving waters.

The second step is to utilize a more detailed water quality model for the specific body of receiving water and to use more detailed cost information specific to the region under study. The two-step process enables a sequential series of investigations. If the results of the first step indicate that the particular goals can not be achieved, then the more detailed study need not be undertaken. However, if the initial step with simple models and preliminary cost data produces results which appear promising, more detailed region specific studies may be undertaken. An example is presented which show the application of this two-step process for investigation of nutrient removal from Saginaw Bay, Michigan.

THE SAGINAW BAY EXAMPLE

Saginaw Bay is part of Lake Huron bounded on three sides by the State of Michigan. Lake Huron is one of the three Upper Great Lakes and is jointly shared by the United States and Canada. Figure 1 shows the location of Saginaw Bay both with respect to the State of Michigan and also with respect to Lake Huron. As shown in Figure 1, Saginaw Bay is a shallow extension of the west side of Lake Huron extending into the lower peninsula of the State of Michigan. Historically, the inner portion of Saginaw Bay has had poor water quality. It has been characterized by high algal concentrations and low water transparency and has been generally described as eutrophic. Historically, these water quality problems have limited the value of the inner bay for recreational and water supply purposes. Efforts to reduce nutrient loadings have concentrated upon large point source discharges. While these efforts have reduced the total nutrient loadings to the bay, the water quality has not improved to the point where the trophic condition would be considered mesotrophic; the inner bay remains eutrophic. The present total phosphorous loadings to the bay result from agricultural non-point sources, point sources, and atmospheric deposition. Approximately 60% of the total phosphorous loadings now come from agricultural non-point sources. Whereas, only 20% arise from point source discharges. Multi-objective programming techniques are applied to illustrate the display of trade-off choices for decision-makers for further improvement in water quality in Saginaw Bay. In particular, the implications of trade-offs between water quality and further reduction of total phosphorous from point source discharges

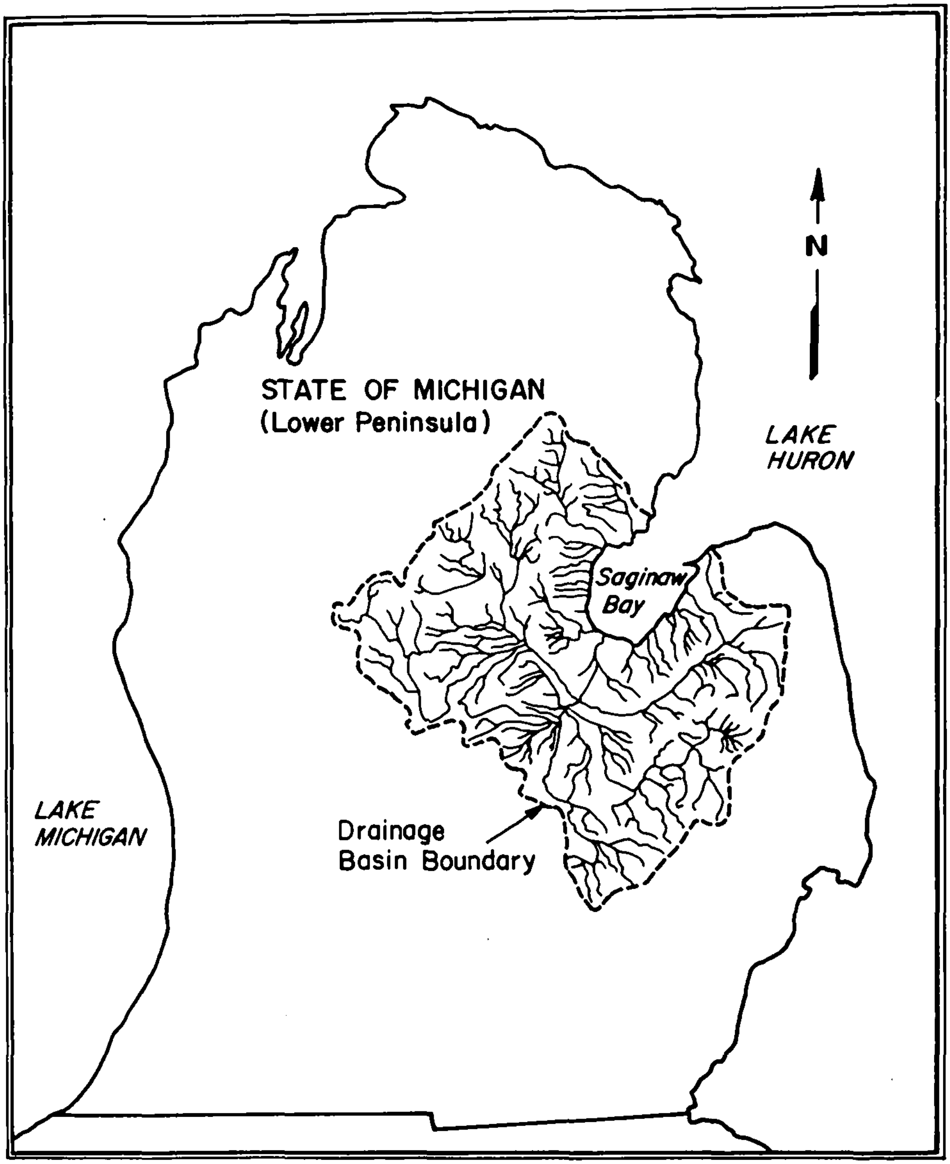

Figure 1: Inner Saginaw Bay Drainage Basin

and/or reduction of total phosphorous from agricultural non-point sources are presented.

Step 1: Preliminary Analysis

A phosphorous budget model for the Great Lakes was utilized initially to obtain phosphorous concentrations from total loadings and other relevant information [6]. The phosphorous concentrations obtained from the budget model were utilized in a second model [7] to obtain probabilities of trophic status as a function of phosphorous loadings. Elements from these two models have been combined in order to establish a means to obtain the change in water quality of Saginaw Bay - expressed as the probability of eutrophic conditions versus total phosphorous loadings from all sources to Saginaw Bay.

Next, information [8] upon treatment alternatives and the cost of such treatment to meet phosphorous objectives in the Great Lakes was utilized to formulate a multi-objective programming model with two objectives. The first objective was a cost objective. It was to minimize cost of treatment for nutrient (phosphorous) removal. The second objective was to minimize the probability of eutrophic conditions in Saginaw Bay. It is important to state that these models and cost data utilized in Step 1 should be considered as preliminary results only. The information contained in the output of the Step 1 analysis served as an indicator to the decision-makers as to whether or not more detailed studies were warranted. Figure 2 presents a set of non-inferior solutions obtained from a preliminary analysis. This preliminary information suggests that improvements in water quality can be made. The trade-off shown is a cost trade-off. As the probability of eutrophic conditions decrease, the annual cost of treatment to remove nutrients from point sources, rural non-point sources, and urban non-point sources increases. Based upon the results from the preliminary Step 1 multi-objective programming model, one may undertake a more detailed analysis with more complex models and site specific cost information.

Step 2: Detailed Analysis

In the specific case of Saginaw Bay, a more detailed analysis has been performed to evaluate cost effective measures to reduce basin phosphorous loads to the bay and to improve water quality in the Bay [9] [10]. This specific detailed study focused on evaluating the cost effectiveness of conservation tillage to reduce Saginaw Bay Basin phosphorous loads and improve water quality as compared to further reductions in municipal point source loads. The reason for the focus on conservation tillage arose from the fact that up to approximately 60% of the present total phosphorous loadings to Saginaw Bay comes from agricultural non-point sources. The detailed study had several specific objectives including the following:

(A) Determine the costs and effectiveness of conservation tillage in the basin to reduce phosphorous loads to the bay.

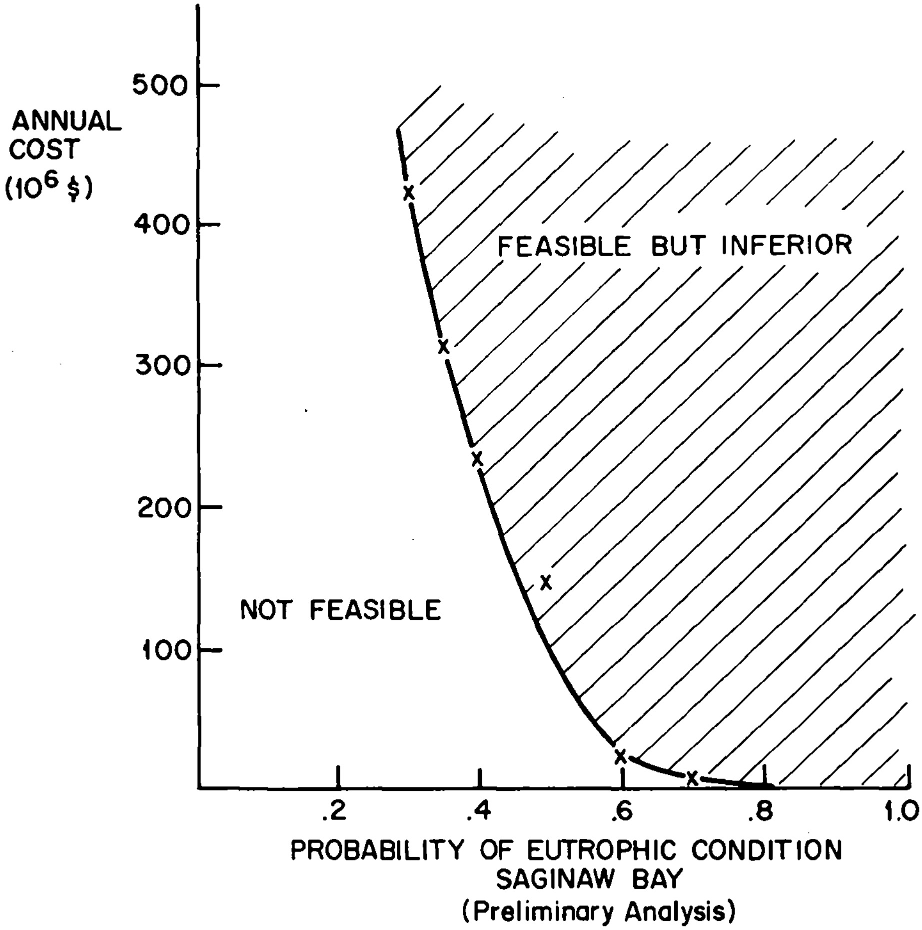

Figure 2. Tradeoff Between Cost of Treatment and Probability of Eutrophic Condition

(B) Determine the costs and effectiveness of additional municipal point source treatment in the basin to reduce phosphorous loads to the bay.

(C) Compare the cost effectiveness of conservation tillage and further municipal point source treatment towards reducing pollutant loads.

Figure 3 summarizes the results of this more detailed analysis. The implementation of conservation tillage in the agricultural area of the drainage basin is estimated to produce a net increase in farm income of $24.3 million per year [9]. The information presented in Figure 3 provides a trade-off between the additional cost of municipal point source control to achieve a specified total phosphorous target in the Bay versus implementation of conservation tillage as a means to achieve the target concentration. 100% implementation of conservation tillage would achieve a target total phosphorous concentration of roughly 19 μg/l. It would cost an additional $1.4 million/year to achieve a comparable target concentration for total phosphorous by increased point source control. Recall that the implementation of conservation tillage is estimated to generate an increase to net farm income of $24 million/year. The total phosphorous concentration of 20 μg/l is at the boundary between eutrophic and mesotrophic conditions. Finally, Figure 3 further indicates that if the decision-makers wish to further reduce the target phosphorous concentration loads below 18 μg/l then measures other than additional control of municipal point source discharges will be required. For example, a 40% reduction in agricultural non-point source nutrient loadings would be required to achieve a target total phosphorous concentration of 18 μg/l. Full implementation of conservation tillage in this region can be expected to reduce the agricultural nutrient load by sixteen percent.

OBSERVATIONS

Examples have been presented to illustrate the application of multi-objective programming models to rural water management. The examples have been focused upon the trade-offs between the minimum treatment cost objective and the improvement in water quality objective. A more detailed analysis has shown the trade-off between alternative treatment strategies to achieve target goals of total phosphorous concentration in receiving waters.

It is expected that these multi-objective technique for planning and analysis can be applied to additional topics of interest to rural water management. Erosion control and the contamination of both surface and ground waters by topics of interest to rural water management. Erosion control and the contamination of both surface and ground waters by toxic chemicals utilized by farmers are two areas which should lend themselves to application of multiobjective planning and analysis. Recently, ground water contamination has been specifically considered for multi-objective analysis [11].

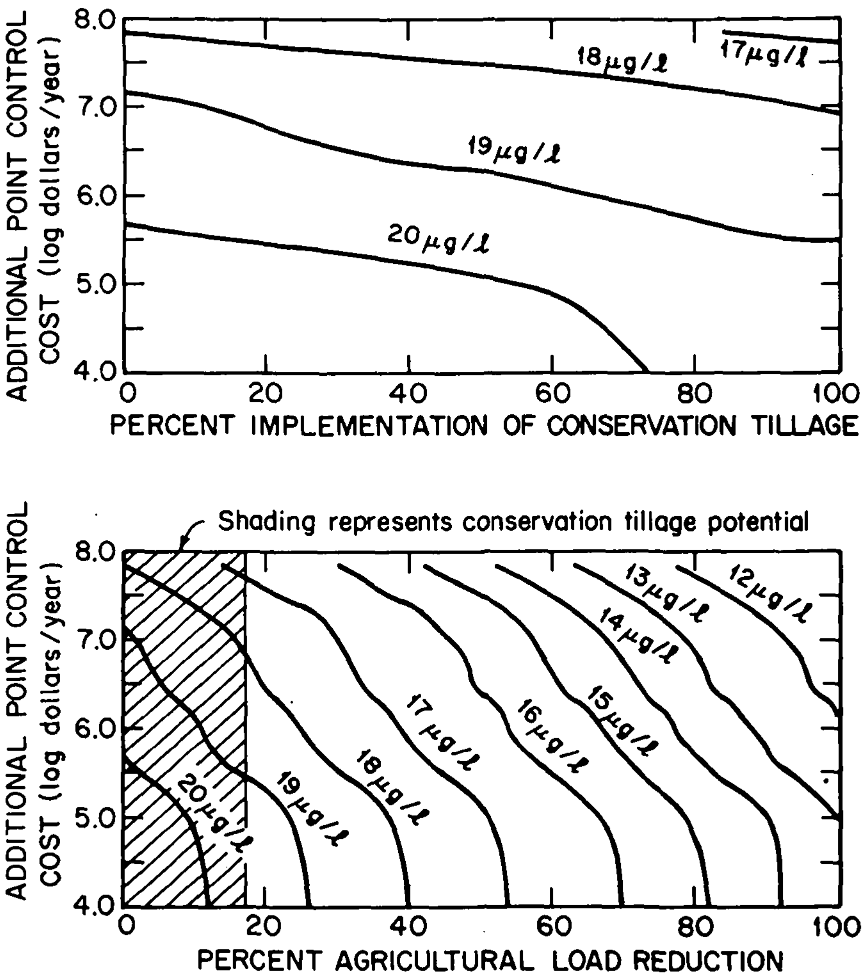

Figure 3: Tradeoff Between Point and Agriculture Non-point Source Phosphorous Controls Towards Meeting Average Total Phosphorus Targets

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Paper number 85 Aspect number 3

EVALUACION DE PROGRAMAS DE ABASTECIMIENTO DE AGUA POTABLE RURAL

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RESUMEN

El presente artículo entrega una metodología para la evaluación de proyectos de abastecimiento de agua potable rural en Chile. Las comunidades rurales chilenas se definen como aquellas que poseen menos de 3.000 habitantes. Sus abastecimientos de agua potable son diseñados por el Gobierno Central y generalmente operados por la comunidad.

En este trabajo se ha desarrollado un esquema de evaluación para cualquier comunidad rural chilena. Se ha supuesto que existe una fuente de agua de calidad satisfactoria y que cumple con los requisitos de calidad de agua estipulados en las normas chilenas referentes a aguas para fines potables. Como estas comunidades generalmente son bastantes remotas, un problema frecuente que enfrentan es que carecen de personal técnico idóneo para la mantención y operación de equipos mecánicos por lo tanto el sistema de tratamiento propuesto incluye sólo un equipo básico de cloración.

El proyecto sujeto a evaluación consta entonces de obras de captación de agua, tendido de la red, estanques de almacenamiento, sistema de cloración y programas de control de calidad. La ventaja principal del método propuesto es que con pequeñas modificaciones éste puede ser aplicado a cualquier comunidad real.

INTRODUCCION

Existen en Chile, aproximadamente 1.100 poblados rurales, cuyas poblaciones fluctúan entre 150 y 3.000 habitantes cada uno, sumando un total de más de 600.000 personas que no disponen de agua potable en su hogar, ni en ningún lugar del pueblo, debiendo idear cada familia algún procedimiento que le permita abastecerse de ella. Esta situación, ha producido en los habitantes de dichas comunidades, entre otros efectos,un menor nivel de vida que sus compatriotas residentes en comunidades que cuentan con agua potable.

La preocupación gubernamental por remediar esta situación, dotando a estos poblados de servicios de agua potable, se ha ido materializando desde el año 1964 hasta la fecha a través del "Programa Nacional de Agua Potable Rural", el cual está destinado a satisfacer esta necesidad. La instalación del servicio de agua potable a una comunidad rural, cuenta con apoyo financiero del Banco Interamericano de Desarrollo, BID.

El presente trabajo, tiene por objeto entregar una metodología general de evaluación para dichos proyectos de agua potable rural. El método empleado puede aplicarse a cualquier localidad rural chilena, variando sólo algunos de los parámetros empleados.

ESTUDIO DE COSTOS

Los costos relativos a proyectos de este tipo, pueden dividirse en dos categorías: costos de inversión y costos de operación. Los primeros se realizan en forma inicial y corresponden a los que se detallan a continuación :
- estudio de factibilidad
- dimensionamiento del sistema incluyendo la red de distribución, los estanques de almacenamiento y los equipos de bombeo
- compra de materiales, y
- construcción y puesta en marcha del sistema.

El Servicio Nacional de Obras Sanitarias, SENDOS, es responsable del programa de agua potable rural en Chile y ha definido una serie de normas y reglamentos respecto al dimensionamiento y construcción del sistema, por lo que resulta relativamente sencillo determinar sus costos de inversión. Los costos de operación son todos aquellos en que se incurre durante la vida útil de la instalación. Dentro de éstos, se incluyen, fundamentalmente los costos de mantención, operación (incluyendo cloración del agua) y administración.

No resulta necesario considerar los costos privados tales como la tarifa,ya que estos son sólo un traspaso de dinero de una parte de la comunidad a otra y no son relevantes en una evaluación que considere a la sociedad en su conjunto. En general puede decirse que son costos de estos proyectos , aquellos recursos que la comunidad distrae de otros posibles usos para dedicarlos a la materialización y mantención de un sistema de agua potable rural.

DETERMINACION DE LA DEMANDA

Los beneficios que obtienen los habitantes de una comunidad rural al contar con un sistema que entregue agua potable en su domicilio, son múltiples y muy variados. Entre estos, pueden citarse algunos de tipo netamente sanitarios tales como reducciones de la morbilidad por infecciones intestinales de origen hídrico, y mayor higiene en la preparación y manipulación de los alimentos. Otros son de naturaleza económica, como el aumento de pro -

ductividad a causa de la reducción de la morbilidad de ciertas enfermedades y el ahorro en gastos médicos. Finalmente, también pueden citarse algunos beneficios nacionales, es decir aquellos que afectan al país en conjunto,tales como la disminución de la emigración de los habitantes de las zonas rurales a zonas urbanas con la consecuente disminución de la marginalidad, y el mejoramiento en el nivel de vida de los pobladores en lugares apartados.

Debido a la diversidad, naturaleza e interdependencia de los beneficios antes enumerados, se hace muy difícil cuantificarlos en forma separada y detallada. Es por ello que se adoptó una metodología de análisis que toma en consideración la mayoría de los beneficios, por lo cual los errores que se cometan al evaluarlos serán siempre por defecto, es decir si un proyecto resulta beneficioso, éste será probablemente aún de mayor beneficio pero no a la inversa.

El planteamiento anterior permitió generar una curva de demanda, cuyo objeto fue simular el valor que los usuarios asignan al agua potable para los distintos niveles de consumo. Esta curva de demanda refleja el valor marginal que la comunidad asigna al consumo de cada unidad de agua potable,o sea, el precio máximo que un usuario está dispuesto a pagar por dicha unidad. Se ha supuesto además, que el beneficio marginal que la comunidad obtiene del consumo de esa unidad es igual a ese precio máximo. El beneficio total que la comunidad asigna a cada unidad de agua potable viene a ser la suma de cada uno de los múltiples beneficios que obtiene por disponer de ella.

El procedimiento para elaborar la curva de demanda, no se basa en encuestas ni en otros sistemas empleados frecuentemente para conocer el valor que los consumidores otorgan a algún producto determinado. La razón de lo anterior estriba en el hecho que, al desconocer los pobladores los beneficios que acarrea el contar con agua potable, no pueden asignarle un valor al agua potable. Para muchos residentes de comunidades rurales el vivir sin agua potable es natural, por lo cual sus respuestas a una encuesta serían de poca utilidad.

La curva de demanda se generó definiendo inicialmente dos puntos por la cual ésta debía pasar. Enseguida, se ajustó ésta de manera tal que refleje los supuestos y las imposiciones que se hicieron sobre ella. El punto inicial A, de coordenadas (Qa, Pa) (ver Figura 1), refleja lo que la comunidad está dispuesta a pagar por obtener una cantidad que, aunque mínima, sea suficiente para satisfacer las necesidades básicas de los menores (en este estudio, se consideró la población más propensa de contagio por enfermedades de origen hídrico a la constituida por niños de 0 a 4 años). El valor asociado al punto A, representa el precio máximo del agua potable. La cantidad Qa , definida anteriormente como la cantidad mínima, se calcula a partir de datos estadísticos y demográficos. Así, Qa resulta de multiplicar la cantidad mínima necesaria por niño y por día por el total de población de la localidad y por el porcentaje de menores en ella con posibilidades de enfermar a causa de no contar con agua potable. Por otra parte, Pa, el precio máximo se estima en base a la productividad de los infantes que sobreviven las enfermedades de origen hídrico. Este método para estimar Pa,elimina la subjetividad normalmente asociada a la evaluación de proyectos sociales. Cabe destacar que esta estimación entrega un valor mínimo para Pa desde el punto de vista social, ya que el impacto causado por la muerte de un menor puede,y de hecho lo hace, elevar notoriamente este valor. El valor de Pa obtenido basado en un cálculo de productividad, es posteriormente actualizado a la fecha de nacimiento de la persona, empleando una tasa de descuento adecuada. Se tiene entonces que :

$$Qa = \lambda \cdot N \tag{1}$$

en donde N representa el número de habitantes de la localidad y λ es el factor de proporcionalidad anteriormente definido. La productividad total Vp, por otra parte, queda representada por :

$$Vp = V \cdot T \cdot N \qquad (2)$$

en donde V es el valor de la productividad actualizada de los menores sobrevivientes y T el porcentaje de éstos referidos a la población total N. De lo anterior se deduce que el precio social de las primeras Qa unidades, es $Pa = Vp/Qa$ en $\$/m^3$.

El segundo punto de la curva, B(Qb,Pb) considera un nivel de consumo correspondiente al que se tendrá en promedio cuando esté en funcionamiento el servicio, y depende fundamentalmente de los hábitos de vida de los residentes de la comunidad y de las características de la zona. Qb se estima a partir de datos estadísticos de lugares similares que ya cuentan con conexiones de agua potable. En cuanto a Pb, éste corresponde a la tarifa del sistema y se determina a partir de sus costos de operación. Se considera que éste es equivalente al precio (beneficio) social ya que el usuario puede regular la cantidad consumida, situándose en un punto en que el costo marginal (tarifa) sea igual al beneficio marginal de la última unidad consumida (Qb).

La curva de demanda está, además sujeta a las restricciones siguientes :

i) Se supone que una vez consumidos los primeros Qa m^3, la sociedad como agente económico, reconoce en el consumo de agua otros beneficios adicionales. A consecuencia de ello está dispuesta a pagar por las unidades siguientes de agua potable ya que éstas le aportarán mayor bienestar.

ii) Se ha aceptado que el cambio en el valor, o beneficio, que la sociedad asigna al consumo de agua potable, varía en forma contínua frente a cambios en el consumo. Esto es equivalente a suponer, que no existen factores que produzcan cambios discontinuos en la valoración de la productividad y de los recursos desaprovechados por la sociedad a medida que aumenta el consumo.

iii) Se ha supuesto que para niveles crecientes de consumo, el valor de uso del agua por cada unidad que se agrega (beneficio marginal), disminuye a medida que se satisfacen las necesidades básicas.

Basándose en las suposiciones anteriores, se ha tomado como representativa de la demanda entre los puntos A y B, una hipérbola equilátera (curva de elasticidad constante) cuya expresión analítica es $P = k\ Q^z$, por considerar que es la que mejor satisface las restricciones impuestas. La constante k, incluye todos los factores distintos al precio, y z, la elasticidad, es un parámetro interno del modelo ya que A y B son establecidos previamente considerando las características propias de cada localidad.

La curva de demanda se genera para el año 0 y luego para los n años siguientes de vida útil del proyecto. Esto significa que los costos y beneficios que implica el sistema de abastecimiento de agua potable, se obtendrán durante esos n años, al cabo de los cuales debería, en teoría al menos, reevaluarse el proyecto.

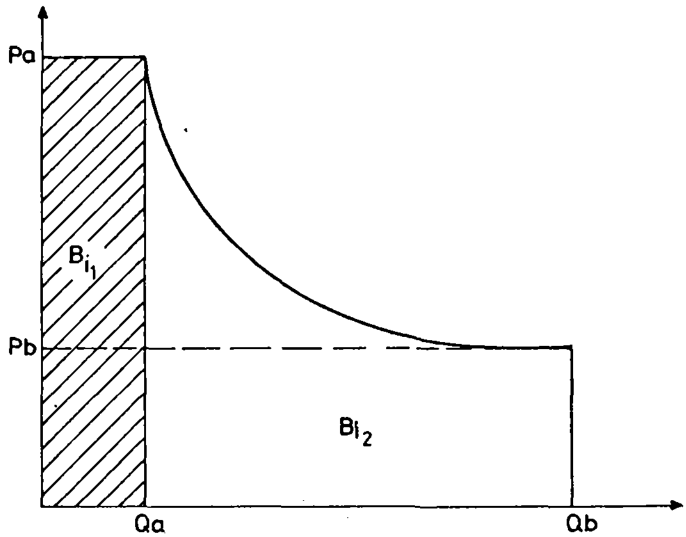

Figura 1. Curva de demanda.

Así, la ecuación (1), al considerar λ constante, permite determinar $(Qa)_i$ para todo el período ya que :

$$(Qa)_i = \lambda \cdot Ni \qquad i = 0,\ 1,\ 2 \ldots\ldots n \qquad (3)$$

en que

$$Ni = N(1+r)^i \qquad (4)$$

con una tasa de crecimiento igual a r, y por lo tanto

$$(Qa)_i = \lambda N(1+r)^i \qquad (5)$$

La ecuación (5) hace que el punto A (Qa_i, Pa) se desplace horizontalmente a un ritmo de crecimiento igual al de la población ya que Pa permanece constante para toda la vida útil del proyecto. Análogamente, se puede demostrar que B(Qb_i, Pb) también varía según el aumento de la población con Pb constante. En el Apéndice 1 se entrega un ejemplo de cálculo, empleando valores típicos para una comunidad rural chilena.

METODOLOGIA PARA EL CALCULO DE BENEFICIOS

Cada una de las curvas de demanda definidas anteriormente, representa el beneficio social obtenido por el consumo de cada unidad de agua, por lo que para calcular el beneficio social que implica el total del consumo, es necesario integrar en todo el rango de consumo, o lo que es análogo, calcular el área bajo la curva de demanda. Para estos efectos, el beneficio social en el año i, Bi, se divide en dos puntos Bi_1 y Bi_2 tal como se ilustra en la Figura 1. El área Bi_1 resulta de multiplicar Qa_i por Pa_i, o sea es el beneficio social proporcionado sólo por esas Qai unidades de agua potable. Por otra parte, Bi_2 queda determinado por el total del área bajo la curva entre Bi_1 y Bi_2, luego el beneficio total para cada año i del proyecto, queda definido por la siguiente expresión :

$$B_i = Pa_i \cdot Qa_i + \int_{Qa_i}^{Qb_i} k_i Q^{z_i}\, dQ \qquad (6)$$

cuya integración resulta en

$$Bi = Pa_i\ Qa_i + \frac{k_i}{z_i+1}\left(Qb_i^{z_i+1} - Qa_i^{z_i+1}\right) \qquad (7)$$

La sumatoria de los Bi, actualizados mediante la tasa de descuento social, entrega el valor actual total de los beneficios sociales.

CONCLUSIONES

La metodología expuesta, fue empleada en la evaluación de un proyecto de agua potable para una pequeña comunidad rural chilena. Se pudo observar que el método entrega resultados confiables y relativamente objetivos, considerando que se trata de una evaluación de tipo social. La generación de la curva de demanda requiere la determinación de varios parámetros, las cuales, a su vez demandan un exhaustivo estudio estadístico de datos de población, mortalidad, consumos de agua potable y productividad. Estos son fundamentales para una buena aplicación del modelo sugerido.

El método de estimación de demanda y de cálculo de beneficios permite, objetivamente, jerarquizar prioridades, de modo que al contar el país con recursos relativamente limitados, se pueden realizar primeramente aquellos proyectos que resulten de mayor beneficio social.

Los beneficios y costos del proyecto fueron comparados entre ellos usando técnicas convencionales de evaluación. Así, se calculó el valor actualizado neto, la tasa interna de retorno y la razón beneficio-costo. También se llevó a cabo un estudio de sensibilidad, llegándose a la conclusión que la tasa social de descuento tiene gran incidencia en la evaluación, más aún que variaciones en los costos de inversión o en los costos de operación.

APENDICE

Generación de la curva de demanda

Punto A

$Qa_i = \lambda\ Ni$
Cantidad mínima de agua para sobrevivir : So (2 litros)
Porcentaje de niños entre 0 y 4 años : Obtenido de datos estadísticos; niños menores de cuatro años en población rural en relación al total de la población rural : Y (en %) (13%)
$\lambda = 0{,}01\ So.Y$
Población total de la localidad en el año i : Ni
$Pa = (Tr - Tu) \cdot Ni \cdot U$
Tr y Tu : Tasas de mortalidad infantil atribuibles a enfermedades de origen hídrico para zonas rurales y urbanas respectivamente.
U = valor presente de la productividad por persona.

Punto B

Qb_i, obtenido de estadísticas de consumo
Pb_i, también basado en un estudio estadístico, además de ciertas imposiciones reglamentarias. En todo caso este valor cubre costos de operación y mantención y considera también la capacidad de pago de los habitantes de la localidad.

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CALCULATION OF FLOOD HYDROGRAPHS USING SATELLITE-DERIVED LAND-USE INFORMATION IN THE DREISAM WATERSHED/S-W GERMANY

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ABSTRACT

Flood hydrograph calculations have been carried out using the SCS TR 20 unit-hydrograph model and a data-bank, describing the properties of the 257 km^2 Dreisam-watershed according to the input requirements of the model. The data-bank contains the input-parameters land-use, soil properties and slope for each point in the watershed with a resolution of 65x 104 m.

The land-use information has been gained through a supervised maximum-likelihood classification of a 4-channel digital LANDSAT imagery using well-known training sites as definition for the land-use classes.

The calculations of the flood hydrographs have been carried out with an uncalibrated model to be able to estimate the accuracy of the calculations for ungauged areas. Results of the calculations for several storms have been compared to measured hydrographs. The peak discharge and the runoff volumes of the calculations lie within a 20 % range around the measured values. Calculations of desing floods using rainfall events of selected return periods correspond well with the design floods estimated from measured data.

The management of the data-bank using spatially distributed parameters makes it possible to simulate influences of future land-cover changes on the flood peaks and volumes. The possible change in the runoff characteristics of the watershed caused by forest damages has been simulated assuming five different scenarios for the damage. The assumptions about the damage distribution include elevation- as well as aspect-influences on the degree of damage and a change in vegetation as well as in soil properties. Assuming these scenarios an increase in peak discharge by a factor of 2.2 to 5 has been calculated.

Keywords: Flood hydrograph calculations, remote sensing, geographical data-banks, deforestation, design flood calculations

INTRODUCTION

One of the aims of the modelling of flood hydrographs during the last decades was to develope models on a, as much as possible, universal basis to be able to extend their application into ungauged areas with a minimum of additional measurements. These models mostly were developed before the first satellites transmitted data about the Earth's surface. They therefore do not have a structure, that is best suited for their application with remote sensing data, which is commonly represented by a spatial matrix of recflection values. The SCS TR 20 in parts offers a structure, that allows the use of remote sensing data or results of the analysis of remote sensing data, like a land-use classification together with other informations about the watershed under consideration.

Remote sensing allows to establish geographical data-banks, which are structured as layers of different parameters in a spatially distributed form (as maps). This concept offers great flexibility and easy access to different input-parameters of the model, like the land-use, soil informations, minimum infiltration rates, storage capacity, slope, aspect, elevation, drainage network etc. This is done in a way, that the different layers are spatially connectable (that means, that there is information on the position of each value in the data-bank) (Ref.1). This also allows flexibility in the simulation of changes and their influence on the runoff characteristics of the watershed.

One great adventage of remote sensing data, espetially of LANDSAT imagery, is their availability for all parts of the world. The properties of the data and its quality does not change in different parts of the world and changes can be detected through successive overflights over the same area.

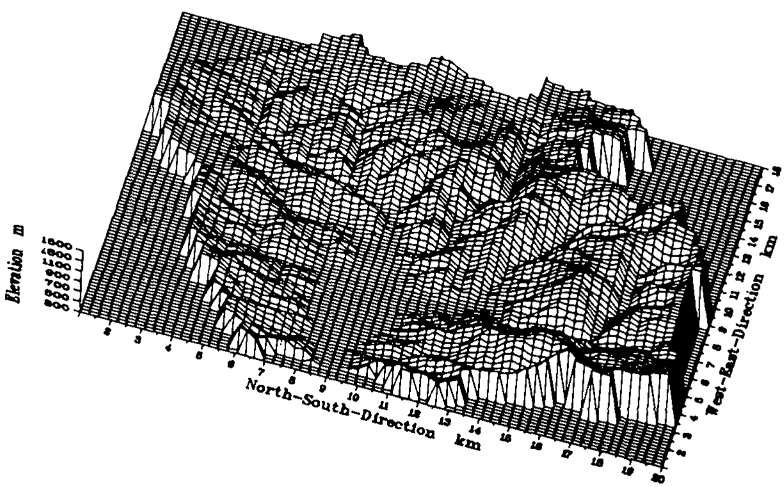

Fig.1: The topography of the Dreisam-watershed

In the course of this text an example will be given of how remote sensing data together with ground based information can be used as input to the SCS TR 20 model. The area under consideration is the Dreisam-watershed (Fig.1) in the Southern Black Forest near the town of Freiburg. It covers an area of app 257 km^2 with an elevation difference of 1200 m. About half of the watershed's area is covered with forest, the other half is covered with meadows and agricultural land (in the lower part of the watershed. The hydrological situation is only slightly altered from the natural condition for the area. The main channel in the plain (Fig.1) has been trained and the population density is low, which leads to a low runoff potential of the watershed.

THE GEOGRAPHICAL DATA-BANK

The geographical data-bank is organized as a 3-dimensional matrix with two spatial dimensions and the third dimension representing the different input parameters. Each point of the matrix has a size of 64x104 m on the ground, so that the whole watershed is represented by app. 36000 points, each having the values of all input parameters at the same time.

Basic input information to the data-bank is a land-use classification of a LANDSAT-2 image, that was recorded on Aug. 9, 1975 (pass 210/row 27). The classification was carried out using the maximum-likelihood algorithm. Well known training areas for the classes high-density-, medium-density- and low-density-residential, conifer-, mixed-, decidious-forest, grass and agriculture were used and each point in the watershed assigned to one of the land-use classes according to its reflection-values in the four spectral channels of the satellite.

The point-by-point classification accuracy when compared to a 1:25000 topographic map (which has to be considered the only alternative for the land-use input to the model) was determined to be 80.5 % when using the land-use classes high-density-, medium-density- and low-density-residential, forest, grass and agriculture. The resolution of the original LANDSAT-data was lowered to 64x104 m to be able to overlay the geocorrected line-printer outputs of the ASTEP-program package (Ref.2) on the 1:25000 topographic map.

As second layer in the data-bank the soil has been classified according to the SCS soil type classification. The SCS soil type classification (Ref.3) classifies each soil into one of 4 classes according to its min. infiltration capacity and storage capacity. As a result of field investigations and an earlier publication (Ref.4) the hillier regions of the watershed were classified into soil class B (min. infiltration cap. 4-8 mm/h, storage capacity 250 mm) whereas the valleys and the plain of Fig.1 were classified into soil class C (min. infiltration cap. 1-4 mm/h, storage cap. 150 mm).

A digital terrain model of the watershed was developed by reading off the elevation for each point in the data-bank from a 1:25000 orographic map. The slope, which is the third layer of data in the data-bank, was calculated for each point as the maximum slope between two opposing neighbours of that point. It was then classified into 26 classes between 0 and 100 % on a logarithmic scale.

In addition to this spatially distributed data of the watershed channel cross-sections were measured at 46 representative points in the watershed. The flow velocities for different discharges at these points were calculated using Manning's equation.

THE SCS TR 20 MODEL

The SCS TR 20 model is based on the unit hydrograph approach. The model needs the following input parameters (Ref.3):

1) the areas of the subwatersheds (since it is not possible to carry out thethe calculations for the whole watershed at one time, the watershed has to be divided into 46 subwatersheds)
2) the time of concentration for each subwatershed, that means, the time the water needs to flow from the hydraulically most remote part of the watershed to the outlet
3) a parameter CN, which represents the storage capacity of the land-use - soil type complex. The values of CN for different land-use - soil type complexes has been tabulated by the SCS for a large variety of land-uses. The values were used as a basis for the determination of the CN-values of the subwatersheds
4) channel cross-sections and area-velocity relations for the flood-routing and the connection of the subwatersheds

The effective rainfall is calculated using the equation:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad (1)$$

where P is the precipitation (in inch), S is the storage (in inch) and Q is the effective rainfall (in inch). S is calculated through:

$$S = \frac{1000}{CN - 10} \qquad (2)$$

The time of concentration Tc for the subwatersheds, into which the watershed has been divided for the calculations (A less than 8 km^2) can be calculated using Eq.3:

$$Tc = \frac{l^{0.8}(S + 1)^{0.7}}{1900Y^{0.5}} \qquad (3)$$

where l is the hydraulic length (in feet), S is the storage capacity of the soil (see Eq.2), Y is the average slope of the subwatershed (in %) and Tc is the time of concentration (in hours).

RESULTS OF THE FLOOD HYDROGRAPH CALCULATIONS

Since the SCS TR 20 model calculates surface runoff it can only be used in the Dreisam-watershed for floods with a return period longer than two years with sufficient accuracy. This is due to the fact, that small events do not produce surface runoff. The raingauge network in the watershed consists of 9 stations of which one station records the rainfall intensities with a resolution of 0.5 hours. The measured values at the raingauges were input to the model. The THIESSEN-method was used to compute areal rainfall and its spatial distribution over the watershed. The calculations were carried out for selected Summer storms. The criteria for the selection of the Summer storms were:

1) the events fall into the period from May to September
2) the events were sufficiently large to produce surface runoff
3) all gauges in the watershed operated at the time of the event

As a result of the calculations Fig.2a+b show the rainfall distribution and the resulting measured and calculated surface runoff.for the 7-29-61 storm. The typical deviation of the calculated to the measured hydrographs in terms of peak discharge and runoff volume are of the order of 15 %. No calibration has been carried out.

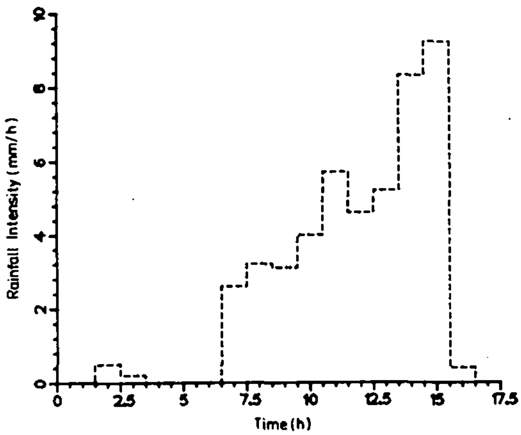

Fig.2a: Measured rainfall-distribution for the 7-29-61 storm

As a result of the frequency analysis of the peak annual Summer runoffs and 24h rainfall values over a period of 30 years the 24h rainfall values for return periods of 2, 5, 10, 50 and 100 years were chosen as input to the model. The SCS-Type II storm distribution was chosen for the temporal distribution of the rainfall. The results of the calculations in terms of peak discharge are shown in Fig.3. It can be seen, that for short return periods there is a great difference between the calculated and the measured peak discharge. For longer return periods this difference becomes smaller. This underlines the influence of the interflow on the peak discharge for events with a short return period.

Assuming, that the basic hydraulic of the surface runoff and the interflow show similar dependencies on the input parameters, the interflow was calculated using the TR 20 model structure and changing the values of the input-

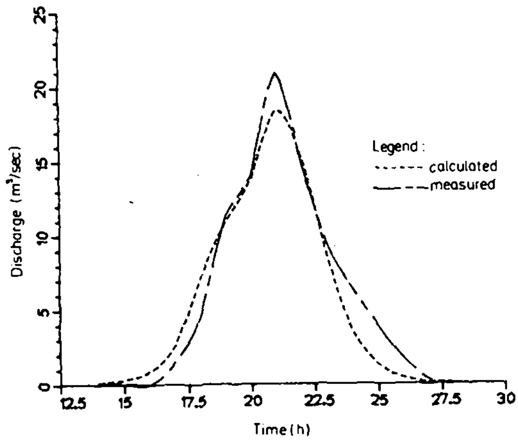

Fig.2b: Measured and calculated surface runoff for the 7-29-61 storm

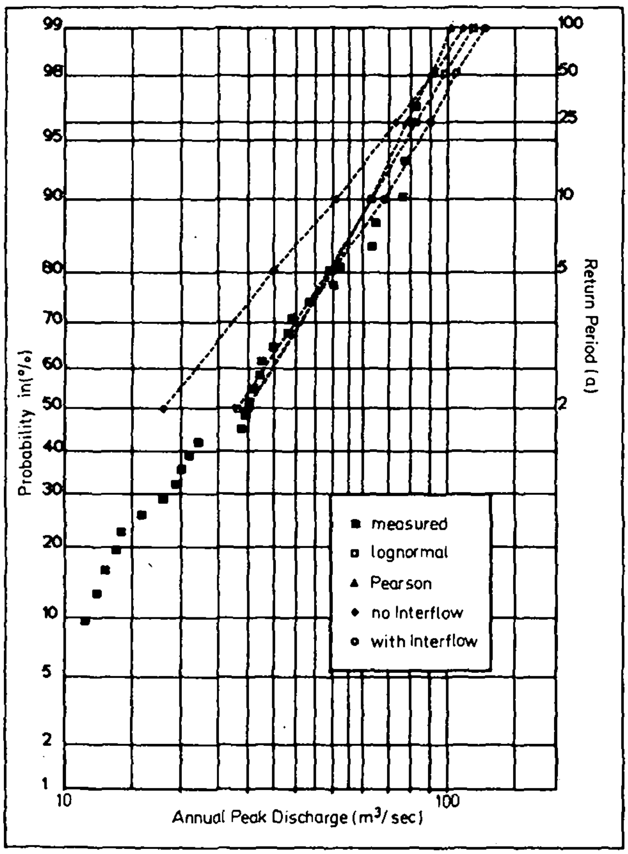

Fig.3: Calculation of peak discharges for selected return periods and comparison with observed data (lognormal- and Pearson-dis. as fit)

parameters. To be able to do this is was necessary to modify the surface runoff CN-values and the times of concentration of the subwatersheds. A closer analysis of the 7-29-61 storm showed, that the timing of the interflow-hydrograph can be accuratelly calculated by multiplying the Tc-values of Eq.3 with a factor of 7.2. The interflow volume is largely depending on the baseflow before the event, which is an indicator for the amount of water, that is stored in the soil. The events, which were used for the calculations of the flood hydrographs showed, that a modification of the surface runoff CN-value through:

$$CN_{Int} = CN_{Surf} + 1.4 + 0.5 \cdot \text{Baseflow} \qquad (4)$$

with the baseflow given in m^3/sec, determines the interflow volume to a sufficient accuracy.

The calculated interflow hydrograph was added to the surface runoff hydrograph. The frequency calculations, that have been carried out using the interflow in addition to the surface runoff are shown in Fig.3. It shows, that the inclusion of the interflow into the calculations considerably improves the results of the peak discharges for short return periods. The results for longer return periods are still within 15 % of the measured peak discharges.

Additional calculations also showed, that the influence of the interflow on the peak discharge of an event decreases with the magnitude of the peak discharge. This is in accordance with the observations.

Fig.4 shows the results of the calculations of the interflow for the 7-29-61 storm. The baseflow before the event was 1.9 m^3/sec, which is almost the average baseflow for the Summer mounths (2.34 m^3/sec). The overall shape of the calculated hydrograph now much better fits the measured total hydrograph, although for this particular event the peak discharge is too large.

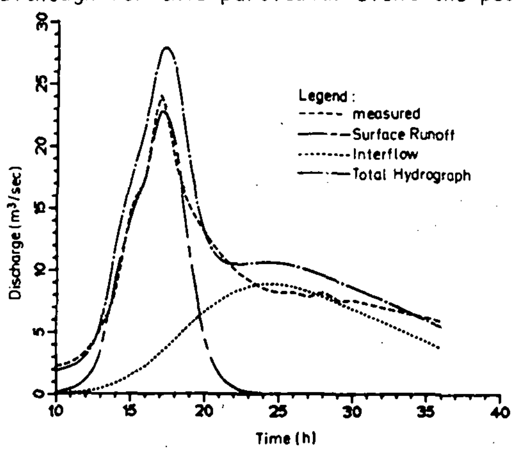

Fig. 4: Comparison of the calculated total hydrograph using interflow calculations with the measured hydrograph

SIMULATION OF THE INFLUENCE OF A CHANGE IN FOREST COVER ON THE FLOOD FREQUENCIES

The forested areas in the Dreisam-watershed are subject to severe damages caused by air pollution and app. 50 % of the conifers are already affected. Therefore it can be foreseen, that a drastic change of the watershed's land-cover will take place in the near future. An attempt has been made to quantify the influences of this deforestation on the future flood frequencies. The following scenarios for the development of the damage were assumed:

1) all presently damaged trees will die, no presently healthy tree will be damaged in the future and the substitute vegetation will be bushes (successfull reforestation). The degree of removal of the conifers is depending on both the elevation and the aspect (max. damage is known to be at an altitude of 900 m on SW-exposed slopes). The degree of damage is assumed to decrease linearly with higher and lower altitudes and also linearly with the aspect changing from SW to NE.
2) same as 1) but substitute vegetation will be grass (no reforestation possible).
3) all conifers will be removed and the substitute vegetation will be grass
4) same as 3) but the soils in the hilly parts of the watershed will be degraded to the SCS soil type C (see Cap.2)
5) same as 4) but all trees will be removed and the substitute vegetation will be grass

For the different scenarios the CN-values and times of concentration were determined using the TR 20 model. The procedure was identical to the one described above. Fig.5 shows the results of the calculations, that have been carried out using the same design storms as in Fig.3. For scenario 1 the 100 years return period peak discharge is increased from presently 110 m^3/sec to 190 m^3/sec. Assuming the worst case, that all trees will be removed and the soil will be degraded through a decrease in storage capacity the 100 years return period peak discharge will increase to 500 m^3/sec according to these simulations.

CONCLUSIONS

It has been shown, that a land-use classification of LANDSAT imagery on the basis of a 1:25000 map scale and a resolution of 64x104 m together with informations on the soils and slopes in the watershed form a geographical data-bank (stored as a 3-dim. matrix) for the use with the SCS TR 20 model. With this structure it is possible to calculate peak discharges with an accuracy of 15-20 % for the Dreisam-watershed. The availability of LANDSAT data on a worldwide basis offers the chance to establish similar data-banks for other parts of the world. Especially a possible future stereo-coverage might extend the applicability of this method by including topography gathered from remote sensing data into the data-bank. Since the tabulated CN-values include a large variety of different land-uses an extension of the application of this method into different climatic regions of the world seems possible.

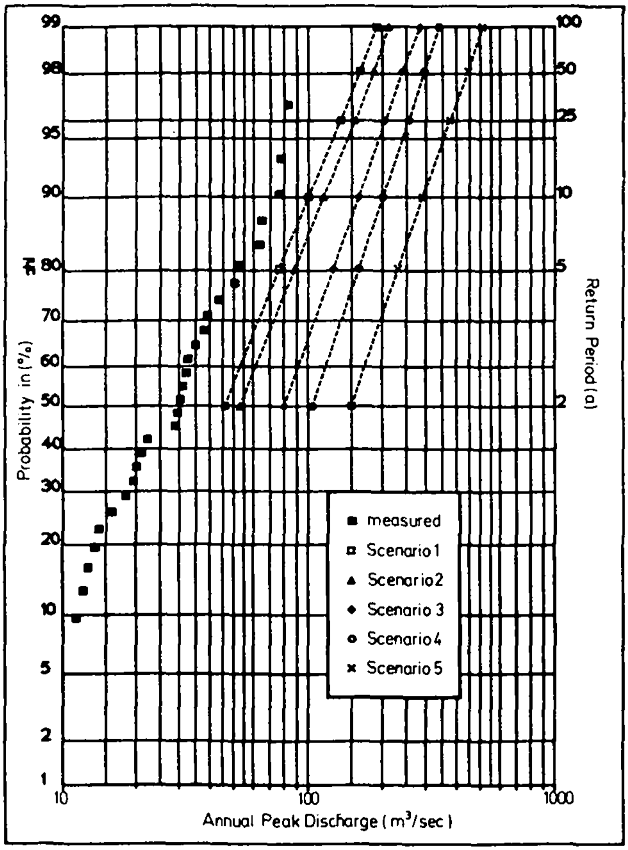

Fig.5: Calculations of the peak discharges for given return periods and 5 scenarios for the future damage of the forest in the watershed

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IWRA Vth WORLD CONGRESS ON WATER RESOURCES
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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 88 Aspect number 10

USE OF PHYSICAL OBJECTIVE FUNCTIONS IN RESERVOIR SYSTEM OPERATION

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ABSTRACT

In most reservoir operation problems several purposes must be included in the objective function. When the purposes are non-commensurate, it is difficult to define a single economic or other objective function. In case of multiple objectives several criteria have been developed to find the optimal or near optimal solution, but the set of non-dominated solutions has to be obtained first through mathematical modelling. In both cases, a model which selects one purpose as the main one to be optimized and the others to be kept in the constraints, can be of significant help. The original reservoir operation problem can be analized in a deterministic or a stochastic framework. The purpose of this paper is to present three examples in which such an approach was successfully applied.

Keywords: Optimization, reservoir systems, reservoir operation, objective function, hydroelectric power, low-flow augmentation, dynamic programming, multiple objectives

INTRODUCTION

Three examples of reservoir operation models will be presented. In all of them a physical objective function, measured in the same units as the main purpose, was defined and optimized through a dynamic programming algorithm. Other purposes of the reservoirs were kept in the constraints and parametric variations were performed in order to obtain the complete set of optimal solutions. In the first example the energy output (in KWh) is to be maximized in a system of hydroelectric power plants whose reservoirs serve other purposes. A deterministic model using historical record of inflows is developed for this case. In the second example, low-flow (in m^3/s) is to be maximized at a control gage located below a system of reservoirs with multiple purposes. Here again, a deterministic model was used for each reservoir in a sequence. In the third example, a reservoir for hydropower production as a main purpose is analyzed in a stochastic framework. In this case the expected value of the energy output (in KWh) is maximized while other purposes are kept in the constraints.

DETERMINISTIC MODEL FOR HYDROPOWER

In this example (Fig. 1) a multipurpose reservoir (Folsom in northern California) was analyzed (Harboe et al., 1970). The reservoir is operated for flood control, mandatory releases, firm water supply and firm on-peak energy production. A dynamic programming model was developed in which all purposes except hydropower were taken care of through constraints of the model. Further, since the maximization of firm on-peak energy was required, a so-called max-min objective function was used. This type of objective function allows the definition of the following operating rule for the reservoir:

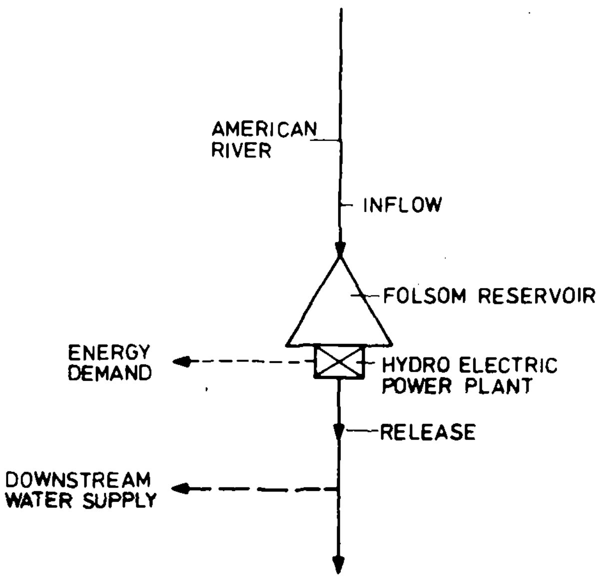


FIGURE 1: FOLSOM RESERVOIR SYSTEM

The release in any month is set equal to the largest of four calculated releases, namely:

- Release necessary to produce the monthly on-peak firm energy (optimal value)
- Release necessary to satisfy the water supply contract (as in constraint)
- Release to satisfy "mandatory releases"
- Release necessary to satisfy the flood control reservation.

This rule can be applied in a simulation of the operation with historical record and all constraints and the energy contract will be met with 100% security. Certainly, several values of the constraints should be analyzed. As an example, Figure 2 presents the variation of the firm water supply constraint and its effect of the firm energy production. From these results, the decision makers can choose the appropriate combination of outputs according to their objectives.

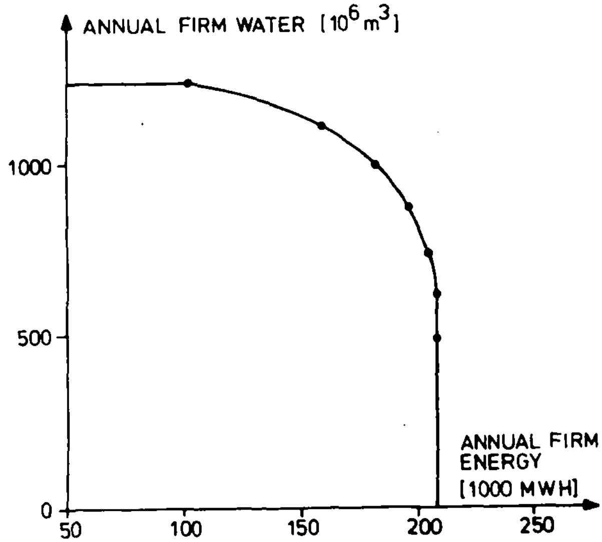
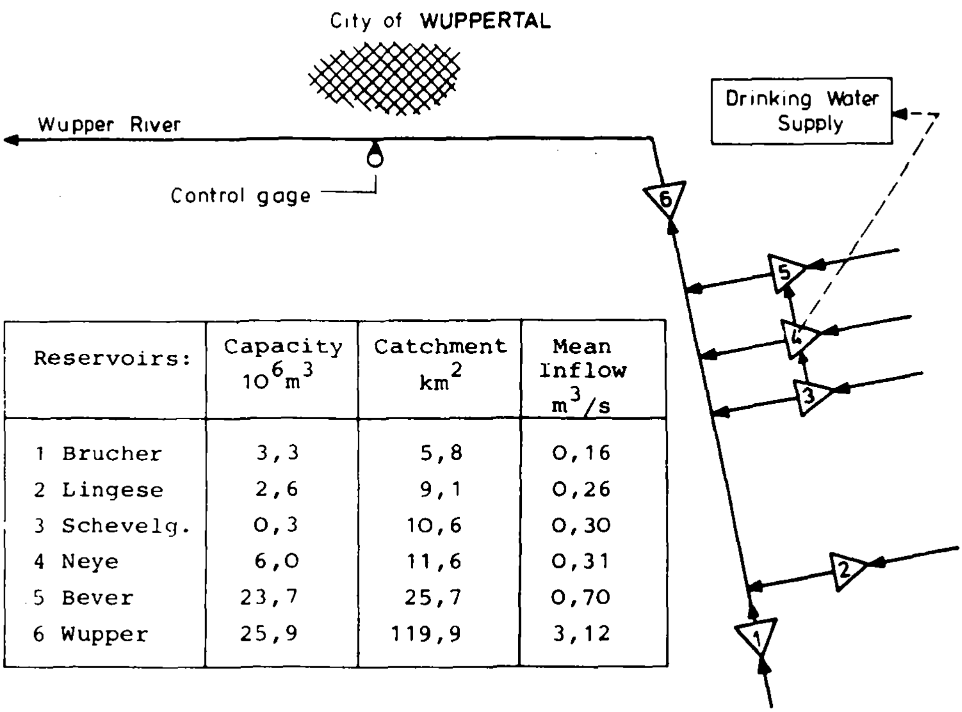


FIGURE 2: FIRM WATER VS. FIRM ENERGY FOR FOLSOM RESERVOIR (TRANSFORMATION FUNCTION)

LOW-FLOW AUGMENTATION

The system is presented in Figure 3. It corresponds to the upper Wupper River in the Federal Republic of Germany (Boehle et al., 1979). The six reservoirs have the following purposes: flood control, minimal releases, recreation and low-flow augmentation at a control gage.



Reservoirs:	Capacity $10^6 m^3$	Catchment km^2	Mean Inflow m^3/s
1 Brucher	3,3	5,8	0,16
2 Lingese	2,6	9,1	0,26
3 Schevelg.	0,3	10,6	0,30
4 Neye	6,0	11,6	0,31
5 Bever	23,7	25,7	0,70
6 Wupper	25,9	119,9	3,12

FIGURE 3: THE WUPPER-RIVER SYSTEM

A sequential application of dynamic programming models to the reservoirs was performed (Boehle et al., 1982). Low-flow was chosen as a main purpose and thus maximized, while other purposes were satisfied through constraints.

The deterministic model yields the results shown in Table 1. Again, an operating rule can be developed for each reservoir (Harboe, 1983).

TABLE 1: OPTIMUM AUGMENTED FLOWS AT CONTROL GAGE

<table>
<tr><td colspan="2" rowspan="2"></td><td colspan="2">Optimum minimum flow at control gage</td><td colspan="2">Length of historical sample: 32 years</td></tr>
<tr><td>$10^6 m^3$/month</td><td>m^3/s</td><td>Augmented years</td><td>Augmented months/year</td></tr>
<tr><td colspan="2">Flow from catchment between Wupper reservoir and control gage</td><td>0.588</td><td>0.23</td><td>-</td><td>-</td></tr>
<tr><td rowspan="4">Flow augmented by reservoirs</td><td>Bever</td><td>3.958</td><td>1.53</td><td>31</td><td>130/31=4.2</td></tr>
<tr><td>Bever
Schevelinger</td><td>3.998</td><td>1.54</td><td>30</td><td>120/30=4.0</td></tr>
<tr><td>Bever
Schevelinger
Lingese</td><td>4.339</td><td>1.67</td><td>30</td><td>125/30=4.2</td></tr>
<tr><td>Bever
Schevelinger
Lingese
Brucher</td><td>4.744</td><td>1.83</td><td>30</td><td>124/30=4.1</td></tr>
<tr><td colspan="2">Flow augmented by Wupper reservoir</td><td>8.666</td><td>3.34 (x)</td><td>32</td><td>250/32=7.8</td></tr>
</table>

(x) Higher levels (4 or 5 m^3/s) can be attained with historical record but with probabilities less than 100 %.

If these rules are used in a simulation with the same inflow record as used in the optimization, no failures to meet all purposes will occur. Changing the flood control reservation as a parameter the transformation function in Figure 4 can be obtained (Harboe et al., 1981).

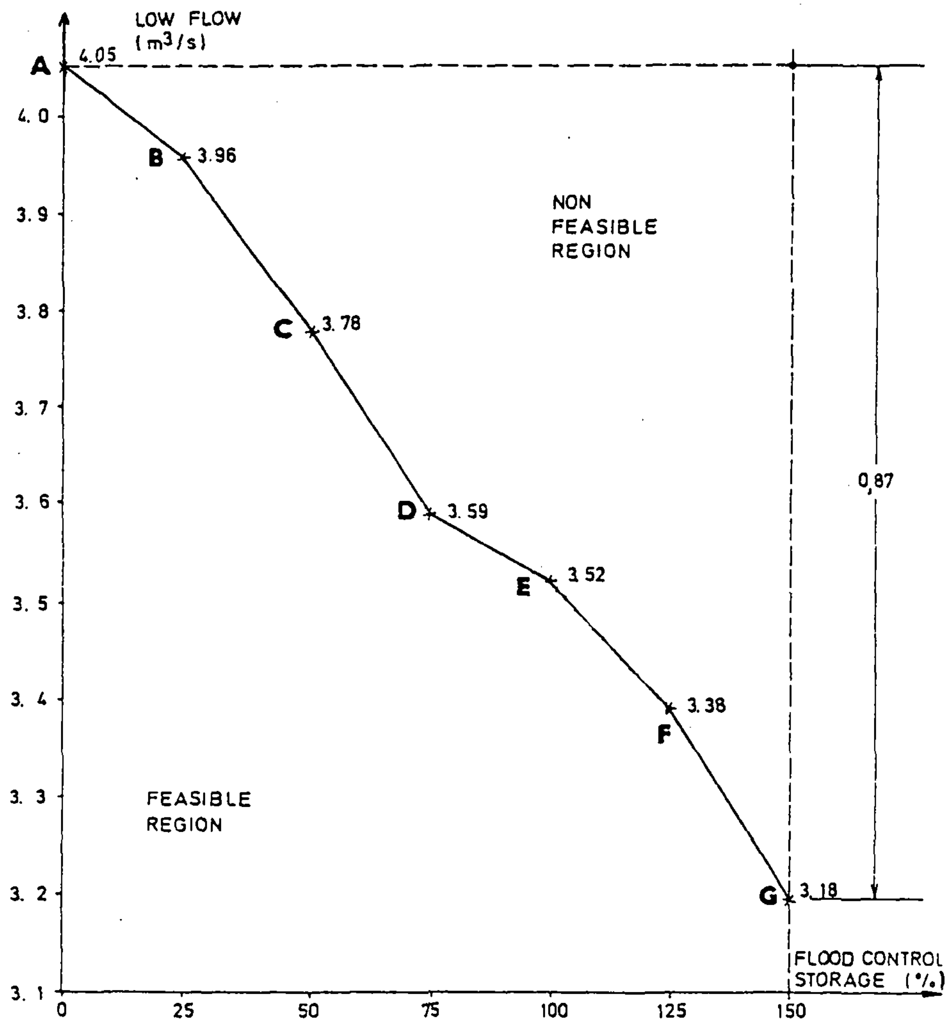


FIGURE 4: LOW-FLOW VS. FLOOD CONTROL STORAGE AS PERCENTAGE OF PRESENT VALUE

STOCHASTIC MODEL FOR HYDROPOWER

For this presentation, the Lech-River System in the Federal Republic of Germany was selected (Fig.5). A reservoir with fifteen hydroelectric power plants is operated for multiple purpose, namely:
flood control, recreation, low-flow augmentation and, of course, hydropower production. A stochastic model was developed in which only the probabilities of the inflows to the reservoir were used (Harboe, 1977). For other lateral inflows only mean values were considered. As a result operating rules for the reservoir were obtained (Fig. 6), which maximized the expected value of the energy production. The actual energy production can be obtained through simulation with the optimal operating rule with a given hydrological sequence. An example is shown in Figure 7, where a simulation between 1955 and 1975 was performed (Harboe, 1976). As a comparison, the actual energy produced during those years, was about 5% lower

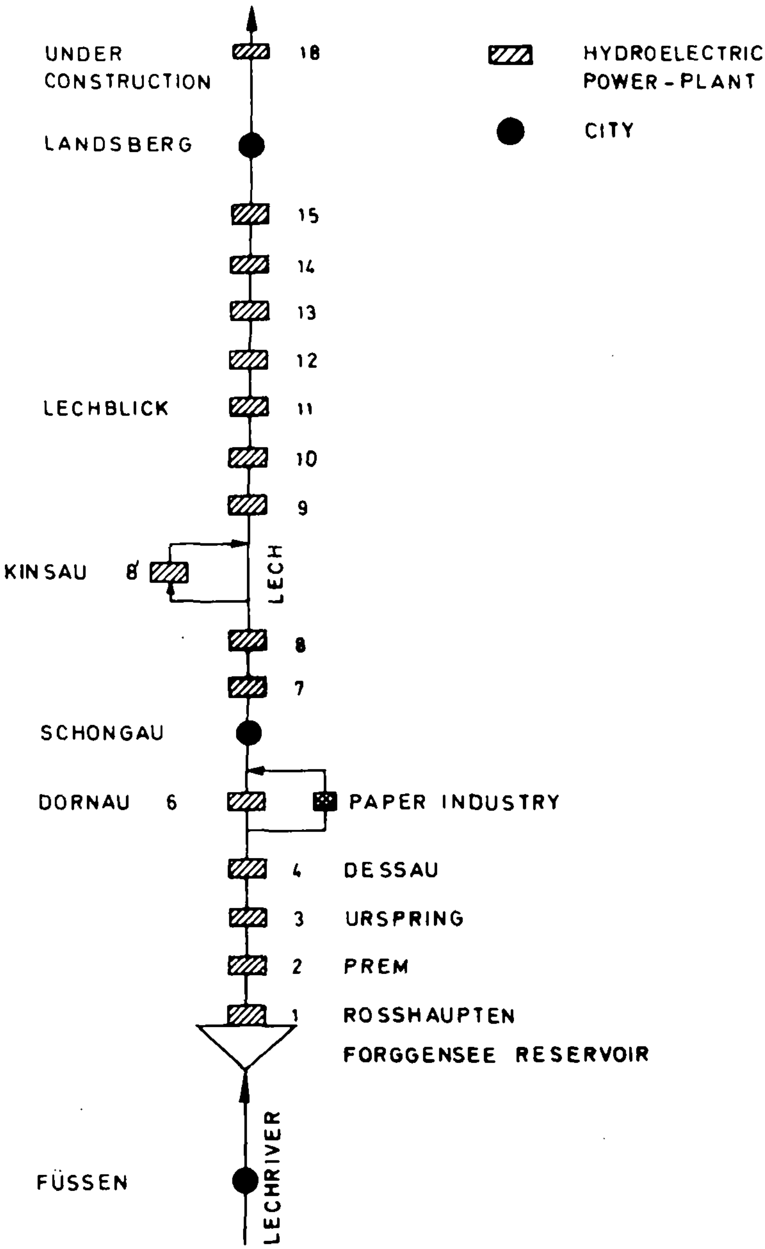


FIGURE 5: THE LECH-RIVER SYSTEM

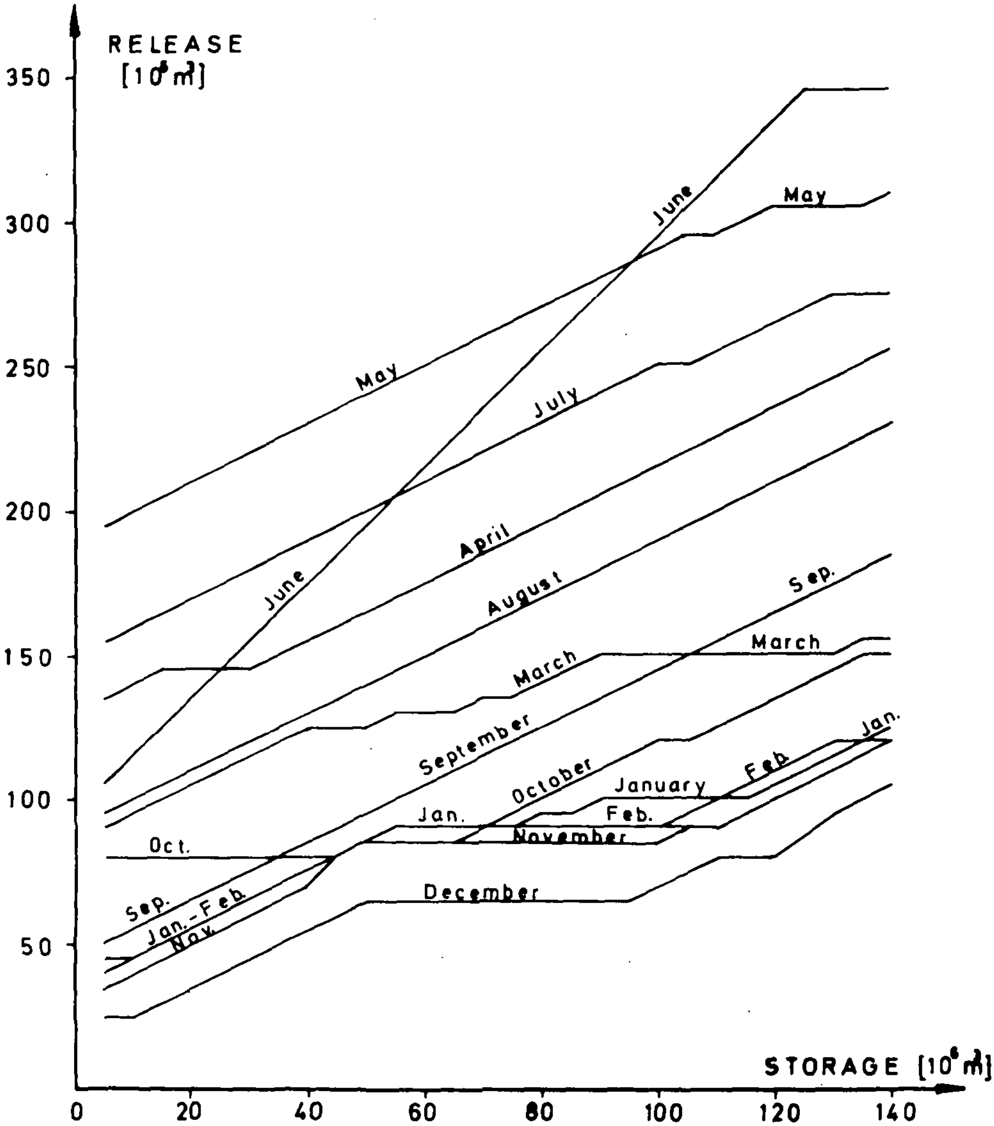


FIGURE 6: STATIONARY OPERATION POLICIES (EXAMPLE FOR MEDIUM INFLOW IN PREVIOUS MONTH)

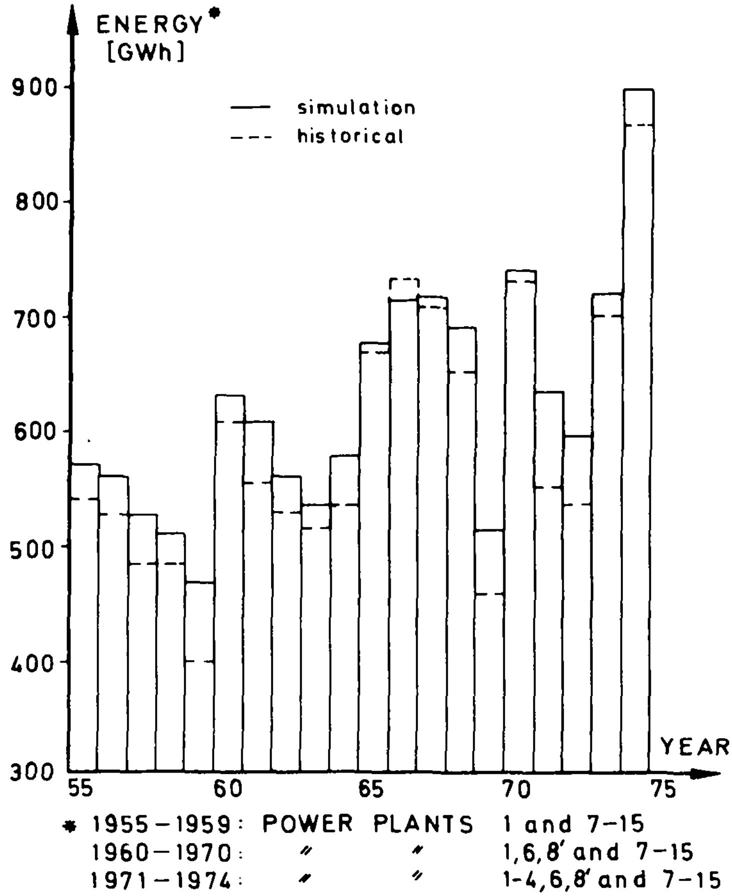


FIGURE 7: COMPARISON BETWEEN SIMULATED AND HISTORICAL YEARLY ENERGY PRODUCTIONS

CONCLUSIONS

Three examples have been presented in which the idea of using a physical objective function was applied. The mathematical models allow in each case a formulation of reservoir operating rules which can be implemented in simulations without forecasting of inflows, i.e. real time operation on a monthly basis. Similar models could be developed for shorter time periods. The deterministic models (hydropower and low-flow) has both to satisfy firm output (guaranteed deliveries) which led to a max-min objective function. The same type of model would apply for irrigation and water supply as long as firm outputs are required. The stochastic model (for hydropower) yields operating rules to be used in simulation models or in actual future operation of the reservoir. Since the expected value is optimized, the results can only be shown through simulation.

All results are useful for analysis with multiple-objective decision making criteria, which is always necessary in multi-purpose reservoir operation problems.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 96 Aspect number 2

HYDROLOGICAL MODELLING AS A TOOL FOR WATER RESOURCES MANAGEMENT IN RURAL AREAS

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ABSTRACT

In recent years mathematical hydrological models have been developed and extensively applied for a number of purposes. In projects concerning for instance irrigation, water supply, flood forecasting, flood control and reservoir operation mathematical models have become indispensable tools. The future will show increasing requirements for application of hydrological models in water resources management in rural areas. If these requirements are to be fulfilled optimally, it is necessary to engage local engineers in the modelling work. Therefore, training and transfer of know-how is an important aspect of project implementation.

In the present paper the types of hydrological models required for reaching the various goals within water resources management in rural areas are described and illustrated in case studies, and general experience gained within modelling transfer and training is outlined.

INTRODUCTION

The term 'hydrological models' as used in the present context comprises deterministic types of rainfall-runoff models and may be divided into the three following categories mentioned in increasing order of complexity

- empirical (black box or 1. generation type)
- lumped conceptual (grey box or 2. generation type)
- distributed, physically based (white box or 3. genereation type)

At the present state of development the empirical models (like the unit hydrograph method and rational formula) and the lumped conceptual models (of which the Stanford model is the best-known), may be categorized under the heading 'traditional methods'. The distributed, physically based models, although having passed their teething troubles, still need to prove their superiority in practical applications.

APPLICATION OF HYDROLOGICAL MODELS

The considerable success for lumped, conceptual models in a wide range of applications indicates that many of these models are physically sound. In addition, these models distinguish themselves by their simplicity.

Simple models are easy to understand, modest in computer requirements, and cheap and easy to run. After being trained local hydrologists can easily run the models at their own computer facilities.

However, when the hydrological cycle is subject to human interference or when a realistic physical representation of flow is required, e.g. studies of water quality or soil erosion, the conceptual representation is no longer adequate. No doubt, this demand combined with the increasing knowledge of the physics of the hydrological process, the increasing information potential provided by new measurement techniques (e.g. remote sensing), and the explosive progress within computer technology will result in a significant increase in the number of applications of the more physically based, distributed models within the very coming years.

The Danish Hydraulic Institute has at its disposal a range of different models, covering the entire range from the very simple approach to what is believed to be one of the most advanced model types available at the market today.

The following contains a description of some of these models including examples of applications. Included is also the experience gained at DHI within modelling training and transfer to developing countries, which is considered to be essential to meet the increasing demand for advanced methods in water resources development.

MODEL TYPES

The NAM Model

NAM is an abbreviation of the Danish: "Nedbør-Afstrømnings-Model", denoting precipitation-runoff-model. NAM has been developed at the Institute of Hydrodynamics and Hydraulic Engineering (ISVA), the Technical University of Denmark by Nielsen and Hansen (1973).

NAM simulates the rainfall-runoff process in rural catchments. It operates by accounting continuously for the moisture content in four different and mutually interrelated storages representing physical elements in the catchment. The model operates for most purposes on the basis of daily values of precipitation and temperature together with mean monthly values of potential evapotranspiration.

Until now the model has been applied in about 50 catchments in several countries with climatic regimes ranging from the arctic Greenland to the tropical Borneo.

The specific purposes of the NAM applications range from the standard extension of streamflow series to runoff predictions from ungauged catchments. The latter case is at the limit of the justifiable applicability of a model type like NAM.

The Nam-S11 Model

The NAM-S11 mathematical modelling system for real time flood forecasting consists of four main elements, i.e. the NAM rainfall-runoff model, DHI's generalized river model (System 11), a computerized updating procedure, and a package of data management programmes.

For the purpose of flood forecasting a hydrological model of the NAM-type and complexity is believed to be appropriate, see Kitandinis (1978) and Jørgensen (1981). A more complex conceptual model will probably not yield better streamflow simulations because the main uncertainty in the rainfall-runoff modelling of flood situations usually pertains to the management of the rainfall input.

System 11 is a general mathematical modelling system for the simulation of flows and water levels in rivers, reservoirs, estuaries and canal systems, and is based upon a numerical solution of the general one-dimensional 'Saint-Venant' equations (conservation of mass and momentum).

Several types of hydraulic structures can be described with System 11, for example, reservoirs, dams, weirs and other types of flow regulators. For the flow description over weirs or embankments automatic switching is performed between subcritical and supercritical flow conditions. For further details see Danish Hydraulic Institute (1982).

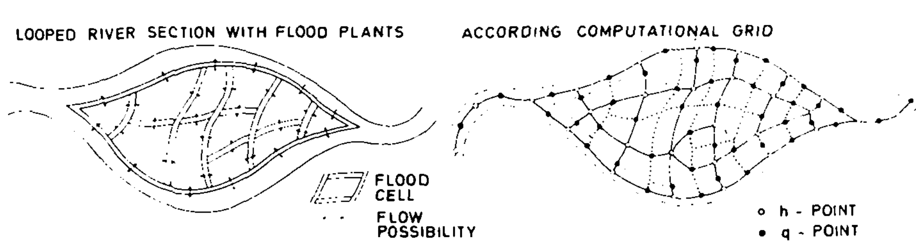

Figure 1 Pseudo 2-dimensional simulation of flood plains, discretized by flood cells between the two river branches.

By virtue of its general formulation, System 11 allows for description of (pseudo) two-dimensional flows over wide flood plains as illustrated in Fig. 1.

The WATBAL Model

The WATBAL (Water Balance) model is a rather simple semi-distributed model being able to utilize spatial distributed information from maps and satelites, as e.g. topography, soil characteristics and vegetation data. As such a model uses more information on catchment characteristics than the NAM-type of model it should be better suited for prediction of runoff from ungauged catchments and the effect of land use changes on the hydrological cycle.

The structure of the WATBAL model is illustrated in Fig. 2. The model operates in a square grid. Different hydrological response units are defined depending on vegetation- and soil type, topographical characteristics, precipitation and basin type.

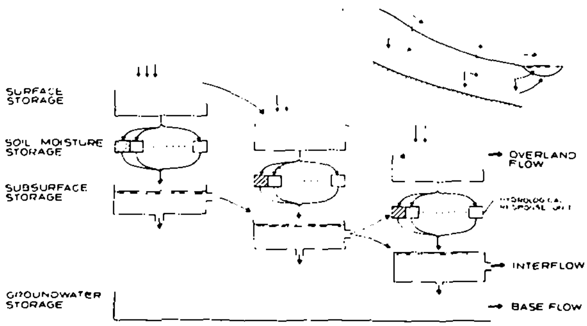

Figure 2 Structure of the WATBAL Model.

For each type of hydrological response unit the model operates with five inter-related storages. i.e. interception-, surface detention-, soil moisture-, subsurface-, and groundwater storages. The exchange between the individual storages is determined on the basis of well-proven physical relationships describing interception, infiltration, drainage and percolation. Exchange between neighbouring surface- and subsurface storages of different topographical zones are accounted for.

The routing from storages to the stream/river is performed using the unit hydrograph method. Channel and reservoir routing may be performed using simple methods, e.g. Muskingum, or by direct coupling to the above described System 11 river model.

The SHE Model

SHE (Systême Hydrologique Europeen) is a fully distributed, physically based model of the entire land phase of the hydrological cycle or any part of that cycle. The SHE-model is developed by DHI, SOGREAH (France) and Institute of Hydrology (U.K.). The model development has been financed. partly by EEC and partly by national research counsils. The structure of the model appears in Fig. 3.

The deterministic, distributed description of the hydrological processes in SHE is based on the equations of flow for water moving over surfaces or through porous media. The equations are solved by finite difference methods.

A two-dimensional overland channel flow model for surface flow is linked to a two-dimensional groundwater model through one-dimensional models of interception, evapotranspiration and vertical flow in the unsaturated zone.The individual submodels can be used independently, or degenerate to simpler versions, depending on the data availability and requirements of a given application.

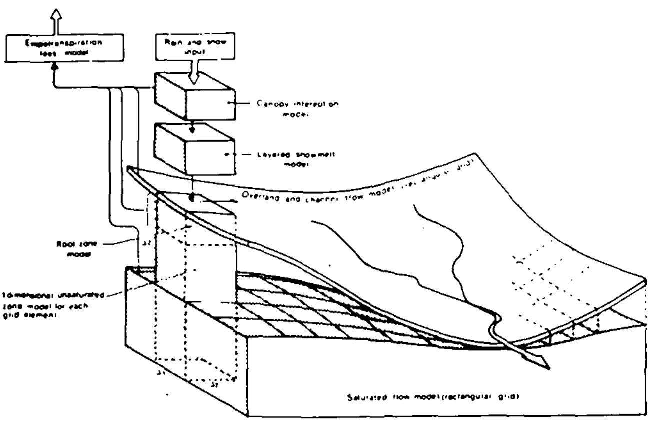

Figure 3 Structure of SHE-model.

An overall controller (the Frame) coordinates and controls the operation and interaction of the system of submodels. For detailed description of SHE, see Jønch-Clausen (1979).

EXAMPLES OF APPLICATIONS

The NAM Model

As a subconsultant to the Danish consulting group Carl Bro-Cowiconsult-Kampsax-Krüger (CCKK), DHI participated in water master planning of three regions in south-western Tanzania in 1980-82. The study which was financed by the Danish International Development Agency (DANIDA) paid particular attention to the supply of water to villages for human and livestock use.

In this connection NAM-modelling of three selected, representative catchments were carried out by the Tanzanian counterparts under DHI supervision. The results of these simulations were very encouraging, see Fig. 4, and proved to be useful for extension of short duration flow records and prediction of streamflow in ungauged catchments as the model parameters varied only little between the catchments. Simulated soil moisture levels corresponded closely with agricultural experience, hence supporting the elaboration of overall water balance maps.

These results, combined with the simplicity and transparency of the model, which the Tanzanian hydrologists competently applied in a very short time, indicate that the NAM-model is a piece of appropriate technology for water resources studies.

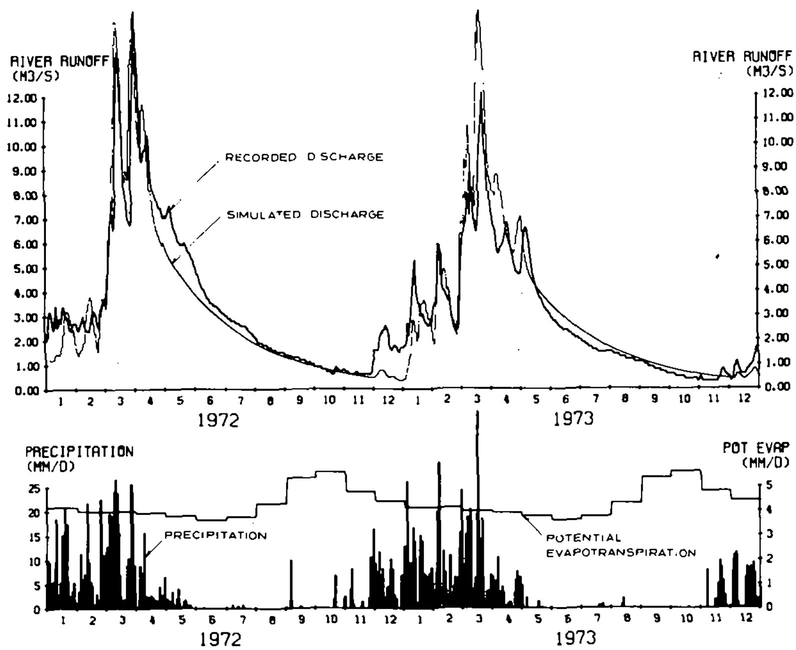

Figure 4 Example of NAM-model calibration for Lt. Ruaha, Iringa Province, Tanzania.

The NAM-S11 Model

In cooperation between the Central Water Commission (CWC) of the Government of India and DHI, jointly financed by the Government of India and DANIDA the NAM-S11 flood forecasting system has been developed and tested on the Damodar River Basin in the states of Bihar and West Bengal.

The model set-up for the catchment upstream of Durgapur Barrage is presented in Fig. 5.The outflows from the 13 NAM subcatchments are taken as inflow to System 11. The model has been calibrated on 1978-80 data and verified on data from the period 1971-77. The results may be characterized as very good; an example is shown on Fig. 6.

Indian engineers have participated in a comprehensive training programme in Denmark and India, and the modelling system is currently being operated by Indian engineers in Delhi, where it is being setup for flood forecasting in the Yamuna River. In addition, CWC and DHI is presently installing NAM-S11 on an indigenous computer in Maithon for real-time testing during the coming 1985 monsoon.

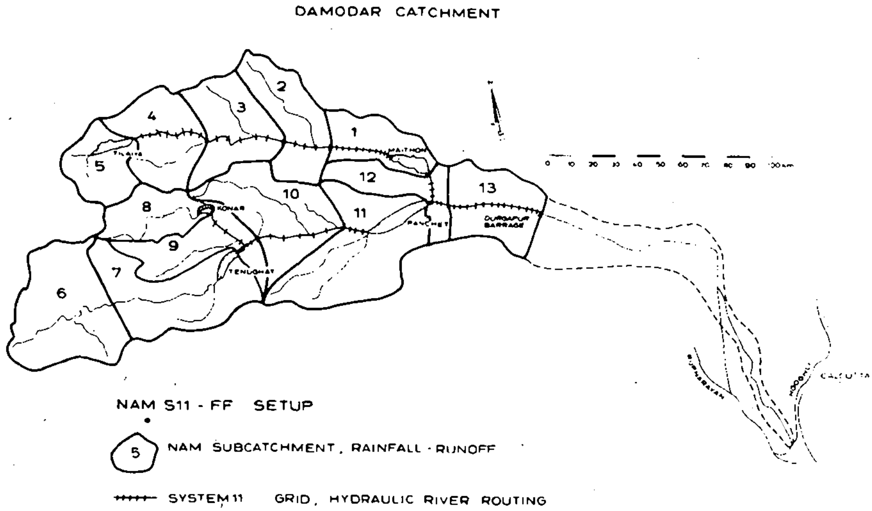

Figure 5 The 13 NAM subcatchments and the System 11 grid for the 20,000 km^2, Damodar catchment.

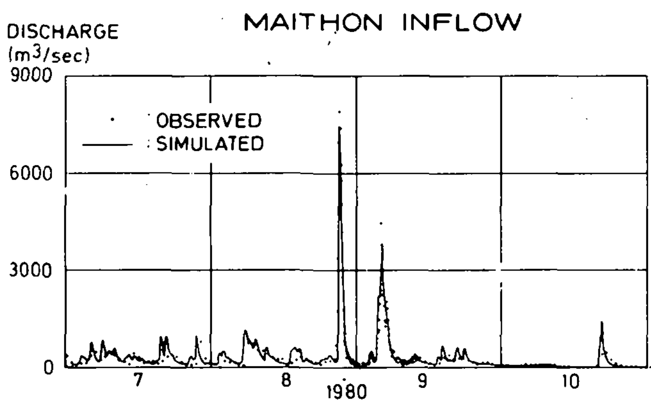

Figure 6 Simulated and observed inflow to Maithon Reservoir during the 1980 monsoon season.

The WATBAL Model

At the time of writing the WATBAL model has not yet been applied in practice. The first practical application is planned for Autumn 1984 in relation to a feasibility study for medium size dams in Zimbabwe (financed by DANIDA), where DHI is sub-consultant to the Danish consulting company

Dangroup. Hopefully, the results hereof can be presented at the congress.

The SHE Model

On catchment scale SHE has been applied to the River Wye catchment (10.5 km^2) in Wales. This study concentrated on the simulation of selected storm flow events, with particular attention to the spatial variation of runoff within the catchment. Another field application is carried out in coorporation with the University of Braunschweig in Germany. Finally, SHE is presently being set up on a major catchment in New Zealand.

The Wye Catchment is a small upland catchment in Mid-Wales (Fig. 7), heavily monitored by Institute of Hydrology in Walllingford. The catchment is characterized by rather steep slopes and altitudes between 340 m and 740 m. On the high plateau, the predominant vegetations are grassland and heaths, whereas mires are dominating in the valley floors.

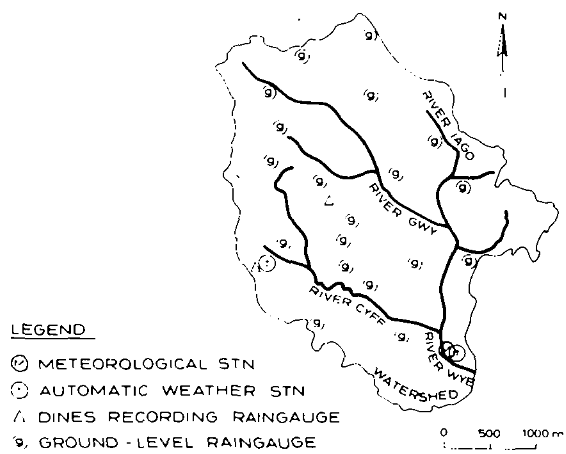

Figure 7 The Wye catchment used for a fixed testing of SHE.

The field tests have focused on the simulation of two individual flood events selected from the stream flow data. The peak of the first flood event, used for calibration, was 1 m^3/s. The second flood event, ten days later, with a maximum peak of 7.5 m^3/s, was used for verification, see Fig. 8.

It is significant that a peak flow of 7.5 m^3/s in this case is predicted rather accurately using a model which has been calibrated on an event with a much smaller peak flow. This is possible because of the distributed physically based nature of SHE, ensuring extrapolation beyond the calibrated range to be much more reliably than similar attempts with traditional conceptual models. Also the simulated and measured hydrographs for the two upland tributaries compare well, showing the ability of SHE to provide an accurate representation of the flow distribution within the cathcment.

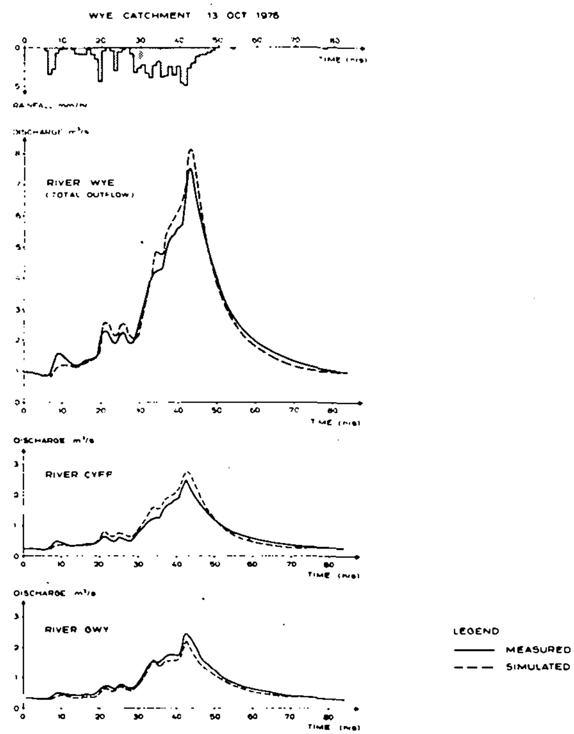

Figure 8 SHE-Verification, Wye catchment

TRAINING AND TRANSFER OF KNOW-HOW

During the last five years, DHI has been engaged in the transfer of hydrological and hydrodynamic models to a number of countries. In most cases, as demonstrated above, the transfer has included training of professionals at receiving institutions in the application and maintenance of the involved computer software.This educational aspect of the transfer process is a prerequisite for a succesful completion of the model transfer.

In a typical model transfer, the institution receiving the model will be guided through an application exercise addressing a relevant problem.This way, model application, the training aspect of model transfer, and the solution of an important problem can often be combined.

The transfer of know-how, as outlined above, thus typically includes the following phases:

* DHI involvement in project formulation and data collection
* training at DHI of selected professionals from receiving institution, including model application to a problem in the recipient country (focus project)

* transfer and modification of computer software to clients' own installation
* continuing training and application
* post-project consulting and gradual phasing out of DHI assistance.

CONCLUSIONS

A number of hydrological models, covering the range from the well-established lumped, conceptual type to the physically based, fully distributed type, has been described.

The traditional types of models have through a number of applications world wide, covering the widest possible climatic range, demonstrated their usefulness for a number of purposes, e.g. extension of streamflow time series, water balance studies and real-time runoff forecasting.

Investigations of the effects of human interference on the runoff process, determination of water quality and estimation of soil erosion, however, require physically based models. Examples of such type of models are given as well as results of first applications.

DHI has with great success transferred several models to a number of countries. The philosophy behind this transfer of know-how is on the-job-training of engineering staff in the recipient country. Through a focus project the training aspect of model transfer, model application and the solution of an important problem are combined. Following the installation on the client's own computer continuing training and application takes place during the gradual phasing out of DHI-assistance.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 98 Aspect number 6

WASTEWATER SYSTEM ALTERNATIVES FOR VILLAGES IN LOWER EGYPT

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ABSTRACT

With the advent of increased irrigation and water usage which followed the completion of the high Aswan dam in 1965, groundwater levels have risen in Lower Egypt, particularly in the irrigated areas of the River Nile delta from Cairo to the Meditteranean Sea, and most significantly in the village areas. With respect to villages, the regional rise of groundwaters has been compounded by the increased underground disposal of wastewaters which has followed improved domestic water distribution systems.

In many villages, the disposal of wastewaters to the ground surface or subsurface has been practiced for hundreds or perhaps even thousands of years. But, now villages may be seen as sitting on a groundwater mound elevated above regional levels, and they are experiencing considerable difficulties with wastewater disposal due to subsurface saturation, high groundwater, emerging surface pools of septic waters, gross groundwater pollution, deterioration of buildings and structures due to moisture absorption, and other related problems. A further complication in certain areas is the existence of a large irrigation canal which usually carries a level of flow above the general elevation of the village and therefore creates a hydraulic gradient of seepage through the dikes toward the village.

This paper describes the above problems and addresses the following:

Typical Village Drainage Engineering Problems

Current Village Wastewater Treatment and Disposal Practice

Wastewater System Alternatives, Rationale and Recommendations

Illustrative drawings are included in the paper.

Keywords: Wastewater disposal, villages, Lower Egypt, high groundwater, drainage problems, wastewater collection, subdrains, combined system, alternatives.

INTRODUCTION

The River Nile delta area and fringes as shown in Figure 1 comprise approximately 22,000 square kilometers. The alluvial valley begins a triangular spread approximately 20 kilometers northwest of Cairo at an elevation of about 17 meters above sea level, with a relatively flat slope of one meter per 8.5 kilometers in a northerly direction. Rainfall in the area is sparse and occurs during the winter season, with annual averages varying from 26 millimeters at Cairo to around 180 millimeters along the Mediterranean sea shore.

Because of the prevailing practice of subsurface disposal of wastewaters in the villages of Egypt, the geology and stratigraphy of the River Nile delta groundwater basin are of importance for purposes of this paper and are briefly described below based on information published by the Egyptian Ministry of Irrigation (1981).

Upper boundary deposits of the delta basin are comprised of a semipervious clay and silt which act as an aquitard or cap over the underlying aquifer. The thickness of the clay cap increases uniformly in a northerly direction from about 3 meters at Cairo to nearly 60 meters at the Mediterranean coast. The basin aquifer is semi-confined and consists of deposits 200 to 300 meters in thickness, mainly of unconsolidated coarse sands and gravels with occasional clay lenses. Sub-soil water contained in the clay cap aquitard creates a shallow water table, whereas the underlying aquifer is under a piezometric head. Both levels fluctuate to some extent during the seasons of the year. Since the river Nile intersects the aquifer formation in most of its reaches, the aquifer groundwater piezometric heads are affected by the water level in the river.

Observations on the aquifer groundwater piezometric heads have been made periodically since 1958 in more than 300 wells scattered throughout the delta area. Figure 2 provides a typical indication of the relative levels for years 1958, 1968 and 1978, with 1958 as a characteristic datum year of the River Nile flood periods prior to the implementation of the Aswan High Dam in 1965. Seasonal fluctuations in aquifer water levels have been considerably reduced in recent years due to the absence of the annual Nile floods, however, a rising groundwater trend is now developing during the summer months due to the increased application of irrigation waters. In general, the aquifer heads are somewhat higher than during the pre-Aswan High Dam flood period, according to data reported by Amer (1981).

SUBSURFACE DRAINAGE PROBLEMS IN EGYPTIAN VILLAGES

Of main concern for the subsurface disposal of wastewaters in the villages is the drainage into and through the clay cap. Drainage waters into these surface deposits are generally collected by ditch drains along the village periphery or manage to percolate through the clay cap at relatively slow rates. The areas surrounding the villages are largely used for irrigated agriculture where subsurface drainage is being widely promoted through the use of tile drain systems discharging to drainage ditches which ultimately carry their flows to the Mediterranean Sea. These subdrain systems provide a relief to the high groundwater problems in the agricultural areas and may have some limited effects extending to village areas in some cases. However, in the villages, the problems of high groundwaters and subsurface drainage are seriously compounded by the increasing disposal of wastewaters into the ground through various seepage methods.

With the increasing development of village water supply systems made possible by the Aswan High Dam and through increased economic aid, water

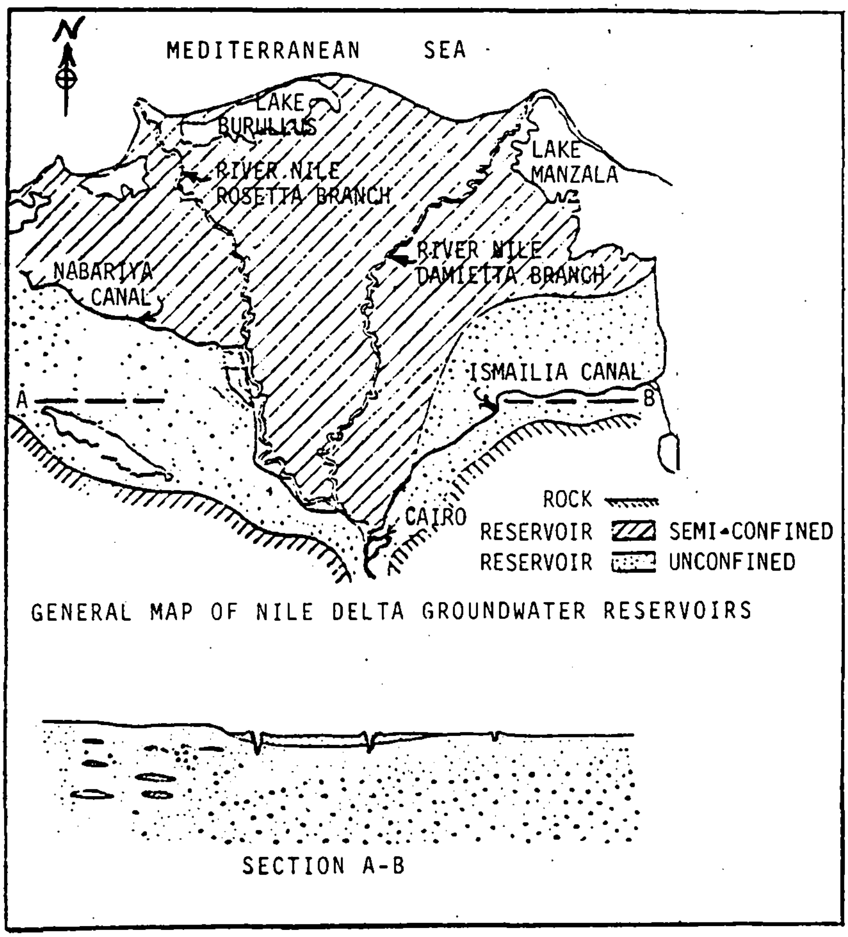

Fig. 1 - RIVER NILE DELTA AREA SHOWING THE GENERAL LIMITS OF THE GROUNDWATER RESERVOIRS

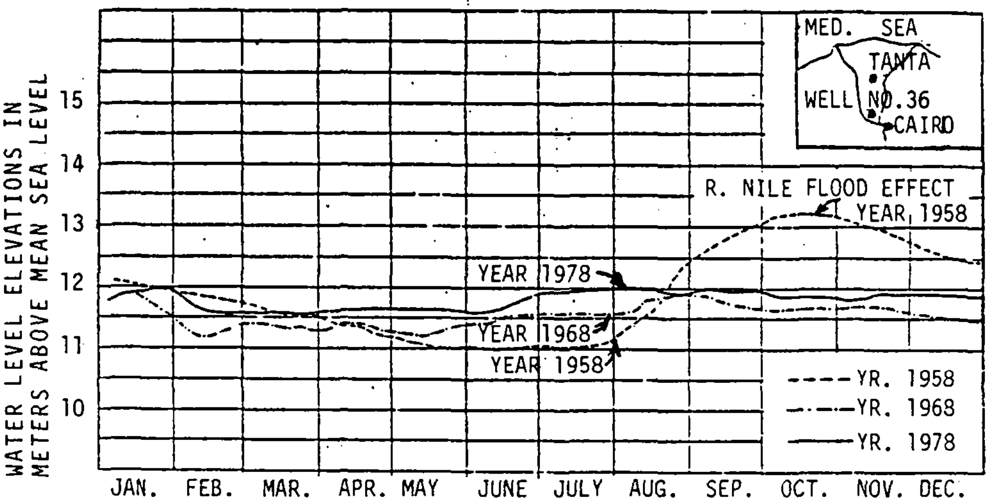

Fig. 2 - RELATIVE WATER ELEVATIONS IN OBSERVATION WELL NO. 36
FOR YEARS 1958, 1968 AND 1978

usage and the associated subsurface disposal discharge have significantly increased in recent years. Consequently, the drainage capacity of the clay cap underlying many villages has been exceeded; the subsurface soils simply are saturated and drainage rates insufficient.

In many villages, the disposal of wastewaters to the ground surface or subsurface has been widely practiced for hundreds and perhaps even thousands of years. But, now villages may be seen as setting on a groundwater mound elevated above regional levels, and they are experiencing considerable difficulties with high groundwater, emerging surface pools of septic waters, gross ground water pollution, deterioration of buildings and structures due to moisture absorption, and other related problems. A further problem in certain areas is the existence of a large canal which usually carries a level of flow above the general elevation of the village and therefore creates a hydraulic gradient of seepage through the dikes toward the village. The typical village drainage problems are generally three-fold:

First, wastewaters must be collected and disposed of utilizing appropriate sanitary engineering practices;

second, and very important from an economic viewpoint, the compounding problem of high localized groundwater levels must be solved in order to prevent damage to buildings and structures; and

finally, certain special canal seepage problems exist which further complicate the above two problems.

CURRENT VILLAGE WASTEWATER TREATMENT AND DISPOSAL PRACTICE

Few villages have sewer collection systems. It is estimated that only about 5 percent of the rural population has access to sanitary waste disposal systems. Although this indicates a general absence of existing sewage systems, a number of villages are currently proceeding with sewer and drainage systems through various forms of governmental aid.

Excreta disposal is mainly practiced on an individual housing unit basis employing cess pits, sanitary vaults with unlined bottoms, septic tanks, holding vaults, and other miscellaneous ground disposal methods. Some of the sewage facilities such as septic tanks and cess pits, are constructed to receive household sullage waters, while other more primitive facilities are unsatisfactory for this purpose and in these latter cases, such waters are thrown out on the roadway or on a convenient area near the dwelling. Tank trucks utilizing hand pumps and vacuum systems currently provide a individual collection service for the removal of wastewaters and sludges from septic tanks, cess pools, and holding tank systems. Generally, the pumpage is indiscriminately dumped in nearby drainage ditches, irrigation canals, roadside areas, and other miscellaneous locations, causing widespread water and environmental pollution. It is to be pointed out that although collective treatment of excreta disposal and wastewaters is not currently extensively practiced in Egyptian villages, it is receiving attention by the Government of Egypt as an urgent environmental problem through recent legislation and other action.

WASTEWATER DRAINAGE SYSTEM ALTERNATIVES FOR VILLAGES

In approaching the design of any sewerage or drainage system for a village or municipality, it is imperative that some concept of an overall master plan be developed as the first step. The master plan must encompass the existing areas to be served as well as future development, so that stages of the system can be engineered and constructed as required. Because of financial constraints, a complete sewerage system is rarely constructed initially for a municipality; normally the system is constructed in stages according to service requirements and as funds become available. Any construction stages of the system must be consistent with the engineering master plan so as to optimize costs and to assure coordinated operation of the system. Three generalized alternatives to the typical village drainage problems are outlined below, briefly discussed, and recommended as applicable.

Alternative No. 1 - Combined Drainage Sewer System with Treatment

Briefly, this approach requires the development of a combined drainage sewer system, including sanitary sewage and collected groundwaters, along with wastewater treatment generally located on the downstream side of a village and effluent disposal to a main drainage ditch, in full compliance with recent Government of Egypt legislation on wastewater disposal.

Figure 3 illustrates the schematic layout of a village combined sewer system with typical treatment and final effluent to irrigation or a drainage ditch. Combined sewage type manholes would be used for receiving groundwater drainage in certain critical areas, as well as for access and transport of sanitary wastewaters through the sewer piping system. A number of low cost treatment methods are applicable as reported by Preul (1983). Where surface water treatment methods are employed, as is generally expected in the delta areas, a lift station will be required at the terminus of the trunk sewer in order to provide sufficient hydraulic head for flow through the plant and discharge into the drainage ditch. Factors in support of this Alternative No. 1 are:

The subsurface soils will be relieved of their sanitary sewage as currently practiced, and this will further relieve the groundwater levels. Experience in certain sewered villages in Lower Egypt, as in the Damietta Governorate, has demonstrated this effect. Engineering calculations of a water balance nature may also be made as a prediction of this effect in lowering groundwater levels in specific cases.

The combined sewage type manholes will serve a dual purpose for sewage and for receiving the infiltration of high groundwaters through perforated tile tubing. In effect, the manhole would serve the added function of a groundwater collection well, strategically located for this secondary purpose.

This approach is consistent with sound sanitary engineering practice and can be engineered using low cost methods.

Factors unfavorable to this alternative are that it may be viewed as costly and will require certain technical services, mainly sanitary engineering, in order to properly plan and design a project.

This alternative is recommended as the best and ultimate approach for combined sewage.

Alternative No. 2 - Continued Subsurface Sewage Disposal Practice With Improved Septic Tanks And Sludge Disposal Handling

This alternative is viewed as an interim approach pending sufficient funding to proceed with Alternative No. 1. Through improved septic tank practice, the subsurface soils and groundwater would be relieved of some of the sanitary sewage currently discharged directly or indirectly to the subsurface through the various seepage disposal methods. It is intended that the improved septic tank practice would include sealing certain tanks, such as false bottomed vaults, along with improved tank truck

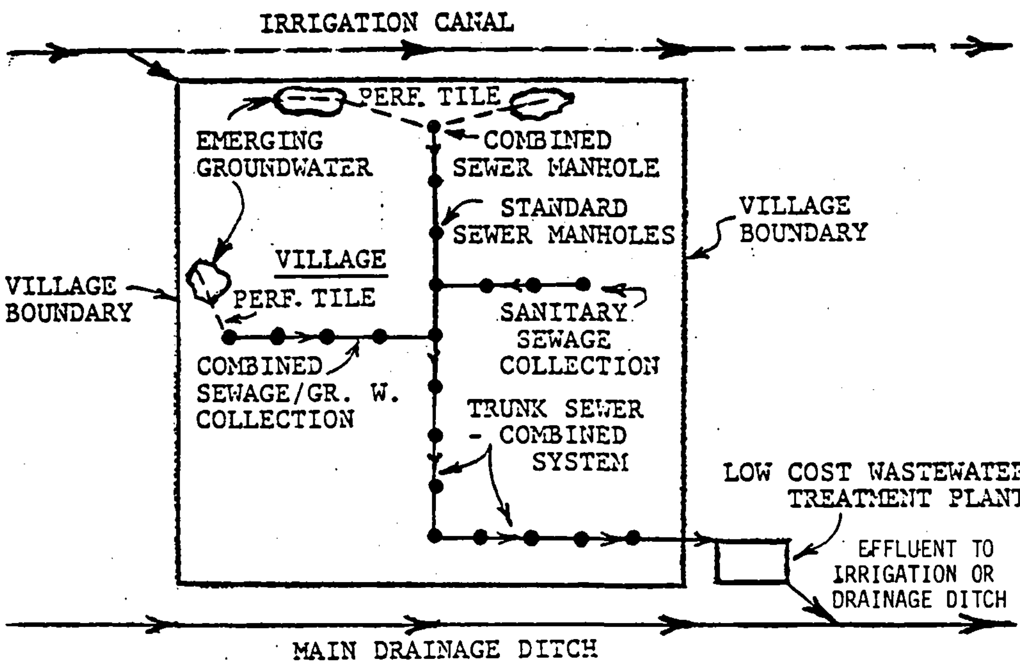

Fig. 3 - SCHEMATIC DIAGRAM OF PROPOSED TYPICAL VILLAGE COMBINED DRAINAGE COLLECTION AND TREATMENT SYSTEM

pumping and regulated disposal of the tank truck discharges to drainage ditches or regulated land disposal areas; proper treatment of the tank truck discharges would be required in order to comply with Government of Egypt public laws. Generally, such treatment would require the equivalent of secondary treatment, i.e. biological treatment and possibly disinfection. Factors in support of Alternative No. 2 are:

The approach follows the current practice and is an improvement pending a better solution.

Current practices can not be immediately changed and will require a transitional period under any scheme and, therefore, at least some public health improvements are advisable.

The costs are relatively low and generally no large financial funding is required until Government of Egypt regulations are rigidly enforced.

The groundwater levels can be expected to be relieved to the extent that direct soil seepage systems are curtailed and to the extent that water usage may be curtailed as a further measure of control. The overall effect on lowering groundwater levels could be significant, but would be directly related to the success of the program.

Factors unfavorable to this Alternative No. 2 are:

It tends to perpetuate an undesirable sanitary engineering and public health approach.

It may foster a complacency with the current sanitary practice, which is not consistent with good public health standards.

It perpetuates a gross contamination of shallow surface waters, with the possibility of deeper water supply aquifer contamination in certain cases, depending on the geology.

The indiscriminate disposal of tank truck pumpage will be difficult to completely control.

The above factors are not intended to be complete, but point out some of the major concerns. This alternative is recommended only as an interim approach where required by financial constraints, until Alternative No. 1 can be initiated.

<u>Alternative No. 3 - Continued Subsurface Sewage Disposal With Addition Of Groundwater Collection System For Lowering Groundwater Table, Treatment, and Effluent Discharge To A Drainage Ditch</u>

This method has been suggested by proponents as a lower cost alternative and therefore should be discussed. The concept of this alternative is to continue the current undesirable subsurface disposal practice utilizing seepage through the subsurface soils as a polluted water transport system to a perforated sub-drain tile collection network and pipeline transport to a treatment plant with disposal to a drainage ditch. The main objective of this approach would be to lower groundwater levels particularly in areas where structural damages, due to moisture absorption, are most significant.

Factors in support of Alternative No. 3 are:

It appears to be a low cost solution.

It would lower groundwater levels in certain areas and relieve structural damage.

The cost of treating the collected ground wastewaters would be expected to be less than for the combined drainage waters of Alternative No. 1.

Factors unfavorable to this Alternative No. 3 are:

It perpetuates a gross contamination of subsurface soils and shallow groundwaters, with the further possibility of contamination of the deeper water supply aquifer. Of particular concern would be certain toxic wastes.

The total capacity of the drainage system is limited by the absorptive capacity and transmissibility of the sub-soil which must act as a transport system for the collection of the subsurface wastewaters. This ultimately would become a severe limitation with regard to future development.

The approach pre-empts the construction of a conventional combined sewer system in the public roads, and unless properly engineered for adaptation, may have to be removed at considerable cost at a later time.

The life of such a subsurface collection system for combined drainage may be limited and doubtful.

Such a system would not provide for the collection of commercial and industrial wastewaters, as these waters would usually require extensive treatment prior to subsurface discharge.

Alternative No. 3 is not recommended and further has not been approved by the concerned ministries in the Government of Egypt.

SUMMARY AND CONCLUSIONS

Because of prevailing wastewater collection problems compounded by high groundwater conditions in the villages of Lower Egypt, conceptual engineering alternatives have been discussed regarding possible solutions. In summary, it may be concluded that the ultimate solution should be a combined sewer system directly receiving discharged wastewaters from users through a conventional pipe network, but with the provision of special combined drainage type manholes to serve the added purpose of collecting groundwaters, thereby lowering damaging groundwater levels. This solution has been advanced and discussed as Alternative No. 1.

ACKNOWLEDGEMENT

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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ETUDE D'UNE PARTIE DU BASSIN HYDROGRAPHIQUE DE LA VESDRE EN VUE DE L'AMENAGEMENT DU COURS INFERIEUR CONTRE LES INONDATIONS ET DE L'INSTALLATION D'UNE MICRO-CENTRALE HYDRO-ELECTRIQUE A CHAUDFONTAINE

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RESUME

La Vesdre, rivière située à l'est de la Belgique, draine un bassin versant de 699 km^2.

On présente dans une première partie, une brève description du bassin, ainsi qu'une étude hydraulique reprenant les précipitations, les crues et le plan de protection contre les inondations.

La deuxième partie de cet exposé est consacrée à l'installation d'une microcentrale hydro-électrique sur le site du barrage Bacquelaine à Chaudfontaine. On y examine notamment la rentabilité et le choix du nombre de turbines.

I. PREAMBULE

Cette communication ne représente qu'une partie de l'étude effectuée pour l'Hydraulique Agricole, qui avait pour objet de définir les travaux d'amélioration nécessaires à la lutte contre les inondations.

II. BREVE DESCRIPTION DU BASSIN DE LA VESDRE

La Vesdre prend sa source non loin de Montjoie et se jette dans l'Ourthe à Chênée; quelques kilomètres plus en aval, l'Ourthe se jette dans la Meuse à Liège.

La direction générale du cours de la Vesdre est E-W; les premiers kilomètres de son cours sont dirigés S-N, puis elle s'écoule vers l'WSW et conserve cette direction jusqu'à Nessonvaux où elle s'infléchit vers l'WNW jusqu'à son embouchure.

Sa longueur totale est de 72,430 km, elle descend du niveau 600 au niveau 67, c'est-à-dire que sa pente moyenne est de 8,2 mm/m.

Les limites N et S du bassin sont à peu près parallèles à la direction générale de la rivière, la limite N à 8 km et la limite S à 14 km environ jusqu'à hauteur de Nessonvaux et à 4 km seulement depuis Nessonvaux jusqu'à Chênée. Sauf dans la partie aval, le bassin est donc plus étendu sur la rive gauche que sur la rive droite.

En aval du confluent de la Hoëgne, l'étendue du bassin est relativement faible. Les principaux affluents se trouvent sur la rive gauche :
la Helle recevant la Soor et la Hoëgne recevant le Wayai étant les plus importants. Tous les affluents sont indiqués au schéma de la figure 1.

Le bassin de la Vesdre s'étend sur 699,07 km^2 au total, soit 400,25 km^2 sur la rive gauche et 298,82 km^2 sur la rive droite. Son affluent le plus important est la Hoëgne avec un bassin versant de 223,37 km^2 soit 32 % du bassin total.

Toute la rive droite fait partie du pays de Herve, sauf en amont de la frontière belgo-allemande d'avant 1914, où s'étend la forêt de Rötgen que nous rattacherons à la région ardennaise.
Tout le cours supérieur jusque Eupen sur la rive droite et jusque Pépinster sur la rive gauche appartient aux Ardennes tandis que la Hoegne est dépendante de la région des Hautes Fagnes. En aval de Pepinster, confluent de la Hoëgne et de la Vesdre, la rive gauche de cette dernière fait partie du Condroz.
Les répartitions du sous-sol et les surfaces sont indiquées à la figure 2.

III. DESCRIPTION DU COURS

Le cours de la Vesdre peut être subdivisé en trois tronçons : le cours supérieur du Stelingberg à Eupen; le cours moyen d'Eupen à Pepinster; le cours inférieur de Pepinster à Chênée.

Les cours supérieur et moyen n'étant pas inclus dans le contrat, on ne s'intéressera qu'au cours inférieur. La vallée n'est plus occupée par une agglomération industrielle, mais plutôt par des prairies, depuis Pepinster jusqu'à Trooz, limite de l'agglomération liégeoise où se reproduisent les mêmes caractères que dans la région verviétoise.

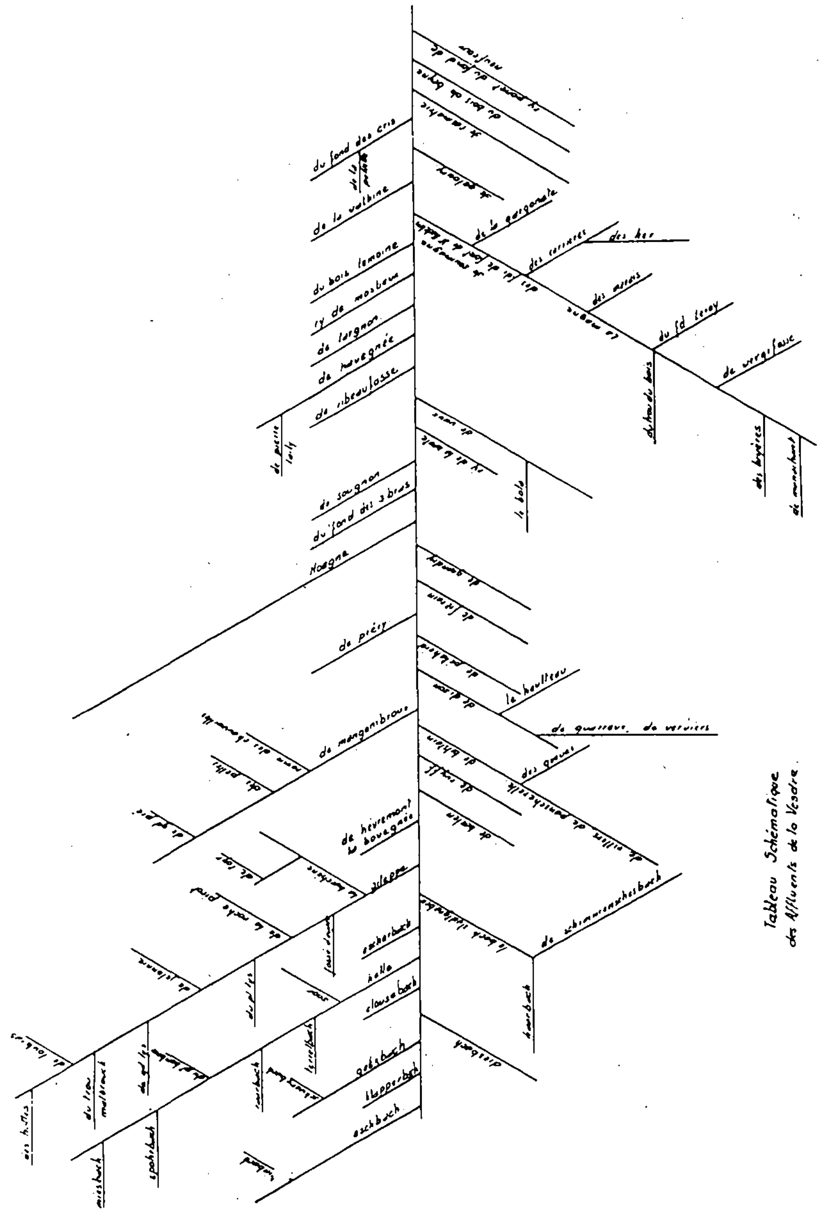


Tableau Schématique
des Affluents de la Vesdre
du fond des cris
de la vatbine
du bois lemoine
ry de mosbeux
de lurgnon
de havegnée
de ribeaufosse
de sougnon
du fond des 3 bras
Hoegne
de préry
de mangombroux
de hevremont
la bovegnée
gileppe
escherbach
holle
clauseback
getzbach
klapperbach
eschbach
de la gargonale
des coriates
des kes
du fd leroy
de vergifosse
des boyeres
la bola
la haulteau
de queureux de verviers
des gueux
de schimmerscheitbach
hoorbach

Figure 1

Région	Surface totale	Prairies, vergers, cultures.	Bois	Inculte	Agglomérations, routes, lacs, etc...
Pays de Herve	174.0 km² 100 %	137,5 km² 79 %	17,4 km² 10 %	1,7 km² 1 %	17,4 km² 10 %
Ardennes à l'ouest de la Gileppe	280,0 km² 100 %	92,4 km² 33 %	134,4 km² 48 %	25,2 km² 9 %	28,0 km² 10 %
Ardennes à l'Est de la Gileppe	189,0 km² 100 %	9,5 km² 5 %	151,1 km² 80 %	18,9 km² 10 %	9,5 km² 5 %
Condroz	56,0 km² 100 %	26,4 km² 47 %	20,6 km² 37 %	0,6 km² 1 %	8,4 km² 15 %
Total	699,0 km² 100 %	265,8 km² 37,9 %	323,5 km² 46,3 %	16,4 km² 6,4 %	63,3 km² 9,1 %

Figure 2.

De Pepinster à Trooz, les barrages sont espacés de 2 km en moyenne, tandis qu'il y en a tous les 750 m de Trooz à Chênée. Ici, actuellement, les eaux n'ont plus aucune utilité industrielle. Elles servaient anciennement comme force motrice et moyens de refroidissement et d'entretien à des usines métallurgiques.

La largeur moyenne du lit atteint environ 40 m dans la région liégeoise. Dans ce tronçon, le seul affluent notable est le ruisseau des Fonds de Forêt qui doit son importance à son orientation ENE-WSW et recoupe ainsi les ruisseaux descendant du plateau de Herve.

La Vesdre se jette dans l'Ourthe à Chênée qui se jette elle-même dans la Meuse à Liège, 3 km plus en aval.

IV. PRECIPITATIONS

Les précipitations moyennes annuelles sont représentées à la carte au 1/100 000 (figure 3) par des isohyètes cotés en mm/an. Dans les limites de la région étudiée, ces isohyètes sont presque rectilignes et orientés SSW-NNE, c'est-à-dire à peu près perpendiculaires à la direction générale du cours. Ils sont approximativement équidistants et cotés de 800 mm à Chênée jusqu'à 1150 mm dans les Hautes-Fagnes (1187 à la Baraque Michel).

On remarque que les précipitations augmentent avec l'altitude, les hauteurs de l'E recevant les vents d'W et de SW chargés d'humidité. La région de Jalhay reçoit environ 25 % d'eau en plus que la région de Liège.

D'après la carte pluviométrique, la hauteur moyenne annuelle sur le bassin de la Vesdre dans son ensemble est de 975 mm.

Les saisons sont caractérisées par leur hauteur de précipitation; la figure 4 donne les hauteurs mensuelles moyennes de précipitation aux différents pluviomètres de l'IRMB.

On remarque que pour former des groupes de mois cohérents au point de vue hauteur de précipitation, il faut rattacher février au printemps.

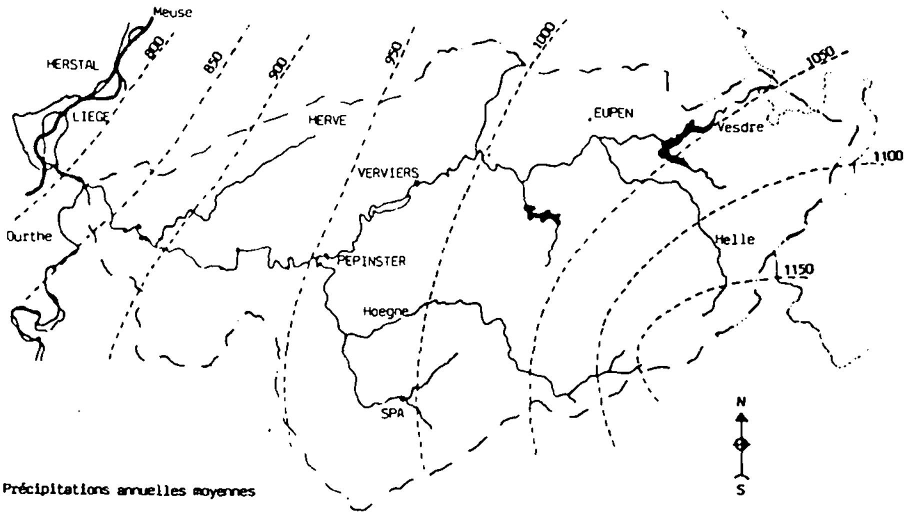


Figure 3

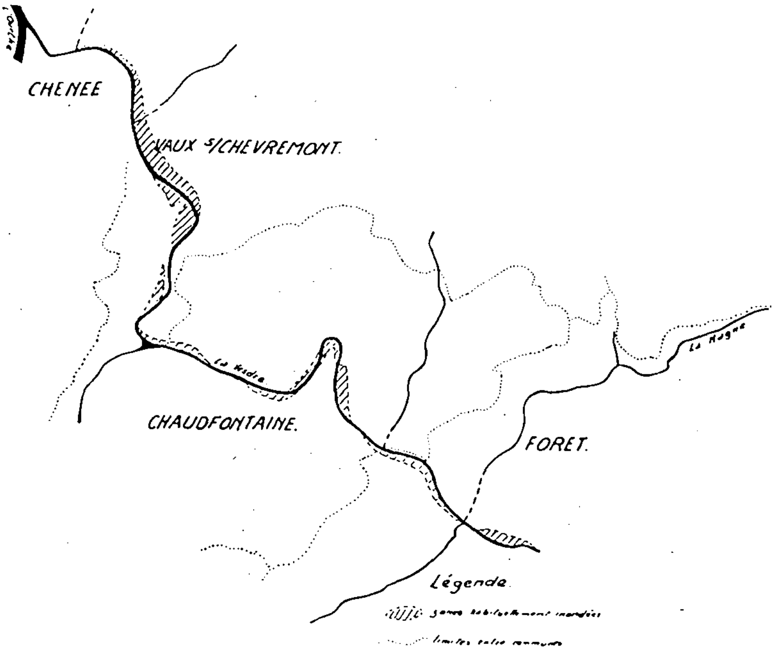
Pluviomètre	D	J	F	M	A	M	J	J	A	S	O	N
1. Barrage Michel	132	125	84	86	91	99	107	124	114	119	113	95
2. Thimister	112	97	68	75	74	75	89	103	96	86	90	85
3. Spa	118	111	81	81	75	75	96	104	98	86	93	95
4. Stavelot	126	115	80	77	75	77	87	98	100	94	90	93
5. Gileppe	125	107	76	82	77	78	86	108	102	96	92	92
6. Membach	116	104	74	83	77	86	99	114	108	104	95	93
Moyenne pr 2/6	119	107	76	80	76	78	91	105	101	93	92	92
Moyenne/saison	113			77,5				99			99,3	
	hiver			printemps				été			automne	

Figure 4

V. CRUES ET PROTECTIONS

La figure 5 indique ainsi les zones habituellement inondées pour la région de Chaudfontaine, selon les observations recueillies par les agents de l'Hydraulique Agricole. De plus, on a effectué une étude statistique des débits à partir des enregistrements disponibles, effectués, depuis 1967 au déversoir de La Rochette dans la traversée de Chaudfontaine pour un bassin versant de 687 km², par le Service d'Etudes Hydrologiques du Ministère des Travaux Publics.
Les résultats des calculs sont donnés à la figure 6.

Crue décennale : 1,8 x 80,80 = 145,43 m³/s
Crue centennale 2,6 x 80,80 = 210,07 m³/s
Crue millenale 3,4 x 80,80 = 274,72 m³/s



zones habituellement inondées

Figure 5

Année	Débit maximum (m^3/s)	Débit minimum (m^3/s)	Débit moyen (m^3/s)
67	85,2	0,93	9,43
68	128,0	2,02	9,7
69	80,5	1,49	10,4
71	28,3	0,585	4,89
72	36,5	1,9	5,42
73	51,5	0,85	7,35
74	105,0	2,59	12,5
75	38,2	2,08	7,89
76	30,0	1,87	5,66
77	38,3	2,7	8,5

Figure 6

On obtient les résultats suivants :

moyenne 62,15 m^3/s
écart type 35,17 m^3/s

Il faut remarquer que la série des débits maximums enregistrés est petite et que dans ce cas il faut être très prudent dans les extrapolations.

Les méthodes statistiques ne rendent pas compte des crues observées où des débits sont supérieurs à 350 m^3/s et en particulier du débit observé en 1934

Ces valeurs peuvent être comparées aux crues d'orages adoptées par l'Hydraulique Agricole qui sont évidemment plus faibles que celles relatives aux crues maximums, car les conditions de précipitations dans ce cas sont localisées sur un bassin versant plus restreint :

Chaudfontaine 138 m³/s
Chênée 140 m³/s

On a adopté comme valeur de crue du projet pour la protection contre les crues, la valeurs centenale de 210,07 m³/s. A partir de cette donnée, les axes hydrauliques ont été calculés de manière à définir les hauteurs des ouvrages à construire ou à modifier, en vue de contenir le passage des crues. Il a été tenu compte des paramètres suivants pour le calcul des axes hydrauliques : largeurs et formes du lit de la rivière, pentes, coefficient de frottement et accidents locaux (déversoirs, coudes, etc...). Les calculs ont été effectués par ordinateur en tenant compte des variations des paramètres pour les sections distantes d'environ 15 m, en mouvement graduellement varié.

Les travaux projetés et dont une partie est déjà en cours de réalisation sont des types suivants : exhaussements de murs de berge existants, construction de nouveaux murs en béton, ou en gabions avec digues imperméables, élargissements, rectifications de courbes, curage, abaissements de crêtes de barrages existants, construction d'un barrage mobile automatique, etc...
Le montant actuel des travaux est estimé à 95 millions soit à environ 15 millions par km de rivière.

VI. INSTALLATION D'UN MINI-CENTRALE HYDRO-ELECTRIQUE

Lors de la reconstruction du Barrage Bacquelaine à Chaudfontaine, l'Hydraulique Agricole a décidé de profiter de cette occasion pour y installer une *micro centrale hydro-électrique.*

Une première approche, tenant compte de la courbe des débits classés, a permis l'évaluation de l'énergie disponible en supposant l'installation d'une machine idéale (rendement 100 %) et une chute constante de 2 m quel que soit le débit. Les résultats des calculs sont donnés à la figure 7.

Coefficient d'utilisation	Débit Q (m³/s)	Puissance P (kW)	Enérgie annuelle (kWh)
100 %	1,75	34	297 840
80 %	3,5	68	476 544
70 %	4,2	82	502 824
60 %	4,9	96	504 576
50 %	5,8	114	499 320
40 %	7,3	143	501 072
30 %	9,9	194	509 832

Figure 7

Ce tableau fait apparaître une énergie annuelle maximale de 500 000 kWh environ. Il semble donc inutile de dépasser une puissance installée de 100 kW. De plus on a effectué une étude de rentabilité en fonction de la possibilité d'installer plusieurs turbines de puissances identiques. Soit 4 groupes de 30 kW maximum (rendement 0,56) :

- la turbine n° 1 tourne en permanence et sa puissance minimum est de 20 kW ($Q = 1,75$ m³/s)
- la turbine n° 2 démarre à 2,6 m³/s et fonctionne 93 % du temps,
- la turbine n° 3 démarre à 5,2 m³/s et fonctionne 57 % du temps,
- la turbine n° 4 démarre à 7,8 m³/s et fonctionne 38 % du temps.

On rappellera que la hauteur de chute est supposée constante.

La puissance de base est de 20 kW pendant 365 jours par année, soit 175 200 kWh. Au-delà, on calculera par paliers de 5 kW en adoptant une valeur moyenne de la fréquence des débits.

En ce qui concerne la production annuelle, il est possible de la déterminer en fonction du nombre de groupe installés (toujours en supposant une hauteur de chute constante) à partir du calcul de la puissance. Les résultats sont les suivants :

1 groupe	260 000 kWh - 10 % pour arrêts	: 235 000 kWh
2 groupes	450 000 kWh - 5 % pour arrêts	: 430 000 kWh
3 groupes	arrêts négligeables	: 550 000 kWh
4 groupes	arrêts négligeables	: 625 000 kWh

Enfin, on a pu calculer le rapport financier annuel de l'installation selon le prix de rachat de l'électricité par l'Association Liégeoise de l'Electricité (ALE). Cette tarification de l'A.L.E. tient compte des heures pleines, des heures creuses, de la fiabilité de la fourniture (Prime de régularité attribuée à partir de la deuxième année).

Le bilan final s'appuyant sur les coûts de l'installation, de raccordement et d'entretien, montre la rentabilité suivante (figure 8).

1 groupe	9,3 %
2 groupes	8,9 %
3 groupes	7,4 %
4 groupes	6,0 %

Figure 8

On constate que la rentabilité diminue lorsque le nombre de groupes augmente. Ce qui s'explique par la sous-utilisation des machines (groupe n° 3 : 57 % du temps - groupe n° 4 : 38 % du temps).
Il semble raisonnable donc de se limiter à un ou deux groupes, soit 30 ou 60 kW effectifs.
La solution d'un seul groupe éventuellement plus important serait la plus simple sur le plan technique.

A titre de prototype, et notamment en vue de vérifier les hypothèses émises dans ce domaine et d'améliorer la technologie existante, la Région Wallonne a confié à un constructeur belge la réalisation d'une turbine de 100 kW au barrage de Bacquelaine à Chaudfontaine.

La figure 9 montre une vue générale de l'aménagement et la figure 10 la turbine à installer.

Figure 9

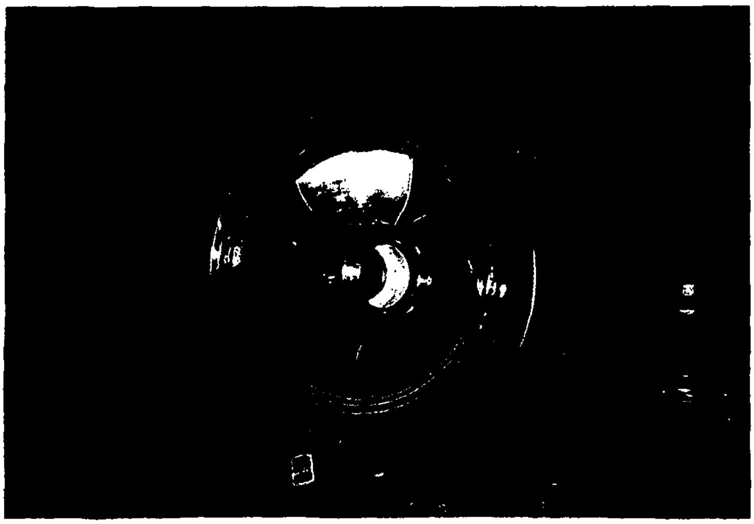


Figure 10

IWRA Vth WORLD CONGRESS ON WATER RESOURCES
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MODELE D'EXPLOITATION DYNAMIQUE DE L'ADDUCTION D'EAU D'EUPEN - LIEGE EN BELGIQUE

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RESUME

L'augmentation de la population desservie en eau par l'adduction Eupen-Liège a engendré une croissance constante du débit véhiculé. Cette adduction initialement surdimensionnée, est actuellement sujette à des aménagements supplémentaires tels que sa configuration future ne permet plus de garantir sa sécurité sans réglementer son mode d'exploitation.

Celui-ci est déduit de la connaissance à chaque instant de la pression et du débit en tout point de cette adduction.

A cette fin, un programme ordinateur a été mis au point. Celui-ci est la traduction en langage machine du modèle mathématique, brièvement rappelé, tenant compte de deux régimes d'écoulement dans les conduites.

I. POSITION DU PROBLEME

I.1. Description de l'adduction Eupen-Thiba

L'adduction Eupen-Thiba est actuellement constituée d'un réservoir (Station d'Eupen) alimentant deux conduites symétriques, reliant Eupen à Goé et se rassemblant en une seule de Goé à Fléron. La conduite à Fléron se divise ensuite en deux vers la prise de Rotheux-Ramet. A l'aval de cette dernière une chambre de rupture de pression de 120 N/cm² est en cours de construction. Le réservoir aval de celle-ci alimentera l'adduction Thiba.

Le tableau 1 donne les caractéristiques des conduites de l'adduction.

Tableau 1 - Longueur totale = 58 650,84 m

Adduction	Tronçon	Nature de la conduite	Nombre	Longueur (m)	Diamètre (mm)	Epaisseur (mm)	Rugosité (mm)
Eupen-Seraing	I	Sidéro-ciment	2	2982,48	1100	100	0,326
	II	acier	2	3009,77	900	10	1,517
	III	béton-précontraint	2	3169,19	900	100	1,517
	IV	sidéro-ciment	1	22371,36	1100	100	0,326
	V	acier	2	14678,04	800	10	1,517
Thiba	VI	acier	2	12440	800	10	1,517

I.2. Evolution du débit d'eau consommé

L'augmentation de la population desservie et l'évolution du mode de vie a engendré une croissance constante de la consommation qui est passée de 10 000 m³/j en 1960 à plus de 60 000 m³/j en 1970 et le besoin en eau n'a cessé de croître.
Le barrage de la vesdre ne pouvant garantir en toute circonstance qu'un débit de 75 000 m³/j, on se trouvait en 1974 dans une situation difficile en ce qui concerne l'accroissement du besoin en eau conjointement à l'occurrence d'années sèches.
Trois solutions ont alors été envisagées :

I.2.1. Solution à cours terme

Construction de deux filtres supplémentaires à la station d'épuration de la Vesdre, qui permettraient d'assurer 82 500 m³/j par le barrage sauf en année sèche. Dans ce cas le débit d'étiage serait assuré par le barrage de la Gileppe et le barrage de la Vesdre fournirait le complément d'eau à traiter. Toutefois, la Vesdre serait à sec jusqu'à Béthane.
Un remède à cette situation consisterait à dédoubler l'adduction entre Membach et la station de la Vesdre donnant ainsi la possibilité, en cas d'insuffisance de la réserve de cette station, de prélever de l'eau brute dans la conduite de la Gileppe et de la pomper par une des conduites jusqu'au bassin de tête de la station d'épuration de la Vesdre.

I.2.2. Solution à moyen terme

On prévoyait de refouler dans les conduites de l'adduction les eaux disponibles au barrage de la Gileppe après traitement dans une station d'épuration pouvant fournir jusqu'à 90 000 m³/j.

I.2.3. Solution à long terme

Construction sur l'Ourthe occidentale d'un barrage réservoir qui permettrait un prélèvement permanent en rivière. L'eau prélevée serait traitée dans une nouvelle station d'épuration comprenant trois modules de 30 000 m³/j. Elle serait construite le long de l'Ourthe à Tilff au croisement de l'adduction.

I.3. Modes d'exploitation possibles de l'adduction

I.3.1. Points d'alimentation de l'adduction

- Le débit total consommé par les réseaux de distribution alimentés est égal ou inférieur au débit disponible à la station d'Eupen. Dans ce cas la station d'Eupen alimente seule l'adduction.

- Le débit consommé est supérieur à celui disponible à la station d'Eupen. Dans ce cas le complément est fourni par la station de la Gileppe ou de Tilff.

- Les débits fournis à Eupen et à la Gileppe sont insuffisants, la station à Tilff fournira alors le complément du débit consommé.

- Il est aussi envisagé d'assurer un débit pour le cas d'une mise hors-service de l'une ou des deux stations amont d'Eupen et de la Gileppe. Ce débit sera fourni par la station de Tilff.

Le tableau 2 montre les différents modes possibles d'alimentation de l'adduction.

Tableau 2

Mode	Station d'Eupen	Station de la Gileppe	Station de Tilff
I	Q_E	0	0
II	Q_E	Q_G	0
III	Q_E	Q_G	Q_T
IV	Q_E	0	Q_T
V	0	Q_G	0
VI	0	Q_G	0
VII	0	0	Q_T

I.3.2. Points de consommation

L'adduction alimente quarante réseaux de distribution d'eau situés entre Eupen et Thiba. Le débit y soustrait varie d'un réseau à un autre selon l'importance de celui-ci. Certains réseaux de distribution sont munis d'un réservoir tampon et d'autres sont connectés directement à l'adduction à l'aide d'une simple vanne ou d'une pompe.

I.4. Necessité d'un programme de gestion

Pour les faibles débits l'adduction surdimensionnée peut être considérée comme un réservoir amortissant sans inconvénient les fluctuations des pressions dues à une manoeuvre d'une vanne ou à un arrêt d'une pompe d'une prise d'eau. Les futures extensions prévues de cette adduction ne permettent plus de garantir sa sécurité sans réglementer son mode d'exploitation. Celle-ci est évidemment déduite de la connaissance à chaque instant de la pression et du

débit en tout point de l'adduction.
A cette fin, un programme ordinateur a été mis au point.
Celui-ci, écrit en FORTRAN, est une traduction du modèle mathématique suivant.

II. MODELE MATHEMATIQUE

II.1. Régime permanent

Pour un écoulement établi dans un réseau de distribution d'eau, les équations régissant le phénomène sont les suivantes :

II.1.1. Equations de continuité

Div $\vec{v} = 0$ soit pour tout noeud I du réseau, on a :
(point de consommation, d'injection ou de connexion des conduites)

$$\sum_{j=1}^{n} Q(I,J) = 0 \qquad (II.1.1.)$$

où n est le nombre de conduites ayant en commun le noeud I
Q(I,J) est le débit dans la conduite I.J, reliant le noeud I au noeud J

II.1.2. Les équations duales

Pour tout noeud (I) du réseau on a :

$$H(I,J) = H(I) \quad \text{pour tout } J \qquad (II.1.2.)$$

où H(I) est la charge au noeud I
H(I,J) est la charge au noeud I calculée en fonction des caractéristiques géométriques et hydrauliques de la conduite I-J.

II.1.3. Les équations caractéristiques

L'équation caractéristique d'une conduite est celle donnant la perte de charge en fonction de son débit :

$$h(I,J) = H(I) - H(J) = G(I,J) \cdot Q(I,J) \qquad (II.1.3.)$$

où

$$G(I,J) = \frac{\lambda(I,J)}{D^5(I,J)} \cdot L(I,J) \; |Q(I,J)| \frac{8}{g.\pi^2}$$

$\lambda(I,J)$ est le coefficient de perte de charge
D(I,J) est le diamètre de la conduite I-J
L(I,J) est la longueur de la conduite I-J
|Q(I,J)| est la valeur absolue du débit Q(I,J) de la conduite

II.1.3. Les équations du régime permanent écrits dans le cas de l'adduction

L'adduction est représentée par n noeuds (points de consommation, d'injection ou de connexions) et n-1 branches. Les noeuds sont numérotés de 1 à n à partir de la station d'Eupen et les branches de n+1 et 2n-1. Cette présentation est décrite par la matrice IA(I,J), telle que :

$$IA(I,J) = \begin{cases} +1 & \text{si le débit dans la conduite I-J part du noeud I} \\ 0 & \text{si le noeud I n'appartient pas à la conduite I-J} \\ -1 & \text{si le débit de la conduite I-J arrive au noeud J} \end{cases}$$

Si les données de l'adduction sont :
- les débits consommés (débits clients) : qc(I)
- les débits injectés : q(I)
- la charge en un noeud au moins : H(I)

les inconnues étant :
- les débits dans la conduite : Q(I,J)
- les charges aux noeuds : H(I)

Les équations s'écrivent :

Equations de continuité

$$\begin{bmatrix} IA(1,1) & 0 & IA(1,3) \\ 0 & E & IA(2,3) \end{bmatrix} \begin{bmatrix} q(1) \\ qc(J) \\ q(n,m) \end{bmatrix} = 0$$

Soit :

$$[IA(1,1)]\, q(1) + [IA(1,3]\, q(n,m) = 0 \qquad \text{(II.1.4.)}$$

$$(E)\, qc(J) + [IA(2,3)]\, q(n,m) = 0 \qquad \text{(II.1.5.)}$$

Equations duales

$$[IA^T(1,1)] \quad h(I) \quad = h(I) \qquad \text{(II.1.6.)}$$

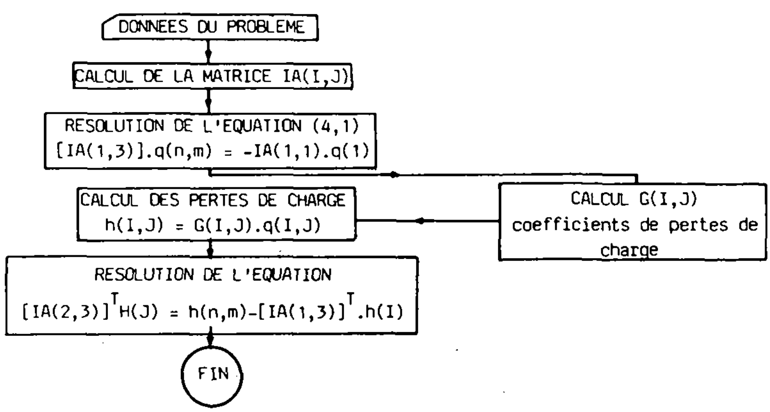
$$[E]\, H(J) \quad = \quad H(J) \qquad \text{(II.1.7.)}$$

$$[IA^T(1,3)] \quad h(I) + [IA^T(2,3)] \quad H(J) = h(n,m) \qquad \text{(II.1.8)}$$

Equations caractéristiques

$$h(I,J) = G(I,J)\, q(I,J) \qquad \text{(II.1.9.)}$$

La résolution des équations de l'adduction s'effectue selon l'organigramme suivant :



II.2. Régime transitoire

Les équations du modèle mathématique sont :

II.2.1. L'équation de Newton

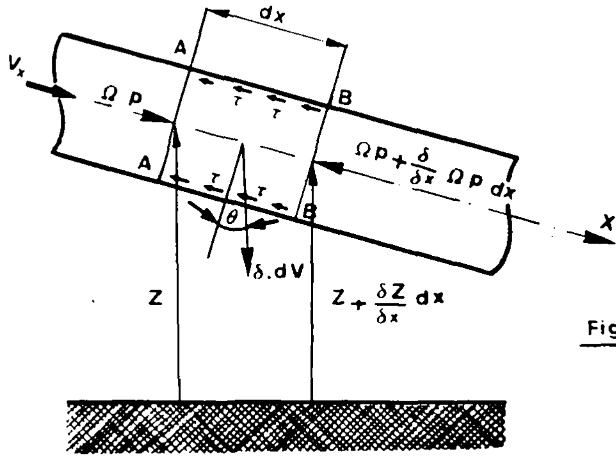


Figure 1

$$\frac{1}{g}\left[\frac{\partial v_x}{\partial t} + v_x \frac{\partial v_x}{\partial x}\right] + \frac{\partial h}{\partial x} = - T \qquad (II.2.1.)$$

où : g est l'accélération de la pesanteur

v_x est la vitesse moyenne dans la section x

$h = \frac{P}{\delta} + Z$

P est la pression dans la section X

δ est le poids spécifique de l'eau

Z est l'altitude

T = frottement par unité de poids

II.2.2. L'équation de continuité

$$\frac{c^2}{g}\frac{\partial v_x}{\partial x} + \frac{\partial h}{\partial t} + v_x \frac{\partial h}{\partial x} = v_x \frac{\partial Z}{\partial x} \qquad (II.2.2.)$$

où

$$c^2 = \frac{1}{\rho(\beta + \frac{D}{E.e})}$$

β est le coefficient de compressibilité de l'eau

D est le diamètre de la conduite

e est l'épaisseur de la conduite

E est le module d'élasticité du matériau de la conduite

Les équations (II.2.1.), (II.2.2.) avec les conditions aux limites et une hypothèse sur le paramètre définissant les forces de frottement constituent le modèle mathématique du régime transitoire.

II.2.3. Méthode d'intégration du système d'équations (II.2.1.) et (II.2.2.)

On a un système d'équations aux dérivées partielles quasi linéaires de type hyperbolique. Une intégration analytique de ce système d'équations n'est

possible que moyennant certaines hypothèses restrictives telles que :
- les pertes de charge sont nulles
- le terme d'inertie est négligeable
- les conditions aux limites sont simples

Dans le cas général, les méthodes d'intégration numérique sont pratiquement les plus indiquées. Parmi ces méthodes nous avons retenu la méthode des caractéristiques, qui permet de tenir compte, d'une façon simple, des conditions aux limites rencontrées en pratique.
Le fondement théorique de la méthode est détaillé dans (3), nous rappelons l'essentiel des résultats.

Les équations de caractéristiques

$$dx = c\,dt \qquad \text{pour } T^+$$

$$dx = -\,c\,dt \qquad \text{pour } T^-$$

Les relations sur la caractéristique

$$dh + \frac{c}{g}\,dv + \frac{\lambda}{D}\,\frac{v|v|}{2g}\,c\,dt + v\,\frac{\partial Z}{\partial x}\,dt = 0 \qquad \text{sur } T^+$$

$$-dh + \frac{c}{g}\,dv + \frac{\lambda}{D}\,\frac{v|v|}{2g}\,c\,dt + v\,\frac{\partial Z}{\partial x}\,dt = 0 \qquad \text{sur } T^-$$

Pour $\frac{\partial Z}{\partial x}$ finie on a :

$$v\,\frac{\partial Z}{\partial x}\,dt = \pm\,\frac{v}{c}\,\frac{\partial Z}{\partial x}\,dt = \pm\,M\,dZ << 1$$

En négligeant $v\,\frac{\partial Z}{\partial x}$ on obtient encore les relations sur les caractéristiques.

$$dh + \frac{c}{g}\,dv + \frac{\lambda}{D}\,\frac{v|v|}{2g}\,c\,dt = 0 \qquad \text{sur } T^+$$

$$-dh + \frac{c}{g}\,dv + \frac{\lambda}{D}\,\frac{v|v|}{2g}\,c\,dt = 0 \qquad \text{sur } T^-$$

En notant par $Q = \Omega\,.\,v$

où Ω est la section de la conduite en x, on obtient le schéma suivant :

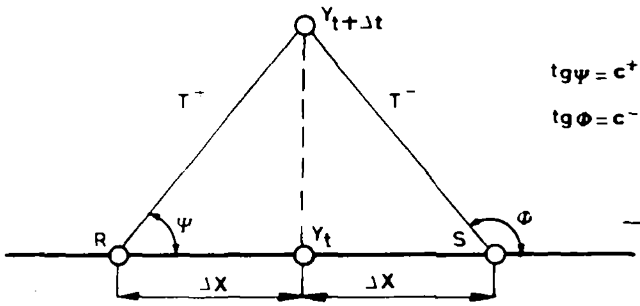


Figure 2

$$h_Y = H_R - B_R . Q_Y \qquad \text{sur } T^+$$

$$h_Y = H_S + B_S . Q_Y. \qquad \text{sur } T^-$$

ou encore

$$h_Y = \frac{B_R H_R + B_S H_S}{B_R + B_S}$$

$$Q_Y = \frac{H_R - H_S}{B_R + B_S} \qquad (II.2.3.)$$

Avec :

$$H_R = h_R + B_R . Q_R - c \frac{\lambda}{D_R \Omega_R^2} \frac{Q_R |Q_R|}{2g} \Delta t + \frac{Q_R}{\Omega_R} \frac{\Delta Z}{\Delta x} \Bigg|_R \Delta t$$

$$H_S = h_S - B_S . Q_S + c \frac{\lambda}{D_S \Omega_S^2} \frac{Q_S |Q_S|}{2g} \Delta t + \frac{Q_S}{\Omega_S} \frac{\Delta Z}{\Delta x} \Bigg|_S \Delta t$$

Ces formules permettent de calculer h et Q au point Y à l'instant t + Δt quand on donne Q_R, Q_S, h_R et h_S à l'instant t.

II.2.4. Conditions aux limites

En ce qui concerne l'adduction Eupen-Thiba les éléments de contrôle sont

- Réservoir de la station d'Eupen où la hauteur $h = \frac{P}{\delta} + Z$ est imposée
- Vannes des utilisateurs
- Vannes de la chambre de rupture
- Pompes des utilisateurs
- Vannes ou pompes des conduites de la Gileppe et de Tilff

Les modélisations de ces éléments de contrôle sont :

Pour le réservoir amont (Station d'Eupen)

Pour $t = t_o$ on a :

$$h = h_o = Cte$$

$$Q = Q(t_o)$$

Pour $t = t + \Delta t$ on a toujours

$$h = h_o \text{ (réservoir)}$$

$$Q = Q(t_o + \Delta t)$$

La valeur du débit Q à l'instant $t_o + \Delta t$ est déduite de la relation sur la caractéristique T^-. (figure)

$$Q(t_o + t) = \frac{h_o - H_S}{B_S}$$

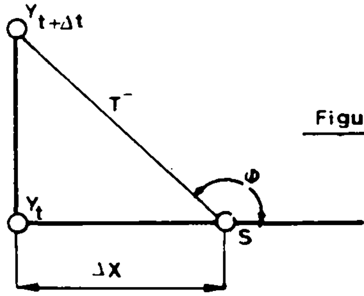


Figure 3

Pour les vannes de la chambre de rupture

En $t = t_0$ on a :

$$h = h(t_0)$$

$$Q = Q(t_0)$$

En $t = t_0 + \Delta t$

La loi d'ouverture ou de fermeture de la vanne $Q = Q(t)$ permet de calculer $Q(t_0+\Delta t)$. La relation sur la caractéristique T^+ (vanne aval) ou T^- (vanne amont) permet de calculer

h en $t_0 + \Delta t$

$$h(t+\Delta t) = H_R - B_R\, Q(t_0+\Delta t) \qquad \text{(vanne aval)}$$

$$h(t+\Delta t) = H_S + B_S\, Q(t_0+\Delta t) \qquad \text{(vanne amont)}$$

Pour les vannes utilisateurs

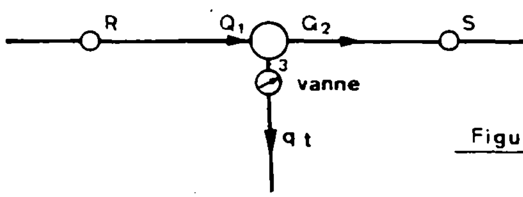


Figure 4

En $t = t_0$ on connaît

$Q_1(t_0)$, $q(t_0)$, $Q_2(t_0)$, et $h(t_0)$ au point I (régime permanent)

En $t = t_o + \Delta t$ on a les relations

$$Q_1(t) = q(t) + Q_2(t)$$
$$h_1(t) = H_R - B_R Q_1(t)$$
$$h_2(t) = H_S + B_S Q_2(t)$$
$$h_1(t) = h_2(t)$$

De ces relations on déduit :

$$Q_1(t) = \frac{H_R - H_S + B_S\, q(t)}{B_S + B_R}$$

$$Q_2(t) = Q_1(t) - q(t)$$

$$h(t) = H_R - \frac{B_R}{B_R + B_S}(H_R - H_S + B_S\, q(t)]$$

où q(t) est la loi de variation de débit de l'utilisateur.

Pour une pompe munie d'un clapet anti-retour

Pour l'étude des pompes on admet la validité des lois de similitude dynamique simplifiées. Des expressions de la puissance (P) et du moment cinétique, on déduit :

$$\frac{dN}{dt} = \frac{P}{I.N}\left(\frac{60}{2\pi}\right)^2$$

c'est-à-dire :

$$N(t_o+\Delta t) = N(t_o) + \frac{P(t_o)}{IN(t_o)}\left(\frac{60}{2\pi}\right)^2 \Delta t$$

Connaissant $P(t_o)$ et $N(t_o)$ on calcule $N(t_o+\Delta t)$. Ayant $N(t_o+\Delta t)$ les paramètres $q(t_o+\Delta t)$, $H(t_o+\Delta t)$ et $P(t_o+\Delta t)$ correspondants sont déduits des relations de similitude.
Les conditions aux limites correspondant à une pompe s'écrivent :

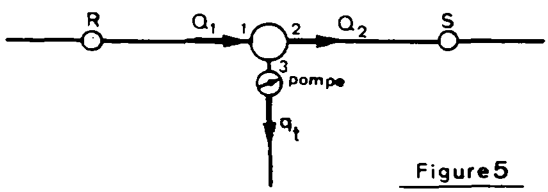


Figure 5

$$Q_1(t_o+\Delta t) = \frac{H_R - H_S + B_S q(t_o+\Delta t)}{B_S + B_R}$$

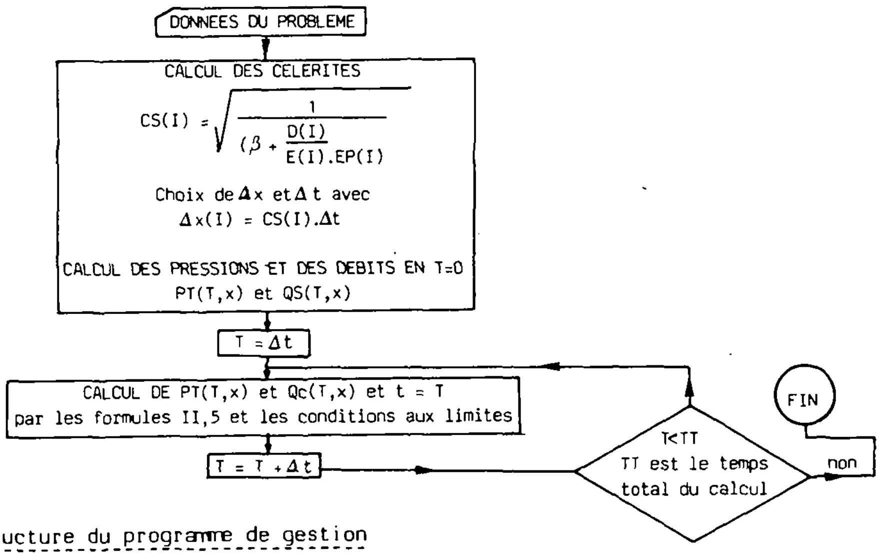
$$h(t_o+\Delta t) = H_R - \frac{B_R}{B_R+B_S}\,[H_R - H_S + B_S\, q(t_o+\Delta t)]$$

$$Q_2(t_o+\Delta t) = Q_1(t_o+\Delta t) - q(t_o+\Delta t)$$

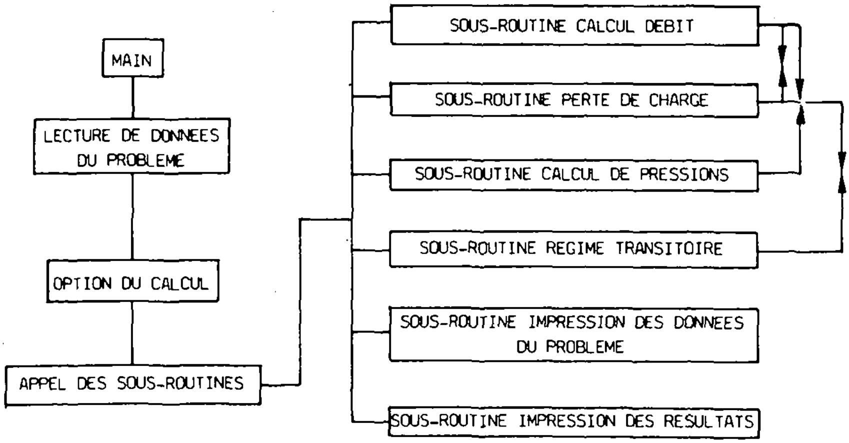
II.2.5. Loi de manoeuvre [q(t) ou N (t)]

Le programme mis au point ajuste toute loi de manoeuvre réelle à une loi analytique obtenue par l'approximation de Lagrange.

Schéma de résolution



Structure du programme de gestion



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CRITERIOS PARA EL ANALISIS Y MANEJO DE LAS INUNDACIONES EN EL NORDESTE ARGENTINO

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RESUMEN

El área afectada por las crecientes e inundaciones en las llanuras argentinas es superior a la extensión de varios países europeos, comprometiendo el desarrollo de decenas de millones de Has. y la seguridad de grandes áreas urbanas.

Su recurrencia en el tiempo y las características singulares de su manifestación, requieren un sistema de manejo multivariable que incluya aspectos técnicos, jurídico-institucionales y socioeconómicos y la definición de estrategias globales y sectoriales a corto, mediano y largo plazo.

Se insiste especialmente en los condicionantes del sistema natural y el contexto socioeconómico sobre los criterios para la concepción, estudio y proyecto de las obras de control y se plantean las pautas básicas para las políticas y estrategias vinculadas con la formación de recursos humanos, ciencia y técnica, participación de la comunidad y relaciones interjurisdiccionales.

Palabras claves: Inundaciones, manejo de recursos hídricos, política hídrica, geomorfología aplicada, sistemas de escurrimiento.

I - CRECIENTES E INUNDACIONES

Parece conveniente diferenciar estos dos fenómenos que, si bien con frecuencia aparecen asociados, obedecen a causas diferentes siendo sus efectos también distintos en cada caso.(12)

La creciente es en escencia un fenómeno hidrológico puro que in dica un aumento significativo del caudal, pero que si se produce en un valle encañonado no se traduce necesariamente en una i nundación o bien ella es insignificante.

La inundación es un fenómeno fuertemente controlado por la morfología que determina la ocupación de extensiones significati-/ vas de terreno por láminas de agua.

En las llanuras esta diferenciación es muy importante por va-// rias razones, entre las cuales podemos mencionar:

1. Existen áreas muy extensas, de miles de km2 sin cursos flu-/ viales definidos, o bien con redes desintegradas, modelos // cribados, o simplemente no hay red y sin embargo las inundaciones originan verdaderos desastres, destruyendo cultivos,/ aislando poblaciones y provocando daños gravísimos a las o-/ bras de infraestructura (16, 8).
2. Los valles de los cursos más significativos presentan plani-

cies fluviales muy amplias (llegando a los 30 km de ancho en el río Paraná) que en crecientes excepcionales se inundan // completamente, generando problemas muy serios en centros urbanos de primer orden como Resistencia, Formosa, Goya, etc., situados en terrazas. (10, 11)

3. En el Noroeste un antiguo modelo de macrotorrentes ha originado una singular morfología que durante las crecientes de / los grandes cursos (Bermejo y Pilcomayo) posibilita que las/ aguas corran a mayor nivel que las planicies laterales, de / manera que al romperse los diques naturales (paleoderrames)/ ellas ingresan a las planicies en forma de láminas de agua / difíciles de controlar. (10)
4. Con frecuencia estas situaciones se producen simultáneamente generando una verdadera catástrofe que puede duras meses por la lenta evacuación de los inmensos volúmenes de agua acumulados, la ineficiencia del drenaje y los remansos que se originan en los cursos afluentes (11, 12).

Resumiendo, podemos decir que en el Nordeste Argentino y en /// gran parte de la llanura en la cual se inserta, se pueden dis-/ tinguir inundaciones de origen pluvial, crecientes con inunda-/ ciones en los valles fluviales, inundaciones por desbordes y todos los tipos mixtos según las circunstancias y la posibilidad/ de superposición de efectos.

II - CONSIDERACIONES SISTEMICAS

A continuación intentaremos plantear algunas consideraciones derivadas del planteo sistémico del problema, que nos permitirán/ comprender los fenómenos y definir políticas y estrategias que/ se adecuen a la realidad.

1°. El área presenta subsistemas hídricos muy diferentes entre/ sí, tanto en cuanto la amplitud de oscilación como en la // magnitud de los caudales y volúmenes transportados.

2°. La observación detallada de la geomorfología indica que se/ encuentran emplazadas en la faja de transición entre la zona intertropical y la templada, que es una de las más sus-/ ceptible a los cambios paramétricos climáticos y antrópicos y por consiguiente de altísimo riesgo. (10)

3°. Que los ecosistemas imperantes en el área se han adecuado a un pulso oscilante de sequías e inundaciones, es decir que/ ellas forman parte inherente al sistema natural y su even-/ tual alteración modificará el comportamiento de todos los / subsistemas que integran el geomorfológico, lo cual podría/ ser más peligroso que la situación actual si no se actúa ra cionalmente. (5, 6)

4°. Que por lo expuesto el pulso hídrico en el espacio y el // tiempo controla la vegetación, los suelos y los procesos de erosión y sedimentación.

5°. Las formas del relieve indican que los subsistemas hidroló- gicos pueden oscilar mucho más arriba de lo conocido, es de cir que las crecientes e inundaciones pueden ser todavía // marcadamente superiores a las registradas y en especial a / las de 1983/84, e igualmente que sequías devastadoras pue-/ den producirse.

6°. La red de escurrimiento controla el comportamiento del sub- sistema hidrológico y por consiguiente, las áreas inundadas e inundables actúan como reservorios permanentes o eventua- les que amortiguan el empuntamiento y están siendo someti-/ dos a fuerte presión antrópica por la extensión de los cul- tivos y la irracional construcción de canales.

7°. Que los diferentes subsistemas pueden llegar a superponer / sus efectos superando la capacidad de control natural por / depresiones y dando lugar a verdaderas catástrofes si no se toma en cuenta dicha superposición en los cálculos de maxi- mización.

III - CONCEPTOS FUNDAMENTALES PARA EL MANEJO

III.1 - Las bases conceptuales

Cualquiera sea aquello que se desee manejar, es necesario aco-/

plarle un sistema tal que la salida del conjunto esté controlada y por lo tanto aparecen una serie de condiciones necesarias/ para lograrlo y para conocer qué podemos esperar del conjunto.

En primer lugar es necesario conocer el sistema que queremos manejar, teniendo presente que dicho conocimiento depende de los/ parámetros socioculturales y tiene por lo tanto un cierto grado de indeterminación conceptual, inversamente proporcional al desarrollo de dicho universo. (9)

En segundó lugar, todo sistema tiene umbrales de equilibrio que pueden ser superados por impactos externos, entre los cuales // puede estar el propio sistema de control y por lo tanto que en/ condiciones naturales la salida tiene un determinado pulso o espectro de comportamiento que se necesita conocer. (12)

En tercer lugar, el sistema de control está condicionado por el tipo de sistema al cual se va a acoplar y por los parámetros socioculturales estableciéndose relaciones de retroalimentación / con este último universo.

En cuarto lugar, el sistema de control tendrá siempre cuatro / subsistemas como mínimo: el energético, el de transformación, / el de transmisión y el de conducción. (12)

Con estos conceptos básicos podemos volver a considerar nuestro tema específico de las crecientes e inundaciones y su relación/ con lo expuesto.

Con relación al sistema que se pretende manejar se pueden sacar las siguientes conclusiones: (4)

- Los estudios básicos, que permiten conocerlo de la manera más definida posible, son esenciales y deben tener una atención / especial, permanente y sistemática.
- La centralización de la información en un banco de datos y // procesamiento es indispensable y deben implementarse todas // las acciones posibles para lograrlo con el máximo nivel tecnológico compatible con los recursos disponibles.
- La normalización de los métodos y procedimientos operativos /

para la obtención y procesamiento de la información es esen-/ cial al conocimiento objetivo del sistema.

- El estudio sistemático de la validez de los modelos conceptua les sobre comportamiento del sistema constituye el eje de la/ cuestión, ya que siempre existirá un grado de indeterminación que reside en la naturaleza misma de las cosas.
- La consideración de los impactos externos, en su tipología, / magnitud y espectro, sobre el sistema deben formar parte del/ análisis de comportamiento y prognosis, instrumentando las me didas de control permanente de los cambios originados por los mismos.

Con relación al sistema de manejo se pueden sacar las siguien-/ tes conclusiones:

- Los cuatro subsistemas mencionados se corresponden para el ca so particular del manejo de las inundaciones con los cuatro / indicados en la misma.
- La energía disponible son los recursos económicos, los meca-/ nismos de transmisión, los recursos tecnológicos y operativos, los de transformación son los recursos humanos y la conduc-// ción constituyen los instrumentos jurídicos y administrativos y la decisión política.

El primer subsistema, como prioridad de eficiencia, es el de // conducción, y de él podemos decir lo siguiente:

- La definición clara y explícita de la política hídrica, sus / bases conceptuales, objetivos y estrategias deben sustentar / la totalidad de las acciones organizativas y operativas.
- La conducción de dicha política debe ser centralizada aún // cuando las unidades operativas estén descentralizadas.
- La elaboración de dicha política deberá acompañarse del ins-/ trumento jurídico y administrativo que permita responder a // los parámetros socioculturales y adaptarse a las característi cas del sistema natural que se pretende manejar.
- El subsistema de manejo debe conocer las implicancias de la /

noción teleológica, asegurando que la disminución de la indefinición (técnica) acompañe a la disminución de la indeterminación (política) con la misma velocidad, para no demorar los tiempos necesarios para lograr los objetivos propuestos.

- Lo anterior implica que debe conocerse el grado de respuesta/ esperable del universo sociocultural y la capacidad de acompañar la decisión política con los proyectos y obras en base a/ los recursos humanos, instrumentales y económicos disponibles.
- El subsistema de manejo debe concebirse con una capacidad de/ oscilación o adaptación tanto más amplia cuanto menor sea el/ nivel de desarrollo del universo sociocultural del cual depende.

El segundo subsistema, en el otro extremo jerárquico, es el correspondiente a los recursos económicos, entendiendo como tales el conjunto de bienes que pueden ser transformados para activar el sistema de manejo, y con respecto a él podemos decir lo si-/ guiente:

- Existe una relación directa y biuniboca entre este subsistema y el de conducción, que debe controlar la dimensión o escala/ del primero para que sea eficiente y operativo.
- La disponibilidad de bienes en el subsistema económico se expresa no solamente en cantidad sino también en espacio y tiempo y por consiguiente es un sector referido a tres ejes coordenados.
- Por lo expuesto es posible expresar derivadas parciales o velocidades relativas de cambio, así como velocidad y aceleración totales.
- La velocidad del cambio posible controla al sistema total y / la posibilidad de acelerar el cambio determina los límites de la decisión política.
- El aumento de velocidad, o la aceleración aumenta la entropía y requiere adecuar los otros dos sistemas: recursos humanos y recursos tecnológicos, produciendo mayor gasto económico

- Los otros dos subsistemas mencionados condicionan la capaci-/ dad de cambio y su aceleración teniendo a su vez tiempos de / respuesta no siempre cortos.
- La disponibilidad de recursos económicos está controlada por/ los universos, que a su vez están ligados por las redes cir-/ cuitales con el subsistema de comando y con todo el sistema / de manejo en última instancia, por lo cual existe siempre un/ grado de indeterminación tanto más alto cuanto mayor sea el / subdesarrollo.

El tercer subsistema es el tecnológico, que recibe influjos directos e indirectos desde los universos y desde los otros sub-/ sistemas, con respecto al cual podemos decir:

- El nivel tecnológico a emplear es un sector multivariable que en última instancia se define en función de la concepción política y sociocultural y no necesariamente por sus parámetros internos.
- El aumento del nivel tecnológico sólo será eficiente si logra disminuir la entropía específica del subsistema económico, si dispone de los recursos humanos necesarios y si la conducción es acertada.
- El aumento del nivel tecnológico provoca aumento del nivel // del recurso humano y simplifica la conducción pero aumenta la necesidad de respuestas rápidas y multivariables en este último.
- El tipo de tecnología a aplicar depende de un condicionante / socioeconómico que determina el nivel de aquélla para adecuarlo a ésta, pero también decisiones políticas en lo atinente a la participación de la industria nacional y/o al grado de empuntamiento de determinados sectores industriales.

El último subsistema lo constituyen los recursos humanos que representan la pieza capital de todo el conjunto, cuya capacidad/ técnica y de conducción es la base del éxito, respecto del cual podemos decir:

- El recurso debe ser preparado y adiestrado para saber armonizar los requerimientos tecnológicos con los económicos, adecuándolos a las condiciones socioculturales del área.
- Los flujos horizontales que permitan las relaciones inter y multisectoriales deben estar asegurados y el personal debe ser entrenado y organizado para que ello se logre con eficiencia y operatividad.
- Los aspectos jurídico-institucionales deben formar parte esencial de la formación y capacitación del personal, así como también aquéllos que permitan la comprensión de la realidad socioeconómica sobre la cual deben actuar.
- La capacitación permanente debe estar formando parte de la política hídrica en todos los niveles técnicos y profesionales.

III.2 - Sobre el tratamiento específico de los problemas

Parece conveniente desagregar los problemas en los siguientes: 1 - Areas de planicies fluviales inundables; 2 - Areas inundadas e inundables por pluviosidad y/o mal avenamiento; 3 - Areas de conoides aluviales sobreelevados; y 4 - Areas con torrentes de llanura.

1 - Areas de planicies fluviales inundables:

- Debe tenerse en cuenta que para situaciones maximizadas, todo el valle fluvial puede ser ocupado por las aguas.
- Se requiere determinar las líneas de iso-riesgo, es decir curvas que definen zonas con semejantes probabilidades de inundación y cotas a alcanzar en cada caso.
- En base a lo anterior, establecer las restricciones de uso del suelo y las normas edilicias que aseguren el salvataje en caso de emergencia y minimicen los efectos negativos.
- Igualmente establecer las responsabilidades de los usuarios y del estado frente a las inundaciones en relación al nivel de riesgo hídrico, diferenciando las tasas y estableciendo las pautas para reclamo y auxilio.
- Estudiar alternativas de obras de infraestructura y su facti-

bilidad en función de la recurrencia y de las características del núcleo urbano a defender.

- Prioritar aquellas obras que aseguren la evacuación de la población y sus servicios esenciales bajo condiciones de inundación maximizada, aún cuando existen obras de defensas para niveles hídricos menores y como seguridad frente a situaciones/ de colapso o sobrepaso.
- Estudiar la factibilidad de accesos elevados, edificación de/ tipo palafítico y otras variantes que puedan emplearse en // reemplazo de polders para poblaciones menores.
- Asegurar que las obras portuarias vinculadas a servicios esenciales estén proyectadas para operar en las peores condicio-/ nes posibles y compatibles con la disponibilidad de recursos.
- Restringir la ocupación urbana de las planicies fluviales al/ mínimo compatible, tanto más cuanto más bajo el nivel tipográfico.
- Asegurar que los terraplenes viales que atraviesen transver-/ salmente el valle no alteren significativamente la sección de escurrimiento ni los filetes líquidos mediante el diseño adecuado de obras de arte debidamente distribuidas y con accio-/ nes suficientes.
- Prestar especial atención a las obras de arte que puedan provocar concentración del escurrimiento con erosiones que pue-/ dan provocar su colapso.
- Analizar detalladamente el efecto de la biomasa flotante a // fin de evitar su embancamiento, empujes y/o erosiones sobre / las obras civiles.
- En los casos en los cuales se recurra a la polderización, deberá analizarse detalladamente la evacuación de los aspectos/ pluviales y de infiltración a los recintos, la elevación de / la freática y el diseño de las obras con relación al escurrimiento.
- Deberá preverse un programa de precisión y alerta así como el

de emergencia frente a colapso o sobrepaso y la debida ins-// trucción a la población en todos los niveles.

- La preparación de técnicos en defensa de ciudades sometidas a emergencias hídricas deberá instrumentarse a fin de disponer/ de personal idóneo para el control y operación de los siste-/ mas de servicios esenciales.
- Deberán estudiarse y ponerse en vigencia los instrumentos jurídicos y administrativos que permitan actuar con eficiencia/ y celeridad en las emergencias.

2 - Areas inundadas e inundables por pluviosidad y/o mal avenamiento:

- Debe instrumentarse un programa conjunto vial e hidráulico // que asegure el aislamiento y/o definición de las cuencas de / llanura, evite la destrucción de obras de arte, asegure el // tránsito y controle el escurrimiento.
- Deberá estudiarse la posibilidad de generar reservorios de poca altura mediante terraplenes viales que disminuyan el empuntamiento de las crecientes, con el agregado de obras hidráulicas para manejo y evacuación de excedentes.
- Los préstamos y cunetas laterales a la obra vial deberán ser/ analizados como posibles sistemas de conducción del escurri-/ miento mediante obras especiales.
- Se tendrá especial cuidado en no aumentar exageradamente la / red natural de escurrimiento para evitar empuntamientos peligrosos.
- El control sobre el uso del suelo y en especial la remoción / de la vegetación natural sobre las divisorias de aguas deberá asegurarse.
- Los caminos transversales a las áreas inundadas e inundables/ deberán tener una rasante superior a la inundación y suficientes obras de arte, debidamente calculadas y localizadas para/ no interferir en el escurrimiento.
- El plan hidrovial deberá diseñar una red troncal de evacuación

frente al máximo riesgo posible para asegurar vidas y bienes.

- Deberá ponerse especial cuidado en la eliminación de los re-/servorios naturales y en la construcción de canales frente al riesgo de aumentar el empuntamiento hidrológico.
- Deberán estudiarse las posiblidades de recintos urbanos, circuitos hidrológicos, embalses escalonados y todo tipo de obra que permita integrar un sistema multivariable de control.

3 - Areas de conoides aluviales sobreelevados:

- Se deberá preservar los derrames laterales que confinan las / aguas, evitando todo tipo de obra o uso del suelo que los rebaje o pueda erosionarlos.
- Se estudiará la posibilidad de localizar los terraplenes viales paralelamente a los derrames como diques laterales de contención de desbordes separados de tal manera que determinen una sección conveniente.
- Deberá analizarse un plan hidrovial que permita controlar el/ desplazamiento de las aguas en caso de desbordes e ingreso en las planicies.
- Las rutas transversales y los cursos secundarios divergentes/ deben ser analizados conjuntamente para encauzar el escurri-/miento, demorarlo, evitar colapso de obras de arte y asegurar el tránsito en las peores condiciones.
- Casi todo lo expuesto en el punto anterior vale también para/ las planicies inundables situadas entre los paleoderrames.

4 - Areas con torrentes de llanura:

- En la cuenca superior se requiere especial atención al control del uso del suelo para evitar erosión y/o aumento de la densidad de avenamiento.
- Se debe estudiar la posibilidad de multiplicar reservorios de cabeceras mediante pequeñas obras de embalse para regulación.
- Habrá que considerar la posibilidad de construir embalses mayores de poca altura en las proximidades del canal de descarga o en éste para disminuir el empuntamiento y servir de paso

a las vías de comunicación evitando las obras de arte.

- Deberá encauzarse el escurrimiento en el cono de deyección mediante canalización o terraplenes laterales de encauzamiento.
- La conducción de excedentes en las planicies terminales deberá hacerse preferentemente por terraplenes semejantes que determinen una especie de canal invertido.
- Ellos originarán recintos que deberán ser estudiados detalladamente para permitir el escurrimiento interno de las aguas / pluviales y su evacuación hacia aquéllas bajo todas las situaciones.
- El diseño de la red vial deberá adecuarse a las obras mencionadas y realizado conjuntamnente con las mismas.

El trabajo es el resultado de las observaciones e investigaciones realizadas en el Centro de Geociencias Aplicadas de la UNNE durante más de 12 años.

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FUNCION DE DISTRIBUCION DE PROBABILIDADES DE CRECIDA A TRAVES DEL METODO GEOMORFOCLIMATICO

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RESUMEN

Un problema de permanente vigencia en hidrología y probablemente el de mayor interés a los fines de diseño de estructuras hidráulicas es el de estimación de crecidas y el de asignación de probabilidades de ocurrencia a las mismas. Es muy frecuente que este tipo de decisiones deban tomarse en regiones con datos escasos, comunes en la mayoría de los sistemas fluviales del mundo, en especial en aquellas áreas rurales de difícil acceso y de baja densidad de población. A Rodriguez-Iturbe et al. le corresponde la gloria de haber desarrollado la teoría del Hidrograma Unitario Instantáneo Geomorfoclimático, el cual constituye la vinculación entre la función respuesta de la cuenca, su geomorfología y el clima. A esta parte medular de la metodología se le ha acoplado un modelo de generación de precipitaciones intensas el cual, sumado al método de infiltración desarrollado por el U.S. Soil Conservation Service permite la generación estocástica de caudales picos. De este modo se logra evitar la errónea y difundida costumbre de asignar la probabilidad de la tormenta de diseño al caudal de diseño.

Palabras clave : Caudales extremos, lluvias intensas, distribución de caudales, geomorfología, hidrología estocástica.

EL MODELO GEOMORFOCLIMATICO

La teoría desarrollada por Rodriguez-Iturbe et al. (1981) en la Universidad Simón Bolívar de Venezuela considera a la función respuesta de una cuenca, hidrograma unitario instantáneo (HUI), como de estructura estocástica y variante con las características de la entrada al sistema hidrológico, la precipitación. El modelo geomorfoclimático, a través de la combinación del clima, representado por la precipitación, y la geomorfología, representada por la estructura de la red de drenaje, permite estimar las funciones de distribución de probabilidades del caudal pico qp y del tiempo al pico tp, del HUI.

Según estableciera Henderson (1963), la forma del HUI no es de importancia a los fines prácticos y es suficiente trabajar con una aproximación triangular. De este modo, la estimación de qp y tp basta para que quede totalmente definido. Tanto qp como tp son variables aleatorias cuyas funciones de distribución de probabilidades dependen de las características geomorfológicas de la cuenca y de la precipitación, especificada esta última a través de la función de distribución de probabilidades de la intensidad y duración de la lluvia efectiva. Las ecuaciones que definen qp y tp según la teoría geomorfoclimática son las siguientes :

$$q_p = \frac{0.871}{\Pi_i^{0.4}} \qquad (1)$$

$$t_p = 0.585\ \Pi_i^{0.4} \qquad (2)$$

donde qp y tp están expresados en hrs^{-1} y hrs respectivamente. Obsérvese que el producto de qp por tp resulta ser constante e igual a 1/2, de lo que se deduce que la relación más probable entre tp y tb, tiempo base del HUI, es 1/4, lo cual es coincidente con lo expuesto por Henderson (1963).

La variable Π_i queda definida como :

$$\Pi_i = \frac{L_\Omega^{2.5}}{i_e\ A_\Omega R_L\ \alpha_\Omega^{1.5}} \qquad (3)$$

donde L_Ω es la longitud en km de la corriente de mayor orden, A_Ω es el área de la cuenca de orden Ω expresada en km^2, ie es la intensidad efectiva de lluvia en cm/h, RL es la relación de longitudes de HORTON, y α_Ω es el parámetro de la onda cinemática definido como :

$$\alpha_\Omega = \frac{S_o^{1/2}}{n_\Omega\ B_\Omega^{2/3}} \qquad (4)$$

siendo S_o la pendiente de fondo, B_Ω el ancho superficial de la sección expresado en m, y n_Ω su coeficiente de rugosidad de Manning.

Henderson (1963) demostró que para un HUI triangular se cumple la relación siguiente :

$$\frac{Qp}{Qe} = \frac{2\ tr}{t_b}\left[1 - \frac{tr}{2tb}\right] \qquad (5)$$

donde Qp es el caudal pico producido por una lluvia efectiva de intensidad constante i_r y de duración t_r, Qe es el caudal de equilibrio $i_r A$ donde A es el área de la cuenca y tb es el tiempo base del HUI triangular o tiempo de concentración de la cuenca.

Partiendo de este enfoque, Rodriguez-Iturbe combinó las expresiones (1),(2) y (5) considerando una lluvia efectiva de intensidad ie y duración te, para obtener la expresión del caudal pico a la salida de la cuenca.

$$Qp = 2.42\ \frac{ie\ A(s)\ te}{\Pi_i\ ^{0.4}}\left[1 - \frac{0.218\ te}{\Pi_i\ ^{0.4}}\right] \qquad (6)$$

donde te está expresado en hrs, Qp en m3/s, A (s) es el área de la cuenca hasta la sección de interés en km2, y las restantes variables en las mismas unidades con las que fueran definidas anteriormente.

Eagleson (1970) obtuvo la expresión del tiempo de concentración de una cuenca simplificada formada por dos planos y un cauce :

$$t_c^{(1)} = \left[\frac{\bar{L}_1\ i_*^{\ 1-m_s}}{\alpha_s}\right]^{1/m_s} \qquad (7)$$

siendo $i_* = \frac{\bar{A}_1}{\bar{L}_1}$ ie, mientras que α_s y m_s representan a los parámetros de la onda cinemática del cauce. Merece destacarse que la ecuación (7) tiene en cuenta solamente el tiempo de viaje por el canal, habiéndose despreciado el tiempo de escurrimiento sobre el terreno. Además la simplificada representación propuesta por Eagleson coincide con el enfoque que la teoría geomorfoclimática hace de una subcuenca promedio de primer orden.

Las ecuaciones (1) y (2) fueron deducidas para una duración de la lluvia superior al tiempo de concentración de la subcuenca promedio de primer orden, ecuación (7), por lo que la ecuación (6) está sujeta a la misma restricción. En la práctica, esta hipótesis no es muy restrictiva ya que las tormentas capaces de producir caudales pico de interés son de duración superior al $tc^{(1)}$, ecuación (7).

La expresión (5) propuesta por Henderson es válida sólo para duraciones de lluvia efectiva menores o iguales que el tiempo de concentración de la cuenca, $tr < tb$, y en consecuencia la ecuación (6) lleva implícita la misma suposición. Según resulta de la teoría geomorfoclimática, el tiempo de concentración resulta ser igual :

$$t_c = t_b = \frac{2}{q_p} = 2.30\ \Pi_i\ ^{0.4} \qquad (8)$$

Vale la pena observar que el tiempo de concentración, lejos de ser una constante, es una función de la inversa de la intensidad efectiva de lluvia.

La ecuación (6), clave en esta nueva metodología, es la que permite obtener los caudales pico Qp asociados a diferentes combinaciones de ie y te.

Las lluvias intensas son la principal fuente de aleatoriedad en la producción de los caudales pico y pueden ser satisfactoriamente simuladas a través de un modelo estocástico de valores extremos. De este modo elegante y sencillo de cálculo a la vez, se evita le equivocada costumbre de asignar los períodos de recurrencia de las tormentas a los caudales pico.

Lo erróneo del criterio habitualmente empleado para la estimación de los caudales de diseño surge a las claras de la expresión (6) pues al resultar Qp una transformación no lineal de la dupla (ir, tr) no es válido realizar primero el análisis estadístico de las precipitaciones intensas para luego transformar a los caudales pico correspondientes.

GENERACION DE PRECIPITACIONES INTENSAS

Existe suficiente evidencia acumulada como para aceptar que la función de *distribución de extremos Tipo I, o de Gumbel, describe adecuadamente al proceso* aleatorio de generación de lluvias intensas. La expresión matemática de la misma es la siguiente :

$$F_X(x) = P(X \leqslant x) = \exp\left(e^{-a(x-u)}\right); \quad -\infty \leqslant x \leqslant \infty \tag{9}$$

siendo a el parámetro de dispersión y u la moda de la distribución. Gumbel demostró que para muestras infinitamente grandes se cumple que :

$$\text{(media)} \quad \bar{x} = u + \frac{\Gamma}{a} \simeq u + \frac{0.577}{a} \quad ; \text{ siendo } \Gamma \text{ la constante de Euler} \tag{10}$$

$$\text{(desvío standard)} \quad \sigma_x = \frac{\pi}{\sqrt{6}\, a} \simeq \frac{1.282}{a} \tag{11}$$

$$\text{(coef. de asimetría)} \quad \gamma_1 = 1.1396$$

Es decir que la función de distribución de Gumbel es de asimetría positiva y constante mientras que sus dos únicos parámetros sólo son función de la media y del desvío standard.

Estos dos estadísticos o, mejor aún, la media $\bar{x}$ y el coeficiente de variación C_V - que es adimensional - pueden ser "ploteados" sobre un mapa de una región extensa, o incluso de un país, para distintas duraciones de lluvias intensas, por ejemplo, 0.5 hr, 1 hr, 3 hrs, 6 hrs, 12 hrs y 24 hrs. Un meditado trazado de isolíneas sobre esa docena de mapas permitirá luego interpolar con confianza, y para cada una de las duraciones de lluvia establecidas, los valores de $\bar{x}$ y C_V correspondientes a un sitio donde no existan registros pluviográficos analizados o que parezcan ser poco confiables por su calidad o escasa longitud de récord.

Si se despeja x de la ecuación (9) mientras u y a se expresan en función de $\bar{x}$ y σx resulta :

$$x = \bar{x} \left(1 - \frac{0.577}{1.282} C_V - \frac{1}{1.282} C_V \ln(-\ln P)\right) \tag{12}$$

En la práctica las muestras son de tamaño finito, por lo que las ecuaciones (10), (11) y como consecuencia la (12) no son estrictamente aplicables.

Existen varios métodos para determinar los valores de a y u. El propuesto por Gumbel se basa en el análisis de mínimos cuadrados de la variable reducida $y = a (x - u)$. Las ecuaciones resultantes que minimizan la suma de los cuadrados de los desvíos perpendiculares a la recta de los valores extremos $x = \frac{1}{a} y + u$ son :

$$u = \bar{x} - \sigma_x \frac{\bar{y}n}{n} \tag{13}$$

$$a = \frac{\sigma n}{\sigma x} \tag{14}$$

Los valores esperados de la media y del desvío standard de la variable reducida, $\bar{y}n$ y σn, sólo son función del tamaño de la muestra y se encuentran tabulados. Es así que la ecuación (12) utilizada para simular aleatoriamente la generación de lluvias intensas se transforma por ejemplo para el caso de muestras de n = 20 años en :

$$x = \bar{x} \left(1 - \frac{0.52}{1.06} C_V - \frac{1}{1.06} C_V \ln (- \ln P) \right) \tag{15}$$

Un algoritmo de generación de números pseudo aleatorios con distribución uniforme en el intervalo 0 - 1, permite concretar la generación de números aleatorios distribuidos según una ley de Gumbel. La misma técnica de generación aleatoria adoptada asegura que no exista correlación serial entre valores de años sucesivos y que se preserven los estadísticos estimados para cada duración. Se logra así generar una matriz precipitación - duración de correlación serial nula entre elementos sucesivos y correlación cruzada nula entre sus vectores. Para conseguir que la misma luzca semejante a las matrices de datos observados, se desarrolló un algoritmo de consistencia que merced al intercambio de elementos por columna consigue la consistencia por filas.

Los estadísticos de las funciones de distribución que describen el proceso aleatorio de generación de lluvias intensas de diferentes duraciones, corresponden a observaciones puntuales. Las técnicas de abatimiento de la precipitación permiten estimar la precipitación media areal que tenga la misma probabilidad de ocurrencia que la lluvia puntual en un punto cualquiera del área. Está implícita, en este planteo, la hipótesis de isotropía o sea que todos los puntos del área posean la misma función de distribución de probabilidades de lluvia.

Si bien existen sobre el tema del abatimiento de la precipitación enfoques bastante sofisticados, se prefirió en este estudio adoptar el propuesto por el U.S. Weather Bureau (1963), resultando el coeficiente de abatimiento sólo función del área y de la duración de la lluvia. La expresión analítica de las curvas cantidad - área - duración del U.S.W.B. resultó del tipo :

$$COEFAB = \eta A^{-\beta} \tag{16}$$

siendo η y β coeficientes función de la duración de la lluvia.

ESTIMACION DE LA LLUVIA EFECTIVA

Decidir qué parte del volumen de agua precipitado escurrirá en forma directa es quizás tan importante como la elección misma de la función respuesta de la cuenca. Ambas decisiones son cruciales por la influencia que tienen sobre el caudal pico.

Afortunadamente la teoría de Rodriguez-Iturbe constituye un sólido fundamento teórico que garantiza una acertada elección del HUI, pero falta aún en Hidrología una teoría de infiltración que, a escala de cuenca, sirva para eliminar los interrogantes a la hora de separar cuánto infiltra de cuánto escurre.

Basándose en la solución de Philip de la ecuación unidimensional del proceso de difusión en medio no saturado, Eagleson (1978) dedujo las ecuaciones que relacionan la lluvia efectiva con la intensidad y duración de la lluvia total, con el contenido inicial de humedad del suelo y con un grupo de siete parámetros que representan las características del mismo. Probablemente el planteo de Eagleson sea hasta el presente uno de los mejor fundamentados, sin embargo, el depender de un número tan elevado de parámetros que habitualmente se desconocen lo hace poco aplicable a los fines del proyecto de estructuras hidráulicas. Por este motivo, y no por causas teóricas, fue que se adoptara como método de estimación de la precipitación efectiva al desarrollado por el U.S. Soil Conservation Service, (1968). Sobre este método existe una larga experiencia de aplicación en todo el mundo y presenta la interesante ventaja, según estudios realizados recientemente en EE.UU., de que la clasificación hidráulica de suelos propuesta por el S.C.S. puede ser realizada a través del procesamiento digital de imágenes LANDSAT.

De acuerdo al S.C.S., la precipitación efectiva se puede calcular a través de la ecuación :

$$R = \frac{(P - 0.2S)^2}{P + 0.8\,S} \qquad (17)$$

donde P es la precipitación y S es el único parámetro que define al suelo, estimado a través del "número de curva" NC. El parámetro S, la infiltración potencial expresada en pulgadas, queda definido por :

$$S = \frac{1000}{NC} - 10 \qquad (18)$$

El "número de curva" NC, valor que se extrae de tablas, depende del tipo de suelo, de su cobertura vegetal, del uso al que está sometido el suelo y de la condición de precipitación antecedente. La infiltración acumulada resulta :

$$F = \frac{(P - 0.2\,S)\,S}{P + 0.8\,S} \qquad (19)$$

Obsérvese que cuando $P \to \infty$, $F \to S$, que es la infiltración potencial. Admitiendo un hietograma rectangular de ordenada i (mm/hr) :

$$F = \frac{(i.t - 0.2S)\,S}{i.t + 0.8\,S} \qquad (20)$$

En consecuencia la tasa de infiltración resulta :

$$f = \frac{dF}{dt} = \frac{S^2\ i}{(it+0.8\ S)^2} \qquad (21)$$

Interesante es observar que la tasa de infiltración inicial, según el método del S.C.S., depende de la intensidad de lluvia pero es independiente del tipo de suelo.

$$fo = \left.\frac{dF}{dt}\right|_{to} = i \qquad (22)$$

El escurrimiento superficial se inicia recién cuando f = i, lo cual ocurre en el instante :

$$to = \frac{0.2\ S}{i} \qquad (23)$$

Por lo tanto, si tr es el tiempo total de lluvia, el tiempo efectivo de escurrimiento será :

$$te = tr - to \qquad (24)$$

y la intensidad de lluvia efectiva promedio resultará :

$$ie = \frac{R}{te} \qquad (25)$$

Al simular con la ecuación (21) la evolución de la tasa de infiltración para distintas intensidades e infiltraciones potenciales, surgió como fundamental el papel que le cabe a la precipitación antecedente.

Los "números de curva" tabulados por el S.C.S. corresponden a la condición antecedente II, o sea aquella que en promedio precede a la ocurrencia de la máxima crecida anual en numerosas cuencas. Siguiendo esta premisa, y para evitar establecer una única condición antecedente para todas las tormentas, es decir, fijar un CN y en consecuencia un S, se supuso que la infiltración potencial era una variable aleatoria de media $\bar{S}$ y desvío standard $\sigma_S = (\bar{S} - Smin)/4$. De este modo se consigue generar valores de S simétrica y estrechamente distribuidos en torno a $\bar{S}$, asegurando así que el 99.99 % de los mismos se encuentre dentro del intervalo $(2\bar{S} - Smin)$ y Smin. La expresión utilizada para la generación es entonces :

$$S = \bar{S} + \sigma_S . u = \bar{S} + \left[(\bar{S} - Smin)/4\right] . \nu \qquad (26)$$

siendo ν una variable aleatoria $\mathcal{N}(0-1)$. El algoritmo empleado para la generación de números pseudoaleatorios de distribución normal fue el de Box y Müller.

EJEMPLO DE APLICACION

El río Matanza presenta inundaciones en su valle inferior que ocasionan e-- normes perjuicios a las poblaciones, industrias y sectores agropecuarios de los Partidos que rodean por el sur y el suroeste a la ciudad de Buenos / / Aires. El río Matanza desemboca en el río de la Plata drenando una cuenca de 2100 km2 con un ancho máximo de 40 km y una longitud de más de 70 km a lo largo del cauce mayor. El relieve es sumamente plano, siendo 38 y 3 me-- tros las cotas máxima y mínima respectivamente. El río tiene un desarrollo de sudoeste a noroeste y en su parte media e inferior divaga por numerosos meandros. La precipitación media anual es de 1000 mm y las tormentas gene- ralizadas son típicamente ciclónicas.

En 1960 la Dirección de Hidráulica de la Provincia de Buenos Aires decidió densificar las redes hidrológicas de observación - 8 pluviómetros, 8 plu-- viógrafos y 3 estaciones de aforo - e iniciar una serie de estudios que cul minaron con el informe de Barbero (1973).

Toda la información básica relativa a la cuenca del río Matanza utilizada en el presente estudio ha sido extraída del citado informe, mientras que los parámetros estadísticos de las lluvias intensas de la Capital Federal (1919-1972) corresponden al estudio de Medina y Moyano (1975). La cuenca usada como ejemplo aquí tiene por cierre la estación de aforo Autopista Teniente Gral. Richieri.

Parámetros geomorfológicos

$A_{(s)} = 1830$ km2 ; $A_{\Omega} = 2100$ km2 ; $L_{\Omega} = 34$ km ; $R_L = 2$

Parámetros de la onda cinemática

$So = .000187$; $n_{\Omega} = .037$; $B_{\Omega} = 200$ m.

Parámetros de infiltración

De la ecuación (17) se deduce que :

$$0.04\ S^2 - (0.8R + 0.4P)\ S + P\ (P - R) = 0 \qquad (27)$$

Conocidos P y R, S se obtiene como solución factible de la ecuación de 2do. grado (27), mientras que CN puede despejarse de la ecuación (18). Del análi sis de crecidas observadas se obtuvo :

Fecha de la crecida	P (mm)	R (mm)	S (in)	CN (-)
6/8/64	69.2	12.9	4.60	68.5
28-29/8/64	100.2	28.8	4.84	67.4
26/8/67	62.5	11.5	4.20	70.4
8-10/10/67	202	122	3.64	73.3

El CN promedio resultante de estos análisis es 69.9, valor muy cercano al finalmente adoptado, CN = 71, extraído de la tabla del S.C.S. para condición antecedente II, suelo tipo C y pastizal natural con buena cobertura. Como infiltración potencial mínima se adoptó la correspondiente a CN = 74.

Parámetros de las precipitaciones intensas

	1 hr	3 hr	6 hr	12 hr	24 hr
$\bar{X}$ (mm)	36.0	51.5	63.4	75.5	80.8
C_V	0.34	0.30	0.27	0.30	0.29

Resultados numéricos

Los cálculos se realizaron mediante una computadora personal HP-86B de 128 Kb y un programa de computación escrito en Basic. Una vez ingresada desde teclado la información básica geomorfológica, hidráulica, de suelos y pluviográfica, se generaron 10 trazas de 40 años. A cada serie se le ajustó una función de distribución de Gumbel estimándose así los caudales pico para recurrencias entre 2 y 1000 años.

Tr \ Traza	I	II	III	IV	V	VI	VII	VIII	IX	X
2	235	259	219	243	249	289	199	236	225	194
5	520	482	474	567	555	654	494	485	506	421
10	708	630	643	782	757	895	689	649	692	571
25	946	817	857	1054	1013	1201	936	858	927	716
50	1123	956	1015	1255	1203	1427	1119	1012	1101	902
100	1299	1093	1173	1455	1392	1652	1300	1165	1274	1042
1000	1878	1548	1692	2116	2015	2395	1901	1672	1846	1504

También se presentan a modo de comparación los caudales pico estimados en el informe Barbero (1973) sobre caudales calculados y observados durante 38 años (1931 - 1968), a los cuales le han sido calculados sus intervalos de confianza o curvas de control.

Tr (años)	L95 %	L90 %	L80 %	L68 %	Qp (m^3/s)	U68 %	U80 %	U90 %	U95 %
2	193	206	221	232	274	315	327	342	355
5	404	427	454	474	548	622	642	669	692
10	531	563	600	629	730	831	860	897	929
25	688	732	782	821	959	1097	1136	1186	1230
50	803	855	916	963	1129	1295	1342	1403	1455
100	917	978	1049	1104	1298	1492	1547	1618	1679
1000	1294	1384	1488	1569	1856	2143	2224	2328	2418

Del análisis conjunto de ambas tablas de valores surge que los caudales pico generados con el modelo geomorfoclimático pertenecen en proporción adecuada al intervalo limitado por las curvas de control, superiores U e inferiores L, correspondientes a los umbrales de confianza establecidos.

CONCLUSIONES

En este trabajo se ha pretendido hacer una propuesta metodológica completa en el campo de la estimación de crecidas de diseño en áreas con datos escasos demostrando que :

a) es factible la caracterización estadística de las tormentas intensas a través de unos pocos parámetros, los cuales pueden ser volcados en un mapa para facilitar su estimación.

b) se debe impulsar la caracterización hidrológica de los suelos de una región o país a través de imágenes satelitarias, como ha sido probado recientemente con imágenes LANDSAT.

c) es conveniente adoptar el HUIG de Rodriguez-Iturbe como función de transferencia entre precipitaciones y caudales.

La aceptación de estos tres puntos implica un avance tecnológico de importancia que significa saltar del semiempirismo a una metodología con un sólido fundamento teórico, y lo que es de sumo interés, de aplicación universal.

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EVALUATION OF THE HEALTH AND SOCIAL IMPACT OF A WATER AND SANITATION PROJECT IN MALAWI

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Malawi

ABSTRACT

The paper reviews an evaluation study of a rural gravity fed water project in Malawi. The evaluation is carried out in three separate areas both before and after the intervention with water, sanitation and hygiene education, which will take place in 1984. Two of the three areas will get water in 1984 while the third area acts as a control. Of the first two areas, one gets a health education and sanitation programme by the Ministry of Health. When evaluating the impact upon health, the indicators chosen are diarrhoeal disease, skin- and eye-infection and nutritional status in 800 children under five years. Morbidity data are collected through fortnightly home-visits with 24 hours recall of symptoms. All children in the study are weighed and measured twice a year and those born after 83-01-01 are measured every second month. On a sample of children a food intake survey is done. Also, a study of the etiology of diarrhoeal disease is carried out. Bacteriological water quality analysis is done of the water sources and of water stored in the households both before and after intervention. It is necessary to complement quantitative data with qualitative, like observation and in depth interviews, esp. regarding personal and environmental hygiene to attain an insight into the relationships between human behaviour, water, hygiene and routes of transmission of diseases. This study will give detailed information about the health and social impact of water and sanitation projects. Although the results are only for the study area, it will hopefully be possible to draw conclusions from the experience of this study for other projects about the design, the methods and the indicators to be used and also to create a model for such evaluation.

1. Background

Provision of water supply for household consumption is among the crucial components in order to improve living conditions in Third World countries. Apart from tedious work connected with the drawing of water from far away sources, the situation is characterized by human sufferings in terms of poor health. At the same time it should be remembered that improvement of water supply in villages may change the social and cultural networks of the villages. Moreover, the very location of the water source within the village may bring water in close contact with pollutants from humans and animals.

Whether improvements in water supplies will give any impact upon the health status of the recipients depends on their potential capacity to control water-related diseases. This capacity is in turn related to the quality of sanitary facilities, personal and environmental hygiene as well as the social, cultural and economic conditions of the population, apart from the water-related aspects. The water-related diseases have been divided into four main categories: water borne, water-washed, water-based and spread by water dependent vectors (Bradley, 1971). Of these four categories, the first two are more directly related to poor and scanty water supply than the last two. The control of these diseases depends both on water-quality and water-quantity. Water-washed diseases may be divided into two sub-groups: infections, which affect eyes and body surface, and diarrhoeal diseases, which are the most important of the water washed diseases, as they constitute a leading cause of child morbidity and mortality.

The most important benefit anticipated when improving water supply is better health. However, it has been maintained that the recent popularity for support to rural water projects among donors and recipient governments is due to the increasing general interest in rural development. Thus, "for the aid donors - national, international and voluntary agencies - rural water supplies are visible evidence that their money reach the rural poor" contrary to much of capital investment and technical assistance (Feachem et al, 1978). Further, "the current enthusiasm has led to a certain amount of wishful thinking about the benefits of villages water supplies, to the extent that rural water supply is sometimes seen as promoting development on its own and not as part of an integrated rural development programme" and "although water supplies are probably a necessary condition for achieving improvements in the rural economy and the public health, they are by no means a sufficient condition" (Feachem et al, 1978).

Therefore, what are the minimum requirements to achieve better health? It is now widely acknowledged that not only water, but also sanitation and hygiene as well as an active involvement of the inhabitants concerned, are important in order to reduce water-related diseases (e.g. Falkenmark, 1982). In case complete water and sanitation facilities were installed, i. e. a water-tap and a flush-toilet for every household, this aim would certainly be met. However, the available resources will only allow limited improvements. Relatively few studies (in Hughes's review (1980) four studies in Africa) have evaluated the impact of improved water supplies upon health with studies both before and after intervention.

Of the 43 studies reviewed by Hughes (1980), several have shown that improved water supplies have some effect on diarrhoeal disease. Hughes suggests that "previous attitudes concerning the lack of documentation of health benefits associated with water and excreta disposal projects may be unnecessarily pessimistic". Young children are those who benefit most. When improved water is combined with improvements in hygiene and sanitation, reductions of 20 to 40 % in diarrhoeal morbidity have been noted (McJunkin, 1982). Often the effect of increasing water availability may be more pronounced than the effect of improving water quality (Falkenmark, 1982).

The minimum amount of water for daily consumption, which is required to prevent illness, is not known. Hughes notes that data from the several studies included in his review "suggest that volumes in the range of 20 - 30 litres per capita and day me be a minimum required to yield reductions in diarrhoeal disease morbidity (Hughes, 1980). McJunkin on the other hand concludes that "50 litres per capita per day should be a minimum goal" (McJunkin, 1982). It is obvious that further evidence is required before any conlusive answer can be given.

Where water of good quality is provided it is often contaminated during collection and storage. A good knowledge about the relationship between water, sanitation, hygiene and health, attained by a sanitation programme and hygiene education, is probably crucial to reduce water related diseases.
The relationships of the factors which influence the morbidity of water-related diseases are complex to determine. Blum and Feachem (1983) identifies eight methodological problems which have limited the value of most of the studies of the impact of water and sanitation projects, including thoses reviewed by Hughes (1980) and McJunkin (1982).

2. Purpose of the study

This research project investigates the importance of household water supply in relation to the social networks of villages in developing countries. It evaluates the impact of improved household water supply in terms of both immediate benefits like better quality of water and shorter distance to water supply, as well as more indirect effects such as improved health and social conditions, such as less time spent to draw water. Fieldwork is done in Malawi between 1982 and 1985.

The purpose of the study is threefold. First, it is to test a number of hypotheses concerning the relationship between various factors, which may influence to what extent there will be an impact on the population. Fig. 1 shows the factors which are believed to be of importance in order to make water projects successful. The relationships which are tested are also indicated with arrows. The testing of these hypotheses is done with statistical analysis (cross-tabulation, correlation- and regression-analysis) in order to determine to what degree the factors are related to health and social conditions.

A second purpose of the present study is to study conditions which are not easily quantifiable but may be better studied with qualitative methods, i. e. by observation and intensive, unstructured interviews and "life-stories". This is the case with among other things the ways people collect, handle and store water up to the time when it is used. This part focuses especially on observing where, how and why water may be polluted between "tap and throat" and also the attitude of people to different types of water-sources, to the water-quality etc. Also, the wide-spread habit of giving water to infants from the first days of life is studied as well its consequences.

A third purpose is to study the etiology of gastroenteritis in children under five years in a rural area in Malawi. This is done during a wet and a dry season both before and after intervention. When the relative importance of different etiological agents has been established, this information, together with what is already known about each agent, will assist in determining possible routes of transmission and in what way and to what extent this transmission is water-related. This will aid in the formulation of health strategies.

3. Methodology of the study

The study is carried out as a 'before and after study' in three areas, consisting of 3, 4 and 5 villages respectively. There are around 150, 170 and 220 households with children under 5 years in each of these areas (the study group). All three were selected to be as similar as possible before the intervention with regard to water supply, environmental location, social structure and educational level, economy and income structure, food production and health services.

The three areas are studied during a wet and a dry season both before intervention with improved water supply (January 1983 - March 1984) and after (June 1984 - May 1985), see fig. 2. In the intervention villages a sanitation promotion programme and hygiene education about the relationship between water and health is provided apart from the improvement in water-supply, in which the population is actively involved in order to be motivated to maintain the water-supply system properly. The other two areas act as control areas. One of them, Area 2, gets improved water supply but no hygiene education or sanitation promotion, while the last area, Area 3, will not have any of these changes until 1985.

There are numerous methodological obstacles when carrying out a survey like this. Thus, a World Bank report writes that "it is extremely difficult to identify and measure exactly the health effects of improved water supply, and there is a limit to the precision attainable" and concludes that long-term longitudinal studies of large size and expense are the only means by which it may be possible to determine a relationship between water and health.

4. Target population, study populatioon and study group

The target population of this study is the Zomba West Rural Piped Water Project in the Shire Valley between Zomba-Malosa mountains and the Shire river. The project provides improved piped water-supply to around 47 000 inhabitants.

Within this target population, the study population for the detailed investigation of the health and social impact of the water-project and the health education project was selected so as to be as similar as possible with regard to physical, environmental, cultural, social and economic conditions. All villages in the area selected were given random numbers. Three starting villages were selected randomly. Then other villages were selected randomly around these three villages, so that altogether 12 villages with around 4 140 inhabitants were included in the study population.

The study population consists of all households and all inhabitants within the three areas selected. During the first two weeks of January 1983 a Village Census of the study population was carried out. The results revealed that the age- and sex-distribution was fairly similar in the three areas (Lindskog and Lindskog, 1983).

The percentage of population in the main ethnic groups were roughly the same for the three areas, although the yaos were a bit fewer in Area 3. This are is also slightly less traditional than the rest. Before the present water project there are no protected water sources within the study population. People use unprotected hand-dug wells, and also rivers and some few springs.

Among the 1 126 households in the study population, those were selected which according to the Village Census had at least one child under 5 years on the 1st of January 1983. This gave a total of 542 households with children under 5 years. These constitute the study group, for which the improvements in household water supply, sanitation and hygiene upon health and social conditions are evaluated. A Household Survey was carried out during the last two weeks of January 1983 which established the cultural, social, economic and environmental conditions of the study population (Lindskog & Lindskog, 1983b). This Household Survey showed that the three areas are fairly similar, especially the age and sex distribution and the level of education.

The total number of children studied in the three areas are around 800 with some fluctuations over time due to movements out from and into the area temporarily or permanently and due to deaths and births in the households originally selected. No new households are included during the study, as this would make the study group much more unstable.

The results below are only for the baseline study, the year before intervention, as the after intervention study is between June 1984 and May 1985. Data for the whole study will be presented at the Congress in Brussels.

5. Results

Between 30th January and 23rd December, 1983, the prevalence of diarrhoea, respiratory tract infections and skin- and eye-infections was investigated through fortnightly interviews in all households of the study group. The survey was done in five periods: season 1 = February - March; season 2 = April - May; season 3 = June - July; season 4 = September - October; season 5 = November - December. Season 1 is warm and rainy, while season 2, 3 and 4 are all dry. It gets cooler during season 2, while season 3 is the coolest of the year. It then gets warmer during season 4. The rains then start during season 5, which is also the warmest time of the year.

Diarrhoeal disease is defined as four or more loose stools or one watery or bloody stool during the 24 hours preceding the interview. Respiratory tract infection includes either runny nose, cough, ear pain and/or ear discharge. Skin-infections are defined as bacterial and fungal infection and scabies.

Eye-infection includes mainly conjuntivitis. The classification is done according to the mother's report of the symptoms which influences the accuracy of the diagnosis although the enumerators check whether the information is plausible by looking at the child.

In all three areas a marked seasonality was found in diarrhoeal disease and skin- and eye-infection. In Area 1 all figures were somewhat lower than in the other two areas. Respiratory tract infection was roughly the same during all the seasons.

5.1 Relationships of morbidity and social and environmental factors

Despite all the different pathogens involved in the transmission of diarrhoeal diseases, they are primarily spread in one way, the fecal-oral route. Different ways of living, such as social, economic and environmental conditions, may influence the transmission of disease. In the present study the prevalence of diarrhoeal disease is related to different household variables. As noted earlier, the results presented here are only for the year before intervention.

A relationship was found between diarrhoea during season 1 and distance from the home to the main water source, when tested with analysis of variance. Children in households which have a longer distance to the water have a higher prevalence of diarrhoea. This was found when children of all areas were analysed together and also when the three areas were analysed separately. For children under three years the same relationship was found. This might be due to variations in amount of water used for households at different distances from the water-source. Unfortunately analysis of data on the amounts of water used has not yet been completed. There is also a strong relationship between skin-infections and distance to water source, so that those having a longer distance to the water have a higher frequency of skin-infections. This was found during all the three seasons. There was no correlation between eye-infections and distance to water source. The relationship between social factors and diseases was generally not very strong. There was a slight tendency for lower prevalence of diarrhoea in those cases where the head of household had some education. This relationship was also found for the education of mothers of children under five years, but none of them was at a significant level. In some of the areas there was a relationship between religion and diarrhoea and between the availability of latrine and diarrhoea. However, it is necessary to do further analysis before any casual relationships can be established.

These preliminary results indicate that there are not one or two, not even a few, variables which explain the relationship between social and environmental factors and morbidity in water-related diseases but the relationship is much more complex with a multitude of factors involved.

5.2 Bacteriological water quality analysis

To be able to draw any conclusions of the impact or lack of impact of improved water supply it is necessary to know the quality of the water. Therefore, during the baseline study examinations of the water have been done to determine the traditional water situation. In September, 1983, which is during the dry and hot season, 168 bacteriological water analyses were done. 106 samples were taken from traditional water sources and 62 samples from water stored in households. The types of water sources are unprotected, hand dug wells (58 samples), rivers (19 samples) and springs (25 samples).

The samples were analysed for indicator bacterias, i.e. total coliforms, fecal coliforms and fecal streptococci with the membrane filtration method. All samples were cultured within 10 hours after collection, most of them within 6 hours. They were cultured for total coliforms on LES endomedium and incubated for 24 hours at 37°C, for fecal coliforms on MFC medium and incubated for 24 hours at 44.5°C and for fecal streptococci on m Enterococcus Selective Agar for 48 hours at 37°C (Geldreich, 1975; Stenström & de Jong, 1982).

The mean value for all water sources for total coliforms was 1930 per 100 ml, for fecal coliforms 820 per 100 ml and for fecal streptococci 470 per 100 ml. Rivers had somewhat worse quality than wells, while springs tended to be better. However, in the whole area there are not so many springs in use. The values are roughly similar to those found in other parts of Malawi through analyses carried out by the water laboratory of the Department of Lands, Valuation and Water (J. Lewis, personal communication, 1983).

Water for household consumption is usually carried from the source to the house in a metal bucket containing 15-25 litres. To prevent the water to splash over the edge of the bucket a plate or some leaves are put on the water surface. At the house the water sometimes is kept in the bucket until it is used but more commonly it is stored in a clay pot. This may be covered or uncovered. It can be put inside or outside the house. Often separate containers are used for drinking water and for water for other purposes. Often people prefer to keep the drinking water inside the house, as it will then keep cooler than if stored outside, at least if the roof is thatched. Also, washing clothes, cooking utensils and bathing (especially women and children) is done at the river. These habits affect the amount of water used at the house.

62 samples were taken from the containers of drinking water in the households. The mean values were about twice as high as the values of the water sources for both total and fecal coliforms, while fecal streptococci were 3.5 times higher in the household samples. 5 household samples had a total coliform count of 20 000/100 ml more than the sample from the water source used and a fecal coliform count between 1 000 and 20 000/100 ml more. 21 samples had a lower bacterial count than the water source, while the storing time of the water was roughly the same as those samples with higher values.

During December 1983 - January 1984, the peak of the rainy season, 117 bacteriological water analyses were done of traditional water sources, while no household samples were taken as there was a limit of the number of samples which could be taken and it was regarded as essential to have a sufficiently large sample of the traditional water sources. The water quality was generally much worse in December - January. The mean value of fecal streptococci was 10 times higher (the median 4 times) than the dry September when analysed for all sources. The mean of total coliforms increased 7.3 times (the median 4.5 times) while the increase was less for fecal coliforms (5.4 and 1.7 times respectively). Feachem (1974) reached similar results. - Of the three types of water sources, the increase was much higher for wells than for rivers and springs when using both total and fecal coliforms as indicators. However, fecal streptococci increased more in rivers than in wells and springs. this was from a much lower level for both rivers and springs than for wells. The size of the sample from rivers was small, only 9. This is due to the fact that contrary to what many "outsiders" expect, people draw less water from rivers during the rains as they then contain a lot of soil. Women therefore walk further to wells and springs to get water during the rains (this observation is supported by the experience of Dr. J. Kalilani, personal communication, 1983). It these results hold for further analysis, they have implications in areas where there is no protected water.

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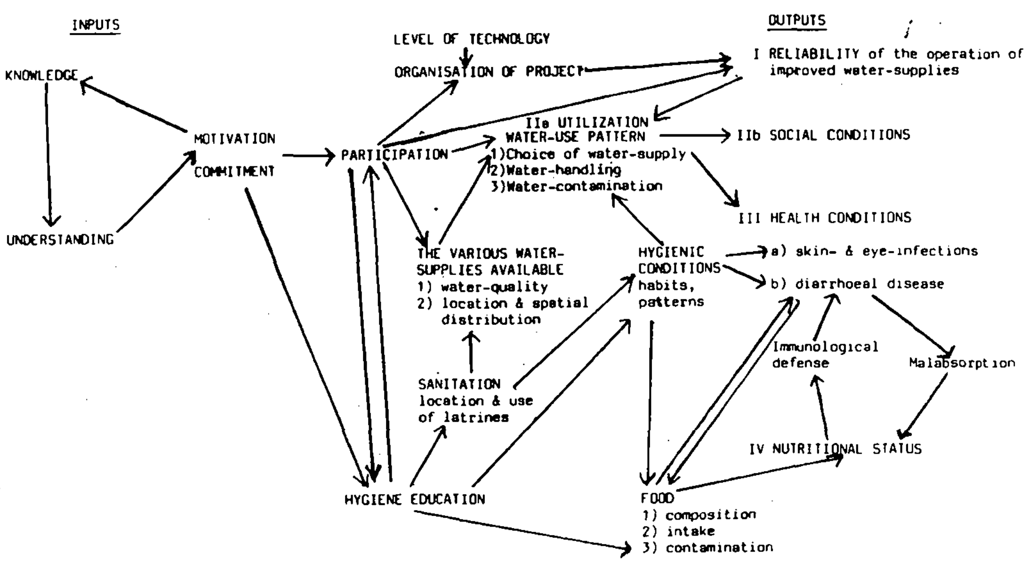
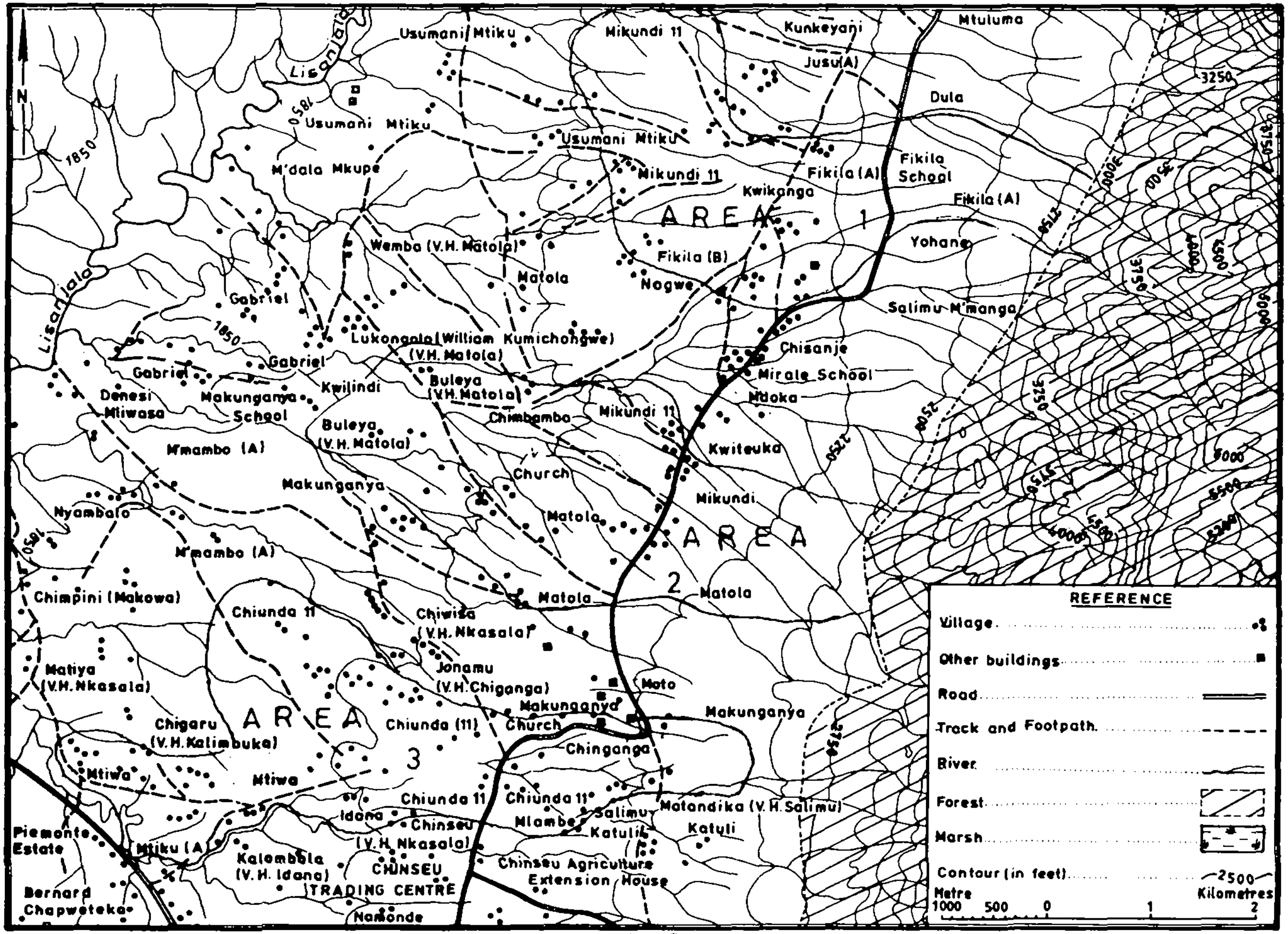
Table 1. Water quality in the project area (coliforms per 100 ml)

Type of water source and season	Size of sample	Total coliforms Mean	Total coliforms Median	Fecal coliforms Mean	Fecal coliforms Median	Fecal streptococci Mean	Fecal streptococci Median
Dry season, all	106	1 930	670	820	230	470	170
Wet season, all	117	14 060	3 000	4 400	380	4 690	700
Dry season, wells	53	2 140	820	950	225	690	230
Wet season, wells	58	24 820	5 000	7 760	1 520	7 820	1 270
Dry season, rivers	19	2 600	1 300	1 140	850	270	100
Wet season, rivers	9	5 230	5 200	1 490	1 300	3 640	1 500
Dry season, springs	25	1 080	240	310	60	150	80
Wet season, springs	36	3 140	835	360	100	1 480	260
Dry season, households	62	3 940		1 800		1 700	

FIG. 1 THE RELATIONSHIP BETWEEN VARIOUS FACTORS WHICH MAY BE IMPORTANT IN ORDER TO MAKE WATER PROGRAMMES SUCCESSFUL

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AREA 1
Yohane
Wemba (V.H. Matola)
Fikila (B)
Matola
Nogwe
Gabriel
Salimu M'manga
Lukongolo (William Kumichongwe)
(V.H. Matola)
Chisanje
Gabriel
Gabriel
Mirale School
Kwilindi
Buleya
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Ndoka
Denesi
Miwasa
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Nyambalo
Matola
AREA 2
M'mambo (A)
Matola
Matola
Chimpini (Makowa)
Chiunda 11
Chiwisa
(V.H. Nkasala)
Matiya
(V.H. Nkasala)
Jonamu
(V.H. Chiganga)
Moto
Makunganya
Chigaru
(V.H. Kalimbuka)
AREA 3
Chiunda (11)
Church
Makunganya
Chinganga
Mtiwa
Mtiwa
Chiunda 11
Chiunda 11
Salimu
Matandika (V.H. Salimu)
Idana
Chinseu
(V.H. Nkasala)
Mlambe
Katuli
Katuli
Piemonte Estate
Mtiku (A)
Kalembela
(V.H. Idana)
CHINSEU
TRADING CENTRE
Chinseu Agriculture Extension House
Bernard
Chapweteka
Namonde
REFERENCE
Village
Other buildings
Road
Track and Footpath
River
Forest
Marsh
Contour (in feet)
2500
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Kilometres

FIG. 2

IWRA Vth WORLD CONGRESS ON WATER RESOURCES
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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 106 Aspect number 6

SURFACE WATER QUANTITY-QUALITY RELATIONSHIPS AND DISSOLVED LOADS IN A RURAL CATCHMENT AREA IN FLANDERS, BELGIUM

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ABSTRACT

In the planning stage of management works for surface water resources it is often required to consider water quality in relation to water quantity to decide on the feasability of a project.

The paper describes a study of the Zwalm river in Belgium, in which the variation of the concentrations of inorganic chemical constituents in the river water with river flow is considered. A mathematical dilution model is presented, relating concentrations of the major ions to river discharge, antecedent flow conditions and time of the year.

It is shown that the response to discharge fluctuations differs considerably for the different solute parameters at the particular measuring station and mean yearly dissolved loads are calculated from the model. The model is able to simulate the observed cyclical pattern of changes of concentrations versus discharge.

Keywords : surface water quality, water quantity-quality relationships, inorganic dissolved load, water quality simulation models.

INTRODUCTION

In the assessment of water resources it is often required to consider water quality parameters. The total pollutional load of a water body is composed of contributions from three sources : direct or 'point' discharges of wastewaters; diffuse or 'non-point' source contributions in runoff and seepage waters; and natural or 'background' source contributions. To a more or less extent, these contributions are influenced by man's activities and, especially in rural areas, point and non-point source pollution require control and management if water quality is to be maintained or improved.
For the evaluation of non-point source pollution of river waters the assessment of concentrations and loads (fluxes or mass-discharges) of solutes is often essential. In most planning studies (having limited time and money budgets) the flow information needed is usually available but sufficient concentration data are frequently lacking : the most common situation is that water flow data are available on a daily or hourly basis, whereas concentration measurements are only performed periodically.
This paper reports on a study of the Zwalm river, Belgium, in which the concentrations of the major inorganic chemical constituents in the river water are modelled as a function of water discharge, antecedent flow conditions and time of the year.

GENERAL METHODS FOR INORGANIC LOAD COMPUTATION

The general expression of the load L of a dissolved chemical constituent, flowing across a cross-sectional area A of a river in a time period T can be written :

$$L = \int_0^T dt \iint_A C(x,y,t) \,.\, v(x,y,t)\, dx\, dy \tag{1}$$

in which x,y are cross-sectional coordinates, t is time, v is water velocity and C is the concentration of the inorganic constituent.
Assuming C to be uniformly distributed in the cross-section, Eq. 1 reduces to :

$$L = \int_0^T C(t) \,.\, Q(t)\, dt \tag{2}$$

with the water discharge : $Q(t) = \iint_A v(x,y,t)\, dx\, dy$ (3)

If daily means values of water flow and concentrations are available, the yearly load can be computed by an approximation of Eq. 2 :

$$L = \sum_{i=1}^{365} C_i\, Q_i \tag{4}$$

In practice however, concentration data are often lacking on a daily basis, since water samples are usually analysed periodically, while daily flow data are mostly available. For the computation of the lacking daily concentration data, two methods can generally be used :
(a) interpollation between the measured concentrations, using different methods. The influence of the choice of different interpollation methods on the results obtained for the total inorganic load is described by Weber et al. (1979). These methods use the additional information contained in the flow data only in a very restricted manner.
(b) departing from the available measurements of concentration and discharge on the same day, a mathematical model is built, relating the concentrations C_i to the discharges Q_i ("rating curve" technique) :

$$C_i = F(Q_i) \tag{5}$$

by which Eq. 4 reduces to :

$$L = \sum_{i=1}^{365} Q_i \cdot F(Q_i) \qquad (6)$$

from which the yearly load can be computed.

GENERAL MODELS FOR THE CONCENTRATION-DISCHARGE RELATIONSHIP

Based on theoretical backgrounds, Hall (1970;1971) and Balland et al.(1979) proposed different expressions for the concentration-discharge relationship (Eq. 5). These 'mixing model'-relationships may be classified into three main types :

$$\text{type I} \quad : C = A \cdot Q^B \qquad (7)$$

$$\text{type II} \quad : C = A + B \log Q \qquad (8)$$

$$\text{type III} : C = \frac{A}{1 + B\,Q^D} \qquad (9)$$

The question which of these models represent best the concentration-discharge relationship in a particular river is very difficult to answer. Analysing data obtained from different river basins, Hall (1971) concluded that in most cases two or more models could fit the data equally well. Considering the inaccuracy of both concentration and discharge measurements, it is unlikely that - on the basis of the measurement data only - one particular type of model would emerge as being much more performing than another one. Therefore, it seems that the choice of the model type can better be made considering all available information about the river and its catchment, while suitable values or expressions of the model parameters can be found by fitting the model to the measurement data.
The model described here for the Zwalm river is an example of this procedure and it is based on a concentration-discharge model of type I.

CONCENTRATION-DISCHARGE MODEL FOR THE ZWALM RIVER

The Zwalm river is a tributary of the river Scheldt, its catchment area is situated about 20 km south of Ghent and the measuring station for discharges and solute concentrations is located at Nederzwalm, 1.7 km from the confluence with the river Scheldt. Some general data about the river basin are given in Table 1, more detailed information can be found elsewhere (De Troch, 1977).

Table 1. General characteristics Zwalm river basin

Basis area : 114 km²			
Geomorphic order (Strahler)	: 5		
Length main water course	: 21.5 km		
Drainage density	: 1.55 km/km²		
Maximum relief	: 146 m		
Mean level	: 61 m a.s.l.		
Land use :			
Agricultural land	: 58 %	Woods	: 9 %
Grass land	: 22 %	Urban areas	: 11 %

The basic data used in this study are daily mean water discharges and concentrations of inorganic solutes in 38 grab samples taken every fortnight. Analyses for the major ions and nutrients were performed using standard methods, after filtration of the samples.
Mean values and standard deviations of the measured concentrations are shown in Table 2.

Table 2. Mean concentrations and standard deviations measured in grab samples.

Element	Mean (mg/l)	St.dev. (mg/l)	Element	Mean (mg/l)	St.dev. (mg/l)
Cl^-	62	10	Na^+	34	14
HCO_3^-	360	68	K^+	11	2
SO_4^{2-}	87	14	Ca^{2+}	118	18
NO_3^-	9.6	7.9	Mg^{2+}	15	2.6
PO_4^{3-}	3.7	2.6			

As a basis for the concentration-discharge model, type I (Eq.7) is chosen. Moreover, it is assumed that the concentrations found in the grab samples are representative for the day of sampling.

STRUCTURE OF THE CONCENTRATION-DISCHARGE MODEL - FIRST STEP

As suggested by Eq.7, a linear regression analysis of $\ln C$ versus $\ln Q$ was first attempted. For all of the inorganic elements examined, still considerable deviations between the regression equation and the observations were found. This indicates that other factors than the discharge may play a role in the determination of the concentrations. These influences are neglected when the parameters A and B are considered as constants.
Further analysis of the values obtained for the regression parameter B led to the introduction of a kind of 'model memory'. Moreover it was observed that the values of B were still somewhat depending on the discharge Q. Finally the constant value of parameter B was replaced by the expression (Ledbetter and Gloyna, 1964)

$$B = A_o + A_f \cdot \ln (AFI) + A_q \cdot Q^N \tag{10}$$

with : B : the exponent parameter in basic equation (7);
AFI : an antecedent flow index, which takes into account the discharges of the days before the sampling day (in decreasing degree of importance);
N : the exponent parameter in Eq. 10;
A_o, A_f and A_q : coefficients to be determined by fitting Eq. 10 to the data.

As will be shown further, the introduction of the model memory by the AFI-index allows the model to simulate the cyclic behaviour of the C-Q relationship. For the AFI-index an expression was chosen, giving the best overall results for all the inorganic constituents considered :

$$(AFI)_i = \sum_{j=1}^{3} \frac{1}{j} Q_{i-j} \tag{11}$$

with i the number of the considered day in the year.

The values of the parameters found for the different elements are shown in the first part of Table 3, for C in mg/l and Q in l/s.

Table 3. Parameter values and efficiency of the concentration-discharge model

C in mg/l Q in l/s	First step : $C = AQ^B$ $B = A_o + A_f.\ln AFI + A_q Q^N$					Second step : $\Delta\% = R\cos\{\frac{\pi}{26}(t_i-\theta)\} + S$			
Element	A_o	A_f	A_q	N	A	R	S	θ	Eff. r
Cl^-	-0.0512	0.00649	1.0579	-0.276	20.09	5.794	0.482	-2	0.78
SO_4^{2-}	-2.0037	0.01936	2.0876	-0.011	54.60	8.032	-0.211	5,5	0.74
HCO_3^-	-0.1329	0.01786	1.7910	-0.277	54.60	7.404	2.239	9	0.91
Na^+	-0.0673	0.00645	2.5438	-0.361	7.389	4.006	2.861	0	0.87
K^+	0.2234	-0.01927	0.3438	-0.153	2.718	14.86	-2.057	-14	0.73
Ca^{2+}	-0.15643	0.01820	1.3632	-0.232	20.09	10.11	-1.318	7	0.74
Mg^{2+}	-0.13857	0.01632	1.3943	-0.244	2.718	7.835	0.268	7	0.70
NO_3^-	-0.22952	0.04834	0.0404	0.236	1.000	-	-	-	0.89
PO_4^{3-}	0.55459	-0.06184	5.9190	-0.439	0.135	-	-	-	0.94

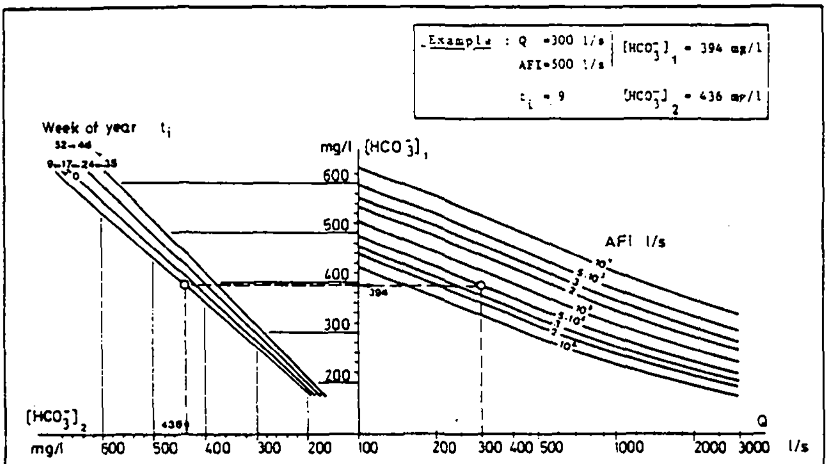
STRUCTURE OF THE CONCENTRATION-DISCHARGE MODEL - SECOND STEP

Analysis of the residual deviations between the concentrations computed by the first step-model and the observed values revealed a seasonal trend in the 3-point moving average residuals. A periodical function (1 year period) could be fitted to the percentic deviation Δ%, by optimization of the parameters R, S and θ :

$$\Delta(\text{in }\%) = R\cos\{\frac{\pi}{26}(t_i-\theta)\} + S \qquad (12)$$

with t_i the number of the week of the year.

The obtained values of the parameters R, S and θ are shown in Table 3, second part. The efficiency of the complete model is also given in table 3 and an example of the graphical representation of the model for HCO_3^--concentrations is shown in figure 1.



Bicarbonate concentration
Figure 1.

SOME RESULTS OBTAINED BY THE MODEL

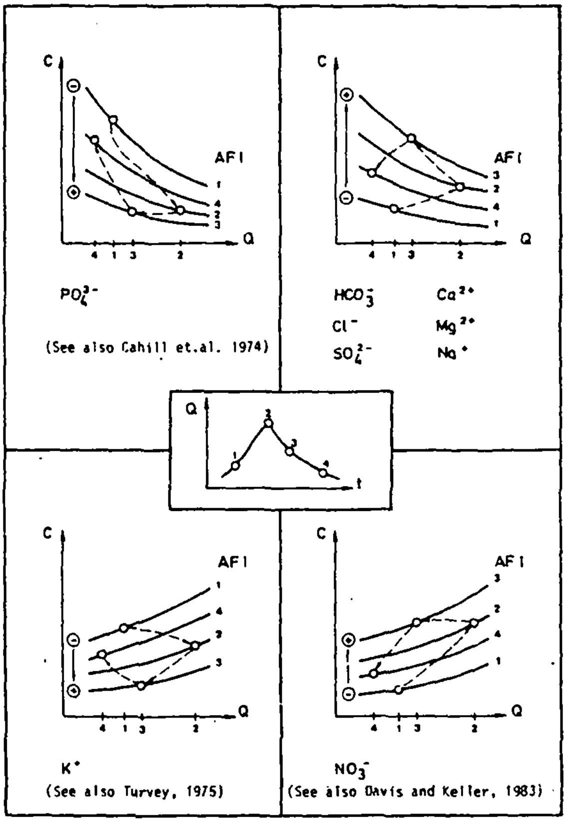
The loads of different inorganic constituents in the Zwalm river have been computed by the model, using the parameters shown in table 3, over a six year period. The resulting mean yearly loads, expressed in $kg.ha^{-1}.year^{-1}$, are shown in table 4, as well as the mean loads obtained from the products of the concentrations measured in the grab samples times the observed discharges. A systematic over-estimation of the load is detected when using the second method, except for nitrate where the load is underestimated, which could be explained by the particular behaviour of the nutrient (enrichment in stead of dilution, see figure 2).

Table 4. Mean yearly loads in the Zwalm river in $kg.ha^{-1}.yr^{-1}$
(1) obtained by the concentration-discharge model
(2) obtained from measured values only

Element	(1)	(2)	Element	(1)	(2)
Cl^-	140	164	Na^+	63	90
HCO_3^-	790	950	K^+	25	30
SO_4^{2-}	224	231	Ca^{2+}	292	314
NO_3^-	50	22	Mg^{2+}	34	40
PO_4^{3-}	6	9			

Other results of the modelling of the mineral content variations and some tentative explanations about the different behaviour of the inorganics are given elsewhere (De Troch et.al., 1976).

Based on the possible combinations of (1) the general trend observed in the concentration-discharge relationship of the elements (decreasing or increasing concentrations for increasing flow with constant AFI) with (2) the way of variation of concentration with AFI, at constant flow, the elements could be divided into four groups, indicated in figure 2.



Loop curves C - Q

Figure 2.

Considering a typical flow variation as shown in figure 2 yields the loop-curve or hysteresis pattern described in recent literature (Davis and Keller (1983), Walling and Webb (1980)). These loop curves are generated by the model memory contained in the AFI-index. Further research on the possible physico-chemical explanation of the observed phenomena is still needed for an adequate evaluation of non-point source pollution of surface waters.

CONCLUSIONS

Overland flow, leaching from agricultural soils and percolation from ground water leads to concentration-discharge relationships in surface water which proved to be specific for the inorganic elements considered.
In the case of the Zwalm river it has been possible to model the concentration-discharge variations by a model relating the concentrations to the river discharge, antecedent flow conditions and time of the year. A dilution effect of increasing flow was observed for HCO_3^-, Cl^-, SO_4^{2-}, Ca^{2+}, Mg^{2+}, Na^+ and PO_4^{3-} whereas an enrichtment was noted for NO_3^- and, to a smaller extent, for K^+. The model can also simulate the loop curves reported recently.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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POLICY ANALYSIS OF WATERMANAGEMENT FOR THE NETHERLANDS (PAWN)

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ABSTRACT

Watermanagement is defined as the care for the control of water, taking into consideration the interests connected. It applies to both surface- and groundwater and to waterquantity and -quality. The interests associated with it vary widely: agriculture,shipping, the processing industry, drinking water companies, power plants etc. are all very much affected by watermanagement measures. Further, the safety against flooding is very important. Last but not least there are the interests of the environment, although no direct financial implication are at stake. It is not possible in practice to satisfy all wishes coming from the many interests. That's why it is necessary to study the most efficient way to manage the water resources. In the PAWN-study much use is made of computermodels, which simulate the watermanagement of the Netherlands. The results of the study have contributed strongly to the design of the watermanagement policy of the Netherlands for the coming 10-15 years.
The used techniques is made applicable in planning studies of water resources management abroad.

Keywords: policy analysis, system analysis, waterresources management, surfacewater, groundwater, waterquality, waterquantity.

INTRODUCTION

In order to design a policy for the watermanagement in the Netherlands in the period 1976-1981 a study was carried out with the following main objectives

- to develop a methodology for assessing the multiple consequences of national water management policies (a coherent mix of measures to improve water management);
- to apply this methodology in generating and analysing alternative water management policies.

The results of the study have contributed strongly to the design of the watermanagement policy of the Netherlands for the coming 10-15 years. A Policy Report is to be published in 1984 by the Minister of Traffic and Public Works. Such a policy document, drafted about once every 10 years, can be considered a master plan for national water management.

About 125 man-years of direct contributions were required for this investigation, carried out by the RAND-corporation (USA), the Delft Hydraulics Laboratory and the National Public Works Departments. The last is the government agency responsible for water resources management and had established the study in 1976. In 1981 the study was ended. The complete rapportage in 21 volumes is available for each interested party.

TEXT OF THE PAPER

The watersystem

In figure 1 a small-scale, slightly stylised map of the Netherlands is shown. The shadowed part is above average high tide sealevel, the rest is lower, up to 6 m below mean sealevel.
In an average year, the total fresh water supply to the Netherlands is 110 x $10^9 m^3$, of which 20 x $10^9 m^3$ evaporates. In a very dry year, supply may amount to less than a quarter of the figure given, while in some dry summer periods, it is much less than these figures would suggest, whereas evaporation and transpiration are higher. This results in severe shortages during the summer months, especially for agriculture. Of the supply, 67 per cent is given by the river Rhine, 8 per cent by the Meuse, 3 per cent by the small rivers crossing the borders in the east and south, and 22 per cent by local precipitation. As can be seen on the map, the Rhine, soon after entering the country, forks twice into, finally, three braches (Waal, Nederrijn and IJssel).

The discharge distribution of the Rhine branches can be influenced by means of weirs. For instance, in order to maintain the minimum water depth for navigation in the IJssel River, a certain amount of water is directed to the IJssel branch flowing into the IJssel lake, which functions as a reservoir. Most of the Rhine discharge flows west where it is mainly used to reduce salt intrusion in the Rotterdam Waterway, thus protecting some important intakes for drinking water supply and agriculture. To effectively combat this salt intrusion the flow past Rotterdam should not be smaller than about 600 m^3/s.
Groundwater is generally of good quality, especially in the higher eastern part of the country. The total amount of fresh groundwater is, however, limited (2 x $10^9 m^3$/year) and quality problems exist in the western part (salt water intrusion). For drinking water purposes, entensive use is made in the western part of the Netherlands of artificial infiltration in the coastal dune area so that the salinization of groundwater aquifers can be kept at an acceptable level.

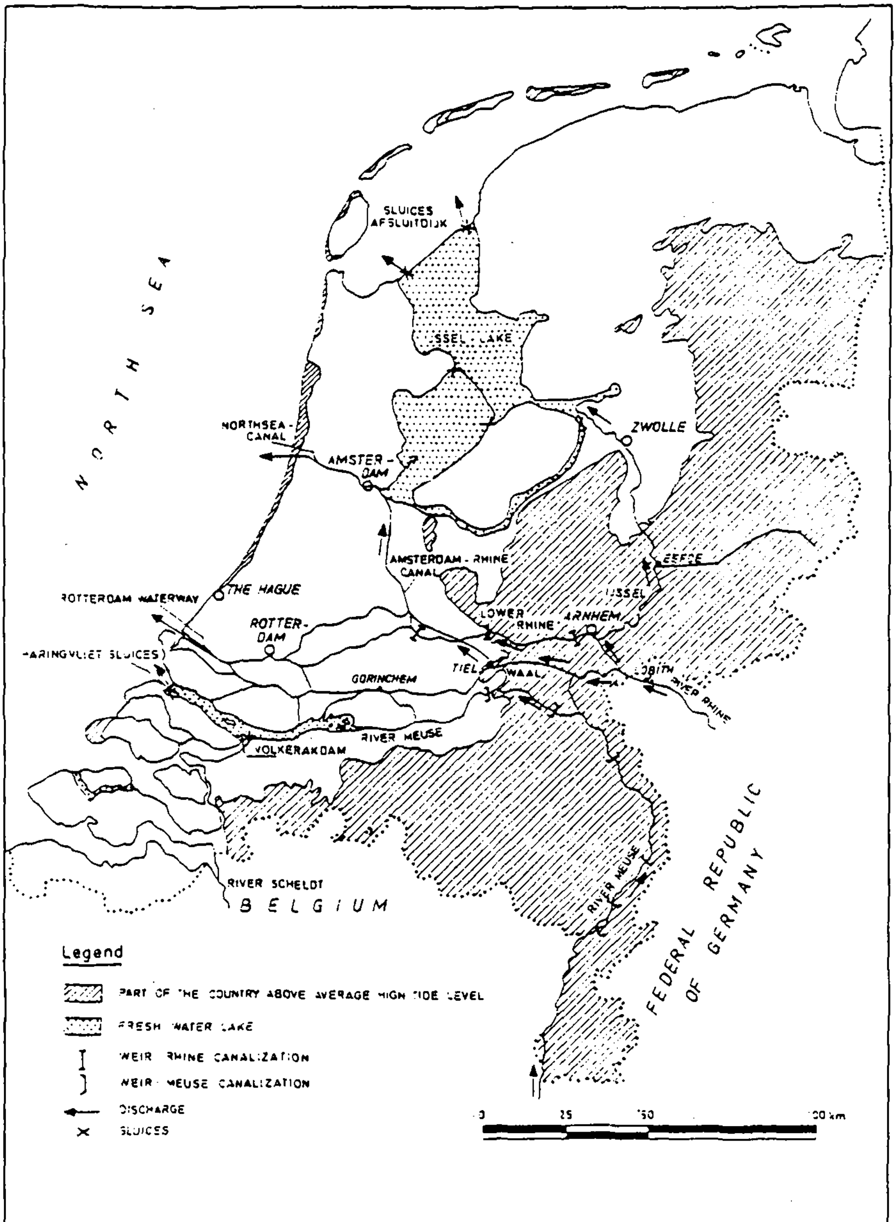

Fig. 1 THE NETHERLANDS, PRINCIPAL ELEMENTS OF THE WATER-SYSTEM

Demand characteristics

Water management in the Netherlands has to deal with a number of different usage categories, such as agriculture, navigation, households, industry, power production (cooling water), and environment (aquatic and terrestrial).

Agriculture is by far most important user of water, both from a quantitative and economic point of view. Most of the agriculture water supply comes from natural sources (rain, soil moisture). In dry periods, water shortages can to some extent be overcome by artificial supply (sprinkling) from either surface water or groundwater sources.

Inland navigation as a means of transportation is extensively used both in the Netherlands and to neighbouring countries like Germany, Belgium and France. Considerable amounts of water are needed to maintain sufficient depth along the most important shipping routes.

Households and industries use relatively small amounts of water. The main sources are groundwater and discharges of the Rhine and Meuse, either directly or after infiltration in the coastal dune areas. Deterioration of surface and groundwater quality is of great concern. Competition exists among drinking water companies, industries and agriculture for the scarce groundwater resources.

Many power plants in the Netherlands are located along the main canals, rivers or lakes to ensure sufficient cooling water. In order to avoid excess of thermal standards the power plants have to take back their production capacity.

The aquatic environment is directly threatened by the pollution of surface water bodies. The most important source of pollution on anational scale is the Rhine which carries a vast amount of pollutants from the industrialized areas of other European countries. Urban, industrial and agricultural waste water in the Netherlands add to the pollution problem. A specific problem is salt intrusion in the lower, northern and western parts of the Netherlands. Pollution problems can, to some extent, be alleviated by flushing surface water bodies with large amounts of water of a relatively good quality. However the desired quantity to meet these flushing demands may not always be available.

Terrestrial environment requirements relate to ground water levels and groundwater quality. As such, the terrestrial environment is another competitor for the scarce groundwater resources. Groundwater quality threatened by the direct pollution of soils from various sources (agriculture, industry, households) or by the deteriorating quality of surface water infiltrating the aquifers.

Problems related to watermanagement

Under average conditions enough water is available in the Netherlands to meet the various demands. Since both supply and demand vary, problems do occur in certain dry periods. These problems may be related to the limited availability of surface water or groundwater, or to the insufficient quality of available water. Also insufficiënt capacity or deficient infrastructure to convey the water to where it is needed may lead to local problems. The lowering of groundwatertables when excess demands for groundwater not are restricted, causes implications for Flora depending on the groundwater.

These problems may cause damage to the agriculture (by dryness or by salinity), to the navigation (by insufficient depth), to the drinking water supply and to the industries (both by restricted availability of groundwater). The power production will become more expansive, while the waterpollution is more difficult to fight against. In some cases there are possibilities to avoid more or less the repercussions by measures (expending infrastructure, changing managemental rules). Most of this measures may cause damage to the environment and nature, because of the altering in flow's, levels, quality etc.

Obviously various usage categories compete for the same water resources. Since a number of users have the option of using groundwater or surface water, competition is not limited to one source of water. Similarly, quantity aspects cannot be considered separately since they are clearly interrelated, both on the supply and demand side. These aspects need therefore to be considered simultaneously in identifying and solving water management problems.

PAWN-approach

The PAWN study concerned all relevant water usage categories mentioned in the previous section. Both quantity and quality, surface water and groundwater were taken into account. PAWN was directed at determining the consequences of alternative water management policies. Such a policy is constituted by a coherent mixture of tactics, where a tactic is defined as a single measure to improve water management.
Three different kinds of tactics were considered:
- technical tactics: changing or expanding infrastructure, e.g. building a canal;
- managerial tactics: changing the operation of water management infrastructure, e.g. different operation of weirs; and
- price and regulation tactics; imposing taxes, levies or quotas to effect water demand or the quality of water discharged.

The general approach with respect to the generation and evaluation of water management policies is reflected in Figure 2, which shows the various stages of analysis. The major stages are screening, policy design and impact assessment.
- In the screening stage, a great number of individual tactics are roughly evaluated to identify 'promising' tactics (mainly based on cost-benefit and implementation criteria);
- In the policy design stage, individual promising tactics are combined in different ways to obtain a number of promising water management policies;
- In the impact assessment stage a more complete and detailed analysis is carried out to determine the multiple consequences of the selected policies (for all relevant sectors of national interest).

Both in the screening and impact assessment stages, the performance of the water management system was assessed with and without the tactic/policy under consideration. In this respect the water management system was interpreted in a broad sense, i.e. it comprised:
- the water in, and on top of the soil (both quantity and quality);
- the infrastructure of waterways, rivers, canals, lakes etc.;
- the system of rules regulating affecting supply and demand;
- the various water users;
- the administrative system.

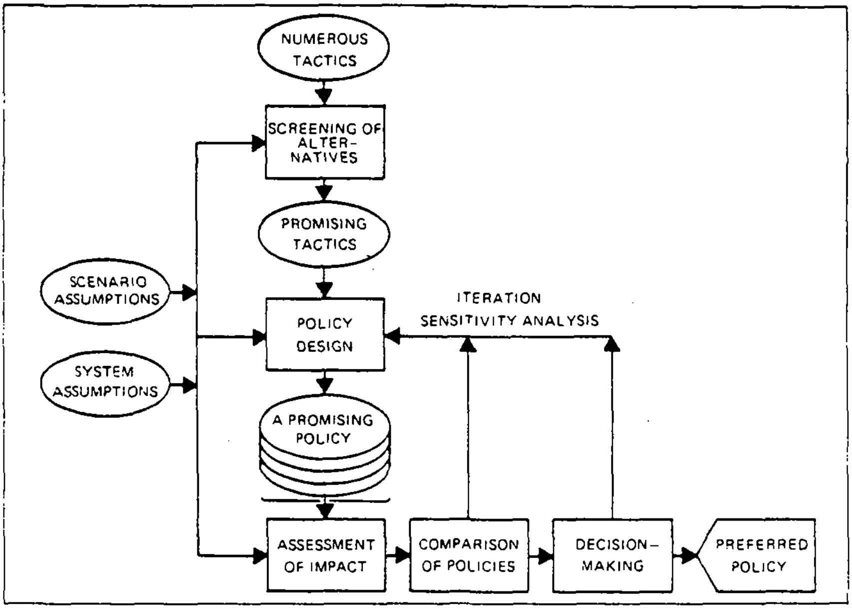

Fig.2. Stages of Policy Analysis in PAWN.

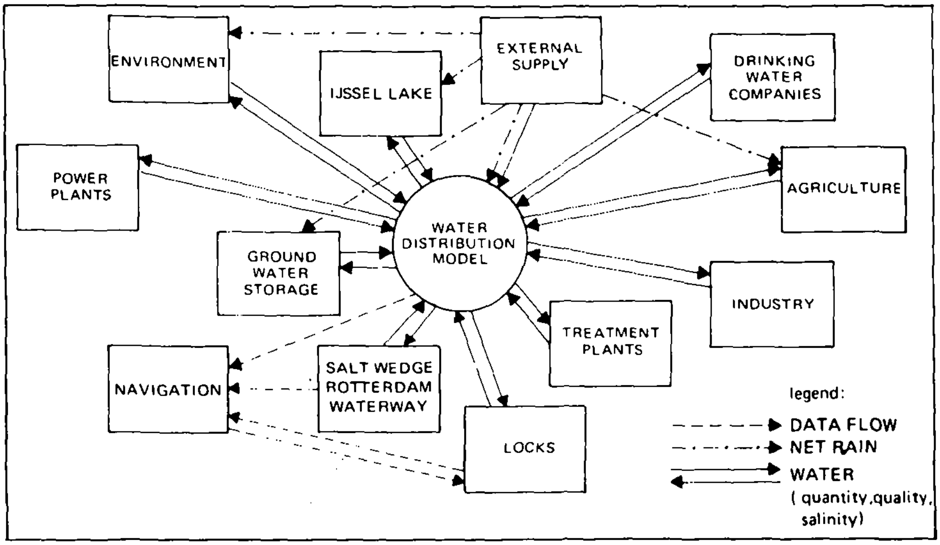

Fig.3. PAWN System Diagramme.

Given these many components to be considered, the aspects of space and time, and the often complex relations existing between the components of the water management system, PAWN made extensive use of mathematical modelling. A survey of the general modelling structure is given in Figure 3, showing the most important components of the water management system and their interrelations. The following models have been specifically developed for PAWN:

- National water distribution:

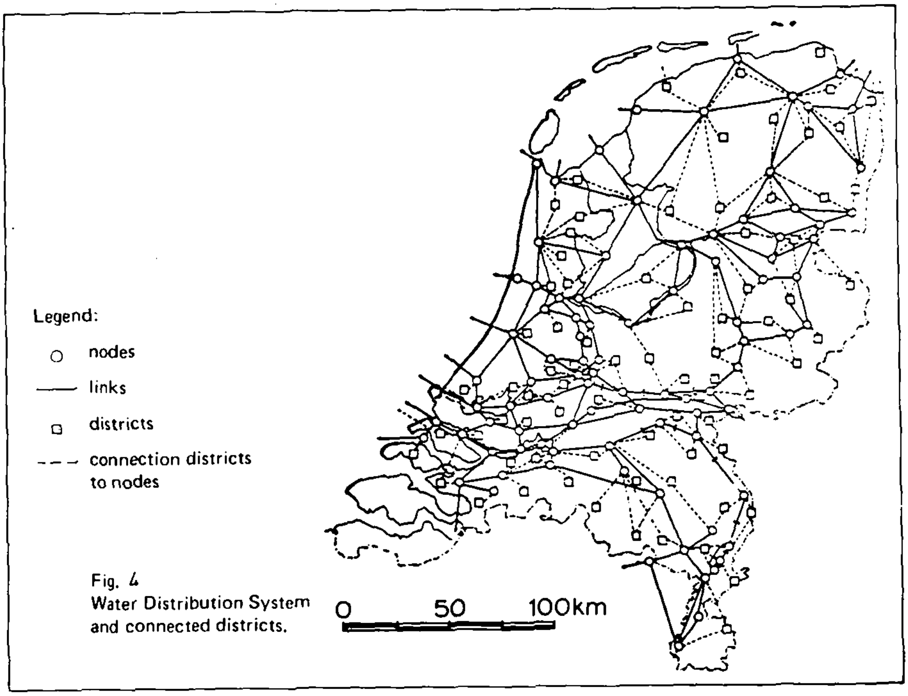

Fig. 4
Water Distribution System and connected districts.

The Water Distribution Model (WDM) is based on a network of links and nodes, representing the main rivers, canals and lakes (see Figure 4). The WDM simulates water distribution among parts of the country and usage categories for discrete timesteps of 10 days by comparing supply and demands and applying allocation rules.

- Agriculture and storage of groundwater:
  The District Hydrologic and Agriculture Model (DISTAG) computeswater and salt balances on the regional level. The country is divided into 77 "districts" which served units to describe local water management. These districts were connected to the nodes of the distribution network of the WDM (see Figure 4).
- Drinking water companies and industry:
  The Response Design Model (RESDM) computes the costs of water supply for drinking water and industry, given the availability of groundwater and surface water.
- Power plants:
  The Electric Power Redistribution and Cost Model (EPRAC) computes the cost of power production given available water flows, while complying with thermal standards.

In addition, sub-studies were carried out for:
- navigation and shipping locks;
- salt wedge in the Rotterdam Waterway;
- IJssel lake;

- external supply (supply of rain and river discharges);
- the aquatic and terrestrial environment.

Each box in the diagram of Figure 3 represents one or more different models or sub-studies. The WDM is in the centre. The models and sub-studies related to specific categories of water usage were integrated with the WDM in one of the following three ways:
- on-line connection with the WDM (e.g. agriculture and groundwater storage);
- pre-processing: use of results of model or sub-study in the WDM (e.g. navigation, shipping locks, drinking water companies and industry);
- post-processing: use of WDM results in model or sub-study (e.g. power plants).

Using the WDM and the related models and sub-studies, the full impact of any particular water distribution could be determined.

Results and conclusions

An important feature of the PAWN study was the creation of a general methodology to assess the impacts of changes in water management on various usage categories. More specifically:
- a set of models procedures to analyse specific water usage categories, other system components and the water management system as a whole;
- an organized data base to be used in the application of the various models.

In this way PAWN not only produced specific results but also provided a useful tool to be used in subsequent studies. Given the fact that water management is a continuous process rather than a one-time effort, this is certainly to be considered a most valuable result.

The actual use of the PAWN methodology yielded a vast number of results, such as:
- a number of cost-benefit analyses of promising options to improve water management;
- priority and allocation rules for water distribution;
- water management policies, inclusive impacts.

Some important conclusions based on these results were:
- Agriculture dominates all other categories in terms of potential benefits associated with the implementation of technical and managerial tactics.
- Sprinkling of agricultural crops seems to be cost-effective in many cases and an increase in the use of sprinkling equipment in the Netherlands is therefore to be expected.
- Many local plans of water boards to enlarge the area with access to surace water seem cost-effective. This and the expected increase in the use of sprinkling equipment may result in substantially higher demands in the future for water for agricultural purposes.
- All of the larger, more expensive, infrastructural works to improve water management were found to be non cost-effective.
- National tactics to improve water quality are not very effective due to the fact that the bulk of the pollution load to Dutch waters originates from the Rhine.
- Stiffer competition for the use of the quite limited groundwater resources among drinking water companies, industries and agriculture is to be expected in future. Imposing tight restrictions to meet desired environmental standards will result in large losses to the various users.

PAWN results and conclusions have played an important role in drafting the

Rijkswaterstaat's Water Management Policy Document. Specific contributions were:

- Many facts, figures and conclusions were either directly obtained from PAWN analyses or computed in a later stage using models and data from PAWN;
- PAWN has highly stimulated the integrated way of thinking expressed in the Document. For the first time explicit attention was given at a national level to the relationship between environmental and groundwater problems and other components of the water management system (e.g. relation between groundwater and surface water);
- PAWN has improved the cooperation between the Ministry of Transport and Public Works (sponsoring the study) and other governmental departments on matters related to water management.

The continuation of the PAWN study

Within the framework of the new legislation in the field of water management (Water Management Act and the Law against Pollution of Surface Water), the Rijkswaterstaat is facing the task of drafting surface water plans (both quantity and quality) for the national waters within the next few years. The Water Management Policy Document provides the overall guidelines for these and other plans (at lower hierarchic levels). This policy Document is to be rewritten every ten years by the Rijkswaterstaat.
The tools provided by PAWN will be used extensively in executing these tasks. As soon as the PAWN study was completed the Rijkswaterstaat started preparations for PAWN II. An evaluation was made of the available tools and data bases and recommendations were made for improvement and extension. Work on PAWN II has started in the second half of 1982 as a joint project of the Delft Hydraulics Laboratory and the Rijkswaterstaat.

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MODELE DE GESTION DES EAUX D'UN LAC SAHELIEN: LE LAC DE GUIERS (SENEGAL)

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RESUME

Le lac de Guiers constitue la principale réserve d'eau douce de surface du Sénégal. Les futurs aménagements du fleuve Sénégal et les deux barrages qui y seront construits auront des effets bénéfiques mais aussi néfastes sur le milieu. Nous proposons un modèle de gestion quantitative de ses eaux, basé sur les différents termes du bilan hydrologique, modèle qui devrait permettre de minimiser les impacts négatifs des aménagements tout en sauvegardant les intérêts des divers utilisateurs des eaux du lac. Le modèle est testé sur six années hydrologiques.

INTRODUCTION

Le lac de Guiers occupe, en rive gauche du fleuve Sénégal, une dépression allongée, étroite et peu profonde (fig.1). Long de 50 km et large de 7 au maximum il s'inscrit dans un quadrilatère entre 15°25' et 16° de longitude ouest, 15°40' et 16°25' de latitude nord. Il constitue la seule importante réserve d'eau douce de surface du Sénégal. A son maximum d'extension, il couvre une superficie de 300 km^2 et contient 720 millions de m^3 d'eau. Divers aménagements successifs ont permis d'en faire un véritable réservoir: endiguement des rives de la région nord et nord-ouest, barrage à son extrémité sud.

Son importance est capitale dans cette région sahélienne qui subit les effets de la sécheresse depuis le début de la précédente décennie. Ses eaux sont destinées en partie à l'irrigation de près de 6 000 ha de canne à sucre exploités par la Compagnie Sucrière Sénégalaise (CSS). La compagnie nationale des eaux y capte et traite quotidiennement 50 000 m^3 d'eau destinée aux grandes agglomérations et à Dakar la capitale. Divers petits périmètres agricoles sont exploités le long des rives, soit par irrigation,soit en culture de décrue. Le lac assure enfin la subsistance des populations de la région, riveraines et sédentaires (pêche, culture vivrière ...) ou transhumantes (éleveurs peuls).

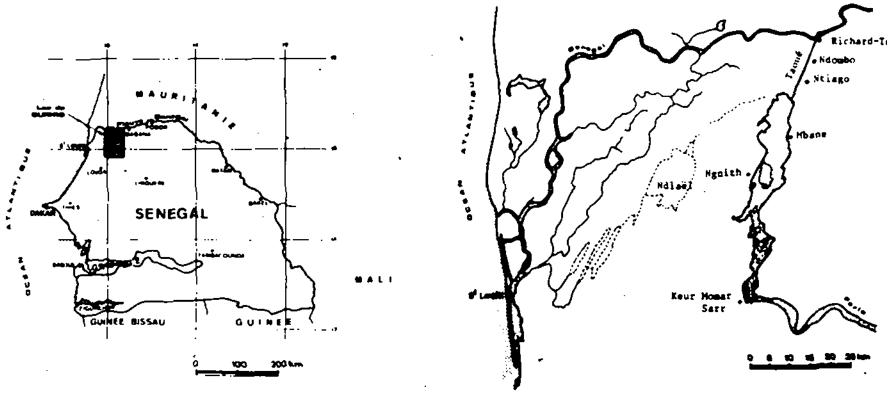


Fig.1 : Carte de situation du lac de Guiers.

APERCU HYDROLOGIQUE

Les relations fleuve-lac .

Le régime hydrologique du fleuve Sénégal est de type tropical pur et se caractérise, dans son parcours aval, par une période de hautes eaux de 3 à 4 mois (juillet à octobre) et une période de basses eaux s'étalant de novembre/décembre à juin/juillet.

A l'heure actuelle l'existence du lac dépend entièrement de la crue du fleuve qui l'alimente. Cette alimentation se fait par l'intermédiaire d'un canal de 17 km, la Taoué limité à son extrémité nord par deux ponts-barrages dont le fonctionnement est schématisé à la fig.2. En période de crue fluviale (t_1) les deux ponts-barrages sont ouverts et le lac se remplit (en moyenne en août et septembre). Dès la baisse du niveau fluvial, le second pont-barrage (b_2) est fermé de manière à empêcher le déversement des eaux lacustres dans le fleuve. Durant la période qui fait suite (t_2) les variations de niveau des eaux sont sous la dépendance de l'évaporation et des pompages de la compagnie des eaux. La CSS s'approvisionne toujours à partir des eaux fluviales; sa station de pompage se localise entre les deux ponts-barrages. Durant l'intervalle de temps t_2, les eaux marines ont remonté le cours du fleuve, ceci lié à la pente quasi nulle du cours d'eau et à ses faibles débits en phase d'étiage. A partir de la mi-février, la langue salée atteint l'embouchure de la Taoué dans le fleuve. A ce moment le premier pont-barrage est fermé (b_1), le second est ouvert et la CSS s'approvisionne alors jusqu'à la fin juillet (t_3) à partir des eaux lacustres.

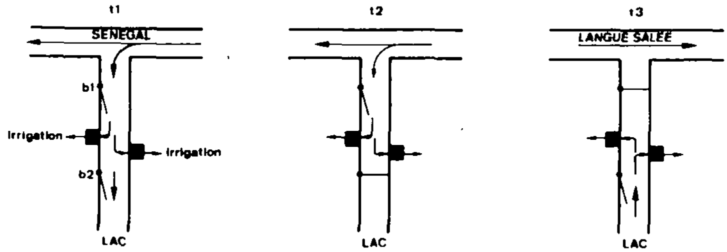


Fig.2 : Fonctionnement du système des barrages de la Taoué en cours d'année.

Hydrologie lacustre

Le contour et la physionomie du lac se modifient sans cesse, ceci lié à un rapport surface/volume élevé. Les variations annuelles et interannuelles du plan d'eau peuvent atteindre 3 mètres, ce qui témoigne de la précarité des équilibres et de la profonde instabilité du milieu.
Le bilan hydrologique du lac, établi de 1976 à 1982 (Cogels et al., 1981; Cogels et al., 1983) et représenté à la fig.3 fait ressortir la part importante des pertes subies par évaporation (79 % et 2.20 m par an) devant celles dues aux divers pompages avec respectivement 19 % pour l'irrigation de la canne à sucre et 2 % pour les prélèvements d'eau de consommation. Les apports sont surtout tributaires de la crue du fleuve (81 %), des précipitations (11 %) et des rejets des zones irriguées (8 %). Les pertes par infiltration sont négligeables vu la nature du fond du lac (argile sableuse).

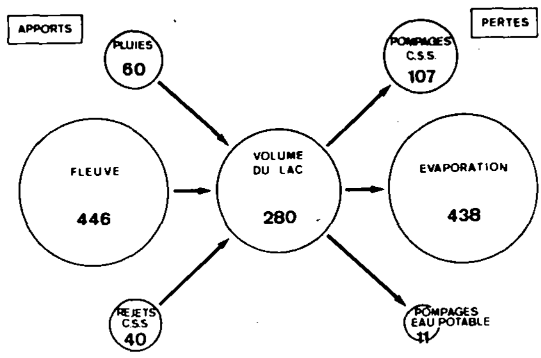


Fig.3 : Bilan hydrologique annuel moyen du lac (1976-82).

Les futurs aménagements de la vallée du fleuve

Dans le cadre de la mise en valeur agricole de la vallée du fleuve Sénégal (300 000 ha de cultures irriguées), deux barrages seront érigés sur le cours d'eau. Le premier à Diama (fonctionnel en 1986) à quelques 30 km en amont de l'embouchure du fleuve. Son rôle est d'empêcher la remontée saline annuelle dans le cours d'eau. Le second à Manantali au Mali (terminé en 1990 ?) 1 200 km en amont, augmentera les débits d'étiage, assurant ainsi l'irrigation toute l'année, la navigation et la production d'énergie électrique.

GESTION QUANTITATIVE DES EAUX

L'extrême fragilité du lac de Guiers est liée aux importantes variations saisonnières du niveau du plan d'eau. Cette instabilité peut, d'une année sur l'autre, ralentir voire même arrêter totalement les principales activités qui se sont développées sur son pourtour (agriculture irriguée, cultures de décrue ...). L'assèchement d'une grande partie du lac comme ce fut le cas en 1978, 1980 et surtout en 1983 ne résulte pas seulement de causes naturelles mais aussi d'un manque de concertation entre les principaux utilisateurs de cette réserve d'eau douce trop fortement sollicitée. Si le ralentissement des activités maraîchères, provoqué par une gestion désordonnée des eaux du lac entraîne à l'échelle de l'année des répercussions fâcheuses sur l'économie du pays, que dire des dégâts irréparables causés à la faune piscicole. A partir de 1986 l'achèvement du barrage de Diama devra permettre un meilleur remplissage du lac de Guiers : le plan d'eau devrait atteindre , voire dépasser, la cote 2.50 m dans la mesure où seront achevés les aménagements, endiguements en rive droite du fleuve et le rehaussement des digues du sud et nord-ouest du lac.
La gestion des eaux lacustres après la mise en fonction du barrage de Diama n'a pas encore été précisée. Deux possibilités peuvent être envisagées : maintien, comme c'est le cas à l'heure actuelle, d'une décrue semi-artificielle du lac ou remplissage épisodique et prolongé jusqu'en fin de saison sèche pour conserver le plan d'eau à un niveau plus constant. Cette seconde solution semble avoir été retenue par les autorités bien qu'elle ne repose sur aucunes données précises du futur statut du lac. Comme nous allons le voir, cette option compte bien des risques et il est en fait possible (après la mise en fonction de Diama) de concilier les multiples intérêts en jeu.

Les intérêts en jeu

-Les cultures irriguées : A partir de 1986, la CSS devrait pouvoir assurer son irrigation à partir des eaux fluviales maintenues douces toute l'année. En dehors de la phase de remplissage du lac, le second pont-barrage pourrait ainsi rester fermé, et la CSS ne serait plus tributaire des eaux lacustres. Les autres petites exploitations agricoles ont le souci de maintenir le lac à un niveau plus élevé que le seuil d'exhaure de leurs installations de pompage. Ce seuil se situe largement en-dessous de la cote -0.50 m. Maintenir le lac à un niveau élevé une bonne partie de l'année ne ne leur procurerait aucun avantage.

-Les cultures de décrue : De par leur nature, ces cultures nécessitent le retrait progressif des eaux. L'alternative du maintien du lac à un niveau assez constant entraînerait leur disparition alors qu'il est très possible de sauvegarder ces exploitations traditionnelles qui ont une importance alimentaire et économique dans la région (1000 $ à l'ha).

-La production d'eau potable : Le seuil d'exhaure de la station de pompage est à la cote -0.80 m, niveau limite impératif à ne pas dépasser. Le maintien du plan d'eau à un niveau élevé toute l'année ne présente ici non plus aucun avantage particulier.

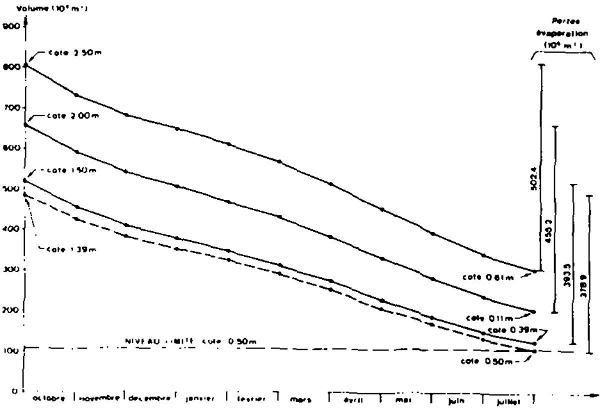
-La faune piscicole : La productivité piscicole des eaux lacustres est bonne : de 1980 (année de très forte baisse des eaux) à 1982, la faune piscicole s'était remarquablement reconstituée grâce au maintien en 1981 et 1982 d'un niveau de l'eau suffisant en fin de saison sèche. Lors d'années d'important retrait des eaux (jusqu'à -1.50 m), elle a cependant fait l'objet d'une pêche surintensive détruisant ainsi une bonne partie du cheptel. Le seuil critique, sans risque de déséquilibre écologique se situe à notre avis à la cote -0.50 m.

Indirectement enfin, les importantes variations annuelles du niveau des eaux observées ces dernières années ont permis de limiter le développement de la faune malacologique et d'éviter ainsi l'apparition de bilharziose dans le milieu lacustre. Cette limitation au développement des mollusques s'avérera encore plus indispensable après la construction du barrage de Diama avec le maintien d'eau douce toute l'année dans le système fluvio-lacustre.

Tout concorde donc pour affirmer qu'il est indispensable de maintenir une importante variation annuelle du niveau des eaux dans le lac, tout en évitant de franchir en fin de saison sèche, une limite inférieure que nous fixerons à <u>-0.50 m</u>. Simultanément, le plan d'exploitation des eaux du lac doit également viser à minimiser au maximum les pertes subies par évaporation.

<u>Plan d'exploitation du lac et modèle mathématique de gestion des eaux</u>

Les taux d'évaporation moyens mensuels du plan d'eau ont été calculés sur base du bilan hydrologique, de 1976 à 1982 (Cogels et al., 1983). L'évaporation totale annuelle du lac est de 2.20 m. Elle est semblable à celle des autres lacs plats sahéliens, lac de Bam en Haute-Volta et lac Tchad, avec respectivement 2.34 m (Pouyaud, 1979) et 2.15 m (Roche, 1980; Riou, 1975). Ceci permet de préciser l'évolution "naturelle" du lac, soumis, durant la période hors remplissage (d'octobre à juillet), aux seuls effets de l'évaporation. Durant cette période de dix mois, la hauteur d'eau totale évaporée étant de 1.89 m et la cote limite inférieure à ne pas dépasser se situant à -0.50 m, le remplissage minimal au 1 octobre devrait ainsi être supérieur à 1.39 m. La fig.4 illustre l'évolution théorique du lac au départ de cette dernière cote ainsi que celle correspondant aux cotes initiales de 1.50 m, 2.00 m et 2.50 m. On observe que plus les cotes de départ (au 1 octobre) sont élevées, plus les volumes évaporés durant la période d'isolement du lac sont importants. Ceci est normal compte tenu du fait que plus le remplissage est important, plus la surface exposée est grande et dans le même intervalle de temps, les pertes par évaporation augmentent. Pour un remplissage à 1.39 m le lac perd au cours des dix mois $380.10^6 m^3$ par évaporation. Au départ de la cote 2.50 m, ce sont $502.10^6 m^3$ qui seront perdus.
Il nous a donc paru important d'établir un modèle de gestion qui, compte tenu des besoins annuels en eau, et éventuellement des besoins futurs, puisse fixer la cote maximale à atteindre au remplissage de manière à réduire les pertes par évaporation, et à pouvoir détourner vers d'autres usages l'eau qui aurait ainsi été perdue.



<u>Fig.4</u> : Evolution du volume d'eau dans le lac, d'octobre à juillet au départ de différentes hauteurs d'eau et pertes totales par évaporation.

Le modèle mathématique de gestion des eaux du lac doit permettre de répondre avec précision aux deux questions suivantes :

1) Compte tenu d'une cote x du lac à la fin du remplissage et connaissant les volumes pompés et rejetés dans le lac, quelle sera la cote atteinte lors de l'ouverture du pont-barrage l'année suivante ? Cette question s'applique à la phase précédent la mise en fonction du barrage de Diama, période durant laquelle il est impossible de régler à volonté la hauteur d'eau maximale lacustre puisqu'elle ne dépend alors que des aléas des conditions climatiques sur le bassin versant et de la crue fluviale qui en résulte.

2) Compte tenu d'une cote limite inférieure du lac déterminée et à ne pas dépasser en fin de saison sèche, et connaissant l'importance et la répartition dans le temps des prélèvements et rejets, à quelle cote doit-on remplir le plan d'eau quelle que soit la date de fermeture du pont-barrage de Richard-Toll ? Cette question s'applique à la période suivant la mise en fonction du barrage de Diama grâce auquel il sera possible de réguler à volonté le niveau d'eau maximal du lac en phase de remplissage et également de prévoir si nécessaire des réalimentations du plan d'eau en cours d'année.

Détermination de l'évaporation journalière sur le lac

Afin de préciser au maximum l'évolution annuelle de la cote du lac, dans les deux cas envisagés, nous allons tenter d'attribuer à chacun des jours séparant la fermeture du pont-barrage de son ouverture l'année suivante, une évaporation "type" en nous basant sur la moyenne mensuelle calculée. La fig.5 explicite la manière dont nous avons procédé. Sur base des moyennes mensuelles calculées précédemment, nous avons, pour chacun des dix mois tracé l'évolution la plus probable de l'évaporation, ceci selon les critères suivants :

-La courbe (en fait un ensemble de droites) passe obligatoirement en début et fin du mois x par les valeurs correspondant respectivement à l'évaporation moyenne des mois x et x-1, x et x+1.

-Au cours du mois x, et selon son évaporation moyenne par rapport à celle des mois x-1 et x+1, la courbe passe par la valeur de la moyenne mensuelle soit au milieu, soit en début ou en fin de mois.

L'équation des différentes droites permet de calculer, pour chaque jour la valeur de l'évaporation correspondante. Les résultats ont été établis pour les dix mois couvrant la période d'isolement du lac, c'est-à-dire du 1 octobre au 31 juillet.

Commentaires

-La moyenne mensuelle de l'évaporation, obtenue par cette méthode graphique (somme des évaporations quotidiennes/nombre de jours du mois) est en bon accord avec celle obtenue sur base du bilan hydrologique; la droite de régression entre ces deux types de calculs est la suivante, pour l'ensemble des dix mois pris en compte :

$$EL_{mG} = 0.982\ EL_{mH} + 0.149 \qquad r = 0.998$$

où EL_{mG} = Evaporation moyenne mensuelle obtenue par la méthode graphique;

EL_{mH} = Evaporation moyenne mensuelle obtenue par le bilan hydrologique.

L'écart entre EL_{mG} et EL_{mH} pour un même mois ne dépasse jamais 0.18 mm/jour.

-Enfin, la somme totale des évaporations quotidiennes obtenues par la méthode graphique pour les dix mois est de 1.89 m soit exactement identique à celle obtenue sur base du calcul du bilan hydrologique.

L'ensemble des droites représentées à la fig.5 et leur évolution dans le temps, semblent donc bien refléter avec fiabilité les variations effectives de l'évaporation journalière.

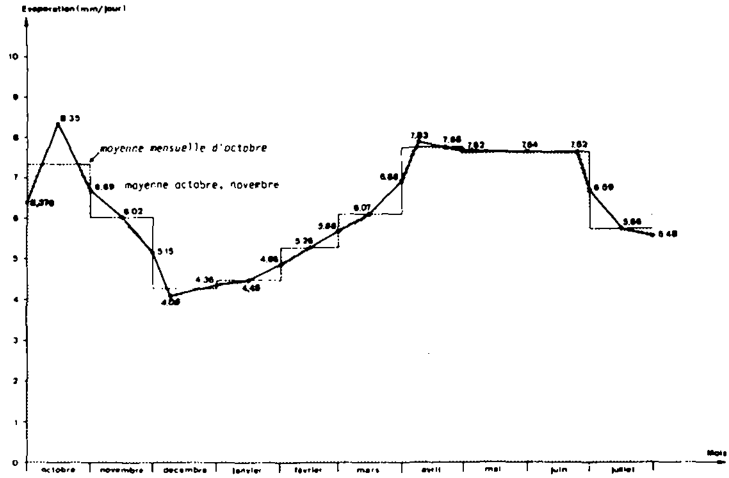


Fig.5 : Evolution moyenne mensuelle de l'évaporation du lac calculée sur base du bilan hydrologique et estimation de l'évolution intra-mensuelle.

Modèle mathématique

-Description

La formule de calcul permettant l'approche de l'évolution de la cote du plan d'eau, au départ d'une cote déterminée s'écrit :

$$H_{n+1} = H_n - \Delta H\ (Vpn) - \Delta H\ (VEn) + \Delta H\ (Vr_n) + \Delta H\ (Vp_{On})$$

où H_{n+1} = cote du lac au jour n+1;

H_n = cote du lac au jour n;

$\Delta H\ (Vp_n)$ = Hauteur d'eau correspondante aux pompages du jour n;

$\Delta H\ (V_{En})$ = Hauteur d'eau correspondante à l'évaporation du jour n;

$\Delta H\ (Vr_n)$ = Hauteur d'eau correspondante aux apports du jour n (rejets dans le lac des eaux de drainage des cultures irriguées);

$\Delta H\ (Vp_{On})$= Hauteur d'eau correspondante à la pluviométrie du journ.

Le passage des volumes apportés ou prélevés (Vp, Vr, Vp_0) à une différence de hauteur d'eau s'effectue par simple division de ces mêmes volumes par la surface du lac le jour n.
Pour la pluviométrie nous tenons compte des apports par ruissellement. Il est bien évident que le calcul de la pluviométrie ne peut s'appliquer lors de l'estimation prévisionnelle de l'évolution de la cote du plan d'eau en cours d'année. Elle doit donc à ce moment être considérée comme nulle, aucune moyenne interannuelle ne pouvant être précisée vu l'irrégularité et la précarité des pluies dans la région. En cours d'année cependant les calculs peuvent être réajustés en fonction des éventuelles précipitations.

-Résultats et commentaires

Le modèle de gestion quantitative des eaux, écrit en langage "Basic" est utilisé sur micro-ordinateur Apple II. Quatre approches du problème sont possibles :

Cas 1 : Calcul de la cote du lac en fin d'année hydrologique (ouverture du pont-barrage), d'après la cote en début d'année hydrologique et en incorporant la pluviométrie dans le modèle.

Cas 2 : identique au précédent mais sans intervention du facteur "pluies".

Cas 3 : Calcul de la cote du lac en début d'année hydrologique (fermeture du pont-barrage), d'après la cote en fin d'année hydrologique et comprenant les précipitations.

Cas 4 : identique au précédent mais sans tenir compte de la pluviométrie.

Les cas 1 et 3 permettent de tester la fiabilité générale du modèle puisque tous les paramètres du bilan hydrologique interviennent et d'apprécier la répétitivité de l'évaporation quotidienne moyenne d'une année sur l'autre. Les cas 2 et 4 sont en fait les deux options intéressant le gestionnaire du milieu puisqu'il s'agit de calculs prévisionnels ne tenant donc pas compte de la pluviométrie. Le cas 2 est utilisable dans la situation actuelle et le cas 4 le sera après la mise en service du barrage de Diama. Enfin la comparaison des cas 1 et 3 avec les cas 2 et 4 permet d'évaluer l'influence de la pluviométrie sur l'évolution annuelle de la cote du plan d'eau, et par là même l'erreur qu'engendre sa non prise en considération. Le tableau 1 regroupe les différents résultats obtenus sur six années hydrologiques (1976 à 1982) pour lesquelles nous disposons des données relatives aux pompages, apports et pluviométrie.

Tableau 1 : Evaluation de la cote du lac (en m) en début ou en fin d'année hydrologique d'après le modèle mathématique.

Année hydrol.	c. obs.	Cas 1 c.calc.	Cas 1 %erreur	Cas 2 c.calc.	Cas 2 %erreur
76/77	-1.04	-1.17	6.02	-1.19	6.94
77/78	-1.14	-1.09	2.13	-1.19	2.13
78/79	-0.50	-0.52	0.94	-0.61	5.19
79/80	-1.10	-0.96	6.51	-0.99	5.12
80/81	-0.65	-0.61	1.78	-0.63	0.89
81/82	-0.57	-0.56	0.43	-0.63	2.58
Moyenne			2.97		3.81

Année hydrol.	c. obs.	Cas 3 c.calc.	Cas 3 %erreur	Cas 4 c.calc.	Cas 4 %erreur
76/77	1.12	1.23	5.09	1.26	6.48
77/78	1.21	1.17	1.70	1.25	1.70
78/79	1.62	1.64	0.94	1.73	5.19
79/80	1.05	0.94	5.12	0.96	4.19
80/81	1.60	1.57	1.33	1.59	0.44
81/82	1.76	1.76	0.00	1.81	2.15
Moyenne			2.36		3.36

(c.obs. : cote observée; c.calc. : cote calculée; %erreur par rapport à la variation totale de niveau).

L'analyse du tableau 1 permet de dégager quelques conclusions :

-L'erreur moyenne par rapport à la réalité est de 2.6 % dans les cas 1 et 3. La fiabilité du modèle utilisé est donc très bonne d'autant plus que le coefficient de corrélation entre les 12 résultats observés et calculés est de 0.998 (c.calc. = 0.99 c.obs. + 0.01).

-La comparaison de ces deux mêmes cas permet de conclure à une bonne répétitivité annuelle de l'évaporation quotidienne du plan d'eau : l'erreur maximale n'étant que de 6.51 % (année 1979-80).

-L'erreur moyenne enregistrée aux cas 2 et 4 est faible (3.5 %) avec un maximum de l'ordre de 7 % en 1976-77. Ceci laisse donc supposer un emploi fiable des deux modèles prévisionnels. Les coefficients de corrélation entre les valeurs observées et calculées aux cas 2 et 4 sont respectivement de 0.947 et 0.969 (c.calc. = 0.93 c.obs. - 0.10 et c.calc. = 1.06 c.obs. - 0.04).

-La pluviométrie intervient peu dans le bilan général. La comparaison des erreurs moyennes entre 1 et 3 d'une part et 2 et 4 d'autre part montre en effet que l'erreur introduite par la prise en considération de ce paramètre n'est que de 1 % environ.

Tel qu'il est programmé actuellement, le modèle permet de suivre avec précision l'évolution du niveau des eaux du lac en fournissant la cote prévisionnelle de 10 en 10 jours. Ceci permet d'éventuels réajustements de niveau par des apports supplémentaires en cours d'année.
La fig.6 schématise l'évolution du niveau du lac, observé et calculé d'après le modèle, au cours de deux années hydrologiques (hypothèse 2) sans apport pluviométrique. On peut remarquer la bonne correspondance entre les deux courbes qui sont presque superposées.

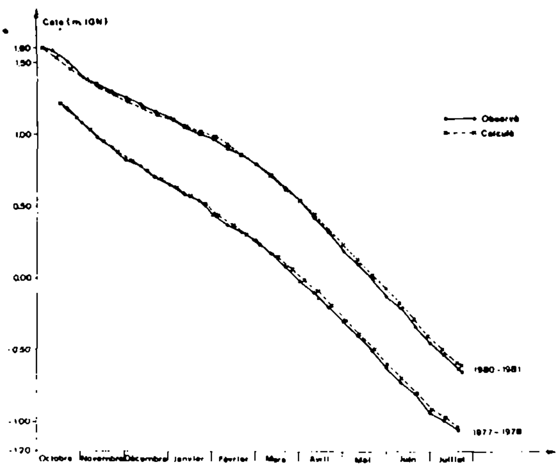


Fig.6 : Evolution observée du niveau des eaux du lac d'octobre à juillet, et calculée d'après le modèle mathématique, pour deux années hydrologiques.

CONCLUSIONS

Notre étude apporte quelques possibilités de réponses aux choix nécessaires dans la future gestion des eaux du lac après la mise en fonction du barrage de Diama. Le modèle devrait permettre d'épargner au maximum l'eau, élément fondamental de la survie dans la région. Il ne faut cependant pas oublier que si la maîtrise de l'eau est entre les mains des hommes qui seront amenés à la gérer, c'est le climat et son évolution dans les années à venir qui restera le facteur clé dans la résolution des problèmes aigus de développement au Sahel.

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WATER TREATMENT BY MEANS OF ACTIVATED CARBON PREPARED FROM LOCALLY AVAILABLE WASTE MATERIALS

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ABSTRACT

Man's natural water supplies are threatened by a large number of nonbiodegradable and toxic organic compounds. Proper environmental standards can often only be attained using tertiary treatment processes, such as adsorption on activated carbon. The latter is an expensive process, owing to the cost of activated carbon and the losses occuring during regeneration. Methods are being investigated at the Free University of Brussels for producing and activating carbon, using various types of waste as a raw material.
Both carbonization and activation have been conducted under the carefully controlled conditions of a fluidized bed reactor. The influence of the following process parameters has been investigated : pyrolysis temperature and time, activation temperature and time and composition of the furnace atmosphere.
The resulting activated carbons should exhibit the following qualities : high adsorption capacity and rate, good resistance to attrition, and the possibility of regenerating the carbon. The properties of the carbon obtained have been evaluated by means of a number of standard tests and adsorption isotherms. In general these properties were comparable to those of commercial qualities.
Some of the activated carbons tested,were evaluated using synthetic and real phenolic industrial wastewaters.

INTRODUCTION

Technologies converting locally available biomass into useful solid, liquid and gaseous products are attractive especially for developing countries.
In many of these countries agricultural and forestry residues, whether scattered in the fields and forests or available at local mills, remain without use. Several of these wastes (e.g. rice husks, sawdust, cottongin wastes, sugar mill wastes, cacao hulls, et.al.) are attractive raw materials for the production of activated carbon, e.g. by means of the activation process described in this paper.
In earlier studies several materials of tropical origin were converted to charcoals and eventually activated in a fluidized bed plant available at the V.U.B.

EXPERIMENTAL

Manufacture

The experimental unit (figure 1) consists of a gas preheater (2), a fluidized bed reactor (3) provided with separate electrical heating blankets in both the bed and freeboard section (4), a screw feeder (1) and a gas cleaning system. The latter consists of two cyclones in series (5), two shell and tube heat exchangers (6), a packed bed scrubbber (7) and a mist eliminator (8).
The reactor is constructed of Inconel 600 alloy and has an internal diameter of 15 cm at the bottom and 30 cm in the free-board area. Fluidizing gas (air and steam mixture) is introduced through a specially designed Inconel distributor plate. The reactor is insulated with Kaowool ceramic fibre. The bed materials consists of a graded sand fraction of 60 - 80 mesh. The main dimensions of the unit are given in table 1.

Table 1 : Main dimensions of the unit.

Diameter of the reactor :	- reactor zone	0.15 m
	- freeboard zone	0.30 m
Height of the reactor		1.00 m
Height of the bed		0.30 m
Steam flowrate		1 - 10 kg/h
Air flowrate		1 - 10 Nm^3/h
Heating capacity :	- preheating section	6 kw
	- fluid bed section	3 kw
Nominal capacity		10 kg/h
Temperature of the bed		up to 950 °C

Raw material

The charcoal used for activation was a commercial quality manufactured from sawmill wastes by Ets. Lambiotte, Marbehan. The proprietary manufacturing process features a vertical carbonizing furnace with forced internal circulation.
Table 2 and 3 give the elementary analysis and the proximate analysis of the two charcoal fractionsused.

Table 2 : Elementary analysis of the different size fractions on a dry and ash free basis.

average diameter (mm)	C	H	O[1]	N	S
3.3	90.0	1.7	8.2	N.D.	0.05
6.7	84.6	2.0	13.1	0.3	N.D.

1 by difference

Table 3 : Proximate analysis of charcoal fractions as received.

average diameter (mm)	3.9	6.7
Moisture (wt %)	7.1	7.5
Ash (wt %)	6.1	5.9
Volatile matter (wt %)	15.7	12.9

Measurements during manufacture.

The effluent gas composition was determined using a dual column Varian 3700 gas chromatograph. The permanent gases H_2, O_2, N_2, CH_4 and CO were measured on a molecular sieve 13 X column. H_2, CH_4, CO_2, C_2H_4 and C_2H_6 on a Chromosorb 106 column. Argon was used as a carrier gas. The bed temperature (12 different points) and the reactor-pressure were measured continuously.

Operating procedure.

The reactor was preheated by means of heating blanckets up to a temperature of approximately 500 °C, which is largely sufficient to ignite the carbon. Once the feeding of charcoal was started, the temperature rose rapidly to the final temperature (800 °C to 900 °C) and only minor corrections by the heating blanckets were necessary to maintain the reactor temperature at a constant value.
About 60 to 90 minutes after the start of the feeding, the reactor conditions were stabilized (for temperature and gas composition see figure 2 and 3). 150 minutes after the start of the feeding the steam and air supply was stopped and the reactor content was cooled to ambient temperature under a nitrogen flow.

Operating conditions.

The following process parameters were varied : temperature, steam flow, air flow and charcoal particle diameter. Table 4 gives a survey of the operating conditions.

Table 4 : Survey of the operating conditions during the continuous gasification of the charcoal.

Code number J x	Temp. (°C)	d_p (mm)	Steam flow (kg/h)	air flow (kg/h)	solid flow (kg/h)	varied parameter
J24	825	3,3	2,3	4,0	1,3	temperature
J28	850	3,3	2,3	4,0	1,3	
J30 }	875	3,3	2,3	4,0	1,3	
J23 }	875	3,3	2,3	4,0	1,3	
J9	825	6,7	2,3	4,0	1,3	temperature
J7 }	850	6,7	2,3	4,0	1,3	
J12 }	850	6,7	2,3	4,0	1,3	
J10	875	6,7	2,3	4,0	1,3	
J11	900	6,7	2,3	4,0	1,3	
J27	850	3,3	1,0	4,0	1,3	steam flow
J32	850	3,3	1,4	4,0	1,3	
J28	850	3,3	2,2	4,0	1,3	
J6	850	6,7	1,0	4,0	1,3	steam flow
J25	850	6,7	1,8	4,0	1,3	
J7 }	850	6,7	2,2	4,0	1,3	
J12 }	850	6,7	2,2	4,0	1,3	
J19	850	6,7	1,8	3,3	1,3	air flow
J16 }	850	6,7	1,8	4,0	1,3	
J25 }	850	6,7	1,8	4,0	1,3	
J8	850	6,7	1,8	4,6	1,3	

Evaluation of the activated charcoal.

The adsorptive properties of the activated charcoal were measured by standard tests such as the Iodine-index (AWWA B604-74), the methylene blue index (US standard and Norit procedure), the phenol value (DIN 19603), the tannin index (AWW) and the BET-surface (VUB). Moreover adsorption isotherms were determined for phenol, pentachlorophenol, p-toluene sulphonate, dodecyl benzenesulphonate and p-chlorophenol using the best samples (J11, J23, J32) of activated charcoal. Selected carbons were also used to determine multicomponent adsorption isotherms of a synthetic and real industrial wastewater containing phenol and hexachloro-endo-methylene tetra hydrophtalic anhydride.

RESULTS AND DISCUSSION

Adsorption capacity of the produced carbon.

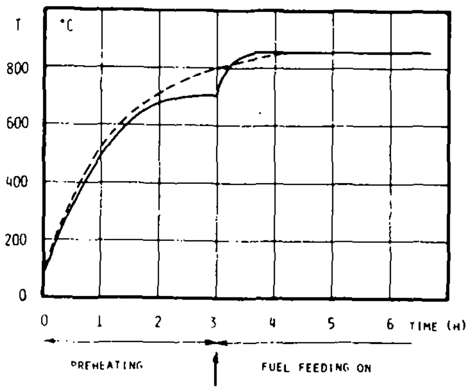
As follows from table 5, the obtained samples show excellent adsorption characteristics in comparison with well-known commercial carbons.

Also for the other mentioned adsorbates, the three selected carbons (J11, J23, J32) showed simular adsorption characteristics and capacities to these of the commercial F400.

Table 5 : Survey of the standard-indexes.

	GAC 40 (ceca)	F400 (calgon)	HD 4000 (Atlas)	J x (own carbon)
S-BET ($m^2\ g^{-1}$)	913	-	-	536-1300
Iodine index	1145	1118	558	717-983
Meth.Blue (NORIT)	12,4	25,3	11,7	17,5
Meth.Blue (U.S.)	170	-	-	89-170
Tannin index*	641	-	-	239-375
Fenol index	39,4	44,2	32,4	32,8-40,6
Freundlich n_{phenol}	3,14	3,05	3,50	3,2-3,90

* The lower the tannin index, the better the adsorption



—— Reactor temperature.
--- Freeboard temperature

Figure 2 : Temperature variation during an experiment.

Influence of the proces parameters on the adsorption capacity.

- Air flow. Within the studied range, no influence on the adsorption capacity was found. It is concluded that all oxygen is depleted while burning off external carbon surface according to a shrinking core model, whereas the internal pore surface is increased by means of the slower gasification reactions involving steam or carbon dioxide.
- Steam flow. When the steam flow is gradually increased, the activity rises to a maximum value, to drop sharply after attaining this maximum. A clear effect of particle diameter on activation and overactivation could not be established on the basis of the results available.
- Temperature. For both small and large sized particles it was essential to use an activation temperature of 850°C or higher, in order to obtain a stepwise increase in activity. The magnitude of the step was smaller

however for the larger particles.

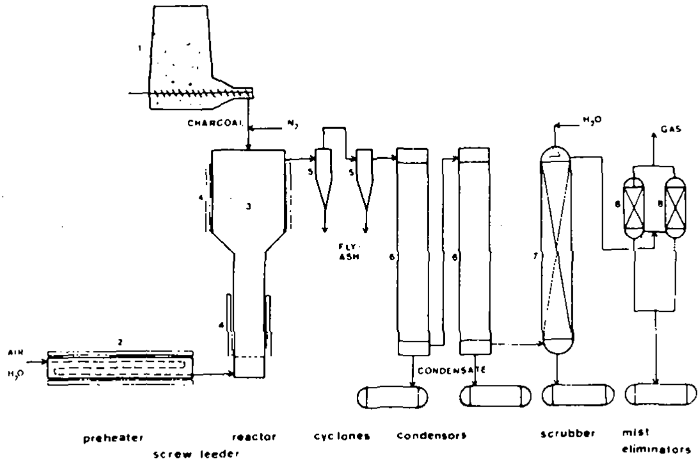


Figure 1 : Diagram of the test unit.

Mass balances and gas compositions.

A general scheme of the in- and outflowing streams is given in figure 4. This leads to the following mass balance.

IN : charcoal (C, H, O, N, Ash, Moisture)
Air (O, N)
Steam (H, O)
Nitrogen (N)

OUT : Gas (C, O, N, H)
Fly-ash in cyclones (C, Ash)
condensate (H_2O)
inerts in sand (<1 mm) (Ash)

The dotted lines represent mass flowrates which are unaccounted for. The total mass balance appears to be very good, with average closure of 99.8 %. Slightly larger deviations were found for the elemental carbon

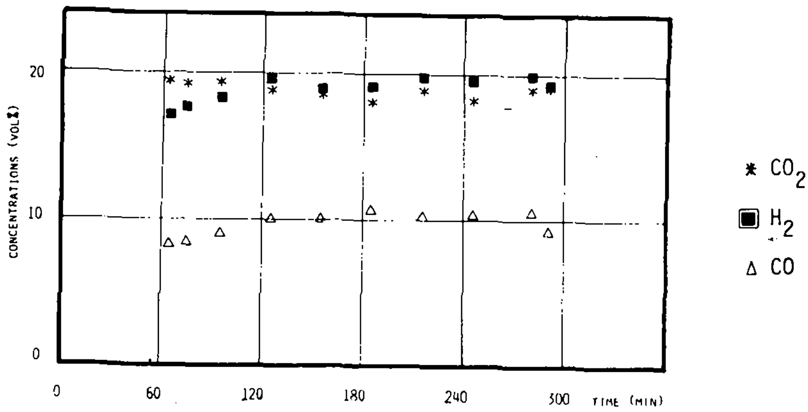


Figure 3 : Variation of the gas composition with the time.

balance, which showed a deficit of 3.6 %. (Table 6)

Table 6 : Closure of the mass balances (total and for carbon in %).

Nr.	Total	C	Nr.	Total	C	Nr.	Total	C
J6	95,6	88,7	J11	102,2	113,8	J24	99,8	89
J7	97,1	94,0	J12	100,9	106,5	J25	101,5	104,2
J8	100,2	100,6	J16	99,6	97,0	J27	100,7	86,8
J9	98,9	94,1	J19	97,9	86,5	J28	98,6	91,3
J10	100,4	104,8	J23	102,6	97,8	J30	100,2	97,0
J32	100,3	90,7	average = 99,8 %, 96,4 %					

The outlet gas composition is mainly determined by the bed and freeboard temperature. At the operating temperatures (800 °C - 900 °C) used in the experiments the water gas shift reaction reaches equilibrium, which fixes the gas composition for a given air, fuel and steam feedrate. (Table 7) The same table also gives the calculated higher heating value of the effluent gases, which is varying between 3.8 and 5.0 MJ/Nm^3. Hence a low BTU-gas is generated as a byproduct.

CONCLUSIONS

The fluid-bed unit, designed at the V.U.B., has proven to be suitable for the production of active carbon from different kinds of waste. Depending on the process conditions we have been able to produce fair to very good active carbon. The carbon proved to be useful in water treatment and is thus a high value product. Also starting from other materials, than this one mentioned in this paper, we obtained usable active carbon. Carbons of lower quality still could be grinded and used as a "low-activity" powdered carbon (P.A.C.T.-system). The production of active carbon from ligno-cellulose waste could be a positive factor in the economy of the developing countries.

Table 7 : gas composition, char holdup, gas velocity and higher heating value.

Exp. nr.	J6	J7	J8	J9	J10	J11	J12	J16	J19	J23	J24	J25	J27	J28	J30	J32
H_2(vol %)	13,3	14,7	13,0	13,5	15,9	17,9	15,7	13,4	15,8	14,5	13,6	14,8	13,8	14,1	15,2	14,5
N_2(vol %)	47,1	36,1	41,3	36,5	35,8	34,5	35,0	39,6	35,7	35,5	36,4	38,8	45,7	37,0	36,1	42,2
CH_4(vol %)	0,7	0,6	0,5	0,5	0,5	0,5	0,6	0,5	0,6	0,5	0,5	0,6	0,5	0,5	0,5	0,6
CO (vol %)	10,9	7,2	7,5	6,3	8,5	10,3	8,1	7,4	8,1	7,2	5,9	7,8	11,3	7,1	8,2	9,5
CO_2(vol %)	13,5	13,3	13,7	13,5	13,1	12,8	13,2	13,8	13,4	13,8	13,6	14,2	13,3	12,8	12,7	13,5
H_2O(vol %)	14,4	28,2	23,9	29,6	26,1	23,9	27,3	25,3	26,3	28,4	30,0	23,8	15,4	28,4	27,1	19,6
Char holdup(g)	819	473	521	644	386	283	578	550	562	346	572	609	676	421	343	748
U/U_{mf}	3,8	4,4	4,6	4,3	4,5	4,7	4,4	4,2	3,5	4,5	4,3	4,2	3,8	4,4	4,6	3,9
H.H.V.(MJ/Nm^3)	3,9	4,1	3,7	3,9	4,5	5,0	4,5	3,8	4,4	4,1	3,8	4,1	4,0	4,0	4,4	4,1

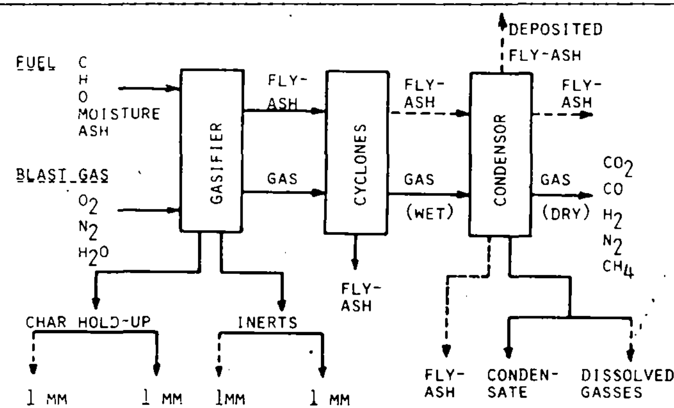


Figure 4 : In and outflowing streams for the gasification unit.

The required installations (gasifiers) are available in various types and sizes; however, local construction and maintenance with available materials and skills will be a problem in some places.

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IWRA	Vth WORLD CONGRESS ON WATER RESOURCES
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	WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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PRODUCER GAS FOR SMALL AND LARGE SCALE IRRIGATION IN RURAL AREAS

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ABSTRACT

In the rural areas of the developing countries, the availability of relatively small amounts of cheap energy is of vital importance. Thermal energy is needed for all kinds of heating purposes related to human activities such as cooking and crop drying while mechanical energy is required for small scale industry and agriculture such as irrigation pumps. Gasification of biomass in relatively simple plant can in some cases be the only realistic solution to these problems.
This paper presents a factual picture of the potential role of producer gas technology in developing countries. After a brief summary of the basic principles and the state of the art in producer gas technology some results obtained from a downdraft gasifier are presented and discussed. The problems associated with operating various engines with producer gas are also adressed with special emphasis to irrigation pumps and systems.

Keywords : Producer gas, gasifiers, engines, energy needs, rural areas, irrigation pumps.

INTRODUCTION

Biomass in the form of firewood, dried crop residues and animal waste is the fuel most extensively utilized in the rural areas of the developing countries. The most common use of biomass is in direct combustion at very low efficiencies in order to supply heat for essential human needs such as food preparation. At Present, the mechanical power needs of agriculture production are largely met through human labour and animal traction.

Energy needs of the developing countries.

Energy is universally recognized as essential for development. Greater amounts of energy are particularly needed in the rural areas of the developing countries in order to : a) provide essential amenities and improve the living conditions of people in remote areas through rural electrification, b) improve agricultural productivity by increasing irrigation capability, c) assist small workshops, d) provide alternative employment and e) improve road transportation.
The above goals can easily be met with the various petroleum products (cerosene, diesel oil, gasoline, fertilizers et.al); however, their costs are becoming economically prohibitive due to the price escalation of the last decade and to the fact that most developing countries have limited foreign exchange reserves.
Biomass conversion technologies which can convert the locally available biomass into useful solid, liquid and gaseous products are especially attractive processes in meeting the current and future energy needs of the developing countries.

Biomass availability.

A renewable energy system to supply the required energy for development is especially attractive. Since most of the developing countries lie in the tropics and already have economics largerly based on agriculture and forestry, the potentiel for biomass as a renewable energy source is great. Further more, a big supply of biomass is already available in the form of agricultural and forestry residues which to a great extent are currently wasted. Some of these residues remain scattered about in the fields and forests whereas others such as rice husks, sawdust, cottongin wastes, sugar mill wastes et.al are concentrated already at the various processing plants, which also are the most likely point of use.

TECHNOLOGY OVERVIEW

Biomass conversion technologies can be classified into three main types : anaerobic digestion, fermentation,pyrolysis and gasification. In the first process, organic materials are broken down by microorganisms in the obsence of oxygen to produce methane, while the second is a micro biological process to produce ethanol form a variety of sugar containing materials; Slesser and Lewis (1979), Wise (1981). Gasification (or starved air combustion) is a thermochemical process with producer gas being the main product.

Gasification/pyrolysis.

Pyrolytic conversion of wood into charcoal by means of simple earth-covered kilns has been widely used for many centuries; however this process is inefficient and does not allow the recovery of liquid and gaseous by products.

Modern processes involve the use of shaft furnace carbonizers with internal combustion and circulation of the evolved vapours, the recovery of which is uneconomic. They can be used to produce charcoal for use in small-scale car or truck-mounted gasifiers. The advantages of charcoal are : a higher calorific value per weight unit, low yields of tars and generally better flow characteristics.
In the fields of gasification and pyrolysis several promising processes have been developed during the past decade : a) continuous low-and high-temperature pyrolysis of wood and crop residues with various retort designes; Goldstein (1981), b) large scale oxygen gasification of wood to produce methanol; Blackadder and Rensfalt (1984), Lemasle (1984), and c) small scale gasification of wood and crop residues; Buekens and Masson (1980), Beenaekers and van Swaaij (1984). From the above only the latter is interesting for application in the rural areas of the developing countries, since high quality technical skills for operation and maintenance are not required in this approach.

Gasifiers.

Producer gas is formed by the partial oxidation of biomass in a gasifier, the most common type of which is a vertical flow packed bed reactor through which oxygen or air for combustion is passed downwards, upwards or accross the bed. The direction of the gas flow defines it as a downdraft, up draft or cross draft producer gas generator; Goss and Coppock (1980), Leuchs (1982). However lately fluidized bed gasifiers have attracted great interest and have been examined in detail; Maniatis and Buekens (1982) while a low cost low capacity fluidzed bed gasifier has been especially designed for the remote areas of the developing world; Flamigan and O'Neill (1984). A Comprehensive review of the state of the art of biomass gasifiers was presented by Beenaekers and van Swaaij (1984) while several authors (Gnmz (1950); Buekens and Schoeters, (1983); Belleville and Capart (1984); Shand and Bridgwater (1984) have developed thermodynamic and mathematical models for the prediction of the performance of the various types of gasifiers, with a good degree of agreement between experimental and theoretical results. A more detailed discussion on the fundamentals of gasification and modelling techniques is beyond the scope of this paper and thus the interested reader should consult the above papers.

APPLICATIONS

Producer gas from any of the above gasifiers can be used to either produce heat through direct combustion or power through an internal combustion engine. However, the former process is the simplest because extensive purification of the gas from tars and ash is not required and presently the most widely used. The main advantage of a gasifier close coupled to a boiler is its ability to operate at higher temperatures than conventional furnaces, which result in enhanced boiler efficiency and output.
On the other hand, producer gas of engine quality has to be clean (tar and ash free), with a rather high calorific value ($\pm$ 4000 kJ/Nm^3) and at as low a temperature as possible to improve the engine's gas intake and power output.
Most engines can be run on producer gas but due to the burning characteristics of the gas only low speed engines provide an acceptable power output. Diesel engines can be converted to gas engines rather easily by lowering the compression ratio and installing a spark-plug ignition. Otherwise a diesel engine can be run with a dual-fuel (diesel-gas) in which case the fuel is composed up to 90 % by gas. This system offers a great flexibility

in case of malfunctioning or maintenance of the gasifiers or even shortage of biomass.

Size classification .

Not every type of gasifier is appropriate for all applications not only due to its characteristics but also due to the size and capacity. A broad size classification is shown in Figure 1.

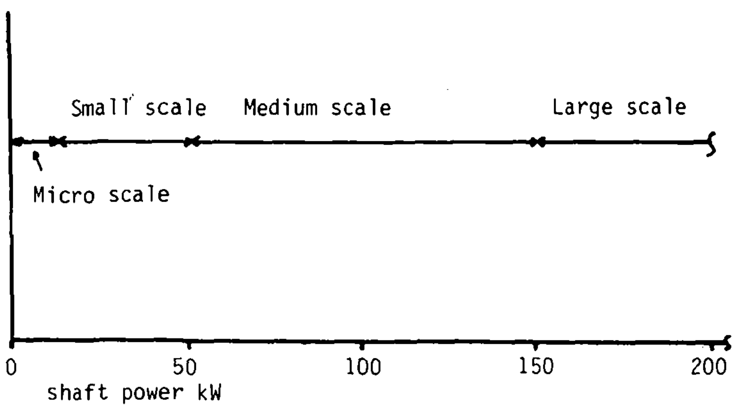


Figure 1 : Scale classification of gasifiers in terms of shaft power.

Large scale (150 kW and above) gasifiers are normally custom built and largely automated units and therefore it will be difficult to construct and assemble them in some of the developing countries. These units normally are an integrated part of bigger installations and are not suitable for rural areas due to the lack of skill and expertise.
In this scale range fluidized bed gasifiers dominate although other specialized fixed bed units are also used.
Medium scale (50-150 kW) gasifiers are to be applied in small to medium size forestry and agricultural related industry such as, wood industry, sawmills, industrial scale cereal mills et.al. Local manufacture is possible in most developing countries while in others parts of the unit can be constructed. In this scale range most types of gasifiers are applicable. Small scale (10-50 kW) units are mainly for rural applications such as ricemills, looms et.al. The equipment has to be cheap, reliable and require little maintenance and operating skills. These types of gasifiers should be simple enough to be constructed locally. Downdraft

moving bed gasifiers dominate this range. Finally the micro scale (1-10 kw) range is for daily life village applications such as irrigation, lighting, and for boat engines. These units must be cheap, transportable and simple in construction and maintenance. Local manufacture is very important to keep the costs low and provide training and knowhow to the inhabitants of the rural areas. Small downdraft units are to be expected in this size although Flanigan and O'Neill (1984), tested a fluidized bed gasifier for microscale applications.
However gasification equipment is commercially available in the large and medium size. On the other hand little interesthas been given to gasifiers for the small and micro scale, which applications are very important to the developing countries. The recent realisations of this prompted greater commercial and technical activity for equipment in this range.

The Engine.

In general, all internal combustion engines can be modified to be operated on producer gas; however the costs for such conversion vary depending on the original design of the engine. Operation with producer gas will result in a certain loss of power. According to Kyellström (1980), the main reasons for the power loss and methods to minimize it are :

- Lower heat of combustion of fuel air mixture; this is primarily important for gasoline engine where the power loss may amount to 30 % for this reason only
- Volume reduction (decrease of molecular number); whether for liquid fuels the volume increases by combustion, with producergas the volume decreases sometimes up to 10 %
- The polytropic exponent during compression is higher for producer gas; this leads to some loss of the working surface of the indicatordiagram.
- Timing advance gives increased compression work; advance timing is needed for producer gas operation because of the lower combustion velocity of the gas
- Pressure losses in the gas generator and intake manifold; a pressure decrease of 100 mmw is equivalent to a power loss of 1 %.

The total decrease in power amounts to about 45-50 % reduction for gasoline engines and to about 20 % reduction for diesel engines. Possible remedies are :

- compression increase
- replacement of intake pipe
- supercharging

The choice of engines for different power ranges has been discussed by Breag and Chittenden (1979). It was reported that spark ignition engines are preferable for sizes below 200 kW and that dual fuels diesel engines are preferable for sizes between 200 kW and 3mW. Gas turbines are recommended above 3 mW. However for this size, steam turbines become a comptetitive alternative.

EXPERIMENTAL RESULTS

The basic system for operating an irrigation pump or station with producer gas is shown in Figure 2 while Figure 3 presents the main equipment for that purpose.

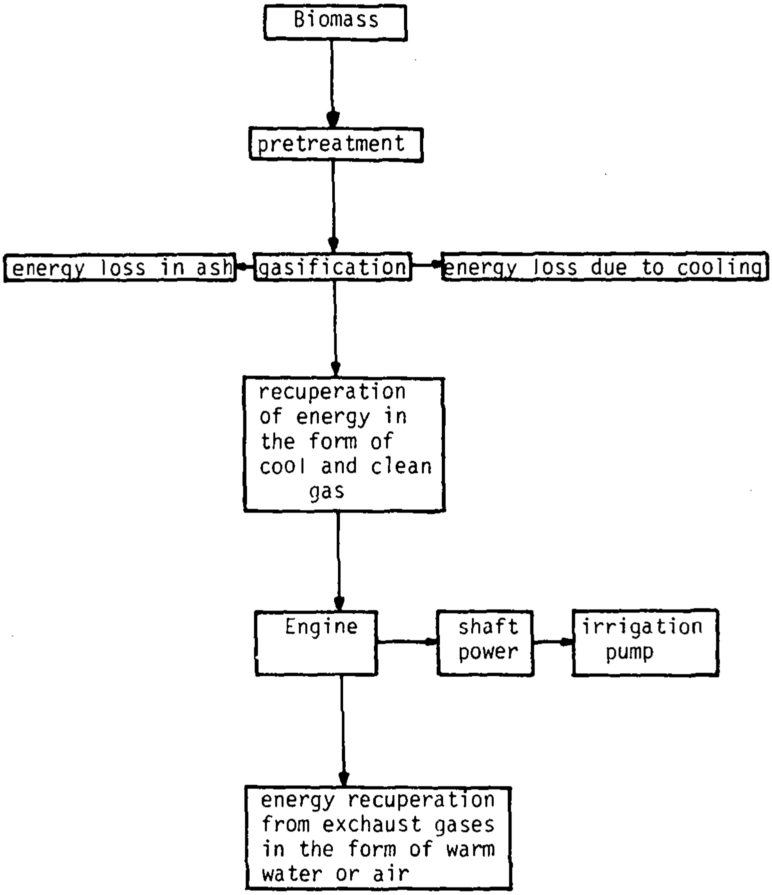


Figure 2: irrigation pump operating with producer gas

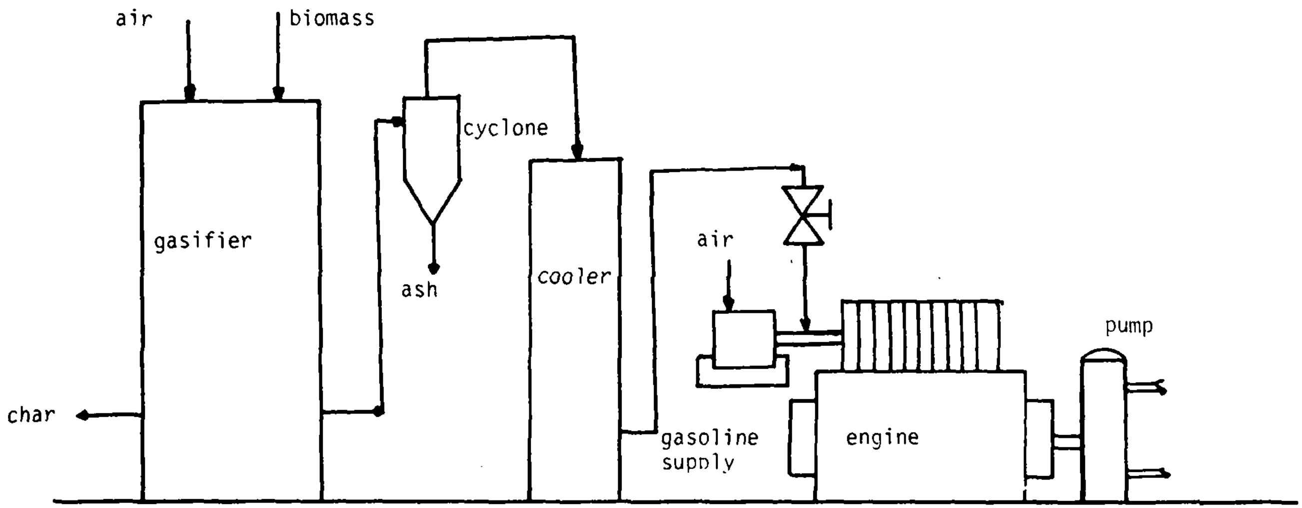
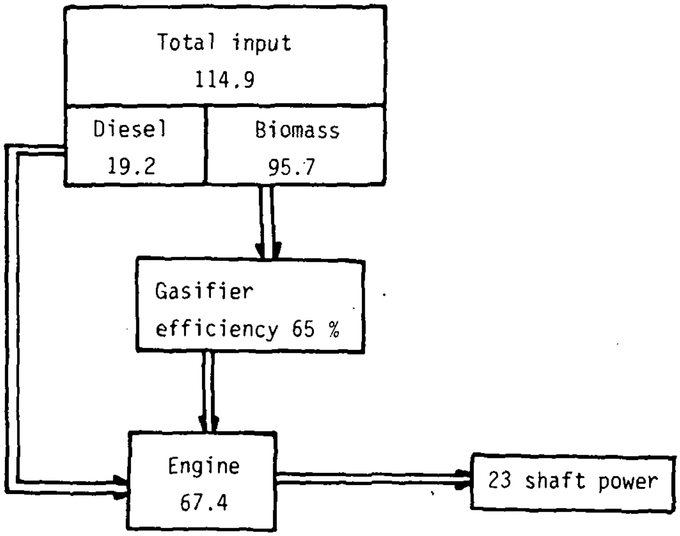


Figure 3 : gasifier and engine plant



All units in kWH

Figure 4 : Results on an engine run with producer gas

The results presented in Figure 4 were obtained by Vyncke Warmtetechniek of Hareelbeke Belgium during a R & D program in collaboration with the V.U.B. A downdraft moving bed gasifier of 25 kg/h biomass capacity was used coupled to a 3 cylinder Lister diesel engine of 20 kW output. This gasifier however is rather big for irrigation purposes in rural areas but could be used in association with another energy demanding unit (a small factory, for example).

CONCLUSIONS

Gasification of carbonaceous residues is a viable solution to the energy demands of rural areas. Gasifiers are currently available in various types and sizes. However special care has to taken so that gasifiers destinated for the rural areas are easy to construct and maintain with locally available materials and skills.
Internal combustion engines can operate with producer gas after certain modifications but with a loss in power. There is a little experience on engines run on producer gas and the problems associated with malfunctions and maintenance.

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L'EPURATION EN ZONE RURALE, ETUDE EXPERIMENTALE D'UN SYSTEME MIXTE

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Belgium

ABSTRACT

Semi-natural purification by intermediate technology ("Bertrix Project"). This experimental station has been chosen as a pilot project by the Belgian Government (Region Wallonne). The latest trend in natural wastewater treatment is to remplace traditional "stabilization ponds" by basins with emergent vegetation. The present experiment will use the complementary properties of aerated lagoons and such basins in order to reduce the total area of the installation. The aim of the emergent vegetation basin is not only to remove nutrients (N, P) and bacteria, but also to contribute largely to the removal of the organic matter.

The following botanical aspects will be studied thorougly :

- the vegetation as support for microorganisms;
- mineralization at root level;
- nutrient storage and removal.

The economic aspect of the project will be also examined.

L'étude du présent projet a reçu l'appui financier du Ministère pour l'Eau, l'Environnement et la Vie Rurale de la Région Wallonne.

Ce projet pilote pour la Région Wallonne est destiné à mettre en place une station d'épuration expérimentale pour une population variant entre 7.700 actuellement et 8.500 équivalents-habitants dans vingt ans dont 1.000 en provenance de l'abattoir communal.

Le site retenu est situé dans la province de Luxembourg, au lieu-dit "Les Grands Prés", le long de la route Bertrix-Cugnon, entre celle-ci et le ruisseau.

Il se compose de prairies et de terrains plantés d'épicéas. D'une étendue de plus ou moins 4 ha, il mesure environ 480 m de longueur (le long de la route) et 160 m (max.) de la route au ruisseau. Les dénivellations sont d'environ 13 m le long de la route et 16 m (max.) de celle-ci à la berge du ruisseau.

Notons que le réseau d'égouttage est du type unitaire : il comporte des parties relativement anciennes et peu étanches. De plus les eaux parasites (drainages, sources) représentent un volume d'eau important et conduisent à prévoir plusieurs déversoirs d'orage. Les eaux qui arriveront à la station seront donc généralement des eaux diluées.

PRESENTATION DU PROCEDE

1. Remarques préliminaires

Le problème de l'épuration des eaux usées en zone rurale ne se pose pas toujours dans les mêmes termes que l'épuration des collectivités urbaines. Les différences sont dues notamment aux caractéristiques propres des eaux usées rurales, à savoir : une charge essentiellement organique : une grande variation des charges et des débits : une dilution liée au caractère unitaire du réseau et accentuée par rapport aux eaux urbaines par les imperfections du système et les habitudes acquises précédemment : captage de sources, drainage de terrains, etc.

De plus le choix des systèmes épuratoires a, jusqu'à présent, été surtout guidé par les performances de ceux-ci en ce qui concerne l'abattement de la charge organique. Cependant, en zone rurale, où les rejets s'effectuent souvent dans de petites rivières, la qualité bactériologique (eaux de baignades) ou chimique (eutrophisation) des eaux rejetées peut poser des problèmes tout aussi graves que ceux liés à la seule charge organique.

Enfin les rendements relevés sur des petites stations de type "classique" indiquent que les performances théoriques de ces sytèmes sont souvent différentes de la situation rencontrée sur le terrain. Les raisons en sont diverses et notre but n'est pas de les discuter ici. Cependant il apparaît évident que, pour ces petites installations, la fiabilité de fonctionnement est plus importante que les rendements héoriques que l'on pourrait espérer.

Ces diverses raisons ont conduit des chercheurs de différents pays à

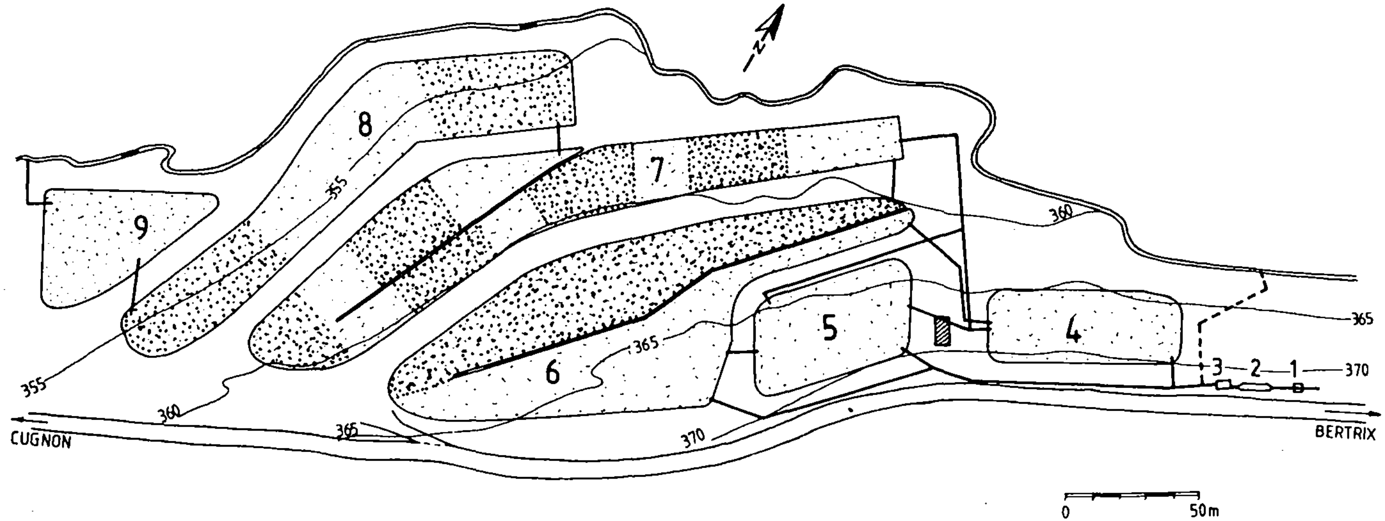

Fig. 1 : Schéma de la station expérimentale de Bertrix

étudier des solutions techniques mieux adaptées aux eaux usées rurales. Ces sy tèmes sont donc appelés à travailler avec des eaux souvent diluées, à subir sans trop de dommages des variations importantes de charge et de débit, à être très fiables et demander un entretien n'exigeant pas un personnel très qualifié. C'est ainsi qu'est apparu le lagunage. En fait, ce vocable recouvre des techniques multiples et des procédés parfois très différents dans leurs principes et dans leur comception. Les tendances les plus récentes dans cette épuration plus extensive conduisent à adopter des macrophytes pour en améliorer l'efficacité.

Le défaut que l'on attribue le plus souvent à ce type de système est l'utilisation d'une surface au sol importante.

Le but de l'expérience qui sera poursuivi à Bertrix consiste à coupler un systéme semi-intensif et un système plus extensif de manière à réduire la surface au sol tout en répondant aux mêmes exigences qu'un lagunage courant.

Le problème de l'épuration n'étant pas seulement technique mais aussi économique, nous v ulons également démontrer qu'une telle installation conduit à des frais d'investissement et, surtout, d'exploitation, bien moindres que les procédés classiques.

LES DIFFERENTES ETAPES DU PROCEDE

Pour décrire les différentes étapes du traitement des eaux usées nous suivrons le cheminement de celles-ci (fig. 1).

La convention F.U.L. - A.I.V.E. prévoit que le débit de 6 Q 18 doit être traité physiquement. Nous avons donc un dégrillage classique, (1) à nettoyage manuel. L'eau arrivera ensuite apres passage dans un dessableur (2) ensuite sur des grilles autonettoyantes (3). Ces grilles très fines remplaceront avantageusement un système de microtamisage ou un décanteur primaire : en effet, elles n'ont aucune pièce mobile et n'exigent pas de système de pompage.

Les déchets seront évacués, via une trémie, vers un bassin de réception où ils pourront être extraits par un grappin monté sur un camion. Les eaux qui auront accompagné ces déchets seront évacuées vers les lagunes aérées. Rappelons que l'écoulement des eaux se fait de manière gravitaire, aucune pompe n'est prévue pour assurer l'écoulement normal des eaux.

Nous avons ensuite le dernier déversoir d'orage qui est destiné à limiter le débit à traiter par voie biologique, soit 3 Q 18. Les eaux en excès sont envoyées directement à la rivière.

Les eaux passent ensuite dans un canal Venturi qui permettra les mesures de débit ainsi que les mesures physico-chimiques et biologiques nécessaires aux expériences ultérieures.

Les eaux sont ensuite acheminées, toujours de manière gravitaire vers deux lagunes aérées (4) et (5), chacune d'un volume de 3.850 m3. Dans un souci expérimental la première de ces lagunes sera aérée par 3 turbines d'une puissance de 4.4 kw. La seconde sera équipée d'un système d'insufflation linéaire d'une conception nouvelle, dont l'air est fourni par 2 surpresseurs d'une puissance de 6.8 kw.

Ces deux lagunes aérées pourront être alimentées tant en parallèle qu'en série, ce qui permettra de comparer ces deux systèmes de manière rigoureuse et sur un même effluent.

L'eau circulera ensuite dans une série de bassins plantés (6) (7) et (8) de macrophytes : différentes espèces de plantes pourront y être utilisées et testées. La géométrie des bassins a été étudiée pour éviter les zones mortes et leur profondeur pourra être modifiée, dans une certaine mesure, grâce aux moines de communication. Ces mêmes moines permettront la vidange complète des bassins à des fins de réparation, d'entretien, de curage, de plantation ou d'expérimentation.

Le dernier des bassins plantés (8) ne le sera qu'aux deux extrémités : la zone profonde du milieu, ainsi que le tout dernier bassin de la station seront des zones a microphytes. Ces deux derniers bassins pourront faire l'objet d'essais de pisciculture.

Il faut insister sur le fait que les lagunes à macrophytes et microphytes n'effectueront pas un traitement tertiaire. En effet le dimensionnement des lagunes aérées et la dilution des eaux permettent d'attendre un rendement sur la charge organique situé entre 50 et 60 % suivant le débit. Les bassins suivants doivent donc éliminer pratiquement la moitié de la charge organique restante, tout en réalisant un abattement du point de vue chimique (N et P) et bactériologique.

ASPECTS EXPERIMENTAUX

S'agissant d'une station expérimentale, divers objectifs doivent être poursuivis :

Prétraitement

L'objectif de la phase de mesure expérimentale pour cette étape, consistera essentiellement au test de 3 grilles autonettoyantes de maillage différents

- pour chacune d'entre elles, des essais de longue durée devront permettre de préciser les rendements de fonctionnement dans les conditions de fonctionnements variables
- les rendements seront établis par rapport à différents paramètres dont MES, DCO, DBO
- les aléas de fonctionnement seront mis en évidence (colmatage, gel, ...) dans les mêmes conditions de travail.

Lagunage aéré

Dans une première étape, une étude de l'hydraulique des bassins sera menée à l'aide de traceurs. Cette approche permettra de mieux connaître les modalités de mélange et de transfert propres à chaque lagune et induites par les différents systèmes d'aération (turbine, insufflation linéaire). Les points différents de mesures représentatifs pourront être choisis à partir de cette première étape; tant dans la masse d'eau qu'au niveau du fond pour les sédiments.

Une série de mesures sera menée pour préciser l'efficacité des systèmes d'aération. Des mesures en continu de la teneur d'oxygène, de la température seront menées dans les lagunes pour différentes conditions météorologiques (température, humidité, vent).
Les résultats de ces mesures seront pris en parallèle avec les consommations spécifiques des appareillages afin de préciser leur rendement énergétique.

Pour différentes conditions de fonctionnement (débits d'entrée variables), des mesures de l'activité biologique seront menées dans les lagunes aérées.

Des mesures de respiration dans la masse d'eau et au niveau des sédiments devront être effectuées en même temps qu'un suivi précis des paramètres pH, conductivité, oxygène, température pour l'effluent caractéristique de la sortie de ces lagunes aérées.
La variabilité de la qualité de l'effluent devra nous permettre en outre d'apprécier le pouvoir tampon de ces bassins vis-à-vis des variations de l'effluent d'entrée.

L'étude des corrélations de l'ensemble des paramètres à l'entrée dans les bassins et à la sortie, devrait permettre de déterminer les facteurs les plus utiles pour le contrôle ultérieur de ces installations.

Lagunage non-aéré

Comme dans les bassins aérés, une étude hydraulique préalable devra permettre de connaître le comportement de la masse d'eau dans ces lagunes (étant donné cependant la nécessité d'assurer une bonne reprise des macrophytes avant la mise sous eau, cette étape ne pourra pas se faire immédiatement pour chacune des lagunes).

Pour les différents points de mesure représentatifs de la masse d'eau et des sédiments les variations des paramètres O_2, pH, température, conductivité, seront observées en liaison avec des mesures de productivité (plancton et macrophytes).

Les éléments Azote et Phosphore seront l'objet d'une attention particulière à ce niveau du traitement. Des analyses des différentes formes de ces éléments permettront d'établir des bilans précis de leur évolution, pour

diverses conditions climatiques et hydrauliques.
Ces résultats devront être mis en relation avec les charges surfaciques et massiques de l'installation afin de distinguer autant que possible les interactions des conditions "naturelles" d'une part et des conditions "anthropiques" d'autre part.

L'objectif à moyen terme de ces observations résidant dans la recherche des conditions de fonctionnement optimales pour ces lagunes (charge surfacique maximale aux différents stades de traitement), un examen particulier de l'adaptation des plantes, de leur compétition et de leur rôle devra être mené en parallèle.

Traitement des boues

Un aspect fondamental tant du point de vue technique qu'économique de la gestion des stations d'épuration de cette taille est le traitement des boues. C'est la raison pour laquelle trois dispositifs différents seront testés à cette occasion

- deshydration sur container
- deshydratation sur lit de séchage
- deshydratation en lagune à boues

Les boues pourront être traitées aux polyelectrolytes.

Vu l'imprécision des estimations que l'on trouve dans la littérature il est essentiel de chiffrer l'accumulation des boues dans les différents bassins.

Pour satisfaire à ces objectifs expérimentaux, des moyens importants doivent être mis en oeuvre. De nombreux by-pass permettent de modifier le trajet des eaux usées et de choisir ainsi différents modes de fonctionnement. Des appareils automatiques de mesure pourront être placés à différents endroits pour effectuer des mesures en continu afin de pouvoir calculer des bilans précis. Pour les autres mesures, des échantillonneurs automatiques pourront prélever des echantillons avec des fréquences programmables.

Les paramètres classiques de l'épuration seront mesurés sur ces eaux aux différents stades du traitement
(MES, DBO, DCO, NH4, NO3, NK, PO4, PK, Bactéries totales,
Bactéries fécales...)
De la sorte le rendement d'épuration de chacune des étapes de l'épuration pourra être calculé et une comparaison entre les différentes filières permettra de définir le traitement optimum.

Aspects budgétaires

L'ensemble de la station représente un coût d'investissement d'environ 41.000.000 FB.

Compte-tenu des surcoûts importants liés aux objectifs de recherche, l'investissement est moindre que celui de systèmes conventionnels.

Cependant c'est surtout sur les frais de fonctionnement d'une telle station qu'on peut arriver à des différences considérables par rapport à d'autres systèmes.

L'expérience pilote porte donc autant sur l'étude du fonctionnement de ce système mixte que sur l'évaluation coût-performances de celui-ci. Le projet definitif de cette station a été déposé auprès des autorités compétentes en juin 1984. La réalisaton des stations devrait débuter dans le courant de 1985. Certaines collectivités n'ont pas attendu la fin de la période d'expérimentation pour adopter une technologie proche de celle exposée ici. Des exemples pourront être donnés au cours de l'exposé.

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LA GESTION DES RIVIERES EN PERIODE D'ETIAGE ET LES PREVISIONS HYDROLOGIQUES

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RESUME

Le bassin hydrologique de la Meuse est le plus important de la Belgique. Le débit de la rivière est extrêmement variable à proximité de la région liégeoise : 3000 m^3/s en période de crue exceptionnelle et souvent inférieur à 30 m^3/s pendant des mois d'étiage. Pour répondre à tous les besoins en eau de la population, de l'industrie et de la navigation, le débit de la Meuse doit être maintenu à 50 m^3/s en lâchant à un ou à plusieurs barrages-réservoirs le complément nécessaire. La présente communication expose le problème et traite les facteurs intervenant dans la détermination de la capacité des réservoirs à construire et de l'importance des lâchers d'eau à effectuer pour soutenir le débit naturel en période de sécheresse. Les limites opérationnelles et les possibilités de gestion sont examinées et leur lien avec la qualité des prévisions hydrologiques est établi. Nous démontrons aussi, qu'en première approximation, la capacité des réserves à établir doit être au moins de 150% du volume net du déficit de l'étiage le plus faible.

ABSTRACT

The most important hydrological basin in Belgium is that of the river Meuse. The discharge in the Liège industrial region is extremely variable reaching 3000 m^3/s during floods, but falling often below 30 m^3/s during droughts. To provide a satisfactory supply of water for the industry, population and navigation, a minimum discharge of 50 m^3/s must be maintained by releasing the necessary quantity from reservoirs. The paper describes the problem and discusses the criteria governing the volumes of reservoirs to be constructed and the discharges that should be released to maintain an acceptable flow in the river during drought periods. The operational limits and the management factors are examined and their close dependence on the quality of hydrological forecasts is shown. It is established, that the capacity of the reservoirs should be at least 150% of the net deficit in volume of the worst known draught.

INTRODUCTION

En Belgique on se plaint souvent à cause des nombreuses périodes pluvieuses. Il est vrai, qu'il pleut souvent, mais cela ne veut pas dire qu'il pleut beaucoup. La pluviosité annuelle moyenne est de 800 mm à 900 mm, descendant pendant les années sèches à 450 mm.

La Belgique étant un pays hautement industrialisé et de population dense (315 habitants au km²), la consommation en eau est en augmentation constante, et la gestion des ressources d'eau est d'actualité économique et politique depuis de nombreuses années.

LA CONSOMMATION D'EAU

Il est toujours intéressant d'examiner les prévisions d'il y a 15 ou 20 ans.

Un rapport du Commissariat Royal au Problème de l'Eau [2] a estimé en 1966 la consommation comme suit (en 10^6 m³) :

	1964	1980	Augmentation
Eaux de distribution	285	578	103 %
Eaux industrielles	277	455	64 %
Agriculture	133	235	77 %
Totaux	695	1268	82 %

En 1970, la Commission Interministérielle de l'eau [3] a modifié les prévisions pour 1980 (en 10^6) m³ :

Eaux de distribution	660	132 %
Eaux industrielles	900	225 %
Agriculture	165	24 %
Totaux	1725	148 %

Toutes ces prévisions n'ont évidemment pas pu tenir compte de la dépression économique et de la stagnation industrielle que nous avons connue depuis 1970, et qui doivent certainement influencer la consommation.

En outre, la Belgique a connu en 1976 une année exceptionnellement sèche. Pendant et en conséquence de cette sécheresse, de nombreux fermiers ont investi dans une installation d'irrigation. Ces installations continuent à fonctionner, augmentant la consommation agricole prévue.

En 1978 Wathelet [9] donne pour la consommation de l'eau potable une estimation de 615.10^6 m³, ce qui est assez proche aux prévisions de 1970. Aussi en 1978 Atquet [1] donne pour les eaux de distribution :

1973	540.10^6 m³
1990	724.10^6 m³

Concernant la consommation globale, on peut estimer qu'elle a doublé depuis 1965 et sera en l'an 2000 150% de la consommation de 1980.

LES RESSOURCES EN EAUX SOUTERRAINES

Mignion [7] donne une indication de la proportion de la consommation fournie par des ressources souterraines. En comparant ses données aux prévisions citées ci-avant, on peut constater que les ressources souterraines ne peuvent pas suivre la demande, et la Belgique doit tourner de plus en plus vers les eaux de surface (en 10^6 m^3) :

	1976	1980	1990	2000
Consommation	615	660	724	990
Ressources souterraines	416	428	440	445
Pourcentage	68%	65%	61%	50%

On peut remarquer que le pourcentage des ressources souterraines est assez élevé en Belgique, mais pendant que l'augmentation de la consommation entre 1980 et 2000 est estimée au moins à 50%, l'accroissement de la capacité des ressources souterraines pour la même période n'est que de 4%.

RESSOURCES DES EAUX DE SURFACE

Dans les années à venir, la gestion des ressources en eaux en Belgique doit être basé sur cinq éléments :

- amélioration du rendement des ressources souterraines
- accroissement de l'utilisation des ressources superficielles
- économies en consommation
- traitement intensif des eaux usées
- recyclage des eaux industrielles.

Nous allons examiner dans la communication présente le problème de la gestion des eaux superficielles, car il est évident que les ressources souterraines ne sont pas capables de suivre l'augmentation de la consommation.

Nous trouvons trois bassins hydrographiques en Belgique (Figure 1) :

- le petit **Bassin Cotière** de la rivière Yser (2240 km^2 ou 8% de la superficie du pays)
- le **Bassin de l'Escaut** (12240 km^2 ou 42%), ayant une pluviosité moyenne de 800 mm, répartie sur environ 160 jours de pluie par an. L'Escaut est une rivière à marée jusqu'à Gand et son débit moyen d'eau douce à Anvers peut être estimée à 80 m^3/s. Les eaux du bassin de l'Escaut sont fortement polluées, elles conviennent seulement à la navigation et à quelques usages industriels.
- le **Bassin de la Meuse** (14630 km^2 ou 50% du pays) ayant une pluviosité de presque 1000 mm, répartie sur environ 200 jours de pluie par an. Les précipitations sont fréquentes mais de faible intensité (seulement 20 à 40 jours avec plus de 10 mm de pluie) et elles sont très irrégulières.

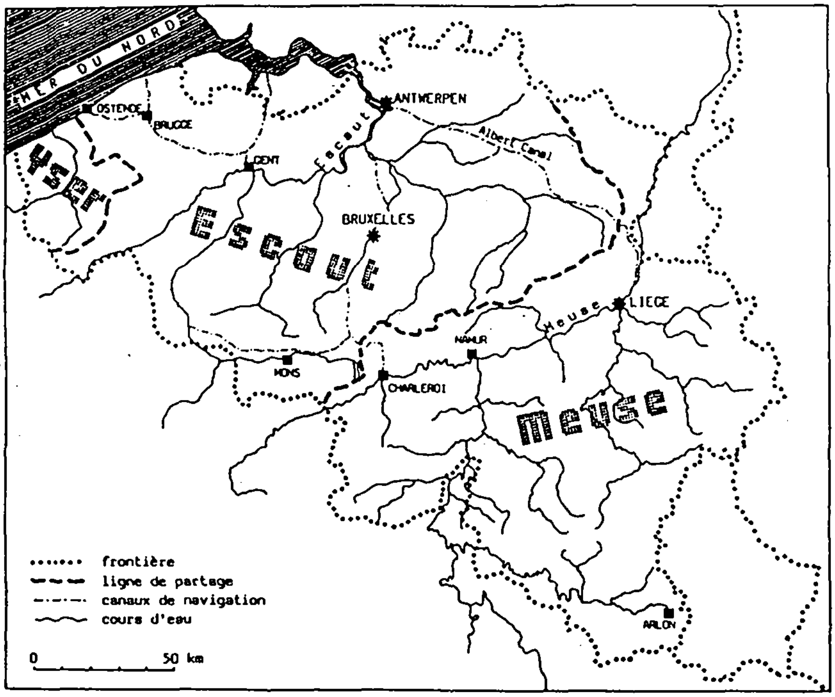

Figure 1. Bassins hydrologiques

PERIODES D'ETIAGE

C'est cette irrégularité des précipitations qui est à l'origine de la plupart des problèmes hydrologiques et particulièrement de celui d'assurer un débit minimum admissible à la Meuse pendant des périodes d'étiage.

Si le débit moyen de la Meuse, étant en période normale de 273 m³/s, est suffisant pour les besoins actuels et futurs, pendant des années sèches il peut tomber sous la valeur de 30 m³/s, ce qui est nettement insuffisant. Au courant des dernières 60 années, le débit est descendu sous 50 m³/s au moins en 18 occasions, les années les plus sèches étant :

Année sèche	Débit minimum (m³/s)	Durée d'étiage (jours)	Déficit de volume à 50 m³/s (10^6 m³)	Déficit moyen de débit (m³/s)
1921	31	128	145	13
1934	40	83	60	8
1947	28	122	137	13
1949	47	38	25	8
1955	44	75	50	8
1959	23	129	110	10
1962	35	16	6	4
1964	13	116	170	17
1971	13	79	74	11
1973	30	55	44	9
1974	36	11	6	6
1975	33	49	16	4
1976	17	164	284	20
1977	38	11	5	5
1979	39	16	6	4
1981	23	17	33	22
1982	13	73	20	3
1983	15	96	28	3

L'eau de la Meuse doit satisfaire une très grande variété de besoins :

- fournir de l'eau à la population et aux industries du bassin;
- assurer un débit garanti au Canal Albert, une voie navigable pour convois de 9000 tonnes reliant Liège à la mer via Anvers;
- fournir de l'eau à d'autres régions du pays y compris Bruxelles;
- maintenir un débit et une qualité d'eau convenable au passage de la frontière entre la Belgique et les Pays-Bas.

Voici quelques exemples des consommations spéciales des eaux du bassin de la Meuse :

- prises d'eau pour alimenter Bruxelles, une des dernières construites prenant 2 m³/s. Ces eaux quittent définitivement le bassin de la Meuse;
- la centrale nucléaire de Tihange, en amont de Liège, évapore par ses tours de refroidissement quelque 3,5 m³/s pendant les étiages, quand la centrale fonctionne en circuit fermé;
- la voie navigable du Canal Albert a besoin d'un débit moyen de 13 m³/s. Ce débit quitte aussi définitivement le bassin de la Meuse;
- 16 m³/s est déchargé par des écluses vers le Pays-Bas.

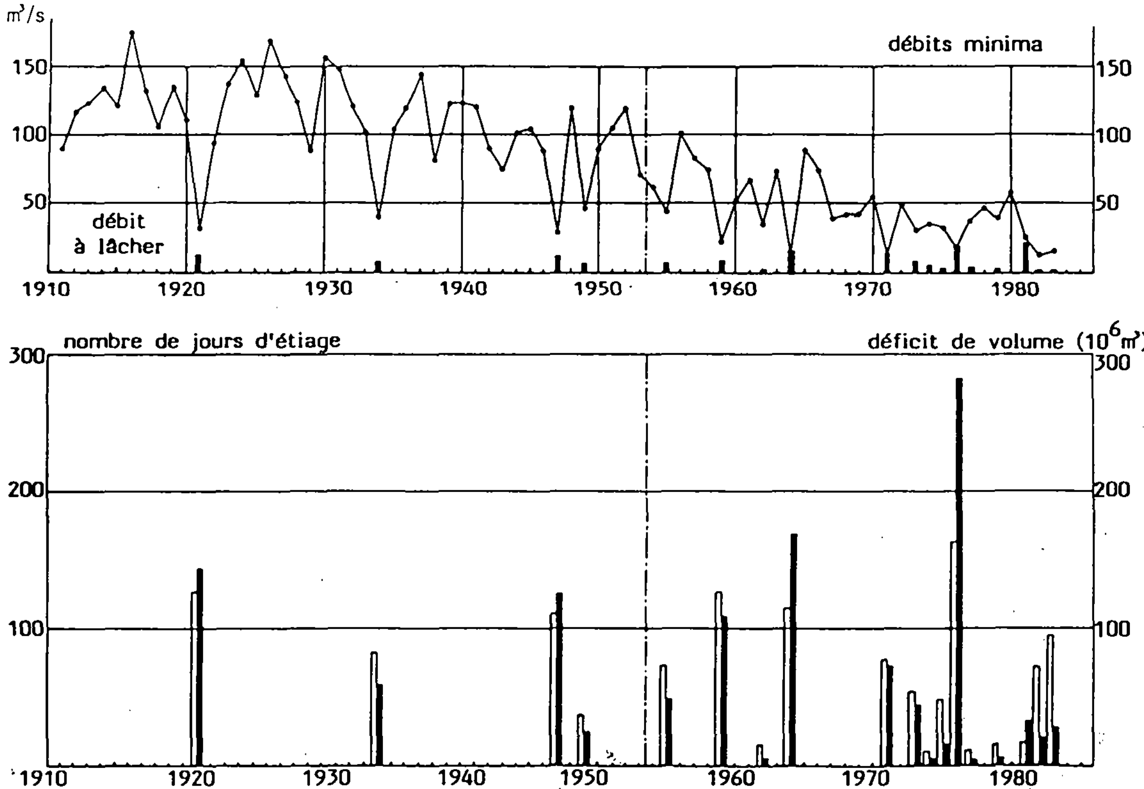

Figure 2. Débits minima. Débit moyen à lâcher. Durée d'étiage. Déficit de volume.

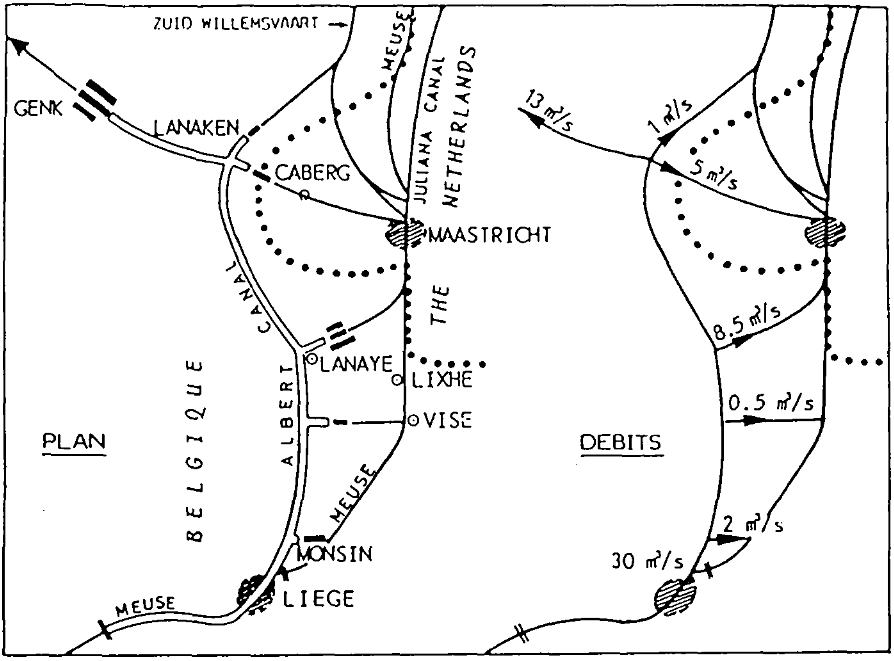

Figure 3 - Répartition des débits en aval de Liège

RESSOURCES EN EAU POUR L'AN 2000

En se basant sur des considérations que nous ne tenons pas de discuter ici, il a été estimé qu'il faut assurer dans la Meuse à Liège un débit minimal de 50 m³/s.

Un certain nombre de sites ont été étudiés dans la partie supérieure du bassin de la Meuse, en vue d'y construire des barrages-réservoirs (Figure 4). Ces réservoirs pourraient régulariser le débit de la Meuse et soutenir le débit d'étiage en lâchant le complément nécessaire pour maintenir 50 m³/s à Liège.

Toutefois, comme les sites les plus favorables sont à une distance considérable de Liège, l'eau lâchée à ces barrages peut mettre jusqu'à deux semaines pour arriver à destination. Cela veut dire que la gestion de l'étiage doit être basé sur des prévisions hydrologiques, car les lâchers doivent débuter deux semaines avant que le débit à Liège descende sous 50 m³/s. En outre, si le débit naturel de la rivière est inférieur à 50 m³/s, mais en augmentation, pour éviter tout gaspillage d'eau, nous devons choisir judicieusement le moment où les lâchers sont interrompus. En effet, si aux barrages on n'arrête le lâcher d'eau qu'au moment où le débit naturel de la rivière atteigne 50 m³/s, le volume correspondant à deux semaines de lâcher serait gaspillé, ce qui peut dépasser 20 millions m³. Cela montre l'importance d'une prévision hydrologique de qualité en temps réel.

En ce qui concerne les réserves d'eau à constituer, le premier et plus important critère est le déficit de la rivière, dans notre cas de la rivière Meuse à Liège. Les projets, étudiés entre 1965 et 1976 ont prévus une capacité nécessaire

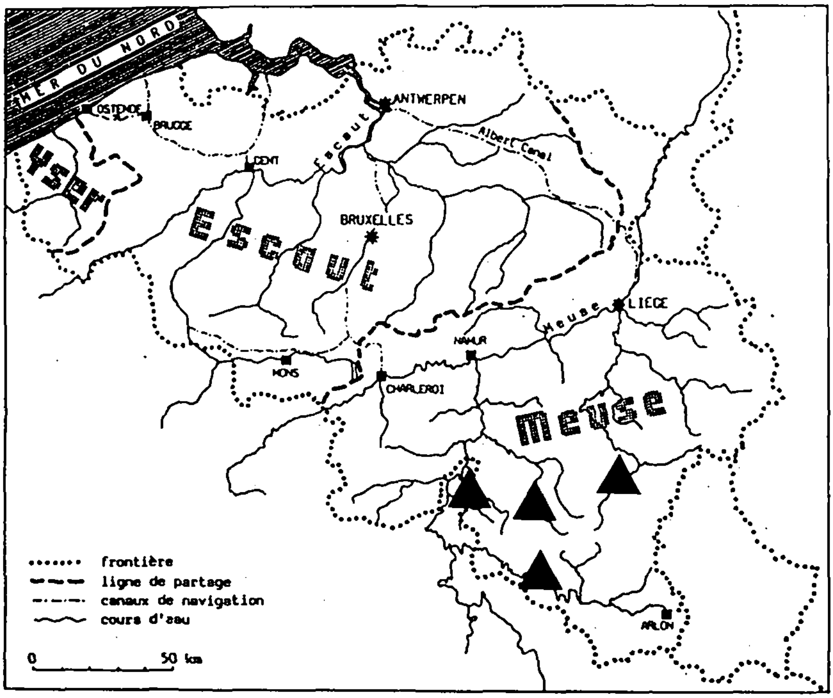

Figure 4. Sites pour barrages-réservoirs

de 200 millions m^3. Effectivement, on peut constater à la figure 2, que le plus important déficit jusque 1975, celui de l'année 1964 était de 170 millions m^3. A cette quantité net nous devons ajouter diverses pertes, telles que l'évaporation à la surface du réservoir et de la rivière, infiltrations et consommations en route, ainsi qu'une réserve qui doit rester derrière le barrage en permanence et pouvant être estimée à 10 % de la capacité totale. Tout compte fait nous arrivons effectivement à une capacité nécessaire de 200 millions m^3. Mais à la figure 2, on peut aussi voir que le déficit en 1976 a largement dépassé celui de 1964, nécessitant une quantité net (sans perte et réserve permanente) de 284 millions m^3, *ce qui rend caduc les projets prévoyant un total de 200 millions m^3 bruts.*

CRITERES DE GESTION DES ETIAGES

Les considérations traditionnelles de la capacité nécessaire et du déficit de débit ont été abondamment débattues au courant des années passées. Des consommations et déficits peuvent être mesurés ou estimés, des sites pour barrages-réservoirs peuvent être étudiés et comparés du point de vue technique, économique et écologique, mais il nous semble qu'un certain nombre de critères ont été jusqu'ici négligés.

Pour une gestion convenable des étiages des bassins, il faut prendre en considération quatre facteurs (pour illuster notre raisonnement, nous donnons des valeurs pour la Meuse) :

- *débit minimum naturel*
- nombre de jours d'étiage (au-dessous de 50 m^3/s)
- volume à fournir pour maintenir le débit minimum admissible pendant la durée de l'étiage
- débit moyen à lâcher pour maintenir le débit minimum admissible.

Le **débit minimum** minimorum d'un étiage n'a pas beaucoup de signification, bien qu'il fournit un diagramme impressionnant. Mais si ces débits ne persistent qu'un ou deux jours, ils ne justifient aucune intervention.

Par contre, si le débit reste pendant des longues **périodes** sous sa valeur minimale admissible, d'importants **volumes** seront nécessaires pour soutenir ce débit.

Un autre facteur important est **le débit moyen à lâcher** pour assurer un débit minimum admissible pendant la durée de l'étiage. Ce débit de lâcher va influencer la capacité nécessaire, car le décalage en temps entre un lâcher et son effet beaucoup plus en aval ne permet pas de suivre de près les variations horaires et même journalières du débit naturel.

Autrement dit, la capacité des réservoirs doit être considérablement supérieure au volume net nécessaire à maintenir le débit minimum admissible de la rivière et cela avant toute majoration pour pertes et réserves permanentes.

Si, par *exemple, le débit minimum admissible est 50 m^3/s et le débit naturel* de la rivière est autour de 35 m^3/s pendant plusieurs jours ou semaines, un complément de 15 m^3/s serait lâché des réservoirs. Mais si au milieu de cet étiage le débit naturel de la rivière remonte à 50 m^3/s ou plus pendant un jour ou deux seulement, suite à une averse isolée, les réservoirs devraient continuer à lâcher le débit moyen de soutien de 15 m^3/s à cause du retard entre une modification du débit au barrage et son effet à des centaines de kilomètres plus en aval.

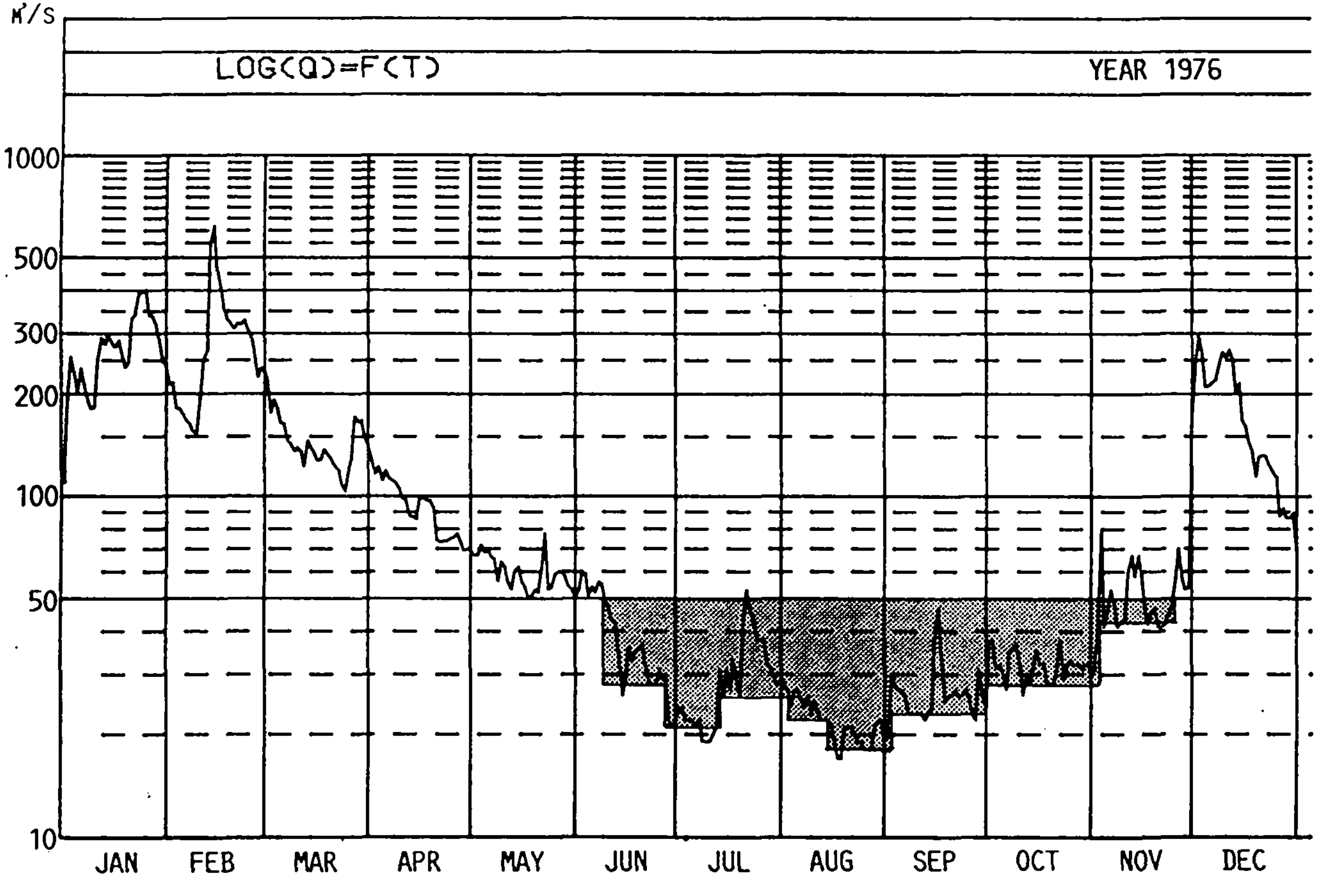

Figure 5 . Diagramme des débits naturels et des débits à lâcher en 1976.

Prenant l'exemple de l'année 1976, la zone tramée à la figure 5 représente le volume net nécessaire pour maintenir le débit à ou au-dessus de 50 m³/s. Cette solution admet qu'occasionnellement, au courant de 9 jours isolés, le débit descende sous 50 m³/s. Dans ce cas, que nous pouvons considérer minimal, le volume net à fournir à Liège est de 340.10^6 m³, un accroissement de 20 % sur le volume initial du déficit de 284.10^6 m³. Prenant en considération des pertes et réserves permanentes, nous arrivons à une capacité de 400.10^6 m³ à installer dans le bassin de la Meuse, juste le double de 200.10^6 m³ qui ont été envisagés dans les projets existants. Le tableau ci-après montre le calcul du volume net à fournir pour l'exemple de 1976 :

Période	Débit naturel moyen (m³/s)	Déficit sous 50 m³/s (m³/s)	Nombre de jours	Déficit de volume (10^6 m³)
Juin 9 - Juin 27	28	22	19	36,1
Juin 28 - Juil 12	21	29	15	37,6
Juil 13 - Août 3	26	24	22	45,6
Août 4 - Août 14	22	28	11	26,6
Août 15 - Sept 1	18	32	18	49,8
Sept 2 - Sept 30	23	27	29	67,7
Oct 1 - Nov 1	28	22	32	60,8
Nov 2 - Nov 24	42	8	23	15,9
Juin 8 - Nov 24, 1976				340,0

CONCLUSIONS

Ce raisonnement est applicable à tout bassin hydrologique où le débit naturel doit être soutenu par des lâchers à un ou plusieurs barrages-réservoirs. Il est le résultat d'une approche réalistique des possibilités et limites opérationnelles d'un tel système. Il est aussi démontré que le volume des réserves à établir est étroitement fonction de la précision des prévisions hydrologiques. Il doit être, en première approximation, au moins de 150% du déficit net en volume de l'étiage le plus faible.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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WATER RESOURCES MANAGEMENT IN MEDELLIN, COLOMBIA

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ABSTRACT

The City of Medellin, located in the Aburra Valley in the Colombian Andes, is a metropolitan area of 2.3 million people. With the population of the drainage basin projected to grow to 3.1 million by the year 2000, water supply and wastewater management problems are paramount. Severe flood damage, extensive erosion, and considerable sediment transport contribute to the area's water problems. Planning for the valley is presently focused on wastewater management, since only 40 percent of the population is currently sewered and there are at present no wastewater treatment facilities.

This paper presents the results of special studies conducted for the Empresas Publicas de Medellin as part of the Rio Medellin Sanitation Project. The studies involved an integrated assessment of the effects of water supply diversions, hydroelectric power diversions, point source discharges, solid waste practices, and nonpoint runoff on achievement of beneficial uses in the Rio Medellin. Potential beneficial uses of the river (including recreation, water supply, aesthetics, and irrigation) were developed to evaluate pollution control alternatives. Relative impacts of each factor were evaluated using a steady state river quality model, supported by comprehensive water quality sampling programs and hydrologic data analyses. River water quality was predicted under both high and low flow regimes, assuming various combinations of dilution water, point source controls, and solid waste controls. An evaluation of in-stream mining of sand and gravel was performed to determine the sediment contribution and resultant water quality degradation due to such practices. This effort was supported by a sediment sampling program, aerial photo interpretation, and an analysis of sediment transport capacity. Sensitivity analyses using the stream model were performed for each pollutant source under varying river flow conditions.

Keywords: Water pollution, river quality, flood control, nonpoint sources, water supply, wastewater management, erosion, solid waste.

INTRODUCTION

The City of Medellin, located high in the Aburra Valley of the Colombian Andes, is the center of a metropolitan area of about 2.3 million people. Although undeveloped land in the valley is limited, the population of metropolitan Medellin is expected to increase to over 3.1 million by year 2000, with the population of the Aburra Valley predicted to reach 4.0 million. Figure 1 shows the Aburra Valley, the City of Medellin, and other communities in the valley.

Geographically, Medellin is located about 7 degrees north latitude at an elevation ranging from 1,400 to 3,000 meters (m) above sea level. As in many tropical areas of higher elevation, it has two wet and two dry seasons. The economy of the area is driven by a substantial industrial base that includes heavy equipment fabrication, textiles, mining, chemical production, and breweries. Agriculture is also important to the valley.

Because of the abundance of high quality water resources in neighboring undeveloped river basins, Medellin has supported its growth over the decades through transbasin diversions. Such diversions for water supply provide about 75 percent of the total demand of the valley at present. This percentage will increase over the next 10 years as additional diversions from adjacent watersheds are constructed for water supply and hydropower purposes.

Because the water of the Rio Medellin, which bisects the city, has not been necessary for water supply, the river and its tributaries have been employed as receiving streams for all the raw wastewater of the domestic, commercial, and many industrial discharges of the valley. Although 96 percent of the city is served by sewers, only 50

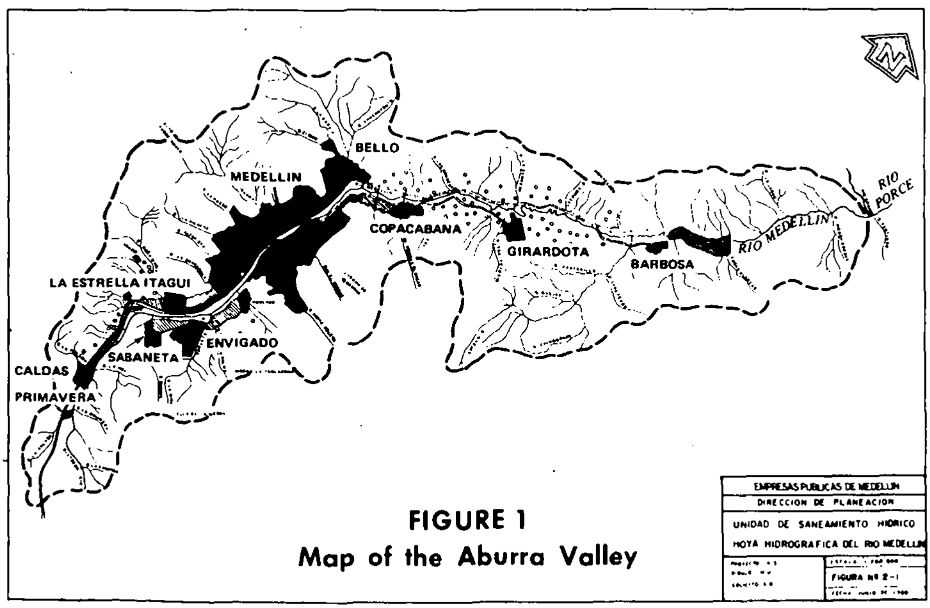

FIGURE 1
Map of the Aburra Valley

percent is connected to a collection system, which runs parallel to the Medellin tributaries and the river itself. Areas outside this collection system discharge directly to the river or its tributaries. The heavy pollution loads in the river now preclude its use for water supply, or virtually any other beneficial use. In addition, the natural aesthetic beauty of the valley is compromised by the odors and visual impact of the river. Visual blight, caused by the uncontrolled dumping of solid waste into the river, has also resulted from the perception of the river as merely a waste transport system.

Other problems also inhibit usefulness of the river. The hydrography of the Aburra Valley is such that large amounts of sediment are delivered to and transported by the Rio Medellin. Some of this material is deposited within the stream and has provided attractive locations for mining of gravel from the bed and banks of the river. Such sand and gravel mining can destroy the hydraulic and aesthetic characteristics of the river and/or its tributaries, while the poor water quality contributes to health problems among workers.

Also, a disadvantageous combination of land use, rainfall characteristics, topography, and other factors make urban nonpoint source pollution a major deterrent to achieving beneficial uses of the river.

In combination, the above factors provide a formidable obstacle to water quality improvement. Nevertheless, the Empresas Publicas de Medellin--responsible for water supply, electric services, and wastewater management for the valley--has begun a major program to restore, step by step, the uses of the river. The following sections describe the methodologies and results of special studies performed as part of an overall prefeasibility level analysis of alternative controls for improvement of river quality.

PROJECT APPROACH

Potential beneficial uses of the river, along with associated water quality criteria, were identified to allow evaluation of pollution control alternatives capable of supporting various uses. Relative impacts of each factor were evaluated using a steady state river quality model, supported by comprehensive water quality sampling programs and hydrologic analyses. River water quality was predicted under both high and low flow regimes, assuming various combinations of dilution water, point source controls, and solid waste controls. An evaluation of in-stream mining of sand and gravel was performed to determine the sediment contribution and resultant water quality degradation due to such practices. This effort was supported by a sediment sampling program, aerial photo interpretation, and an analysis of sediment transport capacity.

Beneficial uses and criteria

To direct public works funds to projects that achieve the greatest improvement in river quality and resulting beneficial uses, two series of activities were accomplished in parallel during the study.

First, desirable beneficial uses of the Rio Medellin were identified and then linked to necessary water quality criteria. For instance, the recreational use, as defined by local recreation practices (which, it must be realized, are somewhat different from definitions of recreation in other areas or countries), was deemed to be technically attainable and was assigned BOD5, dissolved oxygen, pH, and solids criteria necessary for sustaining that use. Preliminary criteria for uses ranging from drainage to full body contact were developed.

A separate source of benefits related to river quality pertained to improvement of public health. In a separate study, health benefits were estimated for a specified level of in-stream pollution indicators, including coliforms. Preliminary results of this study indicate, however, that public health benefits would be low relative to other categories, primarily because the local population's activity near or in contact with the river has been almost eliminated due to its poor quality and offensive odors. The exception is the relatively small number of laborers who manually mine sand and gravel from the river.

Concurrently with assessing possible use benefits, a program was initiated to ascertain the actual quality of the river and relative pollutant loads contributed by various sources. Several activities were necessary to understand the processes occurring in the Rio Medellin and its tributaries.

Hydrologic studies

Both low- and flood-flow frequency analyses were performed using the stream gaging data accumulated over the study area. Although the period of record was short (averaging approximately 16 years), the areal coverage was good and enough correlation was present to provide flow data for subsequent special studies. Because a large percentage of river flow is wastewater fl w originating from interbasin diversions, flow records had to be separated into natural and man-generated components to determine actual frequencies. Also, for flood flows in ungaged subwatersheds, synthetic methods applicable to the Aburra Valley had to be developed.

Water quality sampling

The dual wet and dry seasons in the valley required that existing in-stream quality be determined via sampling programs for both wet and dry conditions. Dry weather sampling runs were conducted over a 3-month period, both to characterize general river quality and to provide data for calibration and verification of a computer simulation model of the river. Wet weather water quality sampling included both in-stream and land surface sampling necessary for characterization of nonpoint source pollution loads. Land surface sampling involved measurements of overland and gutter flow from multiple land use types. Suspended sediment sampling was conducted during both low and high flow periods at several sites, as described below. Additional sampling programs were conducted to characterize wastewater discharges from domestic, commercial, and industrial sources; to determine amounts and composition of solid waste generated in the valley; and to estimate pollution loads due to surface runoff and groundwater seepage from the municipal solid waste landfills.

Water quality modeling

The computer model QUAL-II was adapted for project use to quantitatively predict the response of important water quality parameters to future control of pollution sources. QUAL-II is a steady-state, deterministic model developed under the sponsorship of the U.S. EPA. It is a comprehensive and versatile stream water quality model that can simulate up to 13 water quality constituents in any combination desired by the user.

The model is applicable to dendritic streams that are well mixed. It assumes that the major transport mechanisms--advection and dispersion--are significant only along the main direction of flow (longitudinal axis of the stream or canal). It allows for multiple waste discharges, withdrawals, tributary flows, and incremental inflow. It also can be used to estimate dilution flows required for flow augmentation to meet any prespecified dissolved oxygen level.

Hydraulically, QUAL-II is limited to the simulation of time periods during which the stream flows in the river basin are essentially constant. Input waste loads must also be held constant over time. QUAL-II can be operated as a steady-state model or a dynamic model. Dynamic operation makes it possible to study water quality (primarily dissolved oxygen and temperature) as it is affected by diurnal variations in meteorological data.

QUAL-II can be very helpful as a water quality planning tool. It can be used to study the impact of waste loads (magnitude, quality, and location) on in-stream water quality. It can also be used in conjunction with a field sampling program to identify the magnitude and quality characteristics of nonpoint source waste loads. By operating the model dynamically, diurnal dissolved oxygen variations due to algae growth and respiration can be studied. Dynamic operation also makes it possible to trace the water quality impact of a slug loading, such as a spill, or of seasonal or periodic discharges. Other advantages of the model are that it is relatively inexpensive to run, it has optional meteric and English versions, and it is easily linked with an interactive plotting package, which allows direct graphic presentation of results.

Analysis was performed with the verified model to determine alternative point source abatement strategies for a planning period that extended to the year 2000. Preliminary results indicated that a configuration of four treatment plants sited along the Rio Medellin, as shown in Figure 2, would meet selected water quality criteria for the year 2000. The proposed configuration consists of two large secondary plants and two additional plants defined as providing preliminary treatment (screening, grit removal, and chlorination).

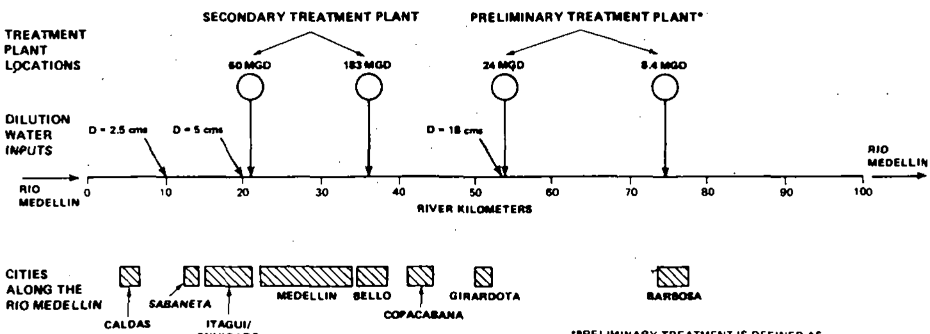

FIGURE 2

Point Source Abatement Strategy

SPECIAL STUDIES

Predicting future water quality conditions required many assumptions and projections regarding the expected "state" of the environment in the year 2000. The following were the major assumptions made for the initial Baseline Simulation:

- A flow likely to be exceeded 75 percent of the time was assumed as the design flow.
- Seventy-five percent of the domestic, industrial, and commercial wastewater in the valley will be collected and treated, with 95 percent collection and treatment of industrial waste from several major industries in the valley.
- High quality dilution water will be available to the Rio Medellin at specified locations.
- A solid waste management program that essentially eliminates pollution from solid waste dumping will be implemented.
- Nonpoint sources of pollution will be minimal under baseline conditions.

To better define the ability of the point source abatement program to meet water quality criteria and achieve beneficial uses, special studies were conducted to determine contributions from various pollution sources and the impacts of the assumptions stated above.

Low flow studies

The selection of a design flow for use with the QUAL-II model was a major factor in the analysis. It is more difficult and costly to achieve in-stream quality during low flow periods. A low design flow means that treatment plant effluent is a greater percentage of in-stream flow. Currently, no local institutional or legal/regulatory constraints exist in Colombia to guide the selection of this flow.

An analysis of extreme low flow events, the usual procedure for determining a design flow, was not possible due to the short length of records available. The period of record was 2 to 10 years, with an average of 6 years at each gage. Therefore, for purposes of screening abatement strategies, a flow exceeded 75 percent of the time was assumed for the analysis.

To demonstrate the sensitivity of this assumption on predicted water quality, the QUAL-II inputs were modified to reflect a more severe flow condition, the average annual 14-consecutive-day low flow. The reduced flow corresponds to an exceedance frequency of 90 percent. The effect of modifying the design flow results in minimal impact on oxygen reserves due to the large quantities of dilution water and sanitary contributions to the total flow in comparison to the smaller "naturally" occurring base flow portion. The predicted maximum DO sag level was less than 0.5 mg/l more severe. The predicted in-stream BOD5 concentrations are only minimally affected.

With a knowledge of local hydrology, coupled with the low sensitivity of water quality to changes in the base portion of the design flow, it was evident that further work in establishing a more restrictive or reliable design flow was not a high priority. Water quality was not particularly sensitive to changes in this factor.

Collection system studies

As indicated earlier, though there are no treatment facilities in the Aburra Valley, portions of Medellin and the surrounding municipalities are sewered. In 1957, a plan was completed for construction of a sanitary collection and treatment system for the City of Medellin. Presently, approximately 50 percent of the 152 kilometers of major interceptors originally planned are constructed. A major problem is that many of the trunks and smaller domestic lines are not connected to the main interceptors, and necessary manholes and cleanout points are not built.

A large-scale program for additional sewers, system connections, manholes, and overflow outlets is necessary to achieve 75 percent wastewater collection in the valley. A separate special study was conducted to determine the collection and transport system needs of the city. Theoretically, wastewater from any industry or residence can be collected if cost is not a factor. The intent of the study was to determine the collection percentage most feasible from an engineering and cost standpoint. The topography of the region, as well as the condition of existing dwellings, precludes economical collection of a certain percentage of waste flows. Many of the houses in the valley, especially in the upland areas, are without plumbing or public water supply.

The QUAL-II model was used to test the sensitivity of predicted river quality to uncertainty in the collection percentage. The ability to meet minimum water quality criteria was shown to be strongly influenced by the amount of sewage that can be collected and treated before release. As a result, a program to collect at least 75 percent of the wastewater flows, especially in heavily populated Medellin, is a critical factor in achieving water quality goals.

Dilution water studies

Current planning calls for the diversion of water from the neighboring Rio Grande watershed to the Aburra Valley for hydroelectric power, water supply, and river dilution purposes. The proposed diversion includes the building of gravity tunnels to the municipalities of Bello and Girardota. The plan also includes construction of two hydroelectric power plants as well as a water treatment plant. An average of 35 m^3/sec (1,236 ft^3/sec) would be diverted by the year 2000. Although approximately 2 m^3/sec (424 ft^3/sec) would be used to meet increasing water demand, the remaining 23 m^3/sec (812 ft^3/sec) would be discharged into the river as dilution water after power generation. The Baseline Simul tion accounted for the above by establishing inflows to the Rio Medellin of 5 and 18 m^3/sec (176.6 and 636 ft^3/sec) at river kilometers 30 and 54, respectively. An additional 2.5 m^3/sec (88 ft^3/sec) will be available from another source at kilometer 20.

Consequently, the availability of dilution water (and hence river quality) is dependent on completion of the proposed diversion plan. Should a decision be made not to build the Rio Grande Diversion Project, the future quality of the Rio Medellin could be significantly changed. To test the sensitivity of water quality to the possibility of no dilution water, all dilution flows were eliminated from the QUAL-II model and a new simulation was run. The BOD5 concentrations remain elevated through the City of Medellin and only begin to decline downstream as the organic load is diluted, river flows increase, and BOD5 inputs are reduced. Upstream of river kilometer 20, predicted water quality is the same for the baseline and modified simulations. The simulation of DO is shown to be very sensitive to the availability of dilution water. Without added dilution water, a major zone of anaerobic conditions is likely to develop, even after construction of expensive collection and treatment facilities.

The importance of dilution water to meeting future water quality criteria is therefore large, particularly near the population center of Medellin, between river kilometers 15 and 35.

Solid waste studies

In 1980, the total quantity of solid waste produced in the Aburra Valley was estimated to be approximately 460,000 metric tons. The collection, treatment, recycling, and disposal of this solid waste is a complex problem in Medellin and the surrounding municipalities. Although a significant portion eventually reaches a controlled landfill or the composting plant for ultimate disposal, in many areas of the valley the location and methods of disposal are determined by convenience. A common practice is to dispose of solid waste on the banks of the Rio Medellin or its tributaries. Eventually, disposal sites adjacent to water courses extend into the channel and waste is carried downstream during high flow periods, causing widespread visual and odor problems and degradation of in-stream water quality.

Implementation of a solid waste management program is intended for Medellin. For planning purposes, it was assumed that such a program would essentially eliminate solid waste as a source of water pollution. Therefore, the estimated contribution of solid waste to the total pollution load to the river from all sources was excluded as input to the QUAL-II simulation for the Baseline Simulation.

However, this assumption involves a significant change in solid waste disposal practices in the valley. Solid waste recycling provides income to thousands of people throughout the valley. Changing methods of disposal may have serious socioeconomic impacts, and therefore implementation of a program to eliminate solid waste pollution may be difficult to achieve.

As a result, the sensitivity of future water quality to a failure to implement a solid waste program was also tested with QUAL-II. Solid waste is assumed to enter the river as leachate from solid waste sites (a relatively minor source of pollution), through direct dumping, and by entrainment with the runoff. Modeling results show elevated BOD5 concentrations at almost twice the baseline concentration along the entire modeled portion of the river. The dissolved oxygen sag is approximately 1 mg/l more severe at the point of minimum DO concentration. Recovery of oxygen to concentrations similar to the baseline condition occurs downstream of kilometer 51 (mile 32) because of the addition of 18 m^3/sec (636 ft^3/sec) of water for dilution. Meeting water quality criteria is thus shown to be very dependent on an effective solid waste management program.

Nonpoint pollution studies

Nonpoint pollution occurs primarily during rainstorm events when accumulated pollutants on urban and rural land surfaces are carried to receiving streams. Hence, this source of pollution is largely dependent on regional rainfall patterns, topography, and land use. Since the design flow assumed for the Baseline Simulation was a dry weather condition, nonpoint pollutant loadings would be minimal and were, therefore, excluded as input to QUAL-II. An important concern to planners, however, is whether nonpoint pollution in combination with point discharges would be significant enough that water quality criteria would be violated during wet weather.

A special study to determine the pollutant load from nonpoint sources relative to point sources was conducted using QUAL-II. Simulations showed BOD5 concentrations to be slightly higher in wet weather relative to the baseline condition. Below river kilometer 40, concentrations were similar to baseline conditions because of the large quantities of dilution water. Dissolved oxygen levels, however, were higher throughout the study area. The sag during wet weather was more than 1 mg/l less severe when compared to baseline conditions.

Interestingly, wet weather conditions do not seem to seriously affect the opportunity to achieve water quality criteria. Results indicate that for most water quality parameters, nonpoint pollution would have little impact on achieving water quality criteria at the levels indicated.

Cumulative impacts of pollution sources

In the studies described above, changes to the baseline conditions included individual variations to flow, percent collection, water for dilution, nonpoint source pollutants, or solid waste pollutants. In a separate simulation, the cumulative effect of the assumptions was evaluated by modifying several assumptions simultaneously to reflect a potential "worst case" future water quality. The simulation was based on the following assumptions, modified from the Baseline Simulation:

- The design flow was changed to the lower flow (90 percent exceedance).
- All dilution water inputs were eliminated.
- No abatement to the solid waste dumping was assumed.

The results clearly demonstrate the importance of both dilution water availability and solid waste abatement. A DO concentration of zero occurs for approximately 50 kilometers of the river. BOD5 demands exceeding 30 mg/l are predicted for approximately 88 kilometers of the 100-kilometer Rio Medellin. Thirty milligrams per liter of BOD5 was the maximum permissible concentration to meet water quality criteria. This concentration was exceeded in all except the most upstream reach. A peak of 130 mg/l resulted in the City of Medellin, where organic loading contributions to the river are the greatest. These results again illustrate the importance of understanding all factors influencing water quality.

SAND AND GRAVEL MINING STUDY

The considerable growth and construction that has occurred in the Aburra Valley over the last 20 years has demanded large quantities of raw materials, particularly sand and gravel for concrete, brick, and roads. A major source of this sand and gravel has been the beds and banks of the rivers in the valley. A special study was conducted to determine the effects of this sand and gravel mining on the quality, hydraulics, and beneficial uses of the river.

Erosion and sedimentation are continually occurring processes in the Aburra Valley that have produced many of the major features of the landscape and fluvial systems. Sediment is delivered to the river system through a number of natural mechanisms, including surface erosion, landslide activity, channel degradation, and streambank erosion. Man's activities, such as construction, agriculture, timber harvesting, and mining, also contribute sediment. The mining of sand and gravel from river beds and banks is one such practice that affects the supply and deposition of sediment in the Rio Medellin and its tributaries. The approach used to investigate water quality aspects of sand and gravel mining in the Rio Medellin is the subject of another technical paper.

Results of this study indicated that shore mining may measurably impact water quality, specifically suspended solids concentrations, in the immediate vicinity of the mining operation. The relative impact of this increase in suspended solids is greater in undeveloped watersheds at higher elevations where initial water quality is better. Sampling data indicated that, for an entire tributary and for the Aburra Valley, shore mining presently contributes only approximately 3 percent of the total suspended solids. It does not appear that implementation of an extensive point source pollution control program will change that percentage significantly.

The impacts of shore mining on the hydraulic characteristics and aesthetic value of a river reach are as important as the water quality impacts. Regardless of water quality, reaches destroyed by mining operations could not be considered aesthetically beneficial. Uncontrolled mining can also result in structural problems, including scour around bridge abutments and undercutting of roadways adjacent to the watercourse.

In summary, the physical impacts of shore mining appeared to be more important to the achievement of beneficial uses than the water quality effects.

SUMMARY

Results of the prefeasibility study showed that accomplishment of several major programs over the 20-year planning period would improve beneficial uses of the Rio Medellin and reverse the historic trend toward degradation of river water quality. Programs required to provide an aesthetic use of the river include improvement of the solid waste collection and disposal system, completion of the Rio Grande diversion project, collection of at least 75 percent of the wastewaters in the Valley, construction of two secondary and two preliminary wastewater treatment plants, and control of sand and gravel mining in important reaches of the river. A key aspect of the potential success of this comprehensive program is the transbasin diversion of water--the primary purpose of which is water supply and hydroelectric power, but a major beneficiary of which is improved water quality in the Rio Medellin.

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POSTGRADUATE EDUCATION OF HYDROLOGISTS AN INTERUNIVERSITY PROGRAMME

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ABSTRACT

The Interuniversity Postgraduate Programme in Hydrology (IUPHY), located at the Free University Brussels (VUB), is oriented towards students from developing countries. IUPHY has a formal instruction programme over one or two years. At the beginning of the first year an evaluation of prerequisites is made and examinations are scheduled throughout the academic year. This allows for a step by step screening, guidance and interaction with the students of different background. The first year leads to a Diploma in Hydrology, the second year to a M.Sc. Degree in Hydrology. The programme offers basic courses, three orientations (surface water hydrology, groundwater hydrology and water pollution control), seminars, field training and the M.Sc. Thesis. Continuation is possible with a four year programme leading to a Doctorate (Ph.D.) in Sciences, Agricultural Sciences or Applied Sciences. Specific assets of the programme are believed to be 1° a thorough and practical training in computer programming and 2° an in-depth training of operational research techniques applied to water resources management.

INTRODUCTION

Only a decade ago, professional hydrologists were people with a university degree in a traditional field such as civil engineering, agricultural engineering, geology or geography. Through the International Hydrological Decade (IHD) - which started in 1965 and was coordinated by UNESCO - hydrology became recognized as an independent professional career and various possibilities for the training of hydrologists have been developed. Nowadays, postgraduate courses exist in many countries (UNESCO, 1983) which is an evidence of the need for specialized training in modern hydrological techniques.

This need was also felt in Belgium where the Belgian National Committee of the International Hydrological Programme of UNESCO made a recommendation to the Belgian Universities to establish a postgraduate programme. Actually, three programmes exist : one programme is supported by the French-speaking universities; another programme is in Dutch and taught at the Free University Brussels (Vrije Universiteit Brussel - VUB); the third programme is the Interuniversity Postgraduate Programme in Hydrology (IUPHY) which is located at the Free University Brussels and has English as the medium of instruction. Under the coordination of the Flemish Interuniversity Council (VLIR), the programme is supported by the Universities of Antwerp (UIA), Ghent (RUG) and Leuven (KUL). IUPHY receives also the co-operation of the Royal Meteorological Institute (KMI). It is oriented towards students from developing countries and therefore sponsored by UNESCO and supported financially by the Belgian Administration for Development Co-operation (ABOS). The programme is organised annually and starts in September.

This paper outlines the programme of IUPHY, its characteristics and experiences. Full details about the programme are available from the authors and can also be found in Van der Beken (1985).

PROGRAMME OUTLINE

IUPHY has a formal instruction programme over one or two years which is open to candidates with a B.Sc. or B.Eng. degree or equivalent degree.

<u>The first year</u>, leading to a Diploma in Hydrology if all requirements have been met succesfully, includes the Basic Courses (410 hrs), listed in Figure 1, and Field Training (30 hrs).

PREREQUISITES

		minimum hrs attended
PRE 1	Mathematics	120
PRE 2	Physics	90
PRE 3	Mechanics	90
PRE 4	Fluid mechanics and general hydraulics	45
PRE 5	General chemistry, including analytical chemistry	90
PRE 6	Principles of geophysics, geology, geomorphology and soil science	45
PRE 7	Surveying	35

BASIC COURSES

		hrs
BC 1	Open channel hydraulics	40
BC 2	Hydrometry	20
BC 3	General meteorology and climatology	20
BC 4	Hydrometeorology	20
BC 5	Surface water hydrology	40
BC 6	Groundwater hydrology	60
BC 7	Probability and statistics	60
BC 8	Systems approach to water management Part I : Introduction	40
BC 9	Water quality, water treatment and water supply	40
BC 10	Introduction to computer science and the use of hydrological databases	30
BC 11	Introduction to computer programming	40
	total hrs :	410

Figure 1. LIST OF PREREQUISITES AND BASIC COURSES

The prerequisites indicate those courses the candidate is required to have covered during undergraduate education and which are believed indispensable for full understanding of the Basic Courses. Candidates with a B.Sc. non-engineering degree will probably not have been trained in general mechanics nor in fluid mechanics, neither will they have been introduced in surveying techniques. B.Sc. candidates in Physics, Mathematics or Chemistry will not have covered a course in general geology,etc. Eventually, a candidate may therefore be requested to extend his knowledge on one or more of these subjects. Moreover, it has been experienced that the Prerequisites Mathematics (PRE 1) and Fluid Mechanics (PRE 4) organized as lecture courses early in September and obligatory for all candidates, provide for a very helpful start to the first year programme.

The evaluation of the prerequisites in November and the spreading of the examinations of the Basic Courses (January, April, June/July - first session - final examinations and September - second session and final examinations) allow for a step by step screening, guidance and interaction with the students of different background.

The second year of the programme, leading to a Master's Degree in Hydrology, includes Optional Courses (minimum 190 hrs), listed in Figure 2, Field Training (90 hrs), Seminars (30 hrs) and the M.Sc. Thesis.

OPTIONAL COURSES

		hrs
	Orientation A : Surface Water Hydrology	
OC-A1	Synoptic meteorology	15
OC-A2	Advanced hydraulics	35
OC-A3	Advanced surface water hydrology	35
OC-A4	Watershed management	35
	Orientation B : Groundwater Hydrology	
OC-B1	Flow through porous media	35
OC-B2	Groundwater abstraction techniques	15
OC-B3	Hydrogeology, geophysical prospection	35
OC-B4	Applied mathematical methods	35
	Orientation C : Water Pollution Control	
OC-C1	Water chemistry and water quality	15
OC-C2	Water quality models	35
OC-C3	Hydraulics of waste water collection and water supply	35
OC-C4	Waste water purification and water treatment	35
	Common to all orientations	
OC-D1	Systems approach to water management Part II : Basic concepts	35
OC-D2	Statistical applications in hydrology	35
OC-D3	Teledetection techniques	35
OC-D4	Damsites and drilling techniques	35

Figure 2. LIST OF OPTIONAL COURSES

Optional Courses are divided into three orientations (surface water hydrology, groundwater hydrology and water pollution control). Each student makes a selection of Optional Courses under the constraints that at least one orientation is covered completely and the subject of the M.Sc. Thesis is in line with the orientation selected.

The Field Training aims to give the students the opportunity to learn techniques of hydrometry. Excursions in Belgium and abroad are organized and allow the student to get acquainted with different water resources systems. The students should not select their field training programme too uni-directional : a large number of hydrometric techniques should be selected. During the second year field training tutors are helping the students. The students are expected to have frequent contact with these tutors for organizing their field training programme and for the contents of their field training reports.

The M.Sc. Thesis is the backbone of the second year programme. The research project must be selected at the end of the first year. Ideally, the student selects this research project from his own experience in his country and with the help of his supporting organization.

Outstanding applicants with a M.Sc. Degree in a branch closely related to water resources can be admitted to a four year programme of study and research leading to a Doctorate (Ph.D.) in Sciences, Agricultural Sciences or Applied Sciences. This programme is comprised of a 12-month pre-doctoral training of intensive course-work and seminars.

PROGRAMME OBJECTIVES

Education at a university requires a succesful combination of teaching and research activities, both considered equally important especially at the Master's level. Ideally, a mutual interaction between the 'transfer of knowledge' (teaching methods and techniques) and the 'advance in science' (research improving these methods and techniques) takes place.

It is IUPHY's commitment to confront the Master's candidates with a number of water resources research projects whereas an active role is required in his/her Thesis project. The latter eventually allows them to gain experience in the four main items any scientific work is usually based upon :

i) MEASUREMENT
ii) ANALYSIS
iii) SYNTHESIS
iv) APPLICATION

In their list of curricula and syllabi in hydrology, Chandra and Mostertman (1983) redefine this main objective of education in hydrology as to develop competence for the measuring, handling and analysis of hydrological data, as well as the proficiency in the application of such information for the planning and design of water resources projects. The way this competence and proficiency is to be transmitted to the IUPHY-students has been the subject of changes in both the general programme outline and in the contents of more specific courses.

The first year programme aims to give all students the basic background before further studies in depth in one specific orientation. Understanding multidisciplinary problems is an important asset in water resources management; likewise, the training in statistical and systems analysis techniques. The use of a computer is of course a necessity and students are trained from the very beginning of the programme to handle hydrological data and to use them in practical computer programs.

The growing importance of water resources systems analysis dictates the need to include systems analysis topics as part of the postgraduate programme. Principles are taught in the course "Probability and Statistics" (BC 7). Auto regressive, moving average processes and concepts on reservoir management are given in the course "Systems approach to water management, part 1" (BC 8).

The increasing use of digital computers in the process of storage and analysis of hydrological data has led to the introduction of two Basic Courses on computer programming. In a first course, "Introduction to computer science and the use of hydrological databases" (BC 10), a brief summary of computer history and a description of both hard- and software elements of a mainframe computer allow to familiarize the student with the computer facilities the University offers. In the second course, "Introduction to computer programming" (BC 11), the Fortran 77 programming language is taught, the choice of which has been based primarily upon the fact that most of the existing hydrological programs are written in this language. However, the BASIC language is also introduced in view of both its use on some programmable pocket-calculators and the expanding market of micro-computers.

This first year allows students to end eventually their hydrological training with a Diploma in Hydrology, without further specialized studies leading to a M.Sc. degree with research thesis.

During the second year of the programme the student must choose an orientation, i.e. a series of courses designed to provide the student with competence in a particular area. Three orientations (surface water hydrology, groundwater hydrology and water pollution control) are available, but the student is allowed to take, concurrently, optional courses 'common to all orientations' or select courses from other orientations.

The Optional Courses will continue to be the media for introducing new ideas and are taught in a different way since the students gained easier access to computers. Moreover, seminars, mostly by invited lecturers, will give broadness as well as training in specific areas.

Field training is an important part of this second year programme. Students must be able to guide and train technicians for field measurements. Therefore, they must gain experience in the field.

The M.Sc. Thesis is a culminating experience, allowing synthesis and integration of the knowledge gathered during the programme course. It is a personal project of the student with the help of an advisor and approval of a supervisor. Students are expected to choose a project related to an issue in water resources of their own country. Therefore, they should prepare data and all useful information as soon as possible by contacting the relevant authorities of their country. The student's advisor and supervisor will help in exploring the potential for such data collection.

EXPERIENCES

The number of hours for each course is divided into theoretical lecture hours and exercises, depending on the choice of the lecturer. All classes must be attended regularly and practical sessions are always obligatory. It is expected that the students start with the exercises during the scheduled sessions and ask questions if necessary, in other words, that they prepare the exercises and finish them at home. A concise report, in clear writing, of the solved problem must be presented during the next session. The experience as already described by Custiodo and Martin-Arnaiz (1975) is that the exercises are best solved if they are printed and prepared with guides to the method used to arrive at the solution. Later on, the solution to the problems are handed out. A first draft document of 'Problems in Hydrology, annotated', has been written by the co-author De Ketelaere (1984).

It has become commonly accepted that the use of computers in teaching hydrology brings positive effects which influence both the learning process and the future professional activity of the studens. The principle of unifying theory and practice can be applied by solving real-world problems using an extensive database (Elias, 1981).

In this context, HYDROBASE - a simple database for hydrological data (Vanouplines et al., 1984), developed at our Laboratory proved to be very helpful. HYDROBASE allows the students to create their own datafiles either in the frame of a practical session or in view of their MSc research work.

The most important experience was that the students themselves asked to increase the number of class hours devoted to exercises or laboratory work of the first year of the programme.

Laboratory experiments, especially those with the Basic Hydrology System (Sellin and Treleaven, 1975) allow to visualize hydrological phenomena. Because of the high interest of the students in laboratory experiments as well as in Field Training - most students, although working in Water Resources Institutes, recognize they never carried out field work before - it was decided to appoint Field Tutors to guide the second year students in their Field Training programme. Guided training in a Consultant's Office is considered equally important. IUPHY appreciates very much the help of several governmental institutions and water related agencies in Belgium for providing training facilities.

Another very useful experience is the flexible programme section 'Seminars' : visiting lecturers are welcome guests who can bring advanced research results, practical experience or special topics such as "sociological problems' or 'legal aspects' of water resources systems.

In the future it is our intention to have 'common project discussions' organized like seminars but focussed on a more active participation of the students.

CONCLUSIONS

The world water problems are a tremendous challenge for man. Food production, drinking water supply, sanitation and industrial development are key factors in the world economy and welfare.

All of them rely directly on water. However, in many countries, water is not available in sufficient quantity and quality. The training of the students from different disciplines in one of several area specialities of water resources development is therefore a necessity. The aim of IUPHY is to make a real contribution to this training.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 127 Aspect number 5

A REGIONAL GROUNDWATER FLOW MODEL BASED UPON THE VARIABLE SOURCE AREA CONCEPT

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ABSTRACT

A model is presented for groundwater flow, on a regional scale, based upon the variable source area concept. This implies that the catchment is divided into recharge areas, characterised by diverging groundwater flows and deep groundwater levels, where all netto precipitation is recharged in the groundwater reservoir, and discharge areas, characterised by shallow groundwater levels almost coinciding with the groundsurface, where all excess water, netto precipitation and possible converging groundwater flow, is drained away by the river system. The division between discharge and recharge areas is variable and depends upon climatological, topographical and hydrogeological conditions. Such a formulation makes it possible to take into consideration the very complex and heterogeneous hydrological conditions that usually exists in a large river catchment. The model was applied to the Demer catchment with an area of 2240 km^2 . The results are very satisfactory and give a complete picture of the hydrological situation in the area. This type of model can be of great importance for the study of many hydrological problems in the screening and planning phase, such as groundwater abstraction, pollution control, storm flow and soil drainage.

INTRODUCTION

Numerical modeling of groundwater systems is a relative new field. Since the mid-1960's, significant progress has been made in the development and application of numerical models for groundwater-related resource management (Bachmat et al., 1980). The techniques for simulating groundwater movement on a digital computer are well developed, see for instance Remson et al. (1971), Pinder and Gray (1977), Cheng (1978), and Wang and Anderson (1982). The general approach is to solve the flow equations, describing the dynamics of the groundwater system, by means of finite-difference or finite-element techniques. In order to obtain a unique solution, appropriate boundary and initial conditions are necessary. In many small scale problems these conditions can easily be quantified or estimated. However for regional problems this is not the case, because many conditions are unknown or are difficult to translate into mathematical form, due to the complexity of the hydrological situation. Consider for instance the complicated relation between precipitation, surface runoff, infiltration, groundwater flow and groundwater discharge to the river system in a watershed with large heterogeneity in soil types, land use, and topographical and geological conditions. In this work a computer model, based upon the concept of the variable source area, is presented that is capable of solving such complicated systems.

THEORY

The variable source area concept

In the classical Horton concept, it is assumed that rainfall events generally exceed infiltration capacities of the soils, resulting in overland flow. In such case, the response between precipitation, runoff, and infiltration is very complex, due to the great heterogeneity in soil types that usually exist in a watershed and the very irregular pattern of precipitation in both time and space. However, recent field studies revealed that overland flow due to exceedence of infiltration capacity, is a relatively rare occurence, especially in humid and dense vegetated basins. It was found that overland flow originates from wetlands in the low lying areas adjacent to the rivers, as direct precipitation cannot infiltrate into these saturated soils, due to high water table elevations and rising groundwater flow conditions (Freeze and Cherry, 1979). These wetlands expand and contract during and following rainfall events under the influence of the subsurface flow system. The resulting variation with time in the size of the areas, contributing to overland flow, is referred to as the "variable source area concept". The model that will be developed is based upon this concept.

Flow equations

In a regional groundwater basin, the groundwater flow can be assumed to be largely horizontal

$$\bar{q} = - T \text{ grad } h \qquad (1)$$

with $\bar{q}$: flux over the entire thickness of the groundwater basin (m^2/day)
T : transmissivity, assumed to be independent of the groundwater table elevation (m^2/day)
h : hydraulic potential or piezometric level (m).

Combined with the equation of continuity, the equation for steady groundwater flow becomes

$$\frac{\partial}{\partial x}\left(T\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T\frac{\partial h}{\partial y}\right) + N - Q = 0 \qquad (2)$$

with N : recharge (m/day)
Q : groundwater outflow to the river system (m/day)
x,y : coordinates (m)

This equation will now be interpreted according to the variable source area method. In a catchment two areas can be identified :

1° Recharge areas, where no surface runoff occurs. These areas are characterised by diverging groundwater flows and deep watertable elevations. The natural recharge equals the total precipitation minus the evapotranspiration. The surface outflow is zero. Hence, the flow equation is Eq.(2) with Q=0, and N equal to the netto precipitation.

2° Discharge areas where the watertable elevations are near the soil surface such that overland flow and surface runoff is possible. All excess water, netto precipitation and possible converging groundwater flows are drained away by the river system. The flow is governed by Eq.(2) with the hydraulic potential, h, equal to the maximum possible watertable elevation, H, depending upon the local topography, and with N equal to the netto precipitation, such that the only remaining unknown in this equation, the surface outflow, Q, can be calculated explicitely. The discharge areas can be further subdivided into completely saturated discharge areas, when the surface outflow exceeds the netto precipitation, Q>N, (converging groundwater flows), and partly saturated discharge areas, where only part of the netto precipitation is drained away by the river system and the remainder is recharging the groundwater reservoir, Q<N (diverging groundwater flows).

The division of the catchment into recharge and discharges areas is variable, depending upon the climatological, topographical, and hydrogeological conditions.

Numerical solution

The flow equations are solved with a finite difference technique. The catchment is devided into a large number of small squares, with side Δx. Averaged values of the parameters are considered in every square. A typical square, denoted by index 0, is shown in Fig. 1, with its four neighbouring squares, denoted by indexes 1, 2, 3, and 4.

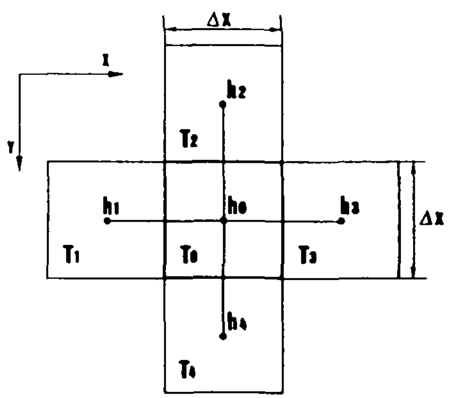

Fig. 1 : A typical element of the numerical model

The flow equation is discretised as follows :

1° recharge areas

$h_o < H_o$ (3)

$Q_o = 0$ (4)

$$h_o = (N_o \Delta x^2 + \sum_{i=1}^{4} \tau_i \, h_i)/\sum_{i=1}^{4} \tau_i \qquad (5)$$

with τ_i the harmonic mean of the transmissivities of square 0, and square i :

$$\tau_i = 2T_oT_i/(T_o+T_i) \qquad (6)$$

The use of harmonic means is very handy for describing imprevious boundaries; when $T_i = 0$, it follows that $\tau_i = 0$ and no flow occurs between square 0 and its neighbouring square i.

2° discharge areas

$$h_o = H_o \qquad (7)$$

$$Q_o = N_o + \sum_{i=1}^{4} \tau_i (h_i - H_o)/\Delta x^2 \qquad (8)$$

This system of equations, Eq. (3), (4), (5), (7) and (8), can be solved with the aid of computer in an iterative way; the division into recharge areas ($h_o < H_o$, $Q_o = 0$) and discharge areas ($h_o = H_o$, $Q_o > 0$) results automatically. After convergence, the groundwater flows can be calculated as :

in the x - direction

$$q_x = [\tau_1(h_1-h_o) - \tau_3(h_3-h_o)] \, \Delta x/2 \qquad (9)$$

in the y - direction

$$q_y = [\tau_2(h_2-h_o) - \tau_4(h_4-h_o)] \, \Delta x/2 \qquad (10)$$

APPLICATION

The Demer catchment

The model was applied to the Demer catchment, a basin of 2240 km^2, situated in the north-east of Belgium (Fig. 2). A detailed description of this catchment is given by Bronders and De Smedt (1984).

The relief varies between 15 m, above sea-level, in the north-west, to about 170 m in the south. The soil types vary from sand to sandy loam and loam, going from north to south. All soils are well drained, except in the depressions, where soils are saturated and sometimes marshy. Underneath a Quarternary layer with a varying thickness of 0 to 20 m maximum, chalk, sand, clayey sand, clay and silt layers alternate with varying thickness up to a depth of several hundreds of meters. The hydrogeological characteristics of these layers are well known.

As an illustration, Fig. 3 shows the geological situation along a cross-section going from north to south.

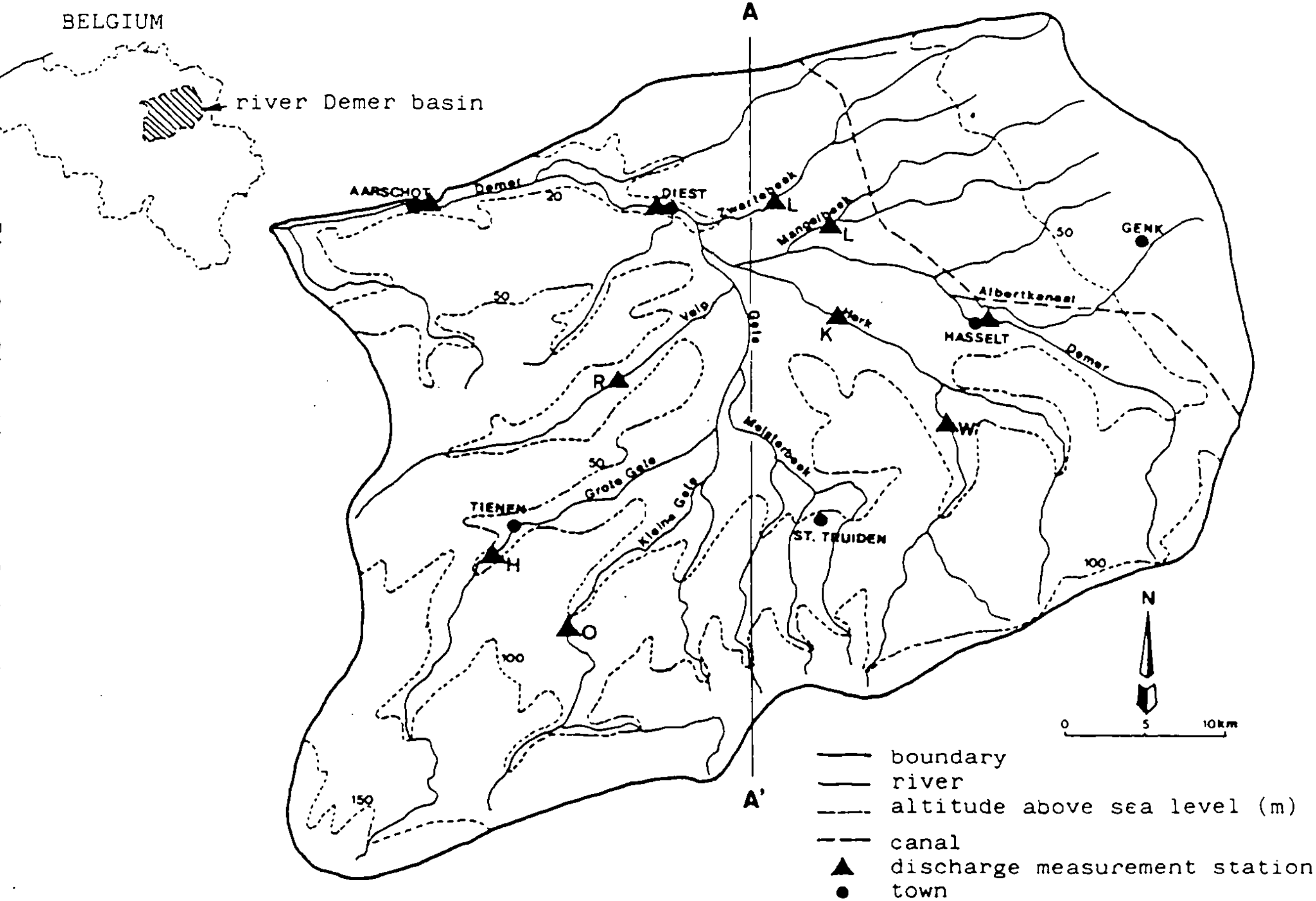

Fig. 2: Situation map of the river Demer basin.

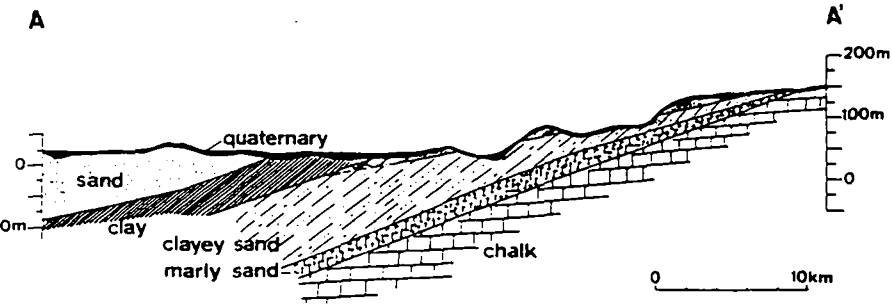

Fig. 3 : Geological cross section from north to south (Section A-A')

Reasonable estimates of the hydraulic conductivities of these layers are available (Loy and De Smedt, 1978), ranging from about 70 m/day for the Quaternary sand to nearly zero for the clay layers. Transmissivity values of the phreatic groundwater reservoir were calculated by summation of the thickness times hydraulic conductivity of the different layers up to a depth of 100 m or until a clay layer was reached. In this way, the catchment could be divided into 17 zones with more or less constant transmissivities, ranging from about 300 m^2/day to 2000 m^2/day.

Climatological data are available at the Royal Meteorological Institute. The annual precipitation and evaporation of a free water surface, according to the Penman formula, are given in Table 1, for the period 1973-1978.

Table 1. Annual precipitation, Penman evaporation and discharge of the river Demer basin.

year	precipitation (m)	evaporation (m)	discharge at Aarschot (m)
1973	0.660	0.660	0.159
1974	0.978	0.628	0.243
1975	0.627	0.653	0.191
1976	0.505	0.752	0.116
1977	0.819	0.570	0.178
1978	0.692	0.569	0.192
mean	0.713	0.638	0.180

River discharges are measured at several locations, shown in Fig. 2. The total discharge of the Demer catchment can be taken equal to the measurements at Aarschot; these values are also given in Table 1. It can be assumed that for calculating the hydrological water balance only precipitation, evapotranspiration and discharge have to be taken into consideration; other terms as groundwater in- or outflow to other catchments, groundwater pumping, and storage are negligible over longer periods. Hence, the mean annual netto precipitation, over the period 1973-1978, can be taken equal to the mean annual discharge of 0.180 m or

5.10^{-4} m/day. The mean annual evapotranspiration, over this period, is equal to the total precipitation minus the discharge, or 0.533 m; this is 84 % of the Penman evaporation for a free water surface, a value comparable to other water balance studies in Belgium (for instance Van der Beken, 1977).

The Demer catchment was divided into squares with sides of 2000 m. In every square, average values of the parameters were specified. Transmissivity values were taken as explained before.The netto precipitation, N, was taken equal to 5.10^{-4} m/day. The maximum possible watertable elevation, H, was taken equal to the topographic elevation, averaged visually in a square, minus 0.5 m.

Results

Details of the computer program can be found in Bronders (1983). The program solved the finite difference approximation of the groundwater flow equation in 81 iterations, obtaining an accuracy in the piezometric levels of 10^{-4} m. Results are shown in Fig. 4. In this figure the lines of equal hydraulic potential are shown, together with the groundwater fluxes and the areal distribution of completely and partly saturated discharge areas.

The calculated piezometric levels were compared with measurements available at the Belgian Geological Survey. For the lower parts in the NW and centre of the catchment the agreement was very good, with differences less than 3 m. In the higher topographic parts, difference were larger, up to 8 m. No data are available for the groundwater flow. With these results, presented in Fig. 5, a clear picture is obtained of the groundwater flow conditions in the catchment. In the NE part, large groundwaterflows, in the order of 2 to 6 m^2/dag, occur in the NE-SW direction. These flows converge to the river Demer and its nothern tributaries. In the S part of the catchment, the groundwater flow is mainly directed S-N, with large flows in the order of 6 m^2/day. Here, the groundwater flows converge to the southern tributaries of the Demer river. In the middle part, groundwater flows are low, in the order of 1 m^2/day, because of the presence of a large clay layer at shallow depth. No preferential directions are present; all groundwater is flowing to the nearest river.

The total discharge areas amount to 28 % of the total catchment area; completely saturated discharge areas make up 23 %, while the partly saturated discharge areas are only 5 %.

Discussion

The concept of the variable source area is clearly illustrated when the calculations are repeated for other values of the netto precipitation. Fig. 5 shows the ratio of discharge areas and completely saturated discharge areas versus the total catchment area, as a function of the netto precipitation.

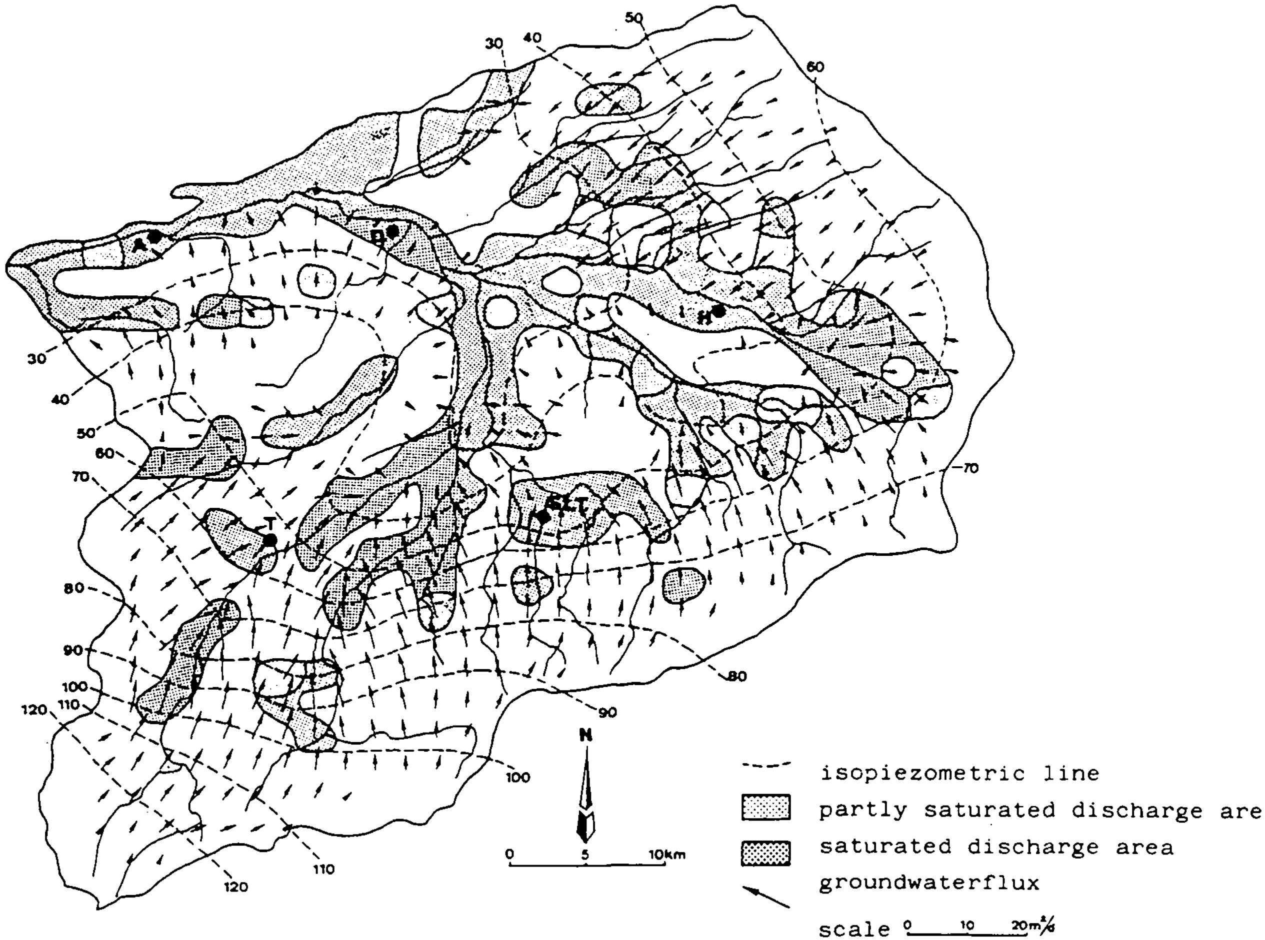

Fig. 4: Calculated groundwater levels and flows, and distribution of discharge areas.

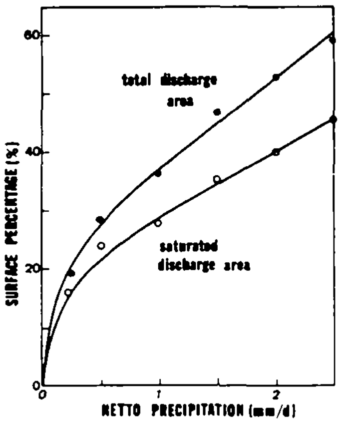

Fig. 5: Total discharge areas and saturated discharge areas, expressed in percentage, as a function of netto-precipitation

These results are of great value for the prediction of stormflows. According to the variable source area concept, the contribution of a storm to the direct discharge of a river system will be equal to the precipitation volume falling on the saturated discharge areas. For example, from Fig. 6 it can be seen that the direct runoff resulting from a storm will vary from 17, to 23 and 27 %; in function of the preceeding hydrological conditions, characterised by a mean netto precipitation of respectively 2.5 10^{-4}, 5.10^{-4} and 10^{-3} m/day. The direct storm flow can even amount to 40 % in case of a very wet preceeding period of 2.10^{-3} m/day netto precipitation.

With the obtained results, it is also possible to divide the catchment into subcatchments, in each of which discharge and recharge areas can be identified and discharges can be calculated. E.g. for all the existing river gauging stations, the average annual discharge was calculated with the model, and compared to the values measured during the period 1973-1978, as shown in Table 2.

Table 2. Observed and calculated discharges for the period, 1973 - 1978, in the river Demer basin

station	river	observed mean (m^3/s)	calculated (m^3/s)
Aarschot	Demer	12.34	12.82
Diest	Demer	10.81	11.47
Lummen Lummen	Zwartebeek Mangelbeek	1.59	1.21
Ransberg	Velp	0.40	0.74
Kermt	Herk	1.23	1.31
Wellen	Herk	0.42	0.48
Opheylissem	Kleine Gete	0.64	0.48
Hoegaarden	Grote Gete	1.00	0.82
Hasselt	Demer	2.47	2.01

It can be seen from this table, that a very good agreement exists between measured and calculated discharges .

Finally, it should be pointed out that this model is also very useful for problems of ground water recovery and pollution control. The model gives a clear picture of the groundwater levels and flows. Discharge areas have usually converging groundwater flows, high groundwater elevations, and consequently will be characterised by a quick leaching of pollutants, in contrast with recharge areas, characterised by diverging groundwater flows, low groundwater elevations, and long pollutant flowpaths with large residence times.

CONCLUSION

Equations were established for the stationary horizontal groundwater movement on a regional scale, based upon the concept of the variable source area. An advantage of this formulation, is the automatic interaction of the ground and surface water in the so called discharge areas, the size of which depends upon climatological, topographical and geological conditions. The model was applied to the Demer catchment, by means of a numerical solution technique based upon the finite difference approximation. The model makes it possible to take into consideration the very complex and heterogeneous hydrological conditions that usually exists in a large river catchment.

The results give a clear indication of the groundwater levels and flows and the resulting discharge to the river system. This can be of great importance for the study of many hydrological problems as for instance groundwater recovery, pollution control, flood prediction and soil drainage. As such, this model enables the study of the complete hydrological situation in a catchment, which no other technique can provide. The present model can be improved by taking into account, in- and outflows to other catchments, groundwater abstractions, spatially varying precipitation, and most important of all, non steady state conditions, such that the relation between precipitation, evapotranspiration and groundwater flow and storage can be investigated in time.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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WATER RESOURCES MANAGEMENT FOR VILLAGE WATER SUPPLIES IN AL MAHWIT PROVINCE, YEMEN ARAB REPUBLIC

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ABSTRACT

The growing population and the decrease of spring discharge have led to bottlenecks in the water supply of villages and dispersed settlements in the Al Mahwit Province. The economic development of the country, extensive road construction last years and the introduction of adapted technologies have contributed to the reduction of this bottleneck. Considering the natural ecological conditions the possible solutions for five villages in typical locations are presented: pressure or gravity pipe schemes, water truck systems and for the zones with more severe aridity, a prototype of a bacteriologically clean cistern that works without outside energy supply, has been developed.

Keywords: Yemen Arab Republic, hydrogeology, groundwater exploration, natural conditions, adapted technologies, village water supply, pressure pipe schemes, clean cisterns.

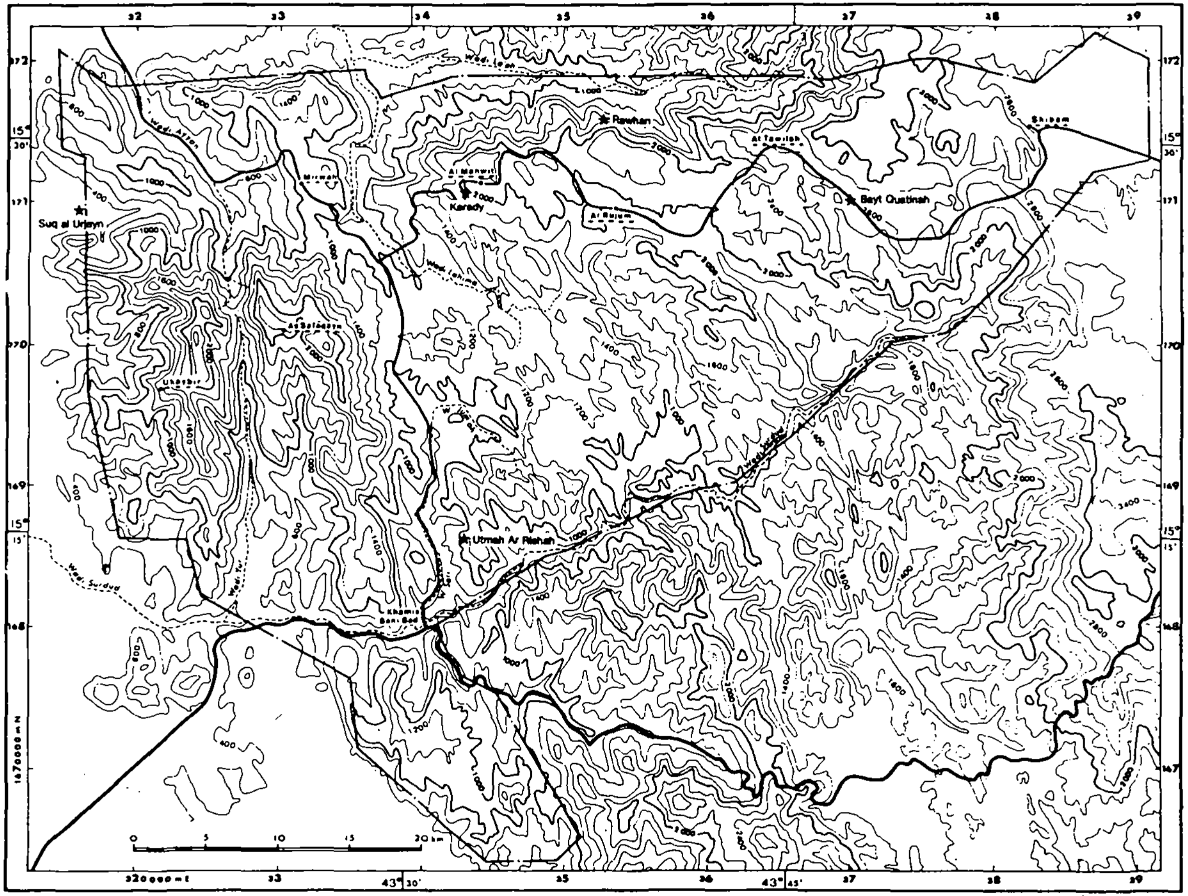

Fig. 1: General map of Al-Mahwit Province

INTRODUCTION

The Province of Al Mahwit stretches from the cliffs of Kawkabahn in the east to the Tihama Desert in the west. The mountains in the east exceed 3000 m in altitude whereas the valleys at the fringe of the Tihama Desert lie below 400 m. The topography of the settled areas in the mountain regions favours rapid run-off over the partially bare slopes and the exposed rocks. Therefore, stream discharge can only be observed in the larger wadis of the deeply incised valleys.

Due to defense requirements, villages and dispersed settlements are usually situated on the highest points of ridges, crest, or on table-like plateaus. They depend on small springs, the discharge of which is nowadays insufficient. The location of the settlements on the top of an extensively dissected landscape results in a severely limited water supply.

As no piped distribution system could be found, spring or cistern water is carried in plastic cans or metal buckets by women and girls. Normally this has to be done several times a day according to the demand of the family. Over longer distances donkeys are also employed for carrying two plastic cans of 20 l each. In some cases drivers who own a four wheel drive pick-up have a small metal tank installed (appr. volume 0,7 - 1,2 m^3) and sell the water by full tank load or by plastic cans. The safety and hygiene of this sort of supply is unsatisfactory.

If one aims at an improvement of the water supply systems, one has to concentrate upon the bigger villages because a more consistent settlement situation can be expected. Dispersed settlements are often abandoned or move down to the wadis, so that in these cases mobile water supply systems have to be devloped.

All measures should be adapted from the point of view of their economy. This is only possible if the water supply schemes that one developed are adapted to the natural ecological conditions. A sound knowledge of the rainfall pattern as well as the water bearing strata is indispensable and therefore is one of the points of emphasis.

CLIMATE

The climate of Al Mahwit and the surrounding areas does not fit into any simple climatic pattern, because it lies somewhat outside the range of the main climatic regions, and is essentially transitional and governed by the the relative prevelance of air masses coming from at least two different directions. During the summer months a westerly maritime influence with monsoon air masses and moderate daily temperatures predominates. Continental air masses from the east prevail in September and October. Clear skies, a fall in humidity, and a greater diurnal range of temperature can be observed.

The distribution of rainfall is determined by the topography and exposure to the rain-bearing winds. Deeply incised valleys which run north-south in the rain shadow have relatively small average rainfall (300 mm) because they are sheltered from the prevailing rain-bearing winds by a range of mountains.

In contrast to this, the windward slopes exposed to the west can have extremely heavy rainfall, owing to the forced ascent of moisture-laden air. In these areas we have calculated precipitation of 800 - 900 mm. However, mountain regions with an elevation of 1 500 m are to some extent moister than the precipitation figure might suggest. This is due to an inversion of temperature causing fog or cloud which may persist for several days delivering excess moisture to a particular strip of land. For these altitudes we presume moisture conditions corresponding to an annual average precipitation of 1 000 mm. We have to expect interannual deviations from these figures of ± 60 %. The probability that any region will have an absolutely rainless year is extremely low.

During the last 15 years the rainfall was distinctly below average and therefore influenced the recharge of the springs unfavorably.

HYDROGEOLOGY

The lithology of the project area is characterized by sandstones, limestones and marls of Mesozoic Age deposited over the levelled Palaeozoic Basement Complex.

During the Tertiary tectonic activity led to tilting of blocks to the south and south-west. To the Red Sea step faults could be found, so that the general direction of the water movement is to the south-west.

Amran Series

One of the formations particularly important in the context of our question is the limestone of the Middle to Upper Jurassic, called Amran Formation. The outcrops conform with the general tectonic pattern. The formation comes to the surface at the north facing cliffs leading to Wadi La'ah and is there exposed in full thickness of 350 to 380 metres. Several other exposures running north-west south-east can be found over the Province.

The Amran Limestone comprises a series of greyish black limestone interbedded with marly limestone. The uppermost strata, the so-called "Unnamed Formation" has a typical lagoonal facies. It consists of a series of clay, fine-grained sandstone, marl interbedded and interfingered with lignite and gypsum lenses. In some places thickness of this aquiclude/ aquitard is less than 20 metres.

From the hydrogeological point of view the Amran Series are poorly productive, yielding only a few litres per second with a large drawdown. Even in the north of YAR where Amran Limestones are disturbed by a number of faults, fractures and volcanic intrusions, a great number of villages experience the most serious water shortages in the YAR.

The capacity of some springs at the cliff which faces towards Wadi La'ah is usually very low and several of them have only seasonal flow.

Only in areas where the fractures, fissures and solution openings are well developed and an aquitard follows at a depth of 100 to 180 metres drilling in the Amran Series should be considered.

Tawilah Mejd-Zir Formation

During the period, lasting from the Lower Cretaceous up to Palaeocene, continental sediments were deposited. The crossbedded sandstones are flesh-coloured, coarse grained, quartzitic and rather compact. Towards the top of the Formation, the grain becomes slightly more shaly.

The thickness of the whole Formation may reach 400 metres, especially where it has been protected from erosion by the volcanic cover. But in the parts west of Ar-Rujum, erosion has removed substantial parts of that Formation. Some places show nothing but a thin veneer of sandstone over the Amran Limestone which underlie this Formation.

Of the identified groundwater units the Tawilah Mejd-Zir Formation is the most interesting with regard to the extraction of groundwater. However, one should stop drilling if the uppermost strata of the Amran Formation is reached. If not, a substantial loss of groundwater may be expected in the swallow holes of the subjacent limestones.

Transmissivity shows large variations between 10 m^2/d and 1 000 m^2/d corresponding to permeability between 0,08 and 9 m^2/d. According to the anisotropy of the aquifer, storage coefficients are also to be expected in a wide range from 0,1 up to 30 %. The average effective porosity will be about 5 to 6 per cent.

South-east of Al Mahwit town, where the Formation is overlain by volcanic rocks of substantial thickness, some of the upper clay layers of Mejd-Zir or volcanic strata may act as aquitard or aquiclude. In these areas groundwater is stored under elastic conditions.

Yemen Volcanics

The Yemen Volcanics, deposited from Late Oligocene to Early Miocene, are the second important hydrogeological unit. The Formation comprises several lithologic units which are in parts fractured, porous and permeable.

A number of low capacity springs on mountain slopes in Jebel Hufash and Jebel Milhan yield water for domestic use. Only in extraordinary cases do springs in mountain regions yield enough water for irrigation purposes.

In valleys and the southeastern area of the Province where the volcanics dip to the south-west, dug wells can be found. These wells produce only low amounts of water for domestic supply and villages when they lie on higher locations of mountainous areas. However, the capacity of dug wells constructed along the wadi floors are comparatively much higher in yield. A lot of these hand-dug wells are equipped with a small pump used for the irrigation of crops grown nearby.

Drilling in Yemen Volcanics may be successful. As we know from drilled wells in mountain plains, namely Wadi Thulla, Sana'a Basin and other places, Yemen Volcanics have the capacity to support irrigation. Output of these drilled wells ranges from 3 to 30 l/s.

In the south-east of the Province one can expect artesian aquifers. There Yemen Volcancis in alternated stratification, dipping to the south-west, create water conditions with a certain hydrostatic pressure. We expect several aquifers in different units of this Formation.

Alluvial Deposits

Alluvial Deposits are very scarce in the project area. They are exposed in the form of a thin alluvial cover in intramountane plains and along the Wadis. These deposits comprise boulders, gravel, clay, sand and silt.

The alluvial sediments of the valley floors are usually not very thick. Handdug wells constructed in these sediments are normally not deeper than 20 m and can therefore produce only a limited amount of water.

PARTICULARS OF FURTHER LOCALITIES STUDIED

After discussing the hydrogeological conditions in the Province, we want to point out by means of different case studies how the present water supply of villages can be improved.

Rawhan

- Location: levelled area in the steep slope leading to Wadi La'ah, conditioned by clayey, morphologically soft strata.
- Rock type: Amran Limestone.
- Water supply: by two small perennial springs.
- Type of spring: overflow spring, intensified by fissures.
- Discharge: 2,5 l/minute.
- Water quality: good, except for nitrite (0,04 mg/l).

1. Improvement: Increasing of discharge by digging back and better capture of the springs. Possible discharge: 5 l/minute.

From the collection chamber a gravity pipeline is to be constructed over a distance of about 800 m to the centre of Rawhan. This will reduce the walking distance for women and girls without having a destructive effect on the necessary social communication at the water place.

2. Improvement: Further increase of discharge by the capture of storm run-off and continuous infiltration on a small plateau above the slope. Possible discharge depends on the amount of infiltrated water, 7 - 10 l/minute may be expected.

Utmah Ar Rishah

- Location: west-facing slope east of Jebel Hufash
- Rock type: Yemen Volcanics
- Water supply: small springs, cisterns, water tanker
- Water quality: cistern and spring bacteriologically unacceptable, but chemically good.

- Improvement: the abundant supplies of water in Wadi Sari may be tapped at any time in order to supply the mountain settlements around Utmah with drinking water. The water can best be supplied by means of wells dug in Wadi Sari. From there to Utmah Ar Rishah the difference in altitude is appr. 550 m, linear distance appr. 1 500 m. The amount of 50 l/cd for 1 600 people (= 80 m^3/day) will be supplied by a scheme consisting of two steps with a booster station. From the main storage tank situated on the plateau over Arisha public fountains will be served by gravity.

As the abundant quantities of water in Wadi Sari facilitate the supply of the dispersed settlements and small villages of Jebel Masjid lying near Utmah Ar Rishah, a second supply scheme should be constructed. On the basis of a truck supply water could be brought to small metal tanks lying near the road, from where people could carry their daily demand over a very short distance to their houses.

Bayt Quatinah

- Location: south-facing plateau
- Rock type: Tawilah Sandstone
- Water supply: cisterns and minor springs north of an east-west running valley.
- Water quality: as expected, chemically good, but contaminated in open pools and therefore hazardous to health.

- Improvement: Tawilah Sandstone facilitates an improvement of the water supply of Bayt Quatinah by a deep well which should be drilled 40 m below the plateau. From there a storage tank with distribution pipes to public fountains, should be supplied by a pumping scheme. Considering the results of pumping tests run in the Tawilah Sandstone, the water requirements for 800 people with a demand of 40 l/cd (= 0,4 l/sec) should be easily reached by the proposed deep well. If the yield of the bore hole proves to be higher, the demand of some smaller villages in the vicinity could also be satisfied by gravity pipes.

Suq al Urjayn

- Location: Suq al Urjayn is situated on the northern terrace of Wadi Tabab (Jebel Milhan) which leads from Jebel Milhan to the Tihama Desert.
- Rock type: Alluvial Deposits, Yemen Volcanics and Tawilah Sandstone

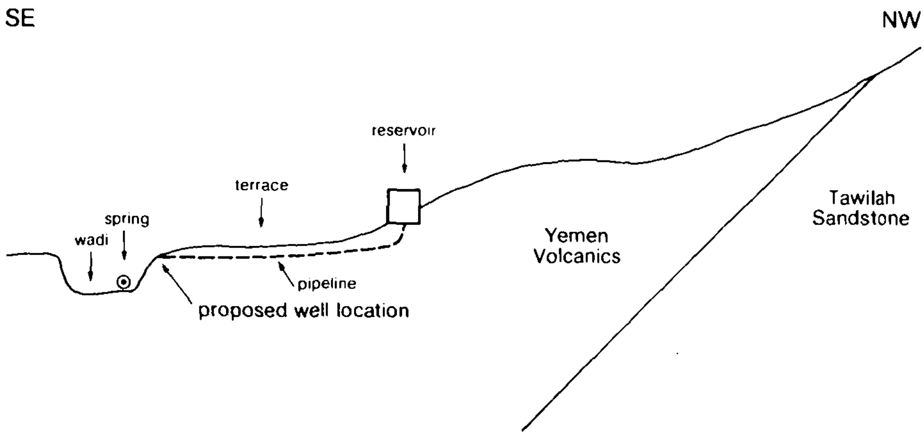

Fig. 2: Cross section at Suq al Urjayn

- Water source: a spring and a well in the wadi bed which are slightly artesian.
- Water quality: chemically good and, if managed correctly, also bacteriologically good.
- Water supply: During the dry season, people from Suq al Urjayn and adjacent villages take their water from the spring in the wadi. During the remaining 6 humid months, however, a water course runs in the wadi and the villagers then use this as their water supply.

At the ford near the lower part of the village we measured a flow of around 20 l/s in August 1983. At the upper end of the village, in contrast, the wadi was dry.

The following explanation of the recharge and discharge situation is rather sophisticated. Due to the narrowing of the valley filling, water does not come out of the Alluvial Deposits, although one might be inclined to think so at first glance. Water is recharged in the Tawilah Sandstone, passing through the Yemen Volcanics by means of unknown underground fissures and faults until it reaches, with slight artesian pressure the main spring point and several secondary points in the wadi bottom, where it then discharges. The artesian pressure as well as the chemical quality are typical for the hydrological situation described above.

- Improvement: Water there is largely sufficient to supply the market place and other surrounding villages. Moreover, a lot of the villages situated in the unfavorable Amran Limestones on the road from Suq al Urjayn to Al Mahwit could be supplied with that large amount of water.

Therefore we suggest a well with a bigger reservoir near the school. The well should be dug on the northern terrace slope only a few metres from the main spring. If possible, the well should have a depth of 25 metres.

From the edge of the terrace, water will then be piped across the square in front of the school to the reservoir which lies in a slightly elevated position on the slope. That point could be reached easily by water tankers which no longer need to go to the Tihama wells to supply water to the mountain villages.

Karady

In several places, where open water courses and springs are not present, like in Karady, people have to take their water from cisterns. The chemical water quality of cistern water is more than excellent (80 to 150 micromhos/cm), but from the bacteriological point of view these cisterns have to be regarded as breeding places of several diseases such as enteric fever or dysentery.

Therefore the prototype of a bacteriologically clean cistern working without an outside energy supply has been developed. The system comprises two reservoirs seperated by a semipermeable wall of filter brick. Both reservoirs are covered with a concrete roof. The second reservoir is equipped with a hand pump, and both with hatches for cleaning.

Hence, if storm run-off starts, water runs over the first sediment basins where boulder, gravel, sand and silt are deposited. These basins are open and must be emptied from time to time. After the last basin, water runs into the first reservoir, where it still contains clayey sediments and bacteria. As the filter wall is permeable, water will penetrate slowly through the filter wall into the second reservoir, reaching it as clean water. The filtering process will of course stop if the water level is identical in both reservoirs.

If the hatch is closed and the water is only extracted by the hand pump and not by a bucket with a rope, one can expect to get clean water over a long period. However, chlorination should be repeated every year after the first flooding. This prototype cistern was first built at the cistern of Karady near Al Mahwit town.

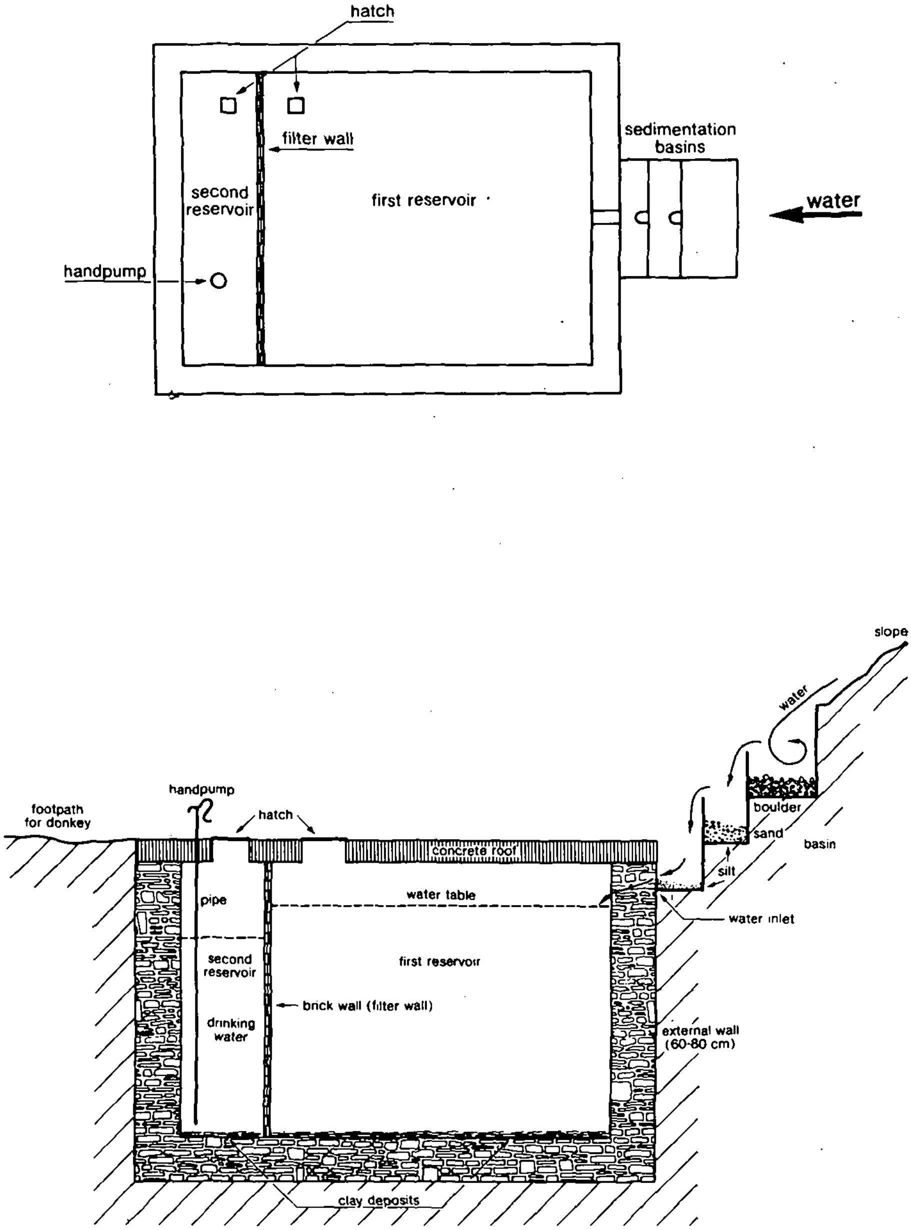

Fig. 3: Ground plan and vertical section of a two-chamber cistern, built at Karady

THE QUESTION OF MODERN RECHARGE

As we know from the latest research results of Sana'a Basin and surrounding area (JUNGFER 1982, 1983 and 1984) waters in Tawilah Sandstone and Yemen Volcanics may be 20 000 years old. The actual replenishment in this region which lies at the border of Al Mahwit Province is rather small.

In the Al Mahwit area, however, one need not worry about the water age, even though we are dealing with the bedrock from the same geological period as in Sana'a. Results from isotopic surveys carried out in Al Mahwit Province serve as evidence for the correctness of the hydrogeological judgement of the situation.

All water samples taken from the project area show a tritium content between 14 TU and 29 TU which is characteristic of a water which is at most 20 years old. Therefore tritium levels which are due to the termonuclear devices since 1952 justify the proposed quantities of water to be extracted.

The stable isotopes deuterium and ^{18}O which are excellent indicators of the climate during recharge time, were analyzed for recent and fossil reference samples, too. They indicate that the waters in the Al Mahwit Province are recharged under the present climate. Therefore we do not expect an exhaustion of the waters to be widespread.

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LA GESTION DE L'EAU ET SES PROBLEMES
Analyse sociologique à partir de l'exemple de quelques communes rurales de la plaine d'Alsace

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RESUME

Le présent papier, s'inscrivant dans le cadre plus vaste d'une réflexion pluridisciplinaire sur la gestion de la ressource eau en Alsace, n'a pour ambition que de mettre en situation réelle, sur la base d'enquêtes sociologiques de terrain, la manière dont le problème de l'eau est perçu par les groupes sociaux. Une simple "administration" de l'eau, indépendante trop souvent du rapport réel des partenaires sociaux à l'eau, à l'environnement en général, ne permet la plupart du temps pas de raisonner en terme de "gestion" globale.

PRESENTATION

Les premières investigations, menées dans la montagne vosgienne (Vallée de la Fecht) aussi bien que dans la plaine (Ohnenheim et Sundhouse) ont permis de dégager peu à peu une problématique qui nous paraît fondamentale et qui s'articule autour de deux éléments :

- d'une part la solidarité amont-aval, visible notamment à travers son disfonctionnement. Peut-être plus accentuée dans la Vallée de la Fecht, elle n'en est pas moins sensible dans la Plaine, dans sa surdimension même.
- d'autre part l'interdépendance quasi-générale au travers du vecteur eau : interdépendance des groupes sociaux (résidents, résidents secondaires, usagers domestiques, pêcheurs...), groupes professionnels (agriculteurs, non-agriculteurs, industriels...) groupes institutionnels (communes, administrations).

Toutes ces données doivent être confrontées dans une réflexion plus large, à la dimension de l'économie régionale, dépendante elle-même de la définition d'une politique régionale de l'eau devant aboutir à une "régulation" de la ressource.

Les enquêtes ici proposées participent à cette démarche, où il était intéressant en outre de vérifier ce qui avait été constaté précédemment dans le Ried (Ohnenheim et Sundhouse - cf Rapport PIREN 1983) où les rapports permanents des paysans à la nature semblaient les autoriser à exercer sur elle des actions régulières (travail du sol, pompage de l'eau, traitements...), actions "légitimes" pour des agriculteurs qui dénonçaient la pression des contraintes économiques qu'ils subissaient et dont le coût social (environnement)semblait revenir aux collectivités !

Si l'eau, à travers sa disposition et sa distribution permet d'être à la portée de tous, elle permet davantage de mettre en valeur les conflits qui opposent des usagers aux intérêts divergents, conflits qui trouveront réponse dans un compromis politique.

Parler de la gestion de l'eau, c'est parler des solidarités et, à travers elles, de la gestion globale ("régulation")de ces solidarités qui ne peut être que de l'ordre du politique.

ABREVIATIONS UTILISEES

CPIE	Centre Permanent d'Initiation à l'Environnement
DDE	Direction Départementale de l'Equipement
FDSEA	Fédération Départementale des Syndicats d'Exploitants Agricoles
PIREN	Programme Interdisciplinaire de Recherches sur l'Environnement
RGA	Recensement Général de l'Agriculture
SEITA	Société d'Exploitation Industrielle des Tabacs et des Allumettes

ILLHAEUSERN

Le choix d'Illhaeusern présente un intérêt certain pour l'étude des rapports de la population à l'EAU.

A la localisation particulière de cette commune dans le Ried, dans une cuvette proche des anciennes zones inondables du Rhin s'ajoute une appartenance administrative qui durant l'Histoire a été l'objet de nombreux conflits. Illhaeusern, après avoir été une annexe de Guémar et intégrée au Bas-Rhin, est passée au département du Haut-Rhin (1795), faisant partie du canton de Riquewihr, puis de celui de Ribeauvillé. De nombreux conflits économiques, mais aussi religieux (commune luthérienne de Guémar, commune catholique d'Illhaeusern) ont amené à une autonomie en 1833. Cette situation explique pour une part la position intéressante de cette commune qui aujourd'hui supporte encore les conséquences de sa position de "frontière politique" entre deux départements. Le partage des compétences administratives (l'Ill est domaniale et dépend du Bas-Rhin à Illhaeusern, commune haut-rhinoise) concernant la gestion de l'eau intervient dans l'inertie déplorée par les habitants victimes des inondations, il permet également d'éclairer le non-engagement politique en ce domaine où l'on préfère entretenir l'idée d'un consensus harmonieux.

Illhaeusern a toujours eu "les pieds dans l'eau". L'économie locale en témoigne. Au siècle dernier, les actions de pêche et de battellerie sont encore florissantes. Les agriculteurs pratiquent la polyculture et souvent sont des "double-actifs" (travail de tissage à domicile, osier...). Au début du siècle s'affirme la spécialisation laitière de l'agriculture par l'utilisation des prairies inondables... mais progressivement les paysans s'intéressent à l'acquisition de terres dans d'autres communes (le vignoble d'abord avec Bennwihr et Ribeauvillé, les champs labourables de Guémar et Colmar). Les années 1960 voient ici comme ailleurs une forte diminution de la population (454 habitants en 1954 contre 655 en 1826) surtout paysanne. En 1968, les agriculteurs ne représentaient plus que 58% des actifs, en 1975, 21%. Les exploitations agricoles se tournent de plus en plus vers les céréales et les cultures spéciales. Aujourd'hui, l'élevage laitier est maintenu par 5 ou 6 exploitants. Les conversions économiques ont été rapidement menées sous l'impulsion d'une politique propice à la céréaliculture et à la faveur d'années "sèches". De sérieux problèmes sont apparus avec le retour d'inondations fréquentes à partir des années 1982-1983.

Les interviews ont été menées auprès d'un échantillon représentatif de onze personnes. Une part importante (sept) a été accordée aux agriculteurs, non pas à cause de leur représentativité en nombre, mais en raison de la place qu'ils occupent dans la gestion de l'espace et de la commune (conseil municipal, présence du leader départemental de la FDSEA). De plus, il s'est avéré que les agriculteurs disposaient d'informations beaucoup plus riches, parfois variées et contradictoires, que les autres partenaires sociaux.

Résidents en général

La résidence à Illhaeusern correspond, comme le souhaite la politique municipale, à une implantation familiale plus ou moins longue. Elle peut être continue (les enfants "construisent" au lotissement) ou discontinue (la parenté ayant hérité d'une maison revient au village). L'installation dans la commune signifie une certaine intégration par la famille. Rares sont donc les "étrangers" (11 au recensement de 1975) nous avons choisi d'enquêter dans une famille ouvrière italienne tout particulièrement touchée par les inondations en 1983 (maison ancienne au centre du village). Dans ce cas précis, le désir de s'installer dans la commune s'explique par un choix délibéré d'habiter à la campagne en méconnaissance des conditions d'habitabilité.

L'habitat de l'échantillon considéré est plutôt ancien (après-guerre) : il a souvent fait l'objet de rénovations pour l'amélioration du confort. Dans 6 cas sur 11 les maisons ont été construites après 1945 : les fondations et cours sont alors généralement rehaussées à la différence des maisons anciennes du centre du village (constructions antérieures à 1930). Pour ces dernières de même que pour quelques maisons provisoires de la reconstruction ("barraques") les inondations ont causé des dégats semble-t-il bien plus importants, les rez-de-chaussée étant au même niveau que les cours généralement inondées. Les habitants de ces maisons que nous avons enquêtés ne sont pas directement originaires de la commune (immigrés, réfugiés de la dernière guerre, ou parenté lointaine). Ces familles sont les plus exposées aux inondations en raison soit de l'ancienneté soit de la vétusté de leur habitat. (1)

Pour ce qui est des constructions plus récentes, nombre d'enquêtés y ont fait référence à titre de comparaison : il apparaît qu'elles ne sont pas plus soumises à des risques d'inondation que la règlementation qui figure au POS est appliquée. Cette règlementation impose la construction de caves au niveau du rez-de-chaussée. Les maisons du lotissement ainsi construites, en tenant compte de l'affleurement de la nappe phréatique, semblent moins exposées aux inondations.

Les usages de l'eau par la population locale sont nombreux et il a semblé paradoxal que des problèmes de quantité ou de qualité puissent se poser considérant les avantages que l'on pouvait espérer. Ces avantages sont de différents ordres, le plus important étant le développement d'une vie économique locale par la présence de l'eau.

Une vie économique qui a connu de profonds changements ces dernières années et les habitants sont là pour rappeler en permanence les usages anciens de l'eau que les contraintes économiques extérieures ont conduit à l'abandon. Illhaeusern fut pendant longtemps une commune vouée à l'élevage et à la pêche d'écrevisses et à la culture de l'osier utile pour la région toute proche du vignoble. Ces activités sont tombées en déclin avec l'intensification de l'agriculture et la détérioration de la qualité des eaux due au rejet de déchets industriels en amont des rivières. L'activité piscicole par exemple a connu une forte diminution (on met en cause notamment la distillerie de Sigolsheim). Il reste un éleveur de truites professionnel à l'endroit dit "du Moulin" et celui-ci connaît des difficultés dans le développement et la vente de sa production (il se convertit progressivement dans la collecte de céréales). La pêche a connu un fort déclin dans la commune à la suite de la baisse reconnue de la qualité des eaux. Habitants, pêcheurs et éleveurs ont rappelé les problèmes sanitaires que soulève cette dégragation. La construction récente de stations d'épuration en amont de la commune (région de Colmar, collines sous-vosgiennes) a eu pour effet de "rassurer" une population inquiétée par la pollution de l'eau. On assiste depuis peu à la

(1) Les constructions de l'immédiat après-guerre sont mal adaptées aux conditions locales. Pour certains, les erreurs commises à l'époque relèvent de l'incompréhension entre architectes "français" et les anciens habitants, ces derniers ayant à subir les contraintes de constructions modernes standarisées.

réapparition de quelques pêcheurs.

En réalité les agriculteurs apparaissent comme les principaux bénéficiaires de l'abondance de l'eau dans cette région. Parmi les actifs agricoles, tous ont eu à leur disposition un ou plusieurs puits servant à l'irrigation des champs de maïs. Il est à noter que c'est le groupe qui semble le moins déplorer une diminution de la qualité de l'eau mais qui a également davantage conscience de l'abondance relative de l'eau en évoquant les périodes successives de pénurie et d'inondation. Pour les habitants les agriculteurs, en position dominante dans la commune, retirent d'abord un "profit" de la situation.

A la différence des usages domestiques de l'eau, les usages économiques n'ont pas vraiment disparu mais se sont adaptés aux transformations des structures économiques en dépit de la dépréciation lente et progressive qui s'est opérée au sujet de la qualité et donc de la confiance en l'eau.

C'est cette confiance que les habitants semblent avoir perdue. Avec l'apparition du sentiment que l'eau n'est "plus propre", les pêcheurs et les baignades disparaissent totalement. Personne depuis plus de 10 ans ne lave encore son linge dans les rivières. Après les inondations qui se sont traduites par une dégradation très nette de la qualité de l'eau courante dans les maisons ("elle n'était même plus bonne pour arroser le jardin"), la méfiance s'est installée et nombre de mères de famille préfèrent l'eau en bouteilles à l'eau du robinet. Certaines ont relevé le développement de maladies infantiles lors des inondations de 1983, d'autres évoquent l'aspect "malsain" de l'humidité installée dans les maisons surtout anciennes. Cette prudence dans l'utilisation de l'eau courante a paru plus particulièrement développée chez les femmes que chez les hommes. Chez ces derniers ce sont les agriculteurs qui sont les plus confiants (ils ont massivement répondu "oui" à la question "Buvez-vous de l'eau du robinet ?")... sont-ils réellement des consommateurs d'eau ?

On pourrait, en évitant toutefois de schématiser, retenir une double sensibilité : d'une part celle des habitants aux problèmes touchant à la qualité de l'eau; d'autre part celle des agriculteurs aux questions relevant de la régulation de la quantité de l'eau disponible. Ces deux points de vue, reposant sur des rapports différents aux usages de l'eau, pourraient paraître contradictoires. Ils semblent cependant relativement bien se compléter à Illhaeusern où l'eau n'est pas l'objet d'antagonismes profonds au sein de la collectivité locale.

1983 - une année exceptionnelle

D'après les habitants, les inondations de 1983, très spectaculaires, recouvrent les aspects d'une catastrophe exceptionnelle. Bien que "habitués à l'eau", certains habitants, parmi les agriculteurs en particulier, acceptent difficilement le caractère "naturel" du phénomène (c'est ainsi qu'il a souvent été présenté par la presse) qui mène à l'indifférence publique. Les risques d'inondation sont généralement reconnus et acceptés par la population locale dans le cadre de limites fixées par la mémoire collective. Un risque "normal" d'inondation dans la commune s'intègre dans l'histoire et la mémoire de l'histoire qui permet son acceptation dans la vie quotidienne. Une inondation à caractère exceptionnel appartient à la grille des limites fixées par la mémoire collective (1919, 1947-48) qui excluent de la normalité le phénomène. Cette attitude de rejet des habitants s'explique également par un sentiment d'impuissance entretenu par la non-prise de responsabilités des pouvoirs publics. Les références à la mémoire collective sont nombreuses dans les tentatives d'explication du phénomène par les habitants, tentatives qui rendent nécessaires la connaissance et la reconnaissance de la "sagesse" des anciens, des dictons et des prédictions de l'Almanach. Ainsi, s'il est connu que pendant les mois en "R" l'eau monte, on se souvient beaucoup plus rarement que "s'il y a des inondations avant l'Avent, l'eau montera encore 9 fois de suite". La constante référence au passé entretient la réification de celui-ci : l'"autrefois" sécurisant et harmonieux est opposé au présent inquiétant et en changement permanent. S'il convient certes de faire la part des choses dans l'interprétation de ce discours polarisé autour du passé et du présent, il faut néanmoins le retenir comme l'expression d'un rapport social des groupes à leur espace à travers les usages quotidiens. De façon unanime, les inondations apparaissent nettement plus "gênantes" pour les habitants aujourd'hui qu'autrefois. Ce sentiment est justifié par l'arrivée plus rapide de l'eau (une journée au lieu de 2 ou 3) aujourd'hui canalisée dans un lit soigneusement entretenu alors qu'"autrefois , on avait le temps de voir arriver l'eau".

Les habitants sont convaincus que certains aménagements (comblement de fossé, canalisation) ajoutés aux transformations des techniques agricoles (suppression du labour dans le vignoble...) ont accentué les risques d'inondations. Cette interprétation illustre parfaitement les contradictions locales qui opposent les usagers du "Haut" et ceux du "Bas" (amont/aval du cours d'eau). Les habitants d'Illhaeusern se sentent victimes de décisions politiques d'aménagement prises ponctuellement en amont des cours d'eau de leur commune, commune qui plus est, se situe aux confins du département.

Cette réaction de la population n'exclut pas la conscience que les usages économiques notamment ne sont plus comparables aujourd'hui avec ce qu'ils étaient d'antan. Autrefois les prés se trouvaient régulièrement dans l'eau et "l'eau était acceptée comme une fatalité"; aujourd'hui, selon les propos de cet agriculteur, "l'agriculture se compare à l'industrie", il en résulte que 50% des prés sont labourés. A cette évolution qui a semblé nécessaire à nos interlocuteurs, bien que davantage subie que voulue, s'est ajouté le sentiment d'une maîtrise partielle des problèmes techniques liés à la gestion de l'eau. Parce que les transformations économiques et techniques ont été subies sous forme de contraintes extérieures, les nuisances créées par elles semblent d'autant moins acceptées. C'est ainsi que, depuis que l'agriculture s'est orientée vers la céréaliculture dans les zones inondables, les habitants, agriculteurs et non agriculteurs, déplorent le développement de maladies, insistent sur l'aspect "malsain" et les odeurs qui se propagent aux pourtours des terres inondées. Dans le rejet de ces nuisances, c'est à la fois une évolution de l'agriculture imprégnée de l'extérieur ("aller vers le progrès") qui est contestée, et des rapports socio-politiques à l'espace local (opposition amont/aval) qui sont mis en exergue.

Il est généralement reconnu à Illhaeusern que ce sont les agriculteurs qui ont été les principaux sinistrés des inondations de 1983. Cependant il faut rappeler que l'ensemble des habitants a été touché par la montée de l'eau : généralement les caves ont été recouvertes d'eau, plus rarement c'est le rez-de-chaussée (anciennes maisons (1)). Parmi les terres ce sont surtout les terres à maïs qui furent immergées : les semis de 2 voire 3 récoltes sont perdus pour les exploitants. Si l'unanimité a pu être soulignée pour évaluer des dommages dont les agriculteurs ont principalement fait les frais, ce sont davantage les conflits et les rivalités qui dominent dans l'estimation et le partage des indemnisations. Les habitants et les agriculteurs s'opposent à ce sujet, les seconds jugeant trop généreuses les indemnisations perçues par les premiers. Dans le groupe des exploitants, il semble que les non-éleveurs (principalement céréaliers) se sentent plus lésés par le partage des indemnisations.

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(1) Seule l'auberge de l'ILL semble avoir été -fort heureusement- épargnée et ce grâce à un solide dispositif de pompes installées dans le sous-sol.

Le débat sur les raisons reconnues (1) par les habitants met en présence des usagers différents disposant d'une information inégale selon leur position sociale. La raison la plus souvent invoquée par les non-agriculteurs est d'ordre "micro-locale" c'est-à-dire résultant d'aménagements "non réfléchis" effectués dans la commune comme la nouvelle route d'Elsenheim qui fait un effet de barrage. Cette raison a été avancée par 6 habitants. Les agriculteurs préfèrent à cette première explication qui leur semble partielle (c'est également "le point de vue des femmes"), celle de la canalisation de l'Ill en amont. Enfin, une troisième catégorie, composée de responsables syndicaux agricoles, d'élus locaux, avance une autre explication qui est celle de l'inertie politico-administrative qui caractérise l'administration haut-rhinoise et surtout bas-rhinoise dénonçant l'absence de concertation régionale. Illhaeusern se situant aux confins des deux départements et en aval des cours d'eau, donne l'impression d'être peu prise en compte dans les décisions techniques locales et dénonce l'absence d'une politique globale.

Parmi les solutions envisagées, deux types de réponses sont fournies par les habitants selon leur accès à l'information et leur degré de participation à la vie locale. La première solution est de l'ordre de la réaction d'une population apparemment victime de l'indifférence et qui se réfugie dans un discours simpliste défendant la volonté de poursuivre la canalisation et l'aménagement de l'Ill. Cette attitude caractérise les habitants des couches les plus populaires (étrangers, retraités non agricoles) et certains agriculteurs. La seconde solution soutient le projet de construction d'un réservoir ou d'un canal de décharge vers la Hardt qui diminuerait la quantité d'eau s'écoulant en direction d'Illhaeusern au moins pendant les périodes les plus humides. Ce dispositif est envisagé comme un système de régulation car les responsables agricoles qui le soutiennent sont bien conscients qu'il ne faut pas organiser l'écoulement trop rapide de l'eau par ce canal, l'eau étant toujours indispensable à l'alimentation de la nappe phréatique nécessaire à l'irrigation des champs de maïs en été. Les défenseurs de la seconde solution se trouvent parmi les responsables municipaux et de la F.D.S.E.A. dont le leader est habitant de la commune. Ce groupe dont le noyau actif se compose avant tout d'agriculteurs, a déjà participé à de nombreuses actions et concertations et se définit par la recherche d'une solution à l'échelle régionale à travers la coopération des administrations des deux départements. L'initiative revient aux professionnels agricoles qui se sont engagés sur ce problème politiquement laissant derriière eux le pouvoir municipal dont on relève la position parfois tiède (2).

Agriculteurs

Chez les agriculteurs on observe un degré d'intensification relativement élevé. En effet la polyculture n'y est plus pratiquée (cf. tableaux) elle a cédé la place à des formes variées d'intensification tournées soit vers un élevage (engraissement bovin, élevage ovin) soit vers une culture de haut-rapport (chou, tabac, légumes). L'intensification par la voie de la spécialisation n'est pas à séparer de la présence dans le village du leader départemental du syndicalisme agricole.

Il y a 20 ans, les élevages spécialisés se sont développés dans la commune en raison de la présence abondante de l'eau : il s'agit de la pisciculture et de l'élevage de moutons. Ces activités sont en relatif déclin au-

(1) Selon l'avis général, il s'agit d'inondations qui sont à l'origine des catastrophes de 1983 et non la montée des eaux de la nappe.

(2) Ainsi ce conseiller général qui juge que des actions d'élargissement du pont et la règlementation concernant les vides sanitaires dans les nouvelles constructions sont des solutions provisoirement suffisantes.

joud'hui (concurrence aigue dans le premier cas, absence de succession dans l'autre). L'élevage laitier prend de plus en plus une forme intensive (6 producteurs dans la commune) et n'est plus associé à la polyculture. Qu'il soit bovin ou hors sol (poulets), l'élevage est considéré comme un choix de développement pour l'avenir d'une partie des exploitations (là où la main d'oeuvre est disponible et où la probabilité de succession est élevée). Les autres exploitants s'orientent prioritairement vers la culture de céréales (maïs) associée soit au chou soit au tabac (brun généralement) (1).

Considérant ces orientations dans les systèmes de production, il n'est pas étonnant d'apprendre que tous les agriculteurs de l'échantillon labourent les prés -une partie au moins- les transformant généralement en terres à maïs. La culture du maïs occupe plus souvent près de la moitié sinon plus des terres cultivées et les surfaces augmentent d'année en année.

Les cultures spéciales (chou, tabac, légumes) ainsi que les terres à maïs ont été gravement endommagées pour la saison 1982/83. Les premières ont très souvent été inexistantes et d'importantes parcelles de maïs couvertes d'eau deux ou trois fois ont été abandonnées par les exploitants. Certains ont réussi après 2 voire 3 échecs successifs dans les semis ou les traitements, à emblaver certaines de leurs parcelles. Le retard et les conditions défavorables du printemps ont conduit à une récolte largement insuffisante et tardive. Découragés, les agriculteurs ne savaient quel comportement adopter ne sachant si leur acharnement à mettre en terre deux ou trois fois de suite les pieds de maïs serait "payant", le cas échéant, reconnu (assurances, indemnisations). Les dommages s'avèrent importants et on estime les pertes de récolte allant de 60 à 80% pour le maïs et les cultures spéciales.

Les échecs successifs dans certaines récoltes conduisent les agriculteurs à douter de l'adaptation des systèmes de culture pratiqués aux conditions naturelles du milieu reconnu humide. Le passage des cultures fourragères et des prairies nécessaires à l'élevage et à la polyculture traditionnelle à la culture du maïs qui occupe une place centrale dans les systèmes spécialisés n'est cependant pas contesté. "Le maïs est toujours risqué ici" nous dit-on mais le choix des techniciens et des professionnels semble clair "la culture du maïs est adaptée aux Rieds mais ce n'est pas une région céréalière".

Le succès des récoltes précédentes et les avantages recherchés dans la pratique des cultures céréalières ont conduit nombre d'agriculteurs à convertir hâtivement les prés en terres à labour. Les remembrements de 1958 et 1977 (2) ont accéléré le mouvement de conversion des exploitations à des systèmes plus intensifs ou simplifiés. Le leader syndical local évoque "les abus" de certains exploitants dans cette conversion des prés en terres à maïs mais cela ne met pas en cause le système de culture et le modèle d'agriculture lui-même. Pour les responsables professionnels agricoles, l'agriculture

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(1) Ces deux cultures sont appelées à diminuer d'importance en raison des problèmes d'écoulement de la production sur le marché. L'organisation des producteurs de choux s'est dissoute en automne 1983 et la SEITA appelle ses membres à la conversion de la culture du tabac brun en tabac blond.

(2) Illhaeusern a bénéficié d'un premier remembrement partiel des terres cultivées par les agriculteurs sur le ban de Colmar en 1958 (1312 ha), suivi d'un second portant sur les terres communales, de Guémar et de Colmar en 1977. Ce dernier remembrement n'est pas apparu comme un facteur aggravant les risques d'inondations pour les agriculteurs; à la différence de celui de Guémar qui est dénoncé pour ses effets négatifs dus à la disparition des anciens fossés.

en tant qu'activité économique appelée à se transformer, n'est qu'un élément de la situation qui conduit à augmenter ou diminuer les risques d'inondation mais en aucun cas, elle n'est responsable de la situation.

Lorsque des transformations sont recherchées, elles ont d'abord trait aux techniques employées et à la variation de l'importance accordée à certaines cultures. Les inondations se traduisent ainsi pour les agriculteurs par un accroissement de la quantité de travaux superficiels nécessaires. Ils apportent des dosages d'engrais plus importants sur les parcelles inondées qui sont également soumises à des traitements herbicides et phytosanitaires plus forts (1). "L'eau apporte la maladie" et les agriculteurs sont à la recherche de "recettes techniques". Le problème le plus aigu se pose pour la culture du chou qui souffre très souvent de l'hernie. Du fait de nombreuses cultures fragilisées, exigeantes sur le plan des apports (fertilisants, traitements) et donc peu rentables (surtout par rapport au maïs) les exploitants ont tendance à simplifier la diversité culturale. Il s'agit plus particulièrement des céréales d'hiver (blé d'hiver), du colza mais aussi du tabac (2) et du chou.

Cette situation explique la tendance croissante à l'augmentation des parcelles en maïs d'année en année (5 exploitants sur 7 prévoient cette augmentation). Les cultures spéciales semblent actuellement en sursis et les blés sont en nette régression. La voie de la simplification se présente alors comme une solution risquée certes, mais peu exigeante en investissement et garantissant un revenu minimum. Si l'accent est mis sur la culture du maïs dans les tendances d'évolution, c'est autant parce que les agriculteurs n'entrevoient pas d'autre solution de rechange à court terme (les cultures spéciales sont en perte de vitesse (3)),que parce que la céréaliculture fonctionne comme un modèle. C'est le problème de la diversification qui se pose de manière aigüe à Illhaeusern et dans le Ried de façon générale.

MUTTERSHOLTZ

Muttersholtz est une grosse bourgade rurale, située en pleine zone inondable dans le Ried central de l'Ill. La commune compte encore une bonne trentaine d'agriculteurs (dont la moitié à temps partiel), mais elle ne peut plus être considérée depuis longtemps comme une commune agricole, même si le maire reste un agriculteur. D'ailleurs, fut-elle jamais agricole, la tradition la définissant plutôt comme artisanale (ateliers familiaux de tissage...) et surtout commerçante (une forte communauté juive autrefois et la Synagogue en témoignent).

Muttersholtz reste marqué par la grande diversité de ses composantes socio-professionnelles (voir tableau) : une minorité ouvrière, presque un tiers d'employés et cadres moyens, davantage de patrons de l'industrie et du commerce que d'agriculteurs, des cadres supérieurs et professions libérales

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(1) Les quantités d'engrais et de traitements employés sont deux à trois fois plus importantes à Illhaeusern qu'à Guémar pour un même agriculteur.

(2) La culture du tabac brun est appelée à disparaître; mais elle ne pourra pas être remplacée par celle du tabac blond d'après les premières expérimentations à Illhaeusern.

(3) Les producteurs de choux ont souvent récemment investi dans l'installation de matériel et ne sont pas prêts à abandonner cette production. De nouvelles orientations de production apparaissent cependant avec le développement de légumes de plein champs.

enfin. (Une grosse entreprise moderne sur place, une vingtaine d'artisans et petits entrepreneurs, neuf commerces et cinq restaurants, deux banques, deux médecins, un masseur-kinésithérapeute, un pharmacien, un chirurgien-dentiste, un notaire, un avocat).

Il est important de faire ce rappel, car la sensibilité commune vis-à-vis du problème de l'eau, des inondations en particulier, s'en ressent. La position de la mairie également s'en trouve affectée car, si les agriculteurs sont par principe les plus directement touchés par le problème des inondations, le Conseil municipal ne peut en faire une priorité compte tenu de la faible minorité que représente ce secteur d'activité. Les agriculteurs de ce fait, seront aussi plus attentifs à respecter les zones réputées inondables.

Les entretiens se sont déroulés auprès d'un échantillon représentatif de l'ensemble de la population, soit une trentaine de personnes spatialement réparties aussi bien dans l'ancien village, les deux lotissements ("*les cigognes*" - "*les acacias*") que dans les deux annexes EHNWIHR et RATHSAMHAUSEN. Il en ressort immédiatement qu'en effet, mis à part certains agriculteurs, les inondations ne posent pas un "problème", si ce n'est l'isolement (au sens étymologique du terme) du village par la submersion de la plupart des routes d'accès. L'écoulement de l'Ill semble parfaitement maitrisé et ici tout le monde sait que les inondations sont plutôt dues à une remontée de la nappe phréatique, phénomène auquel il est impossible de s'opposer. Aucune maison d'habitation, ni ancienne, ni nouvelle, n'a subi de dégats et l'un des interlocuteurs a même expliqué comment, pour sa maison neuve, des repères très précis ont été pris en compte pour maintenir la cave hors d'atteinte d'une éventuelle remontée de la nappe même pour les situations les plus extrêmes. Par ailleurs, dans cette même maison, on a su profiter de la présence de la nappe phréatique en installant une pompe à chaleur eau-eau, source unique de chauffage.

Les inondations du mois de mai 1983 ont effectivement été ressenties comme exceptionnelles.

L'entreprise "Mathis", qui a connu une importante expansion ces dernières décennies, a étendu son emprise sur des terrains reconnus comme "critiques", et elle a subi du coup des dégats relativement importants. Mais il s'agit du seul incident notable au niveau du périmètre bâti du village.

Le Syndicat des digues de l'Ill, dont le siège se trouve à Muttersholtz et qui regroupe sept communes voisines, a en charge l'entretien d'une digue construite au milieu du 19e siècle. Depuis lors ces communes étaient à l'abri des crues même les plus importantes de l'Ill. Malheureusement, la mémoire collective s'était quelque peu estompée, d'une part une section de la digue a été carrément supprimée lors du remembrement, il y a dix ans, d'une commune située en aval. En conséquence, une grande partie des parcelles de Muttersholtz vouées à la production de légumes en plein champ et hors d'atteinte de la nappe a été totalement sinistrée (l'un des agriculteurs les plus touchés a précisément été le maire). Depuis lors cette digue a été rapidement reconstituée. D'autre part, faute d'entretien, cette même digue s'est rompue en aval de Muttersholtz, inondant et détruisant les semis de maïs du lieu-dit "Willerhof", commune de Hilsenheim. Le Syndicat qui vivotait paisiblement depuis trop longtemps, se trouve brusquement devant un gouffre financier et doit faire appel aux deniers publics pour faire face aux responsabilités qui sont (et auraient dû) être les siennes. Mémoire et solidarité se sont trouvées sérieusement battues en brèche devant les événements.

Un fait révélateur de la brusque sensibilité que la municipalité manifeste depuis lors à l'égard des inondations est sa réaction récente face à une initiative unilatérale de la D.D.E. Le pont de l'Ill, permettant la traversée de la rivière à la route départementale sur le territoire de la commune, s'étant révélé vétuste, voire dangeureusement miné par les eaux, a été deposé pendant l'hiver 1983/84. En remplacement, la D.D.E. a commencé les travaux de reconstruction d'un nouveau pont. C'est à ce moment-là seulement que la municipalité s'est rendu compte que le nouvel ouvrage, plus étroit allait créer un goulot d'étranglement, provoquer des inondations en amont en ralentissant l'écoulement, voire menacer le village par le détournement des eaux. Les travaux ayant été arrêtés sur demande de la municipalité, il est à prévoir que les plans du nouvel ouvrage d'art seront révisés.

NON AGRICULTEURS

De manière tout-à-fait générale, chez les non-agriculteurs, le questionnaire et l'enquête ne les concernaient pas. ("Allez voir les agriculteurs ...") Les entretiens se sont déroulés malgré tout, et effectivement le discours des uns et des autres est toujours le même, apparemment celui de la municipalité dont personne ne nie l'action positive dans ce domaine. A part une inondation de cave, sans gravité, personne n'a souffert des dernières inondations. La nappe phréatique est généralement ressentie comme une richesse, on en boit l'eau ("même dans les biberons"), on sait l'utiliser pour des installations de pompes à chaleur eau-eau. Les inondations non plus ne sont pas ressenties systématiquement de manière négative. Certains interlocuteurs ont même parlé de la beauté du Ried sous l'eau, rehaussée par la présence d'une nombreuse faune ailée précisément attirée par l'eau, de parties de patins à glace sur les immenses étendues gelées etc...

Certains soulignent cependant une aggravation des inondations, et l'on accuse pêle-mêle : la commune de Mussig qui a aplani la digue de l'Ill; les agriculteurs qui font de la culture intensive de maïs, suppriment les fossés, retournant les prairies (dont surtout la directrice du "Wilherhof" qui spécule totalement sur la production intensive du maïs); les communes qui ont remembré leurs terres, supprimant là aussi les fossés (il s'agit surtout de Mussig et de Sélestat, Muttersholtz n'est pas remembré); le génie rural qui n'a pas su entretenir les digues, les canaux...

En dernière question, il a été demandé si la présence, à Muttersholtz, de la "Maison de la Nature" et du CPIE (Centre Permanent d'Initiation à l'Environnement) joue un rôle dans le rapport à l'eau. Les réponses n'ont pas toujours été explicites, mais la plupart reconnaissent qu'un travail efficace y est mené et qu'ils y ont visité à l'occasion des expositions consacrées au Ried, sa richesse naturelle, sa flore, sa faune.

AGRICULTEURS

Sur le plan général du rapport à l'eau, celle-ci est véritablement considérée comme une richesse, utile pour les arrosages, on se baignait et on lavait son linge dans l'Ill jusque vers les années 1960. Dans ce sens, la qualité de l'eau est perçue de manière négative actuellement et quelques craintes sont exprimées quant à l'avenir, surtout si cette mauvaise qualité doit s'étendre à la nappe phréatique. Pour ce qui concerne plus particulièrement les inondations, celles-ci sont loin d'être considérées comme un inconvénient majeur. Là aussi, cependant la situation semble considérée comme s'aggravant et l'on incrimine tout à la fois le secteur amont (Remembrement de Sélestat, la digue de Mussig, les travaux routiers sur le nouvel axe Rhin-Vosges, la création d'une zone industrielle de 100 ha à Sélestat mise hors eau, les travaux dans les vallées vosgiennes dont la conséquence unanimement reconnue est l'accélération des inondations qui s'étalent en moins de trois jours de pluie alors qu'autrefois il en fallait au moins six) et dans le secteur aval où l'on reproche au barrage de Krafft-Erstein de freiner l'écoulement des eaux. D'un côté le processus est accéléré, de l'autre côté il est freiné et le Ried central de l'Ill est pris entre les deux.

A partir des questions plus spécifiquement réservées aux agriculteurs dans la mesure où aucun des agriculteurs interviewés n'a spéculé sur la seule production du maïs, il ressort que les inondations sont rarement perçues comme catastrophiques, éventuellement acceptées comme une espèce de fatalité, notamment par un agriculteur qui a perdu en 1983 90% de sa production de légumes en plein champ. Alors même qu'il commençait à sortir de ses dettes... mais le fait qu'il pratique de la polyculture (lait, viande) lui assure une marge de sécurité. Ceux (et c'est la majorité) qui font de l'élevage ne se plaignent pas de la fertilisation naturelle provoquée par le limonage des inondations ("production de l'herbe la plus grasse d'Alsace" dit-on ici). De plus, certains ont rappelé que traditionnellement les agriculteurs de la commune vivaient également de la pêche. Outre cela intervient le fait que la commune tire des bénéfices des baux de chasse dont un tiers revient à la municipalité, au proprata de la propriété des terres. En effet plus de 300 ha (dont 68 ha de forêts) sur les 1130 ha du ban sont des terres communales. La municipalité n'autorise d'ailleurs pas les agriculteurs à retourner les prairies naturelles qu'elle leur loue. Elle reconnait néanmoins que certains paysans ont passé outre !. D'autres chiffres significatifs apparaissent dans les divers renseignements fournis par le R.G.A. (cf. tableaux).

Entre les recensements de 1970 et 1980, la surface des terres labourables n'a guère évolué. La surface des prairies n'a diminué que de 12,5%, le nombre de bovins a augmenté de 21%, ceci montrant bien le sens de l'évolution du système de production actuel.

La sensibilisation vis-à-vis de l'environnement s'exprime également par le fait que, en dehors d'un agriculteur s'étant orienté délibérément vers la production bio-dynamique, les autres agriculteurs, dont notamment le vice-président du syndicat agricole (FDSEA) lui-même, reconnaissent leur intérêt pour une agriculture plus "naturelle" alors qu'il y a dix ans encore ils trouvaient cela "ridicule". Mais cela provient plutôt d'une évolution ressentie comme "normale" des choses que de l'influence, entre autres, de la "Maison de la Nature" ou du CPIE dont aucun agriculteur ne semble reconnaître l'intérêt pour eux. L'agriculteur bio-dynamique s'est révélé d'ailleurs le plus réservé à leur égard.

L'attitude des agriculteurs se révèle finalement beaucoup plus comme une acceptation des contraintes locales, alors même qu'ils reconnaissent les propres limites de leur système de production et qu'ils ne condamnent pas systématiquement ceux qui, ailleurs, ont choisi la production intensive de maïs par retournement des prairies, faisant face quant à eux à d'autres contraintes, d'ordre économique celles-là, et au détriment, disent-ils, de l'environnement.

Double discours, de compensation et de justification, qui révèle en réalité leur situation ambigüe vis-à-vis d'un problème dont ils ne détiennent pas, à leur seul niveau local, la solution.

T A B L E A U X

R.G.A. Recensement général de l'Agriculture
1970/71 - 1979/80.

	ILLHAEUSERN			MUTTERSHOLTZ		
	1970/71	1979/80	Variation	1970/71	1979/81	Variation
Nombre d'exploitations	45	38	- 16 %	45	36	- 20 %
S.A.U. (ha)	773	809	+ 4,5%	608	589	- 3,2 %
Terres labourables	480	701	+ 32 %	370	376	idem
Céréales	—	567		—	252	
Cultures fourragères	—	37		—	87	
S.T.H.	268	99	- 63 %	224	196	- 12,5 %
Vignes	2	1		4	1	
Vergers	2	1		4	13	↗
Effectif des membres de la famille	212	163		187	154	
Salariés	2	3		9	2	
Tracteurs	40	52		42	58	
Bovins	354	267	- 25 %	584	737	+ 21 %
Ovins	433	339		10	6	
Porcins	209	23	- 89 %	163	66	- 60 %

Recensement de la population 1975

	ILLHAEUSERN		MUTTERSHOLTZ	
	Nombre	% de la population active	Nombre	% de la population active
Agriculteurs	44	21,6 %	50	8,5 %
Salariés Agricoles	6		4	
Patrons de l'industrie et du commerce	11	5,4 %	49	8,3 %
Professions libérales et cadres supérieurs	4	1,9 %	22	3,7 %
Cadres moyens	13	6,4 %	49	8,3 %
Employés	18	8,8 %	92	15,6 %
Ouvriers	92	45,3 %	299	51 %
Personnel de service	16	7,8 %	19	3,2 %
Autres actifs	5	2,4 %	6	1 %
POPULATION ACTIVE TOTALE	209	—	590	—
POPULATION NON ACTIVE	309	—	967	—
POPULATION TOTALE	518	—	1557	—

NOTE : Le R.G.A. recense les surfaces exploitées par les agriculteurs, y compris celles se situant sur d'autres communes.

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